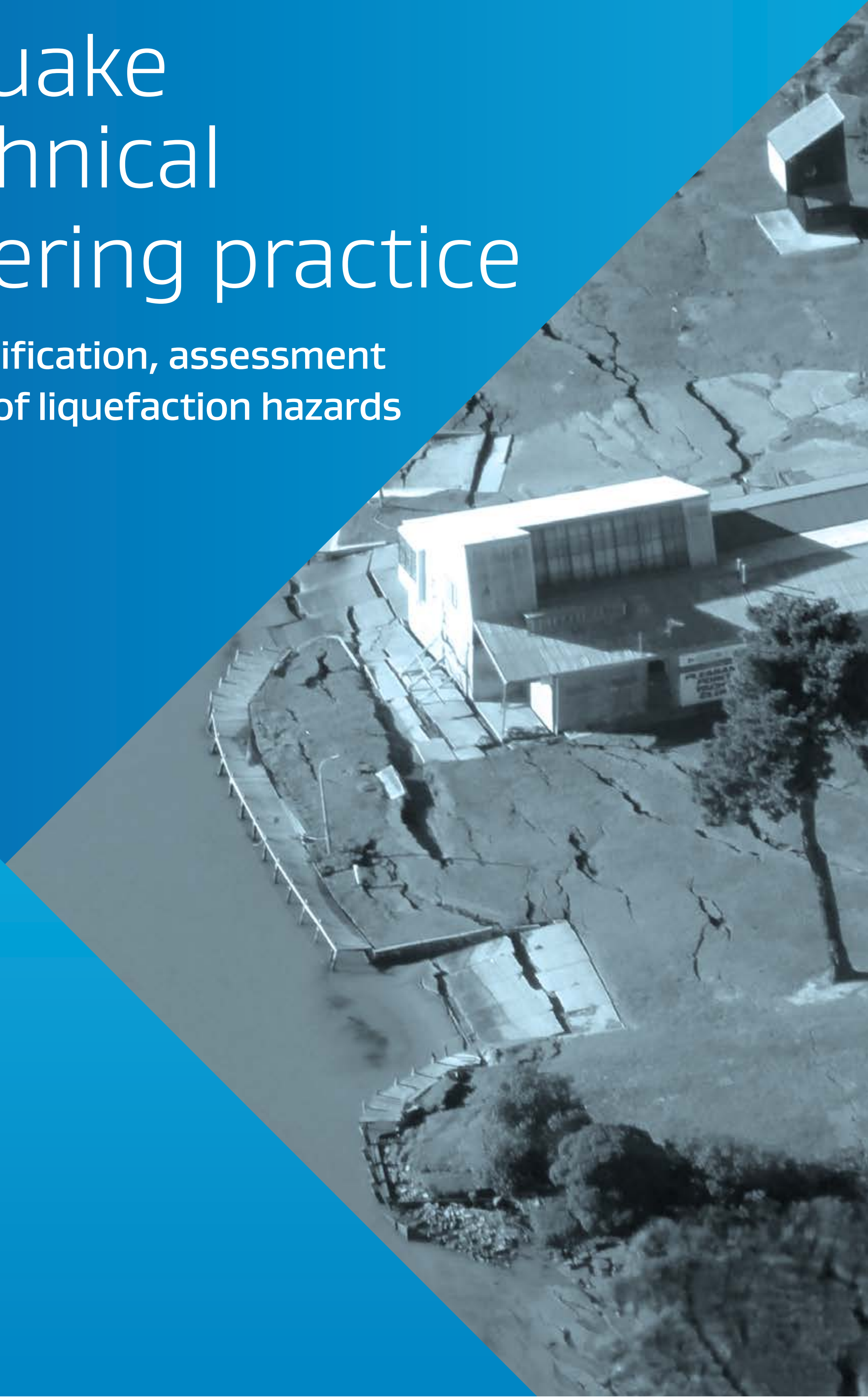
BUILDING PERFORMANCE

Earthquake geotechnical engineering practice

Module 3. Identification, assessment and mitigation of liquefaction hazards

November 2021



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Geotechnical earthquake engineering practice Module 1 — Guideline for the identification, assessment and mitigation of liquefaction hazards

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Preface

This document is part of a series of guidance modules developed jointly by the Ministry of Business, Innovation & Employment (MBIE) and the New Zealand Geotechnical Society (NZGS).

The guidance series along with an education programme aims to lift the level and improve consistency of earthquake geotechnical engineering practice in New Zealand, to address lessons from the Canterbury and Kaikōura earthquakes and the Canterbury Earthquakes Royal Commission recommendations. It is aimed at experienced geotechnical professionals, bringing up to date international research and practice.

This Revision 1 of Module 3 incorporates feedback received from the engineering community from the earlier revision and includes updated information from research and new developments since Revision 0 was published. It should be read in conjunction with other modules in the series:

- Module 1: Overview of the guidelines (incorporating hazard information)
- Module 2: Geotechnical investigations for earthquake engineering
- 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 sequence of strong earthquakes in Canterbury in 2010 to 2011, most notably the devastating M_W 6.2 earthquake on 22 February 2011, the source of which was located within Christchurch, resulted in 185 fatalities and extensive damage to buildings and infrastructure. Widespread liquefaction occurred on several occasions through the city and nearby areas. The damaging effects of this liquefaction included lateral spreading, settlement, foundation failures, subsidence of areas close to waterways, and large volumes of sediment ejecta on the ground surface. Since then, the damaging November 2016 M_W 7.8 Kaikōura earthquake has reinforced the need for robust earthquake geotechnical engineering investigation, design and construction processes.

On-line training material in support of the series is available on the MBIE and NZGS websites, www.building.govt.nz and www.nzgs.org/.

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

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1 Introduction

The Canterbury earthquakes triggered widespread liquefaction in the eastern suburbs of Christchurch, as well as rock slides, rock falls 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 could be attributed to the geotechnical impacts of the earthquake-induced liquefaction and rockslides.

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.

The high seismic hazard and profound relevance 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.

The high seismic hazard and profound relevance of geotechnical engineering were demonstrated in the Canterbury earthquake sequence. 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 100 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 nearly total devastation of the Central Business District with approximately 70 percent of its buildings being damaged beyond economic repair. The total rebuild cost has been estimated to be NZ$40 billion (NZ Treasury, 2014).

The geotechnical aspects and impacts of the earthquakes were of economic and societal significance.

More recently, in the 2016 Kaikōura (M_W 7.8) earthquake, surface fault ruptures and tens of thousands of landslides affected the source region causing severe damage to the transport infrastructure, lifeline networks and farmland. The Kaikōura earthquake caused disproportionate impacts in Wellington, despite the large distance from the source and relatively moderate shaking intensity that was produced in Wellington. One of the most notable impacts of the Kaikōura earthquake was the extensive liquefaction-induced damage at the port of Wellington (CentrePort).

> The 2010-2011 and 2016 earthquakes highlighted the high exposure of New Zealand to seismic geotechnical hazards, landslides and soil liquefaction, in particular.

The main aim of this guidance document is to promote consistency of approach to everyday engineering practice and, thus, improve geotechnical aspects of the seismic performance of the built environment. It is intended to provide sound guidelines to support rational design approaches for everyday situations, which are informed by latest research.

The first edition of the liquefaction guidelines (formerly Module 1 of the *Guidelines*) was published by the New Zealand Geotechnical Society in July 2010 shortly before the Darfield earthquake of September 2010 and was well received and timely, considering the subsequent events. It proved very useful in guiding practice during a period when a very large number of liquefaction site assessments were carried out following the Christchurch earthquakes and widespread liquefaction. The next edition of the liquefaction guidelines (Revision 0 of Module 3) was published in May 2016 as part of a suite of earthquake geotechnical engineering modules. This revision provides updated information from new research and experience.

The science and practice of geotechnical earthquake 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 as time passes.

This document is not intended to be a primer on soil liquefaction — readers are assumed to have a sound background in soil mechanics, earthquake engineering, and soil liquefaction theory, and to be qualified, professional geotechnical engineers.

Neither is it a book of rules — users of the document are assumed to have sufficient knowledge and experience to apply professional judgement in interpreting and applying the recommendations contained herein.

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 geotechnical earthquake engineering practice with a particular focus on soil liquefaction and lateral spreading.

The recommendations in this document are intended to be applied to routine engineering practice by qualified and experienced geotechnical engineers 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.

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 residential dwellings. The intention is that liquefaction hazards should be properly investigated and assessed at the subdivision stage of development. Then, simpler investigations and assessments would be adequate for individual sites. Professional judgement needs to be applied in all cases. The recommendations in this document can often still be applied to small projects without adding a significant cost and complexity.

The topic of site investigation planning and procedures is covered briefly in this document. More detailed information is provided in Module 2 of the Guidelines.

The topic of estimating ground motion parameters is covered briefly in this document. More detailed information is provided in Module 1 of the Guidelines.

The topic of mitigation of liquefaction and lateral spreading is covered briefly in this document. More detailed information on ground improvement as mitigation is provided in Modules 5 and 5a of the Guidelines. Information on seismic design of foundations (including liquefaction considerations) is provided in Module 4.

3 Soil liquefaction hazard

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. One of the principal hazards associated with earthquakes is soil liquefaction. As demonstrated by the 2010-2011 Canterbury earthquakes and 2016 Kaiokura earthquake, soil liquefaction is one of the dominant seismic hazards for urban communities and critical infrastructure in New Zealand.

The principal geotechnical hazards associated with earthquakes are:

› Fault rupture
› Ground shaking
› Liquefaction and lateral spreading
› Landslides.

This Module of the Guidelines is focussed on ground shaking and resulting ground damage, in particular liquefaction and lateral spreading.

3.1 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.

Three characteristics of ground shaking are typically considered in the engineering evaluation:

› Amplitude
› Frequency content
› Duration of significant shaking (ie time over which the ground motion has significant amplitudes).

These characteristics of ground motion at a given site are affected by a number of factors such as the source-to-site distance, earthquake magnitude, effects of local soil and rock conditions, rupture directivity, topographic and basin effects, 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.

3.2 Liquefaction and lateral spreading

The term 'liquefaction' is widely used to describe ground damage caused by earthquake shaking even though a number of different phenomena may cause such damage.

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

Liquefaction represents one of the most severe forms of soil failure and often results in damaging ground deformation and instability. 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 the case 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 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 undrained shear strength may be exceeded 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 8) 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. The summary in Idriss and Boulanger (2008) is helpful in clarifying issues and identifying adequate assessment procedures 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 (FC) of the soil is of lesser importance than its clay mineralogy as characterised by the soil's plasticity index (PI).

Engineering judgment 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 guideline document. 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, which are referenced in Section 7, but are not the subject of this document.

4 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, and the location of all active faults, and the degree of interaction between various fault segments during rupture.

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).

For engineering evaluation of liquefaction phenomena, the amplitude (commonly represented by the largest value of acceleration recorded during the earthquake, ie the peak horizontal ground acceleration, a_{max}) and the duration of ground shaking (related to earthquake magnitude, M_w) are the key input parameters to most common design procedures, with no direct consideration of the frequency characteristics (represented by the response spectrum).

The ground motion parameters at a site to be used for liquefaction hazard assessment 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 probabilistic seismic hazard analysis, PSHA (summarized in this guideline document)*
- *Method 2: Site-specific PSHA*
- *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
Method 3 is practically an extension and enhancement of Method 2.

A more detailed discussion of procedures for estimating ground motion parameters for geotechnical earthquake engineering purposes is provided in Module 1 of the Guidelines.

Several sources of data were used to define the seismic hazard for Method 1 presented in Module 1 (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, site-specific hazard definition was adopted based on results from a hazard study commissioned for this guidelines series.
3. For Auckland and Northland regions, the hazard for return periods RP ≥ 500 yr (ie ULS level and above) was supplemented with load specification corresponding to the lower bound ULS load requirements stipulated in NZTA Bridge Manual (2018; Section 6.2; Table 6.3, p6-6) and NZS1170.5 respectively.
4. Region-specific hazard definition for the Canterbury Earthquake Region based on the interim guidance provided by MBIE following the 2010-2011 Canterbury earthquakes.

For locations within the Canterbury earthquake region the following procedure is required:

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 considered to be the average over the period of 50 years following CES (ie 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 that 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.

5 Identification and assessment of liquefaction hazards

Cyclic behaviour of saturated soils during strong earthquakes is characterized by development of excess pore water pressures and consequent reduction in effective stress. In the extreme case, the effective stress may drop to zero or nearly zero (ie the excess pore water pressure reaches the initial effective overburden stress or the total pore water pressure rises to equal the total overburden stress) and the soil will liquefy.

In these Guidelines, liquefaction refers to the sudden loss in shear stiffness and strength of soils associated with the reduction in the effective stress due to pore water pressure generation during cyclic loading caused by an earthquake shaking.

The mechanism of pore water pressure build-up is governed by a generally contractive tendency of soils (or tendency to reduce in volume during shearing) under cyclic loading. When saturated soils are subjected to rapid earthquake loading, an immediate volume reduction in the soil skeleton (particle arrangement including voids) is prevented by the presence of incompressible water in the voids (pore water) and insufficient time for drainage to occur. The contractive tendency instead results in a build-up of excess pore pressure and eventual liquefaction. In this context, loose granular soils are particularly susceptible to liquefaction as they are highly compressible and contractive under cyclic shearing due to the high volume of voids in their soil skeleton.

It is important to emphasize that the rate of excess pore water pressure build-up, severity of liquefaction manifestation and consequent ground deformation strongly depend on the density of the soil.
In this regard, one can identify ‘flow liquefaction’ as an extreme behaviour of very loose sandy soils in which a rapid pore water pressure build-up is associated with strain-softening behaviour and undrained instability (flow failure).

Flow liquefaction results in practically zero residual strength and large ground deformation. In loose to medium dense sands, liquefaction results in a (nearly) complete loss of effective stress and rapid development of strains in subsequent cycles of shear stresses, but not leading to flow failure. Finally, dense sands exhibit transient liquefaction in which nearly zero-effective stress only temporarily occurs during part of each loading cycle (cyclic mobility), which is associated with a gradual development of strains in each subsequent cycle and limited deformational potential. These effects of soil density on the pore water pressure build-up, mechanism of strain development and consequences of liquefaction should be recognised and accounted for in the liquefaction assessment.

Figure 5.1 illustrates the effects of soil density on the liquefaction-induced ground deformation, where contours of maximum shear strains are shown as a function of cyclic stress ratios (CSR) and penetration resistances (q_{c1Ncs}) (Idriss and Boulanger, 2008). The plot clearly illustrates the significant differences in the maximum strain potential (or consequences of liquefaction) for sand deposits with different densities (ie penetration resistances).

Note

The maximum shear strain values corresponding to low penetration resistances in Figure 5.1 are overly conservative (ie too large) for level-ground free-field conditions, as they have been derived assuming presence of driving shear stresses associated with lateral spreading.

Assessment of the liquefaction hazard and its effects on structures involves several steps using either simplified or detailed analysis procedures. These Guidelines outline some of the available procedures and highlight important issues to consider when evaluating liquefaction susceptibility, triggering of liquefaction, liquefaction-induced ground deformation, and effects of liquefaction on structures. In this document, the term 'simplifioed (liquefaction evaluation) procedure' is used to refer to state-of-the-practice semi-empirical methods for assessment of liquefaction susceptibility, liquefaction triggering, and liquefaction-induced ground deformation.

Figure 5.2 illustrates through a series of flow-charts the principal steps in the simplified liquefaction assessment procedure and highlights important factors to consider in the engineering evaluation. It also outlines the organization of this section in which site characterisation, liquefaction susceptibility, liquefaction triggering, liquefaction-induced ground deformation, residual strength of liquefied soils, system response of liquefying deposits and effects of liquefaction on structures are covered. The final section provides guidance on the use of advanced numerical procedures as an additional approach to simplified procedures in the assessment of liquefaction problems.

Remedial techniques for mitigation of liquefaction and its consequences are briefly addressed in Section 6 of this guideline document, which is followed by brief sections discussing evaluation of clayey soils (Section 7), reclaimed land and constructed fills (Section 8) and volcanic soils (Section 9). Finally, Section 10 provides guidance on best practice considerations in the engineering assessment of liquefaction.

Figure 5.1: Maximum shear strains for clean sands with M=7.5 and σ'_{vc} = 1 atm (source: Idriss & Boulanger 2008)

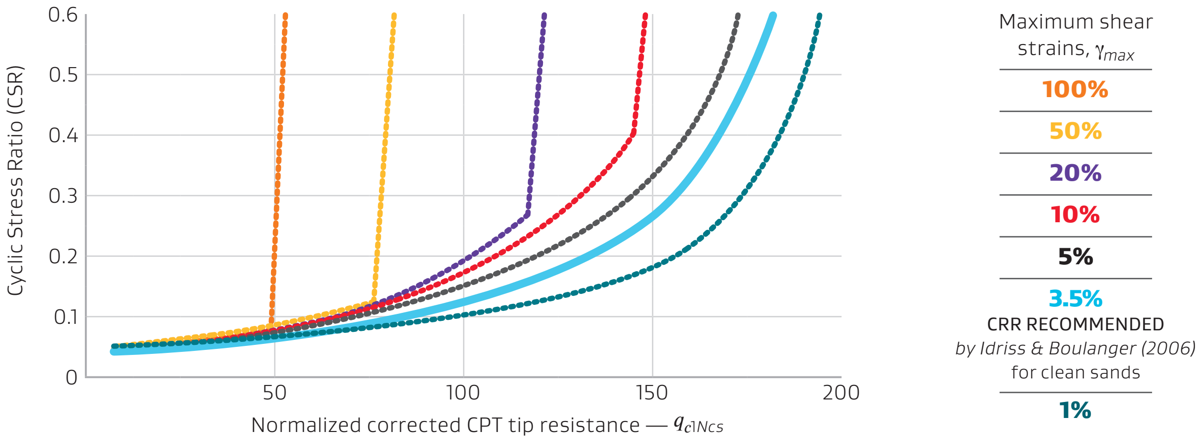


Figure 5.2: Factors to consider in liquefaction vulnerability assessment

a Overview of simplified liquefaction evaluation procedure

Site characterisation and seismic hazard evaluation

↓

Liquefaction susceptibility

↓

Liquefaction triggering

↓

Liquefaction-induced ground deformation

↓

Effects of liquefaction on structures

↓

Liquefaction mitigation

b Ground motion parameters, representative soil profile and liquefaction susceptibility (Sections 4, 5.1 and 5.2)

Ground motion parameters (seismic hazard assessment: a_{max}, M_w; de-aggregated magnitudes; a_{max} and M_w pairs for scenario earthquakes)

↓

Geology, geomorphology and geohydrology

↓

Subsoil stratigraphy and representative soil profile (soil type, thickness, index properties, water table depth, borehole, CPT, SPT, V_s/V_p)

↓

Liquefaction susceptibility (PI-based; I_c-based; age of soils, depositional environment, paleo-liquefaction and other historical evidence)

c Liquefaction triggering and liquefaction-induced ground deformation (Sections 5.3, 5.4 and 5.6)

Level ground conditions

↓

Liquefaction triggering (B&I, 2014)

↓

Effects of fines content, MSF

↓

Uncertainties in *GWL*, a_{max}, FC (*CFC*/I_c), *PI*/I_c

↓

Review and interpretation (critical layers, crust thickness and integrity, homogeneous or interlayered liquefied deposit, system-response effects, severity of liquefaction effects, land damage indices: LPI, LSN, . . .)

d Lateral spreading (Sections 5.4 and 5.5)

Lateral spreading

↓

Free face and topographic conditions; continuity of critical layers; geologic evaluation

↓

Flow-deformation potential (in situ state/loads versus residual strength)

↓

Uncertainties in *GWL*, a_{max}, FC (*CFC*/I_c), *PI*/I_c

↓

Spreading displacements:
– Maximum ground displacement
– Zone affected by spreading
– Lateral and verticaldistribution of displacements within spreading zone (stretch, offsets)

e Effects on structures (Section 5.7)

Quantify effects of liquefaction-induced ground deformation and earthquake loads on deformation and damage to the structure

↓

Consider uncertainties in kinematic and inertial loads; potential instability

↓

Location of structure and foundations in relation to critical layer(s) (liquefaction source and manifestation); impacts on services

↓

Estimated performance versus desired performance for relevant return periods

5.1 Site investigation and hazard identification

Module 2 provides detailed guidance on requirements for site investigations for earthquake geotechnical engineering purposes.

This section provides additional information relevant to investigating sites suspected of being susceptible to soil liquefaction. 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 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 property arising from structural failure and loss of amenity.

Most cases of soil liquefaction have occurred in relatively young (ie Holocene) deposits of poorly consolidated alluvial soils or fills with a high water table (saturated soils). Typically, these are fluvial or constructed fill deposits laid down in a low energy environment and which are normally consolidated. Such sites are often readily identifiable from a basic understanding of the regional geomorphology. Typical sites where liquefaction has been observed include river meander and point bar deposits, lake shore delta deposits, estuarine deposits, beach ridge backwater deposits (beach ridge and dune deposits are usually of higher density and not as prone to liquefaction but may overlie backwater deposits), abandoned river channels, former pond, marsh or swamp, reclamation fills and tailing dams. Such sites should be considered as having a high risk of liquefaction and be subjected to an investigation capable of identifying liquefiable strata.

All sites with potentially susceptible geological history/geomorphology should be considered a possible liquefaction hazard and be subject to a detailed investigation and liquefaction assessment appropriate to the scale and type of development.

New Zealand has a high rate of tectonic movement (uplift mostly) and has also been affected by Holocene sea level fluctuations. The present day surface geomorphology may obscure previous episodes of low energy deposition of liquefiable soils and care should be taken when predicting the likely sub-surface stratigraphy of a site.

Historical evidence for the site should be compiled and evaluated. This includes documents and data on local land use, fills, site features before construction and old river channels, waterways or land features associated with high liquefaction potential, as described above. The historical performance of the site in past earthquake events should be carefully considered in the site evaluation, whenever such evidence is available.

There are numerous case histories where liquefaction has occurred repeatedly at the same location during strong earthquakes. Hence, evidence of liquefaction in past earthquakes generally indicates liquefaction susceptibility of a given site. It is important to note that the opposite does not apply, ie the lack of evidence of historical liquefaction does not imply absence of liquefaction hazard.

Liquefaction can occur within strata at great depths, and this possibility is addressed in the simplified liquefaction evaluation procedure through a set of parameters and empirical relationships as described in Section 5.3. Current state-of-practice considers that for surface structures and shallow foundations the likelihood of surface damage decreases with increasing depth of liquefaction, and therefore liquefaction-related investigations are commonly limited to depths of 20m except for cases in which liquefaction at greater depths is also of particular concern such as thick reclaimed fills, deep foundations, or earth dams.

5.1.1 INVESTIGATION PLAN

The main objective of the site investigation is to identify susceptible soil strata and to evaluate the in situ state of susceptible soils. A suitable investigation should include the following features, as appropriate to the scale and type of development:

- Continuous profile of the subsoil (usually by Cone Penetration Testing (CPT) and/or borehole)
- Measurement of depth to water table
- In situ testing of all susceptible strata (usually by CPT or Standard Penetration Testing (SPT))
- Sampling of susceptible strata
- Composition of susceptible soils (ie grading, fines content, mineralogy)
- Atterberg limit tests for fine-grained soils (PI).

Evaluation of the in situ soil state will typically be carried out by penetration soundings (eg CPT, SPT) for 'sand-like' soils and by measurement of undrained shear strength and sensitivity (eg shear vane) for 'clay-like' soils. Intermediate soils (ie silty soils) can be evaluated with both penetration soundings and strength testing.

Where sampling of loose, cohesionless soils is impracticable because of difficulty retaining material within a sampler, it should be assumed that the soil is susceptible to liquefaction until proven otherwise.

There is often significant variation of subsoil stratification at sites with high-risk geomorphologies. Judgement should be used to develop a suitable investigation plan. Small, undetected lenses of liquefiable soils are unlikely to cause major damage but the risk of damage increases with increasing spatial extent of such deposits. The number of subsurface profiles necessary for a liquefaction assessment will vary with the size, importance of the structure, and spatial variability of the soil profiles at the site. The objective is to develop a geological model and understanding of the site so as to have a reasonable level of confidence of detecting significant liquefaction hazards.

Sampling and laboratory testing (fines content and Atterberg limits) should be carried out for all significant layers of 'suspect' soils that are identified (or, for small projects, where the cost of testing cannot be justified, conservative assumptions should be made).

Comment

For projects where the SPT is being used as the main investigation tool, the recovered SPT split-spoon samples should be used to carry out fines content and Atterberg limit measurements. Fines content measurements are necessary to make significant corrections to the SPT blow count readings. Without making fines content corrections, the liquefaction triggering analysis results may be very conservative. Likewise, without the Atterberg limit measurements it will be necessary to make conservative assumptions regarding the liquefaction susceptibility of the soils.

For projects where the CPT is being used as the main investigation tool, it is still recommended to carry out some drilling to confirm the stratigraphy and to recover samples for fines content and Atterberg limit measurements. Correlations between these soil properties and the CPT are poor in silty soils and may result in less reliable liquefaction triggering assessments unless site specific correlations based on sampling are available (see Section 5.3 for additional information).

5.1.2 INVESTIGATION PROCEDURES

Investigation of sites with liquefiable strata presents special difficulties. Simple procedures such as unsupported test pit excavations and hand augers are usually unable to penetrate far below the water table in loose, cohesionless soils. The Scala penetrometer is insufficiently sensitive and unable to achieve the required depth of profiling, and should not be used for liquefaction assessment. Procedures giving continuous measurement of the soil in situ state (eg CPT) are preferred because complex stratification is commonly associated with high-risk geomorphologies and even relatively thin strata of liquefiable soil may pose a significant hazard in some cases.

Procedures relying on recovery of undisturbed soil samples may fail because of the difficulty of recovering undisturbed samples of loose, cohesionless soils. Methods such as ground freezing may obtain higher quality samples, but also more practical methods for recovering undisturbed samples using 'gel-push' samplers and Dames & Moore (Osterberg-type) hydraulic fixed-piston samplers should be considered (Beyzaei et al., 2015; Stringer et al., 2015; Taylor et al., 2015) although it is noted that these methodologies may be uneconomic for smaller projects.

The following suitable investigation procedures are routinely available within New Zealand:

› Cone Penetration Test (CPT)
› Standard Penetration Test (SPT).

The Cone Penetration Test using an electronic cone (preferably CPTU where pore water pressure is measured), is the preferred in situ test procedure because of its sensitivity, repeatability, and ability to provide continuous profiling and to detect thin strata. Typically for larger projects, the CPT will be used to provide a grid of profiles across a site with a limited number of boreholes to recover samples from strata of interest. Some CPT rigs are able to recover samples using push-in devices. At some sites, susceptible strata will be overlaid by gravelly soils that refuse penetration by the CPT and it will be necessary to pre-drill through these soils.

The Standard Penetration Test (SPT) is performed using a standardised split-spoon sampler within a borehole that is supported with drilling mud or casing (ASTM D6066-11). It has the advantage that a disturbed soil sample is recovered after each test, but has the disadvantage that test depth-intervals are widely spaced and susceptible soil strata may be overlooked. A one-metre (or even 0.75 m) interval in measuring SPT resistance is recommended for collecting data in critical layers. A larger interval may be used in less critical layers, but SPT should be performed when new layers are encountered to ensure each layer has at least one SPT value to characterise it.

The SPT procedure has other technical limitations including relatively poor repeatability, operator dependence and often lack of critical information on the testing procedure (eg energy efficiency specific to the employed testing procedure). SPT energy should be measured and reported to greatly reduce the uncertainty in the collected SPT data and allow for their appropriate use in the evaluation. If the SPT is to be relied upon for an investigation, then the results should be carefully interpreted and corrected according to the recommendations of Seed et al. (1985), as summarised in Youd et al. (2001) and Idriss and Boulanger (2008).

Following the 2010 Darfield earthquake, Swedish Weight Sounding (SWS) testing has been introduced to New Zealand. Initial application and validation of SWS suggest that the test might be useful for quick and relatively cost-effective site investigations of individual residential properties as compared to more robust CPT, and SPT in particular. The SWS has been adopted in Japan as the standard field test for site investigation of residential land. Recently, an improved version of SWS, ie the screw driving sounding (SDS) test, has been shown to be effective for field characterisation in New Zealand (Orense et al., 2019). Further calibration and verification of SDS and SWS is needed in New Zealand in order to develop consistent testing procedures and interpretation of results, and to find an appropriate role for these tests in site investigations of individual residential properties.

Evaluation procedures using profiles of shear wave velocity versus depth are becoming widely accepted. However, shear wave velocity liquefaction triggering procedures are still not considered to be as robust as CPT-based procedures.

Typically, shear wave velocity profiles are obtained using a seismic CPT (a CPT probe with two in-built geophones that are separated a known distance is preferred) and performed in conjunction with a CPT sounding. Shear wave velocity measurements are commonly taken at set intervals (typically 1m) and so do not provide a continuous profile with depth. Seismic CPT procedures using two receivers are recommended since they substantially reduce the ambiguity in the interpretation of shear wave velocity measurements. Other techniques, both intrusive and non-intrusive, are also available for profiling shear wave velocity versus depth.

Penetrometer tests have been shown to be less effective in assessing liquefaction susceptibility in pumice soils. High quality undisturbed samples and specialised dynamic laboratory tests (eg cyclic triaxial or simple shear tests) may be considered for large projects. Shear wave velocity profiling may provide important information in conjunction with other methods for site and soil characterisation, but there is no database of proven case studies for these soils.

More comprehensive information and guidance on site investigations for liquefaction assessment are provided in Module 2 of the Guidelines.

5.2 Assessment of liquefaction susceptibility

This section discusses criteria for assessment of liquefaction susceptibility of soils.

Assessment of liquefaction hazard at a given site generally involves three steps to evaluate:

- Are the soils at the site susceptible to liquefaction?
- If the soils are susceptible, then is the ground shaking of the adopted design earthquake strong enough to trigger liquefaction at the site?
- If liquefaction occurs, then what will be the resulting liquefaction-induced ground deformation and effects on structures?

This section focuses on the first question and discusses two classes of criteria for evaluation of liquefaction susceptibility of soils.

5.2.1 GENERIC SUSCEPTIBILITY CRITERIA

Estimation of site-specific engineering properties of soils and site conditions is a key aspect in the evaluation of liquefaction potential at a given site. Initially, screening procedures based on geological criteria and soil classification are often adopted to examine whether the soils at the site might be susceptible to liquefaction or not. It is also worth noting that previous liquefaction doesn't improve soil liquefaction resistance to future events.

Geological criteria

The age of the deposit, its previous history and depositional environment during the formation of the deposit are important factors to consider when assessing liquefaction susceptibility. Geologically young (ie Holocene) sediments, constructed fills, and soils that liquefied previously in particular are susceptible to liquefaction (Youd and Hoose, 1977; Youd and Perkins, 1978). Most liquefaction-induced failures and nearly all case history data compiled in empirical methods for liquefaction evaluation were in Holocene deposits or constructed fills (Seed and Idriss, 1971; Seed et al., 1985; Boulanger and Idriss, 2008). It has been generally accepted that ageing improves liquefaction resistance of soils, however, ageing effects are difficult to quantify and are usually not directly evaluated in design procedures.

If ageing effects are used to increase liquefaction resistance of soils, then uncertainties in the estimate should be acknowledged and accounted for. Liquefaction has been reported in late Pleistocene sediments (Youd et al., 2003), though such episodes are rare and comprise a small part in the total body of liquefaction case histories. For more important projects, liquefaction in Pleistocene sediments should be checked including likely consequences of liquefaction.

It should be noted that time since last liquefaction event supersedes deposition age. For example, the deposits in Christchurch that liquefied in the 2011 Canterbury earthquakes will be considered as recent deposits in the liquefaction evaluation, as their 'age-clock' was reset in 2011.

There are no widely accepted methods or criteria for quantification of effects of ageing on liquefaction resistance of soils. Several research studies on ageing effects (eg Andrus et al., 2009; Saftner et al., 2015) have shown the following important characteristics:

a. There is a significant uncertainty in the correlations between the shear wave velocity (V_S) and time (t), and the liquefaction strength and time of the deposit.
b. In order to estimate an appropriate age of the deposit for liquefaction evaluation, the shorter time either since initial soil deposition or last critical disturbance (eg liquefaction occurrence) should be estimated.
c. The increase in the shear wave velocity V_S with time is relatively small (eg approximately 8 percent per log cycle of time), which is well within the measurement error even in the case of well-executed and carefully interpreted field measurements of V_S.
d. The derived empirical relationships for evaluation of ageing effects on the liquefaction resistance are based on limited data of clean sands and silty sands, for typical liquefiable soils.

Given the above uncertainties, complexities and limitations of the empirical relationships, the following approach and principles are recommended for the evaluation of ageing effects on the liquefaction resistance of soils:

1. The shorter time since the initial soil deposition or last disturbance (eg historical liquefaction occurrence) should be first conservatively estimated.
2. Empirical relationships could then be used to preliminary estimate the potential gains in shear wave velocity and liquefaction resistance, for the estimated age of the deposit or time since the last critical disturbance.
3. If a notable increase in V_s and liquefaction resistance is indicated from the empirical relationships, then field investigations such as shear wave velocity measurements (followed by an interpretation in conjunction with the penetration resistance, eg an MEVR analysis (Andrus et al, 2009; Saftner et al, 2015) or equivalent) or recovering high-quality samples and cyclic testing in the laboratory should be performed to demonstrate the beneficial effects of ageing for the deposit (soils) of interest.
4. Uncertainties in the estimates of effects of ageing on the liquefaction resistance should be incorporated in the assessment while accounting for the specific ground conditions, soil characteristics, depositional and hydrogeological environment at the site of interest. Input from geological studies and use of historical evidence on the site performance during past earthquakes are essential in this assessment.

A direct use of generic empirical relationships (to increase the liquefaction resistance due to ageing) without the level of scrutiny outlined above is considered inappropriate and could lead to an overestimation of the liquefaction strength of soils.

Compositional criteria

Classification of soils based on soil type and grain-size composition was commonly used in the past for preliminary evaluation of liquefaction susceptibility. Criteria based only on grain size distribution are now generally not accepted for evaluation of liquefaction susceptibility.

Most cases of liquefaction have occurred in saturated, cohesionless, fine sands. However, there is abundant evidence of liquefaction occurring in non-plastic and low-plasticity soils outside this range (eg silts and gravel-sand-silt mixtures). In the 1999 Kocaeli (Turkey), 1999 Chi-Chi (Taiwan), 2000 Tottori (Japan) and the 2010 to 2011 Christchurch earthquakes, extensive liquefaction occurred in sands containing various (including significant) amount of fines. In the 1995 Kobe (Japan) earthquake, massive liquefaction occurred in well-graded reclaimed fills containing 30 percent to 60 percent gravels. Similarly, in the 2016 Kaikoura earthquake, severe liquefaction occurred in gravelly reclamations at the port of Wellington (CentrePort) containing 55-75 percent gravels and 25-45 percent sand and silt. Thus, soil gradation criteria alone are not a reliable indicator of liquefaction susceptibility.

There is general agreement that sands, non-plastic silts, and gravels and their mixtures form soils that are susceptible to liquefaction. Clays, on the other hand, even though they may significantly soften and fail under cyclic loading, do not exhibit typical liquefaction features, and therefore are considered non-liquefiable. The greatest difficulty arises in the evaluation of liquefaction susceptibility of fine-grained soils that are in the transition zone between the liquefiable sands and non-liquefiable clays, such as silts and sands containing low-plasticity silts or some amount of clays. Complex mixtures of gravel, sand and silt are also problematic for liquefaction evaluation as their behaviour depends on the proportion of different fractions in the mix. Similarly, alluvial deposits containing volcanic soils involve various complexities as they have fundamentally different characteristics and behaviour as compared to quartz-based sandy soils that form the empirical database for liquefaction assessment. Liquefaction evaluation of volcanic soils is discussed in Section 9 of this document.

5.2.2 SUSCEPTIBILITY CRITERIA BASED ON GEOTECHNICAL PARAMETERS

There are numerous subtle differences between the undrained responses of sands and clayey soils. Pore pressure rise in clayey soils is typically limited to 60 percent to 80 percent of the effective overburden stress whereas in sands, gravels and non-plastic silts the excess pore pressure can rise to 100 percent and equal the effective overburden stress. Based on principal characteristics of undrained behaviour and relevant procedures for their evaluation, Boulanger and Idriss (2004, 2006) identified two types of fine-grained soils, those which behave:

- more fundamentally like clays (clay-like behaviour) and
- more fundamentally like sands (sand-like behaviour).

Plasticity index-based criteria

The guidelines for treatment of fine-grained soils herein are based on the knowledge and recommendations from recent studies such as those by Boulanger and Idriss (2006) and Bray and Sancio (2006). Liquefaction susceptibility of fines-containing soils (FC > 30%, where FC = percent of dry mass passing through a 0.075 mm sieve) in the transition zone is simply characterised based on the plasticity index (PI) as follows:

- **PI < 7; Susceptible to Liquefaction**: Soils classified under this category should be considered as 'sand-like' and evaluated using the simplified procedure for sands and non-plastic silts presented in these Guidelines.
- **7 ≤ PI ≤ 12; Potentially Susceptible to Liquefaction**: Soils classified under this category should be considered as 'sand-like' and evaluated either using the simplified procedure for sands and non-plastic silts or using site-specific studies including laboratory tests on good-quality soil samples.
- **PI > 12; Not Susceptible to Liquefaction**: Soils classified under this category are assumed to have 'clay-like' behaviour and are evaluated using the procedure outlined in Section 7. Importantly, some clay soils can undergo significant strength loss as result of earthquake shaking, therefore classifying these soils as not susceptible to liquefaction does not imply that they are inherently stable.

The so-called 'Chinese Criteria', which have been traditionally used to determine liquefaction susceptibility of fine-grained soils, should no longer be used (Boulanger and Idriss, 2006; Bray and Sancio, 2006). Current understanding of the seismic behaviour of fines-containing sands is limited and therefore in cases where characterisation of such soils is difficult, the soils should be either conservatively treated (as liquefiable) or detailed laboratory testing should be conducted.

Preferably, soil samples should be obtained from all soil layers of concern so that the fines content and plasticity index can be measured by standard laboratory (index) tests. Samples recovered from the SPT split-spoon sampler are suitable for this purpose.

I_c-based criteria

Where CPT data alone is available, without any sampling, then liquefaction susceptibility may be evaluated by use of the soil behaviour type index, I_c, calculated from the CPT data (Robertson and Wride, 1998, summarised by Youd et. al., 2001). The following criteria are recommended:

- Soils with $I_c \leq 2.6$ are susceptible to liquefaction
- Soils with $I_c > 2.6$ are most likely too clay-rich to liquefy
- Soils with $I_c \geq 2.4$ should be sampled and tested to confirm the soil type and susceptibility (or assumed liquefiable)
- Soils with $I_c > 2.6$, but with a normalised friction ratio F < 1%, may be very sensitive and should be sampled and tested (or assumed liquefiable).

  Here, $$F = \frac{f_s}{q_c - \sigma_{vo}} \times 100 \qquad (\%)$$

  in which f_s = cone sleeve resistance, q_c = cone tip resistance, and σ_{vo} = total vertical stress.

Comment

In practice, $I_c = 2.6$ is commonly used as a threshold for separating between liquefiable and non-liquefiable soils. Using comprehensive data from 15,000 CPTs and 6,000 laboratory tests, Lees et al. (2015) have concluded that the $I_c = 2.6$ cut-off threshold is appropriate for identifying Christchurch soils susceptible to liquefaction.

Deviations from the $I_c = 2.6$ threshold value may be appropriate, but should only be adopted in the liquefaction evaluation if proven by substantial testing of the subject soils and rigorous scrutiny via other susceptibility criteria, or, if there is evidence that $I_c = 2.6$ is not an appropriate threshold based on observed performance.

Robertson (2009), and Robertson and Cabal (2014) provide some updates of the procedure of Robertson and Wride (1998) with regard to the overburden stress correction factor and the threshold I_c value separating liquefiable and non-liquefiable soils.

Reliance on I_c alone, in some cases, may give conservative results in silty soils and supplementary soil sampling and testing is recommended.

For high risk/high consequence projects, I_c should not be relied upon and soil sampling and laboratory testing should be carried out to confirm liquefaction susceptibility.

5.3 Assessment of liquefaction triggering

For all soils identified as susceptible to liquefaction, triggering of liquefaction should be assessed throughout the depth of the layer. There are several approaches available for assessment of triggering of liquefaction.

These guidelines recommend the widely used CPT and SPT-based *simplified procedure* based on the empirical method originally proposed by Seed and Idriss (1971) and Seed et al. (1985), as summarised in the NCEER Guidelines by Youd et al. (2001), and more recently by Idriss and Boulanger (2008), and Boulanger and Idriss (2014). In this revision of the Guidelines, the most recent update by Boulanger and Idriss (2014) is recommended as it offers some additional insights in the liquefaction evaluation and incorporates evidence from recent earthquakes including the 2010–2011 Christchurch earthquakes.

As stated previously, evaluation procedures using profiles of shear wave velocity versus depth are becoming widely accepted, however, shear wave velocity liquefaction triggering procedures are still not considered to be as robust as CPT-based procedures. Other simplified methods based on energy considerations are also available although these methods are not in common usage.

It is essential that whichever method is chosen, it is consistently and rigorously applied following the recommendations of the particular method for each step in the liquefaction evaluation.

In the *simplified procedure* described herein, estimation of two variables is required for evaluation of liquefaction triggering the:

› Cyclic Stress Ratio (CSR), which represents the seismic demand on a soil layer caused by the design earthquake shaking, and

› Cyclic Resistance Ratio (CRR), which represents the capacity of the soil to resist liquefaction.

Comment

The predictive capacity of four CPT-based liquefaction assessment methods was scrutinised in two detailed studies on the Christchurch liquefaction (van Ballegooy et al., 2015; Green et al., 2014). The examined methods were Robertson and Wride (1998), Idriss and Boulanger (2008), Moss et al. (2006) and Boulanger and Idriss (2014). The aforementioned studies used extensive CPT data and detailed documentation of liquefaction manifestation observed in Christchurch after the 2010 to 2011 Canterbury earthquakes. The van Ballegooy et al. (2015) study assessed liquefaction vulnerability based on a comprehensive CPT data set of about 15 000 tests. The Green et al. (2014) study assessed liquefaction triggering based on detailed analysis of 25 well-documented case studies from Christchurch. In both studies, generally consistent results were obtained across the triggering methods, though both studies indicated slightly higher level of accuracy for the Idriss and Boulanger (2008) and Boulanger and Idriss (2014) methods. Further updates and calibration of the triggering relationships for the Idriss and Boulanger (2008) method based on the Christchurch data were included in Boulanger and Idriss (2014). A brief summary of important differences between the method of Idriss and Boulanger (2008) and Boulanger and Idriss (2014) is provided in Appendix A.

The liquefaction triggering factor (F_L) is computed using Equation (5.1):

$$F_L = \frac{CRR}{CSR} \tag{5.1}$$

in which: CRR = Cyclic Resistance Ratio
CSR = Cyclic Stress Ratio

Liquefaction triggering is indicated if $F_L \leq 1.0$. The triggering factor F_L is determined for liquefiable (ie susceptible) soils throughout the depth of the deposit. Methods of calculation for CSR and CRR are given in full detail in Boulanger and Idriss (2014).

During ground shaking, the soil is subjected to cyclic shear stresses. For the purpose of liquefaction evaluation, these cyclic shear stresses are expressed in terms of the Cyclic Stress Ratio (CSR):

$$CSR = \frac{\tau_{cyc}}{\sigma'_{vo}} \tag{5.2}$$

in which: τ_{cyc} = cyclic shear stress
σ'_{vo} = effective vertical stress at depth z

For routine projects, CSR can be estimated using the simplified expression proposed by Seed and Idriss (1971) given in Youd et al. (2001):

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \tag{5.3}$$

in which:

a_{max} = peak horizontal acceleration at the ground surface
(Note: a_{max} is an estimate for the peak ground acceleration at a level site for a hypothetical response without effects of excess pore pressure, liquefaction or surcharges). See Section 4.

g = acceleration of gravity (in same units as a_{max})

σ_{vo} = total vertical stress

σ'_{vo} = effective vertical stress

r_d = stress reduction factor as a function of depth that accounts for the dynamic response of the soil profile

(Note: Boulanger and Idriss (2014) give an updated expression for calculating r_d as a function of depth and M_w)

Values for the peak ground acceleration a_{max} required in Equation 5.3, and for M_w required for calculating r_d, are obtained using one of the methods described in Section 4. Details on the determination of ground motion parameters are given in Module 1.

Whether liquefaction will be triggered or not in a given layer depends both on the amplitude and on the number of cycles of shear stresses caused by the earthquake. In this context, CRR represents a stress ratio that is required to cause liquefaction in a specified number of cycles and in effect indicates the liquefaction resistance of the soil.

For each liquefiable layer consisting of sands, non-plastic silts or fine-grained soils considered to be susceptible in Section 5.2, it is recommended that CRR be estimated using the Boulanger and Idriss (2014) procedures based on penetration resistance (SPT or CPT). The corresponding NCEER (Youd et al., 2001) criteria based on the shear wave velocity (V_s) might be useful for assessment of relatively clean gravels in which penetration tests cannot be performed.

In the simplified procedure, CRR is evaluated by means of semi-empirical charts (relationships) for a magnitude $M_w = 7.5$ event and corresponds approximately to the shear stress ratio that causes liquefaction in 15 uniform cycles. A correction factor, so-called Magnitude Scaling Factor, MSF, is then used to estimate CRR for different earthquake magnitudes or number of cycles.

Comment

The MSF relationship of Boulanger and Idriss (2014) considers not only the effect of increasing duration (or numbers of cycles) with earthquake magnitude but also accounts for differences in the soil response depending on soil density (represented by penetration resistance).
It implies that the MSF varies significantly in dense sands (high penetration resistance), while the variation of MSF with M_w is much smaller for loose sands (low penetration resistance).
Further discussion is included in Appendix A.

Adjustments are also made to CRR for overburden pressure (ie depth, represented by K_σ, overburden correction factor, in the simplified procedure).

Evaluation of depth effects

The simplified procedure is recommended for use up to 15 m depth. If it is employed for greater depths, then additional analyses and considerations should be given to effects of depth and overburden stress on both CSR and CRR.

Comment

In their recent revision of the CPT-based liquefaction triggering procedure, Boulanger and Idriss (2014) indicated that the database of CPT liquefaction case histories is limited to depths less than 12 m with very few data points for depths greater than 9 m. One should also acknowledge that in the simplified triggering procedure, liquefaction at various depths is considered by using a set of parameters incorporating the effects of depth on the seismic demand (stress reduction factor, r_d), penetration resistance (normalizing factors C_N or C_Q) and liquefaction resistance (overburden stress factor, K_σ). There are significant uncertainties with these parameters for depths greater than those covered in the database. With this background in mind, the direct application of simplified liquefaction evaluation procedures should be limited to 15 m depth. Extrapolation to 20 m depth should account for the increased uncertainties at depths greater than 15 m by evaluating the effects of variation in parameters, over the relevant range of their values, through sensitivity studies.

Importantly, empirically-based liquefaction triggering procedures divide case histories into two categories:

› cases where surface manifestations of liquefaction were observed (ie liquefaction cases)

› cases where surface manifestations of liquefaction were not observed (ie no liquefaction cases).

In the latter case, it is possible that liquefaction may have occurred deeper within the soil profile, but a non-liquefiable surface layer could have obscured its manifestation on the ground surface. Further discussion on such cases is provided in Section 5.6. Thus, the simplified empirical method appears to be the most suitable for evaluating shallow liquefaction and for identifying those cases where surface manifestations of liquefaction are likely to occur.

Special expertise and considerations are required for liquefaction evaluation at greater depths such as in the case of deep foundations, earth dams, tailing dams or thick reclaimed fills. In such evaluations, cyclic stress ratios (seismic demand) at depths greater than 20 m should be evaluated using dynamic analyses, including considerations of uncertainties, and variations associated with the ground motion characteristics and dynamic ground response. Effects of large depths and high overburden stresses on the liquefaction resistance of soils should be carefully evaluated using experimental evidence from relevant soils. Consequences of liquefaction should be considered in the context of the particular structure, including stability, deformation and interaction issues.

The following approach is generally recommended in the evaluation of depth effects in liquefaction assessment:

1. Direct use of simplified procedures up to 15m depth
2. Extrapolation of simplified procedures from 15 m to 20 m depth while accounting for uncertainties in estimates of the stress reduction factor (r_d) and overburden correction factor (K_σ)
3. For depths greater than 20 m, uncertainties in both seismic demand and soil response should be carefully evaluated including use of dynamic analysis.

Effects of fines

Values for CRR correlated to CPT and SPT depend significantly on the fines content (FC) of the soil for two main reasons:

› the presence of fines affects the liquefaction resistance of soil, and

› the presence of fines reduces the penetration resistance measured in the CPT and SPT.

> **Comment**
> Further discussion on the effects of fines on the liquefaction resistance of sandy soils including the significant effects of fines content on the penetration resistance of soils — which is inherently embodied in the semi-empirical liquefaction-triggering charts based on CPT or SPT — is given in Cubrinovski et al. (2010) and Cubrinovski (2019).

Accordingly, in the Boulanger and Idriss (2014) procedure, the determination of the FC of the subject soil is critically important to the liquefaction triggering analysis (and also to the determination of liquefaction susceptibility, as discussed in Section 5.2). Adjustments are made to increase the measured values of CPT (q_c) and SPT (N) in fines-containing soils to estimate an 'equivalent clean-sand' penetration resistance using the procedure of Boulanger and Idriss (2014).

For the SPT, it is straightforward to recover the split-spoon samples and have the FC measured in a laboratory (and also the PI for susceptibility determination). For the CPT, no sample is recovered, and therefore Boulanger and Idriss (2014) proposed an alternative approach in which FC is estimated using an empirical correlation between FC and the soil behaviour type index I_c given in Boulanger and Idriss (2014).

The correlation between FC and I_c is weak, and for high risk/high consequence projects, CPT testing should be complemented by drilling and sampling of potentially problematic soils to verify the I_c correlations with FC or determine site specific correlations for each soil layer (and incorporate appropriate FC corrections in the analysis). Where sampling and measurement of FC is not carried out, the sensitivity of the analysis to the FC – I_c correlation should be investigated by varying the correlation-fitting factor according to the recommendation of Boulanger and Idriss (2014).

> **Comment**
> Boulanger and Idriss (2014) recommend varying their correlation-fitting factor C_{FC} over the range -0.29 to +0.29, about equivalent to +/- 1 standard deviation, to test the sensitivity of the analysis to variations in the FC correlation to I_c. Efforts to obtain site specific correlations for C_{FC} should be carried out with care given the difficulties associated with obtaining representative soil samples from the exact locations of specific CPT readings and the generally weak correlation between FC and I_c. Site-specific FC – I_c correlations based on insufficient data may be less reliable than the published generic correlation, which is based on a significant number of data points from various sites and soils. Further discussion of the FC – I_c correlation is given in Appendix A.

Probabilistic assessment of liquefaction

In addition to the conventional deterministic approach, a probabilistic version of the CPT-based liquefaction triggering procedure is presented in Boulanger and Idriss (2014). The conventional deterministic simplified procedure recommended in this Guideline uses a semi-empirical curve for CRR corresponding to a 16 percent probability (ie -1 standard deviation) for liquefaction triggering at F_L=1, considering uncertainty in the liquefaction triggering model alone. A full probabilistic liquefaction hazard analysis will need to consider the uncertainties in the seismic hazard, the site characterization, and the liquefaction triggering model. Note that the uncertainty in the liquefaction triggering model is much smaller than the uncertainty in the seismic hazard, and may often be smaller than the uncertainty in the site characterisation (Boulanger and Idriss, 2014).

For site assessments being carried out for purposes of compliance with the Building Code, it is recommended that the conventional probability of 16 percent be maintained in the deterministic liquefaction triggering analysis.

Liquefaction evaluation of gravelly soils

Procedures for gravelly soils based on large diameter penetration tests (BPT) are discussed in Youd et al. (2001). Effects of grain size distribution on penetration resistance are discussed in Tokimatsu, (1988), Kokusho and Yoshida (1997), and Cubrinovski and Ishihara (1999).

A well-documented case history of gravelly soils from the 2016 Kaikoura earthquake (Cubrinovski et al., 2017) highlights the important role of sands and silts on the liquefaction behaviour of well-graded soils predominantly containing gravels (see Section 8).

Comment

Uniform gravels in which gravel-size particles form the soil matrix have lower liquefaction potential than sandy soils because of their high hydraulic conductivity and greater stiffness and strength. For these reasons, such gravels show greater resistance to cyclic loading including lower rate of excess pore pressure build-up, and smaller cyclic strains. Even when liquefied, such gravels would have a limited strain potential and would not manifest liquefaction instability typical for sandy soils. However, confined gravels and gravelly soils containing significant amount of sands and fines (well-graded gravels) should be considered of similar liquefaction susceptibility as sandy soils.

Penetration tests in gravels should be carefully interpreted to account for the grain-size effects on the penetration resistance. Further discussion on the evaluation of gravel-sand-silt mixtures is given in Section 8.

Shear wave velocity-based evaluation offers an alternative practical approach for assessment of relatively clean gravels.

Note, however, there are indications from a well-documented case history (CentrePort, Wellington; Cubrinovski et al., 2017) that V_s-based liquefaction triggering criteria could be unconservative and may result in an overestimation of the liquefaction resistance of gravel-sand-silt mixtures.

5.4 Liquefaction-induced ground deformation

The significant reduction in stiffness and strength of soils from build-up of excess pore water pressure results in development of large shear strains in the ground during intense ground shaking.

The peak cyclic (transient) shear strains typically range from two percent in dense sands to four percent in loose sands, resulting in large cyclic lateral displacements of the liquefied layer. In the 1995 Kobe earthquake the peak-to-peak lateral displacements within liquefied thick fills reached nearly 1.0 m (Ishihara and Cubrinovski, 2005). These large cyclic lateral movements are important to consider because they may generate large kinematic loads on buried structures and deep foundations. Such lateral loads and effects are especially pronounced in the case of lateral spreading which involves large ground movements of liquefied soils during and shortly after earthquake shaking.

Post-liquefaction behaviour is characterised by a complex process involving dissipation of excess pore water pressure, sedimentation of soil particles, re-solidification and re-consolidation of the liquefied soil eventually resulting in settlement of the ground. Loss of soil volume due to a discharge of liquefied soils (soil ejecta) on the ground surface can also contribute to global and differential settlements. These liquefaction-induced settlements occur during and after the earthquake shaking, and can be significant even for free-field level-ground sites, ie without the presence of an overlying structure.

> Liquefaction-induced settlement should not be misinterpreted as densification of the ground or an indication of an increase in the liquefaction resistance of the liquefied soils. On the contrary, liquefaction usually results in non-homogeneity, weaknesses in the ground (vent holes, cracks and fissures), and a 'weak' post-liquefaction soil fabric with low liquefaction resistance. During the 2010-2011 Canterbury earthquake sequence, many sites in Christchurch repeatedly liquefied in subsequent earthquakes, often exhibiting more severe liquefaction effects in the subsequent events.

Depending on the liquefaction resistance of critical layers, overall deposit characteristics and seismic demand level, the severity of liquefaction manifestation and consequent ground damage may vary over a wide range. The magnitude of liquefaction-induced ground displacements is generally related to the liquefaction triggering factor F_L, calculated as described in section 5.3, and to the overall thickness of the liquefied layer (Ishihara, 1985; Ishihara and Yoshimine, 1992, Iwasaki et al., 1978; van Ballegooy et al., 2014).

Table 5.1 summarises general performance levels for liquefied soil deposits, and their typical liquefaction characteristics. The table provides only a general guidance and is intended to facilitate communication between geotechnical and structural engineers with regard to liquefaction severity and performance levels of liquefied deposits. F_L, LPI and LSN values are only indicative, and particular attention should be given to comments in the footnote, which are important, but not exhaustive as they do not cover all situations.

There are considerable uncertainties regarding the stiffness and strength of liquefying soils, and consequent ground deformation. The magnitude and spatial distribution of lateral spreading displacements are particularly difficult to predict. These uncertainties should be considered in the design.

It is prudent to assume that effects of liquefaction, including consequent ground deformation, can be highly non-uniform (horizontally and vertically) across short distances, and that differential movements, zones of weakness, and irregularity of ground distortion often occur.

Table 5.1: General performance levels for liquefied deposits

	EFFECTS FROM EXCESS PORE WATER PRESSURE AND LIQUEFACTION	CHARACTERISTICS OF LIQUEFACTION AND ITS CONSEQUENCES	CHARACTERISTIC F_L, LPI, LSN
L0	Insignificant	No significant excess pore water pressures (no liquefaction).	$F_L > 1.4$ LPI=0 LSN <10
L1	Mild	Limited excess pore water pressures; negligible deformation of the ground and small settlements.	$F_L > 1.2$ LPI = 0 LSN = 5 – 15
L2	Moderate	Liquefaction occurs in layers of limited thickness (small proportion of the deposit, say 10 percent or less) and lateral extent; ground deformation results in relatively small differential settlements.	$F_L \approx 1.0$ LPI < 5 LSN 10 – 25
L3	High	Liquefaction occurs in significant portion of the deposit (say 30 percent to 50 percent) resulting in transient lateral displacements, moderate-to-large differential movements, and settlement of the ground in the order of 100 mm to 200 mm.	$F_L < 1.0$ LPI = 5 – 15 LSN = 15 – 35
L4	Severe	Complete liquefaction develops in most of the deposit resulting in large lateral displacements of the ground, excessive differential settlements and total settlement of over 200 mm.	$F_L << 1.0$ LPI > 15 LSN > 30
L5	Very severe	Liquefaction resulting in lateral spreading (flow), large permanent lateral ground displacements and/or significant ground distortion (lateral strains/stretch, vertical offsets and angular distortion).	

Notes

1 Liquefaction of relatively thin layers of near-surface soils could be very damaging, and may produce effects equivalent to Performance Levels L3 and L4. Note: such effects of thin layers are not necessarily reflected as an adequate increase in the value of the damage indices (LPI and LSN).

2 A relatively thin liquefied layer with low residual strength could be responsible for lateral spreading and consequent very severe effects (Performance Level L5).

3 LPI (Iwasaki et al., 1978) and LSN (van Ballegooy et al., 2014) are damage indices that quantify liquefaction-induced damage by combining the effects of the severity of liquefaction (value of F_L or FS), thickness of liquefied soils and their location within the soil profile. The threshold values for these indices shown in relation to the performance levels are only indicative values. These thresholds may vary and do not cover all liquefaction cases (scenarios and ground conditions). These indices are typically applied for area-based screening, and in such applications have reasonable predictive capacity, but may mispredict damage/performance for about 20 percent to 30 percent of the cases. Maurer et al. (2014) and van Ballegooy et al. (2014) provide important insights on liquefaction-induced land damage and its interpretation through land damage indices LPI and LSN.

4 All being equal (ie F_L, thickness and location of liquefied layer), liquefaction consequences and magnitude of liquefaction-induced ground deformation strongly depend on the density of the soil. LSN quantifies this effect in a simplified manner. Severity of liquefaction effects decreases with increasing density of the soils, and importantly the mechanism of ground deformation also changes as the density of the soil increases (eg flow liquefaction, zero-effective stress liquefaction, and nearly zero-effective stress transient liquefaction with cyclic mobility are characteristic types of behaviour associated with very loose, loose to medium dense, and dense sands respectively).

5 The LPI and LSN should be considered in the context of particular ground conditions and structure of interest. The ranges provided in the table are based on triggering calculations using Boulanger and Idriss (2014) method, and analyses and interpretation of liquefaction effects in the 2010–2011 Canterbury earthquakes. In the case of overlapping LSN values across different performance levels, the least favourable performance level should be conservatively adopted, unless a specific justification is provided for using more favourable performance level (ie lower ground damage).

6 LPI definition (Iwasaki et al., 1978): $LPI = \int_0^{20} F_1 W(z)\, dz$ where $W(z) = 10 - 0.5z$, z is the depth below the ground surface in meters, $F_1 = 1 - F_L$ for $F_L < 1.0$, $F_1 = 0$ for $F_L \geq 1.0$, where F_L is the liquefaction triggering factor (Section 5.3).

7 LSN definition (van Ballegooy et al., 2014): $LSN = 1000 \int \frac{\varepsilon_V}{z}\, dz$ where ε_V is the calculated post-liquefaction volumetric reconsolidation strain entered as a decimal and z is the depth below the ground surface in meters.

Effects of non-liquefiable crust

For level ground sites, the severity of ground damage caused by liquefaction is affected by the properties and thickness of the liquefied layer, and by the location (depth) of the liquefied layer within the soil profile. Ground displacements and liquefaction-induced damage generally increase with the thickness of the liquefied layer, and with the proximity of the liquefied layer to the ground surface and structure foundations. Shallow liquefaction (associated with large volumes of sand ejecta and ground distortion) was particularly damaging to shallow foundations, roads and buried pipelines (water and wastewater) in the 2010 to 2011 Christchurch earthquakes. Surface manifestations of liquefaction (ground rupture and sand boils) are also influenced by the presence, thickness and characteristics (strength, continuity and integrity) of an overlying non-liquefied crust at the ground surface.

The presence of non-liquefiable crust at the ground surface may reduce the manifestation and damaging effects of liquefaction, as observed by Ishihara (1985) and more recently in the Canterbury earthquake sequence (Tonkin and Taylor, 2014; Cubrinovski et al., 2019). Such beneficial effects of the crust should only be expected in cases where lateral spreading does not occur, and where the crust is sufficiently thick and robust to ensure reduced differential movements for relatively light structures on shallow foundations.

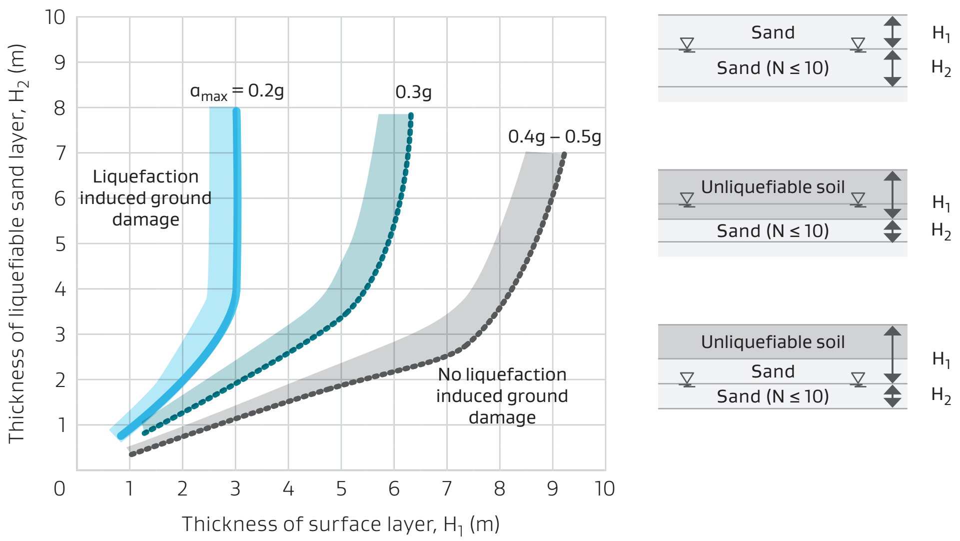
The effects of the crust layer should be considered in conjunction with the response of the whole deposit, and in particular the liquefying layer beneath the crust (see further discussion in Section 5.6). Effects of soil-structure interaction also need to be considered, including loads from the crust on the structure. In this context, effects of the crust layer are not always beneficial, and there are numerous cases in which large lateral loads are applied from the crust on foundations, buried structures and piles during ground shaking, and especially lateral spreading.

> There is evidence from the Canterbury earthquake sequence that an adequate non-liquefied crust at the ground surface, and/or highly inter-layered silty soils in the top 5 to 6 m (consisting of layers of sandy soils and silty soils of higher I_c values), reduced or suppressed the effects of liquefaction and its manifestation on the ground surface (Cubrinovski et al., 2019).

Comments on Ishihara criteria for damaging liquefaction based on crust thickness

Ishihara (1985) developed criteria identifying conditions for occurrence of liquefaction-induced damage based on the thickness of the liquefied sand layer (H_2) and thickness of an overlying crust of non-liquefied soils at the ground surface (H_1). These criteria are expressed in a $H_2 - H_1$ chart in which boundary curves for identification of liquefaction-induced damage are shown for three levels of accelerations (0.2g, 0.3g, and 0.4–0.5g). Because the chart is often used in liquefaction evaluation in practice, it is important to emphasise its key features and limitations.

Figure 5.3: Ishihara's chart for evaluation of effects of crust thickness on liquefaction-induced ground damage (Ishihara, 1985)



The chart was developed based on observations from only two earthquakes:

- 1983 Nihonkai-Chubu earthquake (Japan; M=7.7), and
- 1976 Tangshan earthquake (China; M=7.8).

In the absence of ground motion records, Ishihara estimated accelerations of 0.2 g for the Nihonkai-Chubu earthquake and 0.4 – 0.5 g for the Tangshan earthqauke respectively. Hence, the two lines in the original chart of Ishihara (1985) for 0.2 g and 0.4–0.5g relate directly to the Nihonkai-Chubu and Tangshan earthquake case histories, respectively. The dashed line for 0.3 g in the original chart was obtained by an interpolation and without direct evidence. In summary, on the seismic demand (earthquake loading) side, the chart summarises interpretation for M = 7.7 – 7.8 earthquakes, and includes a rough approximation of associated values of a_{max}.

Another important feature that needs to be acknowledged is that, in the development of the chart, Ishihara considered a relatively simple soil profile of a uniform sand deposit with a non-liquefiable layer or 'crust' at the ground surface. Experiences from the Christchurch earthquakes have shown that other factors such as presence of fines or silty soils, of low plasticity, and highly stratified soils of different liquefaction potential, including non-liquefiable soils, may substantially affect the liquefaction manifestation and associated land damage (Cubrinovski et al., 2019). In this context, the Ishihara chart should not be seen as a generalised criterion that is applicable over a wide range of subsurface conditions.

In summary, the Ishihara criteria were developed based on limited data, and involved multiple simplifying assumptions to arrive at conceptual criteria for general guidance. They were not intended to set or be used as a standard. This intent should be reflected in engineering evaluations referring to this chart.

If at any depth of the investigated deposit the liquefaction triggering factor is $F_L \leq 1.1$, then liquefaction-induced ground deformation and effects of liquefaction on structures should be evaluated.

The magnitude and extent of ground deformation depend on various factors, including initial density of the soil, thickness and location of the liquefied layer within the soil profile, intensity of ground shaking, presence of driving stresses under gravity loads, and drainage conditions. If triggering of liquefaction is predicted ($F_L \leq 1.0$), then both lateral displacement and settlement of the ground need to be estimated. Procedures for estimating liquefaction-induced ground deformation are discussed below.

5.4.1 LIQUEFACTION-INDUCED SETTLEMENTS

Several simplified methods are available for calculation of liquefaction-induced settlements of free-field level-ground sites (eg Tokimatsu and Seed, 1987; Ishihara and Yoshimine, 1992; and Zhang et al., 2002). These methods are compatible with the simplified procedure for assessment of liquefaction triggering described in these Guidelines. The calculation of liquefaction-induced settlements is based on estimation of cumulative vertical strains due to reconsolidation of liquefied soils, which typically range from one percent for dense sands to five percent for loose sands. Hence, thick deposits of loose sandy soils have especially high potential for large settlements. In the simplified method, settlement calculations are performed after the triggering calculations, and use the computed liquefaction triggering factor F_L, penetration resistance of soils and empirical relationships (Ishihara and Yoshimine, 1992; Zhang et al., 2002) to estimate liquefaction-induced settlements.

> The commonly employed Zhang et al. (2002) procedure for calculation of liquefaction-induced settlement is based on the empirical relationships established by Ishihara and Yoshimine (1992).
>
> Note: the relationships were developed from test data on one sand (Fujii sand; Nagase and Ishihara, 1988). Application of these laboratory data to field deposits requires conversion of the relative density to an equivalent penetration resistance of soils (and vice-versa), which is burdened by substantial uncertainties for clean sands, and especially for silty sands, silts and gravelly soils. These uncertainties should be acknowledged and appropriately treated in the engineering evaluation. In this regard, calculating and reporting settlements to the nearest millimetre is inappropriate as it implies a misleading level of accuracy in the calculation. It is much better to report a rounded number and the estimated range of values.

Additional settlements may be caused by shear stresses induced by overlying structures and also by displacement of foundation soils, including loss of soil volume due to sediment ejecta (Bray and Macedo, 2017). When pronounced, such mechanisms produce excessive differential settlements. Areas affected by lateral spreading also commonly exhibit non-uniform settlements due to slumping of soils associated with large lateral movements of the soil towards the waterway. None of these settlement-producing mechanisms (ie settlements due to building effects, ejected soils and lateral spreading) is accounted for in the above simplified methods for evaluation of liquefaction-induced settlement.

In view of the limitations of the existing simplified methods (which only allow for post-liquefaction reconsolidation settlements in level ground free-field deposits), and particularly the pronounced non-uniformity of liquefaction effects and consequent ground deformation, it is rational to consider the calculated settlements based on the simplified methods only as a proxy for the damaging effects of liquefaction rather than as a reliable estimate of ground settlement. Moreover, the difference between the post-liquefaction reconsolidation settlements calculated at one site investigation point and at another site investigation point should **not** be interpreted as representing the likely differential settlement between the two points. Prediction of differential settlements in liquefied soils is particularly difficult and therefore such settlements are often assumed to be proportional to the total settlement (Martin et al., 1999). Evidence from the Canterbury earthquakes (Cubrinovski et al., 2019) shows that the magnitude of differential settlements depends on the depth of liquefaction. Differential settlements are especially pronounced in the case of shallow liquefaction occurring close to the ground surface (refer to Section 5.7 for further discussion).

5.4.2 CYCLIC (TRANSIENT) GROUND DISPLACEMENTS

The empirical procedure proposed by Tokimatsu and Asaka (1998) can be used for a preliminary assessment of cyclic ground displacements in liquefied soils at level-ground sites. The procedure is based on an empirical chart correlating the maximum cyclic shear strain in liquefied soil with the penetration resistance (SPT blow count) and CSR.

Note: this chart is also available in Idriss and Boulanger (2008). Cyclic ground displacements can be calculated by a bottom-up integration of the estimated cyclic shear strains throughout the depth of the deposit (in the same fashion as volumetric strains are integrated throughout the depth of the deposit to estimate liquefaction-induced ground surface settlement in Section 5.4.1). The computed displacement profiles provide estimates for the maximum horizontal ground displacements that develop in the free field during strong shaking, which can be used in the evaluation of the cyclic phase of the response of deep foundations and buried structures based on simplified procedures (eg pseudo-static analysis, see Module 4).

Alternatively, maximum shear strains and consequent horizontal ground displacements throughout the depth of the profile can be estimated using the expressions of Yoshimine et al. (2006), as summarized by Idriss and Boulanger (2008). These expressions allow for calculation of shear strains and horizontal displacements based on either SPT or CPT penetration resistance, and provide strain values similar to the estimates from Tokimatsu and Asaka (1998). In either case, it is important to consider the potential impact of the uncertainties in the estimates of cyclic ground displacements through a parametric variation of the penetration resistance and F_L (in Idriss and Boulanger, 2008) or CSR (in Tokimatsu and Asaka, 1998) within a reasonable range of their values.

5.4.3 LATERAL SPREADING

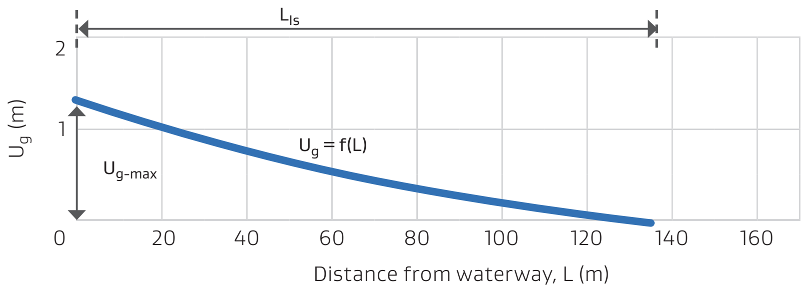
Lateral spreading of liquefied soils results in large permanent ground displacements (both horizontal and vertical), including cracks, fissures, vertical offsets, and overall settlement and slumping of the ground. Large and often highly non-uniform ground settlement occurs due to a characteristic spreading mechanism (mode of deformation) which is in addition to the settlement mechanisms for a level-ground free field site discussed in Section 5.4.1. Lateral spreading displacements occur in sloping ground and are especially prevalent near to free faces such as waterways.

Figure 5.4 illustrates some of the key features and effects of lateral spreading. A lateral spread along the Avon River is shown affecting a building located near the top of the river bank. Permanent ground displacements are largest near the top of the bank (the free face) and reduce with the distance from the river, as shown with the red vectors (arrows) in the figure.

The large differential horizontal displacements indicate an extensional deformation of the ground in the direction of spreading, which causes large cracks to open up in the ground. The cracks generally run perpendicular to the spreading direction (ie parallel to the river). Clearly, structures located within the zone affected by lateral spreading are subjected to significant differential movements and large kinematic loads due to ground movement. For example, the green arrows in Figure 5.4 indicate 'stretch' of the foundation of the building/building footprint in the spreading direction, whereas the ground displacements at the river bank (highlighted in yellow) indicate large displacemnets that will generate correspondingly large kinematic loads on in-ground structures (eg a bridge abutment on pile foundations). Detailed evidence and interpretation of lateral spreads caused by the Canterbury earthquakes is given in Cubrinovski and Robinson (2016), whereas characteristic mechanisms for spreading-induced damage to the Avon River bridges can be found in Cubrinovski et al. (2014a; 2014b).

Figure 5.4: Characteristic features and effects of lateral spreading illustrated on a Christchurch case history (modified from Cubrinovski (2019), Ralph B. Peck Lecture)

Figure 5.5: Principal objectives in the engineering evaluation of lateral spreading: maximum ground displacement, zone affected by spreading and spatial distribution of ground displacements (modified from Cubrinovski and Robinson, 2016)



In the 2010-2011 Canterbury earthquakes, lateral spreading along the Avon River resulted in maximum horizontal ground displacements at the river banks in the order of 1 m to 3 m. Such large permanent displacements occurred in a gently sloping ground with a gradient of 0.5 percent to 2 percent and free face (channel) height of 2 m to 4 m. The zone affected by spreading typically extended inland from the river banks up to approximately 150 m to 200 m from the river. The spatial distribution of lateral ground displacements and density of cracks was variable and largely affected by a complex interplay of the river geometry, topographic features of the site, spatial distribution of geologic units, characteristics and location of liquefiable soils in the profile, and overall deposit characteristics (Cubrinovski and Robinson, 2016).

As illustrated in Figure 5.5, in engineering evaluation of lateral spreading, one needs to estimate the:

1. maximum ground displacements due to spreading ($U_{g\text{-}max}$) at or near the free face
2. zone affected by spreading (or the distance inland from the waterway (L_{lS}) within which spreading displacements are of engineering significance) and
3. distribution of ground displacements within the zone affected by spreading.

These displacement characteristics allow the estimation of ground deformation, differential movements, and kinematic loads due to spreading that are needed in the engineering assessment and design of structures.

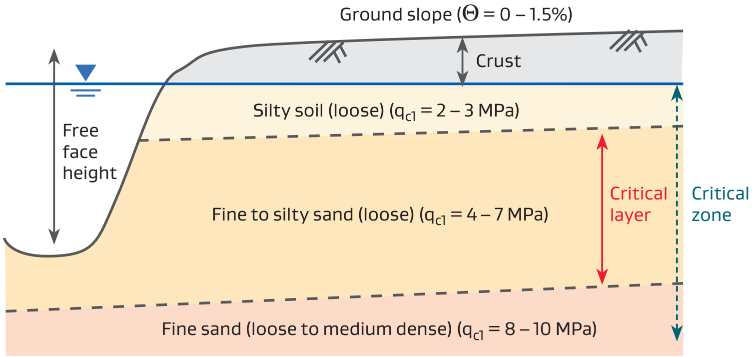
Empirical relationships for estimation of lateral spreading displacements

Several empirical methods are available for evaluation of lateral spreading displacements (Youd et al., 2002; Tokimatsu and Asaka, 1998; Zhang et al., 2004). Using field observations from case histories of lateral spreads caused by liquefaction in past earthquakes, Youd et al. (2002) developed empirical expressions for prediction of lateral ground displacements due to spreading using regression analysis. Factors such as site configuration, SPT resistance, grain-size composition (FC and D_{50}), earthquake magnitude, and site-to-source distance are accounted for in this procedure.

Note: SPT resistance and grain-size characteristics of key layers that contribute to the spreading displacements are crudely characterized in approximate terms.

Zhang et al. (2004) provide an alternative empirical approach for evaluation of lateral spreading displacements based on CPT procedures. Their method first estimates the Lateral Displacement Index (LDI), which in essence is an estimate of the horizontal displacement profile for a level-ground free field site obtained from liquefaction triggering calculation (using F_L and the maximum shear strain, γ_{max}), and then modifies LDI to account for effects of site geometry (ie ground slope or free face) on lateral spreading displacements.

Figure 5.6: Soil profile, site geometry and location of critical layers in relation to free face are important factors to consider in the evaluation of lateral spreading displacements (modified from Cubrinovski and Robinson, 2016)



Comment

The evolution of lateral spreading and resulting permanent ground displacements are affected by a complex interaction of a number of factors related to the liquefaction characteristics of critical layers, location of the liquefiable soils within the profile in relation to the free face, lateral continuity of critical layers, overall deposit characteristics, river geometry, site topography, and ground motion characteristics. Lateral spreading problems are inherently complex, burdened by significant uncertainties and challenging for engineering assessment.

Estimates of lateral spreading displacements involve significant uncertainties in relation to the extent of liquefaction, residual strength of liquefied soils and proportion between shaking-induced and gravity-induced ground displacements. The predicted magnitude and distribution of lateral spreading displacements is highly sensitive to the assumptions adopted in the method and input parameters used in the assessment. Therefore, a systematic approach is recommended in the assessment in which parametric studies are performed considering a range of values for key parameters, as discussed in Section 10.3. In this context, use of multiple approaches in the evaluation of spreading displacements is also useful. Cubrinovski and Robinson (2016) and Little et al. (2021) provide some insights for a systematic evaluation of lateral spreading based on comprehensive studies of lateral spreads observed in the 2010–2011 Christchurch earthquakes.

The accuracy of these simplified empirical methods for estimating lateral spreading displacements is relatively low, and predicted ground displacements are generally within a factor of two (ie 0.5 to 2) of the observed lateral spreading displacements though even larger deviations have often been observed. Generally low levels of accuracy were also observed for the well-documented lateral spreads along the Avon River in Christchurch (eg Little et al., 2021). The low level of accuracy in the predictions, lack of theoretical basis for the empirical methods, and the complexity of lateral spreads in general, emphasises the need to carefully consider and account for uncertainties in the estimates of lateral spreading displacements in engineering evaluations.

Figure 5.6 schematically illustrates a typical soil profile and site geometry (ground slope and free-face height) along the Avon River where large lateral spreads were observed after the 2010-2011 earthquakes. The profile shows a relatively thick critical layer (zone)[1] of low liquefaction resistance. Importantly, the critical layer is located at the level of the free face and bottom of the river channel, which is favorable for the development of large lateral spreading displacement. Once liquefied, these soils are laterally unconstrained and can easily move towards the river.

1 Critical layer in the deposit is a layer of low (often the lowest) liquefaction resistance, which is located below the water table, and is at relatively shallow depth. The critical layer is most likely to be the principal contributor to the liquefaction-induced damage and lateral spreading displacements. Critical zone is of similarly low liquefaction resistance as the critical layer and is located adjacent to or near the critical layer; the critical layer and critical zone are anticipated to interact strongly and develop similar response under earthquake loading (further details are provided in Cubrinovski and Robinson (2016) and Cubrinovski et al., 2019).

The location of the critical layer (zone) in relation to the free face is an important factor to consider in addition to the liquefaction resistance and thickness of the layer. Here, the critical layer (zone) refers to the part of the soil profile that contributes most to the severity of liquefaction and lateral spreading.

The depth over which the lateral spreading analysis is performed is also an important consideration. For free face geometry sites, Chu et al. (2006) recommended that depths to twice the height of the free face (2H) be considered. This is a reasonable and pragmatic approach in the assessment as it focusses attention on the part of the soil profile and site geometry that most affects the lateral spreading displacements.

Vertical continuity of liquefiable soils (discussed in greater detail in Section 5.6) and lateral continuity of the critical layer (zone) are important factors that influence both the magnitude of spreading displacements ($U_{g\text{-}max}$) and the width of the zone affected by spreading (L_{IS}). Thus, it is critical to investigate and characterise the spatial distribution of critical soil layers and lateral extent of geologic units through appropriate geologic interpretation and geotechnical assessment.

Estimation of lateral spreading displacements using the Newmark (rigid-block) method

Earthquake-induced permanent lateral displacements can be estimated using Newmark's procedure for displacement of a rigid body subjected to base accelerations (Newmark, 1965). In this method, yield acceleration is calculated using the limit equilibrium approach, and movement of the slope (earth structure) is then calculated by using acceleration time history records and integrating episodes of movement when the ground acceleration exceeds the yield level. More conveniently, comprehensive suites of earthquake time histories have been analysed using Newmark's method and the results presented as statistical expressions of displacement for different acceleration ratios (eg Bray and Travasarou, 2007; Jibson, 2007). In the calculations, it is important to consider the uncertainties in the estimates of the yield acceleration and residual strength of liquefying or liquefied soils. Additional information is given by Bray and Travasarou (2007), Jibson (2007), and Olson and Johnson (2008).

It is important to acknowledge that the underlying assumption for a rigid-body behaviour of the Newmark method, which assumes a well-defined sliding surface between the moving block and stationary ground, is largely incompatible with the viscous-type of distributed deformation of liquefied soils. This limitation should be considered when evaluating the applicability of the Newmark method to a particular problem. The method may still be useful for cases of abutments/ approaches/embankments overlying well-defined liquefiable layers, and provides a rational approach for quantifying the restraining or 'pinning' effects of the piles and bridge superstructure and consequent reduction of lateral spreading displacements at bridge abutments (PEER, 2011).

5.5 Residual strength of liquefied soils

Residual strength of liquefied soils can be used in the assessment of post-liquefaction stability of sloping ground, risk of bearing failures, and liquefaction-induced lateral displacements.

Experience from previous earthquakes and experimental studies on scaled-down models indicate that residual strength of liquefied soils can be very low. A nearly complete loss of effective stress, which is sustained during the pore water pressure redistribution and groundwater flow, and potential loosening of liquefied soils (void ratio redistribution and expansion) due to upward water flow, are considered the primary reasons for the low residual strength of liquefied soils. In cases where a low permeability layer above the liquefied layer acts as a barrier and prevents upward flow, significant loosening of the liquefied layer may occur at this interface (eg water film effects and void redistribution, Kokusho, (2003)).

There are several empirical relationships currently available for estimating the residual strength of liquefied soils:

› The empirical correlation of Seed and Harder (1990) presents the residual strength S_r as a function of the equivalent clean sand SPT blow count, $(N_1)_{60cs\text{-}Sr}$.
  (Note: the fines content correction for SPT blow count for estimating residual strength differs from the fines content correction used in the liquefaction triggering evaluation, $(N_1)_{60,cs}$).
› Olson and Stark (2002) provide empirical correlations both based on normalized SPT blow count $(N_1)_{60}$ and normalized CPT resistance q_{c1N}. In both cases, the residual strength is defined in terms of a ratio (S_r/σ'_{vo}) or normalized strength.
› Idriss and Boulanger (2008) recommend relationships in terms of S_r/σ'_{vo} based on both $(N_1)_{60cs\text{-}Sr}$ and $(q_{c1N})_{cs\text{-}Sr}$ for two separate conditions:
  – Where void ratio redistribution effects are expected to be negligible (this case should not be used unless it can be shown that void redistribution is not possible).
  – Where void ratio redistribution effects could be significant.

Note

These relationships have been extrapolated beyond the range of available data (eg the relationships are shown with dashed lines for $q_{c1Ncs} > 90$ in Idriss and Boulanger, (2008)) indicating uncertainties and lack of evidence over the extrapolated range. The absence of case histories where $q_{c1Ncs} > 90$ or $(N_1)_{60cs\text{-}Sr} > 15$ (ie denser soils) supports the concept that it is loose soil deposits which are the primary candidates for liquefaction induced instability, but this does not eliminate the possibility that denser soils within the extrapolated range are immune from liquefaction-induced flow failures. Generally, dense soils may liquefy, but they are more likely to undergo cyclic mobility with limited strain potential rather than flow-type of deformation.

It is important to note that the above-mentioned empirical relationships are based on similar data sets and they differ essentially in the interpretation of the case history observations. At a specialised session on residual strength (GEESDIV Conference, Sacramento, May 2008) there was a general consensus that, for the time being, both normalised and non-normalised relationships should be used in parallel. It has been suggested that the normalised form of the residual strength (ie S_r/σ'_{vo}) better reflects the potential strength loss due to void redistribution (Idriss and Boulanger, 2008) and the effects of depth of liquefaction (location of the liquefied layer within the profile) or effective overburden stress. In view of the uncertainties involved in the S_r estimates, it seems prudent to evaluate the sensitivity of the results to assumed range of S_r values, and account for the outcome of such sensitivity study in the interpretation of results and decision-making process.

5.6 System response of liquefiable deposits

Soil liquefaction during earthquakes is a highly dynamic process in which excess pore water pressures rapidly develop and change during the strong ground shaking.

The dynamic response of liquefying deposits involves direct and indirect interactions between various layers that may profoundly affect the evolution of liquefaction throughout the deposit, and may alter the severity of liquefaction manifestation at the ground surface. Such 'system-response effects' (Cubrinovski et al., 2019) are important to consider and evaluate in the liquefaction assessment.

In the 2010-2011 Canterbury earthquakes, vastly different liquefaction manifestations at the ground surface[2] varying from severe liquefaction to no liquefaction manifestation were observed for soil deposits comprising layers with low liquefaction resistance at shallow depths. Sites where vertically continuous liquefiable sandy soils were present in the top 10 m of the deposit, which included a critical layer[3] of low liquefaction resistance at shallow depth, typically showed severe liquefaction manifestation. Conversely, sites with interbedded deposits comprised of layers of both liquefiable and non-liquefiable soils exhibited much better performance and either manifested no liquefaction or substantially reduced effects of liquefaction.

> The overall deposit characteristics, especially stratification and vertical continuity (or lack of it) of liquefiable soils may profoundly affect the liquefaction process including triggering, evolution, and surface manifestation of liquefaction.

Cubrinovski et al. (2019) have identified a number of interaction mechanisms that may either intensify or mitigate liquefaction manifestation at the ground surface. The mechanisms that intensify liquefaction manifestation include:

- Rapid liquefaction of shallow critical layers with low liquefaction resistance during the strong ground shaking
- Additional disturbance and fluidization of the liquefied critical layer due to inflow of water from underlying layers of low-to-medium liquefaction resistance
- Seepage-induced liquefaction of near-surface soils at and above the water table due to inflow of water from underlying heavily liquefied soils
- Strong and unconstrained water flow through liquefiable soils of relatively large thickness that essentially connects the abovementioned three mechanisms and results in a strong and damaging discharge of excess pore water pressures in which liquefiable soils from the entire deposit contribute to and intensify the severity of liquefaction manifestation.

The above mechanisms and consequent response of the soil deposit develop very quickly, typically over several tens of seconds during the strong shaking. These strong dynamic interactions amplify the effects of each mechanism, resulting in a severe liquefaction manifestation.

The mechanisms that mitigate liquefaction manifestation include:

- Liquefaction triggering in deeper layers, with liquefaction resistance similar to the shallower critical layer, producing 'base isolation' effects that substantially reduce the seismic demand (shear stresses) in the shallow part of the deposit. In essence, deep base isolation effects reduce the level of shaking in the shallow part of the deposit.
- 'Grid' effects of interbedded deposits comprising liquefiable and non-liquefiable soils, in which non-liquefiable layers of considerable (cumulative) thickness reduce deformations and pore pressure build-up in the deposit due to:
  1. lateral stiffening effects from horizontally continuous non-liquefiable layers, and
  2. non-liquefiable layers of low permeability soils restricting vertical flow of pore water and thus preventing deeper liquefiable layers from 'boosting' the response of the shallow layers.
- Partial saturation in shallow parts of the deposit beneath the water table where there are interbedded liquefiable and non-liquefiable layers of considerable (cumulative) thickness.

2 Surface manifestation of liquefaction typically involves soil and water ejecta at the ground surface, ground cracks and fissures, ground surface distortion, and differential settlements. The severity of these manifestation features is used to quantify the severity of liquefaction manifestation at the ground surface. Note: absence of liquefaction manifestation at the ground surface does not eliminate the possibility of liquefaction developing at larger depths in the deposit. Uniform global settlement could be caused by deep liquefaction, but such settlement also results due to densification of non-liquefied soils during ground shaking.

3 Critical layer in the deposit, from a liquefaction manifestation perspective, is a layer of low (often the lowest) liquefaction resistance, which is located below the water table, but close to the water surface. The critical layer is the most likely to manifest liquefaction at the ground surface, for a given deposit.

The combined effects of the three mitigating mechanisms above may result in a relatively thick non-liquefied 'crust' that will either substantially reduce or eliminate liquefaction manifestation at the ground surface.

The above mechanisms interact in time and space, and produce strong 'system-response' effects that may either intensify or mitigate liquefaction manifestation at the ground surface. Details on system response of liquefiable soils and guidance for their treatment in the engineering evaluation are given in Cubrinovski et al. (2019), Cubrinovski (2019), and Ntritsos and Cubrinovski (2020).

Cross-layer interactions and system response effects of liquefiable deposits may profoundly affect liquefaction manifestation and liquefaction-induced damage. The activation of specific mechanisms, their interaction and resulting cumulative system response effects depend on the deposit and layer characteristics, ground motion characteristics (seismic demand) and induced soil/deposit response.

In current simplified liquefaction procedures, each layer is considered in isolation, and a factor of safety against liquefaction triggering (F_L), and consequent maximum cyclic shear (γ_{max}) and volumetric (ε_v) strains are estimated independently for each layer. Thus, when calculating F_L, γ_{max} and ε_v for any given layer, the response of other layers, cross-layer interactions within the deposit and system response effects are currently ignored (eg Cubrinovski, 2019).

Given the potential for system response effects to substantially affect liquefaction manifestation and associated damage, it is important to incorporate such effects in the engineering evaluation when they may be present. For routine projects, this could be as an additional consideration following conventional simplified analysis (using engineering judgement). For higher importance or high risk projects, the use of advanced dynamic (effective stress) analyses may be warranted (refer to Section 5.8).

5.7 Effects of liquefaction on structures

There are numerous case histories from past earthquakes demonstrating the significant effects of soil liquefaction on the seismic performance of engineering structures (buildings, bridges, storage tanks, port structures, embankments, levees/stopbanks, and lifelines).

The Canterbury earthquake sequence provided many well-documented case histories on the performance of buildings and infrastructure in a New Zealand natural and built environment. If triggering of liquefaction is predicted and the resulting ground displacements are large, then effects of liquefaction on structures should be assessed and addressed in the design.

While detailed assessment of effects of liquefaction on structures is beyond the scope of this Module, some important issues for consideration in the design of structures at liquefiable sites are briefly discussed below:

› Liquefaction-induced settlements due to re-consolidation of liquefied soils occur in level ground sites irrespective of whether there is an overlying structure. When structures are founded over or within liquefied soils, additional settlements will occur due to shearing stresses induced by the overlying structure, and because of loss of soils beneath foundations due to liquefaction-induced sediment ejecta (Cubrinovski et al., 2011; Bray et al., 2014; Bray and Dashti, 2014). These additional settlements can be of similar magnitude or even greater than the re-consolidation settlements and can be particularly large in the case of heavy structures or where there is considerable sediment ejecta. There are no widely accepted simplified procedures for prediction of structure-induced settlements, however, there are several recently proposed procedures for such evaluation (eg Bray and Macedo, 2017; Karamitros et al., 2013).

In the Bray and Macedo (2017) procedure, total liquefaction-induced building settlement is estimated as a sum of three independent components of settlement:

1 settlement caused by loss of soil due to sediment ejecta

2 volumetric-induced settlement, as computed by the Zhang et al. (2002) method for a level-ground free field site, and

3 shear-induced settlement due to liquefaction in the foundation soils below the building.

Prediction of differential settlements in liquefied soils is particularly difficult and therefore such settlements are typically assumed to be proportional to the total settlement (Martin et. al., 1999). Evidence from the Canterbury earthquakes (Cubrinovski et al., 2011; Bray et al., 2014; Cubrinovski et al., 2019) shows that the magnitude of differential settlements strongly depends on the depth of the liquefied layer. Differential settlements are especially pronounced in the case of shallow liquefaction occurring close to the foundation and ground surface. Deep liquefaction occurring outside the settlement-influence zone results in relatively small differential settlements even if the total settlement is large. Laterally variable subsurface conditions, irregularity in the superstructure (eg complex geometry with asymmetry in the mass and stiffness) and lateral spreading cause large differential settlements.

While shear-induced settlement due to rocking and ratcheting effects occur during the vibration of the superstructure, a substantial portion of liquefaction-induced differential settlements may develop *after* strong shaking, and the relative timing of occurrence of differential settlement should be accounted for when evaluating the capacity of the structure to accommodate such settlements. Given the uncertainty in the estimates of the magnitude and timing of differential settlement, it is recommended to investigate different scenarios in the temporal evolution of settlements. For example:

1 all differential settlements occurring during the strong shaking
2 50 percent of the settlement occurring during the strong shaking and 50 percent post shaking, and
3 20 percent of the settlement occurring during the strong shaking and 80 percent post shaking.

Generally, case 1 is relevant for dense soils, whereas cases 2 and 3 are more realistic scenarios for medium dense and loose soils.

› Large lateral movements from ground oscillation, and lateral spreading of liquefied soils, in particular, are damaging for pile foundations (Cubrinovski et al., 2014a,b). Large lateral loads from a non-liquefied crust layer, kinematic loads due to ground displacement and inertial loads from the building (structure) need to be considered in the assessment of pile foundations. Maximum inertial and kinematic loads may or may not occur simultaneously depending upon characteristics of the ground motion, dynamic characteristics of the site and soil-structure system, rate of pore pressure build-up and soil-pile-structure interaction (Boulanger et al., 2007; Tokimatsu et al., 2005). Various methods for analysis of piles in liquefying soils are available based on the pseudo-static approach (eg PEER, 2011; Cubrinovski et al., 2009). Care must be taken to account for uncertainties in loads and properties of liquefied soils when using these simplified methods. Cubrinovski et al. (2014a) highlight the importance of parameter selection and sensitivity studies in the assessment of a well-documented case history from Christchurch (Anzac Bridge). Detailed guidance for assessment of bridge pile-foundations subjected to lateral spreading is also provided in NZTA (2014b) and NZTA (2018). Module 4 of the Guidelines provides more information.

› Lateral spreading displacements can be very large and highly variable in waterfront areas (the magnitude of these displacements changes rapidly with the distance from the waterfront). Hence, structures founded close to quay walls and revetment lines may be subjected to differential lateral displacements that may stretch the foundation and adversely affect the structure. Interpretation and classification of lateral spreads observed in the Christchurch earthquakes and guidance for evaluation of lateral spreading are provided in Cubrinovski and Robinson (2016). Effects of lateral spreading on bridges including development of a specific mechanism for short-span bridges are summarized in Cubrinovski et al. (2014a,b).

› Liquefaction may cause bearing failures and lead to overall instability with tilting and overturning of structures on poorly designed foundations. Potential punching failures through a surface crust and reduction of the foundation bearing capacity should be considered in the design. Liquefaction in the immediate foundation soils or their vicinity would usually result in excessive transient and permanent displacements/settlements and potential damage to the foundations that could propagate to the superstructure (Cubrinovski and McCahon, 2011; Cubrinovski et al., 2011; Bray et al., 2014; Bray and Dashti, 2014).

- Significant vertical and horizontal ground displacements should be accommodated in the design of foundations and structures in liquefiable soils. If the structure and foundation cannot tolerate the imposed ground displacements, then additional measures such as strengthening of the foundation, ground improvement or structural modification should be implemented. Both the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS) need to be considered separately in the assessment of liquefaction unless the risk of liquefaction or ground damage occurring for the SLS is acceptably low ($F_L \geq 1.2$). Note however that this is only the minimum code requirement, whereas best-practice assessment would holistically consider the seismic performance both between SLS and ULS limits, and also beyond the ULS limit, as discussed in Section 10 and in Module 4.
- Liquefied soils behave as a heavy liquid causing relatively light structures such as buried pipelines, manholes, pump wells and basements to 'float' to the ground surface. Buried lifelines are also subjected to differential movements caused by spatial variability of ground conditions and ground displacements.
- Seepage action, redistribution of excess pore water pressures and rise of the phreatic surface may trigger post-earthquake failures in dams and embankments (Ishihara, 1985). Such water flow effects, and evolution of liquefied zones post-shaking has triggered delayed failures, even up to 24 hours after the main shock in tailing dams (ie Mochikoshi tailings dam case history; Ishihara, 1984).

Inertial loads due to strong shaking (vibration of the superstructure) are significant during the cyclic phase (ie during strong ground shaking and development of excess pore water pressures) but may decrease substantially after triggering of liquefaction because of the reduced capacity of liquefied soils to transfer shear stresses (and accelerations to the surface). Such reduction in acceleration amplitudes post-liquefaction may be pronounced in loose sandy soils but may be negligible or even reversed in dense soils because of acceleration spikes from cyclic mobility associated with temporary dilation during cyclic shearing. The substantial reduction in stiffness of liquefied soils in general, though, leads to lengthening of the vibration period of the deposit which in turn may cause amplification of the response of long-period structures (systems).

Unlike the diverse effects of liquefaction on accelerations of loose and dense soils, softening of the deposit due to excess pore pressures and liquefaction leads to an increase in displacement amplitudes in all cases. Numerical analyses can be used to evaluate these effects in case of important structures.

When evaluating the effects of liquefaction and lateral spreading on pile foundations using simplified analysis procedures, it is important to adopt a consistent scenario with compatible values for the magnitude of ground displacements, soil stiffness and strength properties, and inertial loads from the building. Significant lateral spreading is associated with loose soils and there will likely be a substantial decrease in inertial loads after liquefaction triggering, as well as marked reduction in strength and stiffness of the liquefied soil. Large ground displacements (indicating low stiffness and strength of liquefied soils) are incompatible with high accelerations or inertial loads which are associated with relatively high stiffness and dilation during cyclic mobility or pre-liquefaction soil stiffness.

Note however that high-frequency components of the motion can be transferred to the superstructure through a robust and stiff foundation system even with liquefaction of the surrounding soils, in which case, the high-frequency response of the structure may not be significantly reduced. More detailed guidance on the treatment of kinematic and inertial loads in simplified pseudo-static analyses of piles in liquefying soils is given in Module 4.

5.8 Advanced numerical procedures

Advanced numerical procedures may be appropriate for significant projects or may be justified where the consequences of liquefaction or where dynamic interactions and system-response effects may be significant. Advanced analyses based on the effective stress principle are particularly valuable in the evaluation of the effectiveness of ground improvement and structural strengthening measures for mitigation of liquefaction.

Advanced numerical procedures for liquefaction assessment include total stress and effective stress dynamic analyses. The latter is specifically tailored to analysis of soil deposits, stability of embankments, and soil-structure systems affected by excess pore water pressures and liquefaction, and is the primary tool for detailed assessment of liquefaction and its effects on structures. Effective stress analysis addresses triggering of liquefaction, consequent ground deformation, and effects of liquefaction on structures in an integrated manner, and therefore can provide a more realistic simulation of the complex ground response and soil-structure interaction in liquefying soils, though some limitations must be recognized as discussed below.

First, some advantages of this analysis procedure are listed below:

- The analysis allows detailed simulation of the liquefaction process including build-up of excess pore water pressure, triggering of liquefaction, subsequent losses in strength and stiffness, pore pressure redistribution through water flow and post-shaking dissipation of excess pore pressure. It captures the dynamic nature of the problem and provides realistic simulation of earthquake loads and ground response throughout the depth of the foundation soil by considering responses of individual layers and cross interaction amongst them (base-isolation effects and progressive seepage-induced liquefaction due to upward flow of water; eg Cubrinovski et al., 2019).
- Spatial and temporal variation of ground deformation develops in accordance with changes in soil stiffness and earthquake loads. Thus, in this analysis, both inertial loads due to vibration of the structure and kinematic loads due to ground movements are concurrently considered while accounting for soil nonlinearity and effects of excess pore water pressure on soil behaviour.
- Effects of cross-layer interaction within the deposit and soil-structure interaction are included in the analysis, in which sophisticated non-linear models can be used both for soils and structural members.
- The analysis allows assessment of the effectiveness of countermeasures against liquefaction (ground improvement or structural modification), including their effects on the reduction of ground deformation and seismic performance of structures. It also allows quantification of possible increases in the structural response as less energy will be dissipated in the strengthened and stiffened foundation system.

Practical disadvantages of the effective stress analysis are that it requires:

- Selection of appropriate earthquake records to be used as input motion in the analysis by considering the seismic hazard for the site.
- Two approaches are generally available in setting the key analysis parameters. In the first, more rigorous approach, high-quality site-specific data on the in situ conditions, physical properties and mechanical behaviour of soils (based on field investigations and laboratory testing of soil samples) are required, particularly if the analysis is used to rigorously quantify the seismic performance of important structures. In the second, more generic approach, a less rigorous determination of analysis parameters could be employed using existing empirical relationships and conventional geotechnical data. Recently, user-friendly definition of parameters and analysis procedures have been provided for constitutive models specifically targeting liquefaction problems. For example, Ziotopoulou and Boulanger (2013) have provided guidance on the calibration and use of the PM4Sand model in which only three parameters require input

by the user. Similarly, Ntritsos and Cubrinovski (2020) have proposed a CPT-based effective stress analysis procedure in which all parameters of the constitutive model (Stress-Density Model; Cubrinovski and Ishihara, 1998a,b) have been pre-calibrated on the liquefaction resistance specified in the simplified procedure of Boulanger and Idriss (2014) and, hence, it only requires conventional CPT data as input for the effective stress analysis.

› High demands on the user with respect to knowledge and understanding of the phenomena considered and particular features of the adopted numerical procedures. In addition, numerical analysis procedures require more substantial efforts for pre- and post-processing including visualisation of input data and analysis results.

All analysis methods and constitutive models have limited ability to model certain aspects of soils' behaviour and to simulate complex liquefaction phenomena. Particularly difficult to address are large strain/displacement problems, discontinuities, loss of soil volume due to ejecta, 3-D effects, and similar complex issues. Use of advanced dynamic analyses requires rigorous application of the procedure and a clear understanding of the limitations of the method (Cubrinovski, 2011).

The total stress analysis is an alternative procedure for assessment of the seismic response of ground and soil-foundation-structure systems. This analysis, however, does not directly include effects of excess pore water pressures, and hence requires additional interpretation of non-linear soil behaviour and its simplification for modelling. Total stress analysis is generally not recommended for liquefaction problems, though it could be useful in the engineering evaluation of some problems (eg Newmark type analysis). If total stress analysis is used, due attention should be given to the selection of soil parameters and to the sensitivity of the seismic response on the variation and uncertainty of key input parameters in the analysis.

6 Mitigation of liquefaction and lateral spreading

Liquefaction-induced ground displacements may be large and often intolerable for the built environment.

Ground deformation hazard arising from earthquake shaking (including liquefaction and lateral spreading) should be considered:

› Where failure or excessive deformation of the ground might contribute to loss of life or loss of amenity of a building of Importance Level 2 or higher (refer NZS 1170.0 for definition of importance level)
› Where failure or excessive deformation of the ground is a risk to services to or access to buildings of Importance Level 3 or higher.

Two approaches are generally used to mitigate liquefaction and its consequences:

› soil remediation
› structural modification.

6.1 Soil remediation

Soil remediation methods reduce ground deformation and effects of liquefaction either by preventing, limiting, or slowing-down the development of excess pore water pressure or by limiting the development of shear strains and vertical strains in the ground.

Soil remediation is commonly based on one or a combination of the following:

› Densification (compaction, vibro-flotation, compaction piles, preloading) to increase liquefaction resistance (CRR) and reduce deformability of the soil through increased strength and stiffness.
› Solidification (deep mixing, permeation grouting) through cementation of soils.
› Containment of liquefied soils and limitation of ground deformation by reinforcement (eg slurry walls or soil mixed walls)
› Drainage (prefabricated drains, stone columns) for increased permeability and faster dissipation of excess pore water pressures.

Details on mitigation measures, implementation, and assessment of their effectiveness may be found in JGS (1998), Seismic Design Guidelines for Port Structures (INA, 2001), Martin et al. (1999) and Mitchell et al. (1998). Advanced analysis procedures, and the effective stress analysis in particular, can be used for assessment of effectiveness of countermeasures against liquefaction.

Following the Canterbury earthquake sequence, comprehensive field trials were conducted by EQC to investigate the effectiveness of various liquefaction-mitigation measures specifically for residential buildings and properties (Tonkin & Taylor, 2015).

These benchmark field tests indicate that:

› the effectiveness of many techniques depends on the soil type and ground conditions (eg density, fines content and plasticity of fines, saturation, horizontal and vertical confinement)
› ground treatment procedures used in an inappropriate setting may produce highly non-uniform ground conditions and create weak zones with high potential for liquefaction
› details of ground improvement procedures and their implementation in the field are critically important, hence, calibration, validation and QA are essential aspects of ground improvement and
› ground improvement should be considered in the context of the particular structure and its characteristics, ground conditions and performance objectives; this evaluation should also consider a potential increase in the structural dynamic loads and response during shaking as a consequence of ground improvement.

More detailed guidance on ground improvement is provided in Modules 5 and 5a of the Guidelines.

6.2 Structural modification

Potential effects of liquefaction need to be taken into account and accommodated in the design of the building structure to reduce or accommodate differential settlements and lateral movement.

This is commonly achieved by using a stiff raft system or rigid foundation beams or walls. Also, deep pile foundations with sufficient lateral capacity to resist both inertial loads due to vibration of the superstructure, and kinematic loads due to ground movement can be used. Some examples of engineering design solutions specific to liquefaction are given in the MBIE guidelines for residential buildings (MBIE, 2012) and light industrial buildings (MBIE, 2014).

Alternatively, buildings can sometimes be designed to accommodate expected ground deformations (while still complying with the Building Act), often with pre-identified inspection and repair methodologies (eg Ministry of Education, 2020).

More detailed guidance on foundation design is provided in Module 4 of the Guidelines.

7 Clay soils

7.1 Ground failure of clay soils

Clay soils may significantly soften and fail under cyclic loading but do not exhibit typical liquefaction features and are therefore considered non-liquefiable. Assessment of the cyclic strength ('cyclic softening') of 'clay-like' soils is quite different to the liquefaction assessment of 'sand-like' soils.

Cyclic strength can be assessed by either:

› Cyclic laboratory testing of 'undisturbed' soil samples, or,

› Measuring the monotonic undrained shear strength using standard procedures (in situ, eg field vane, or CPT or laboratory, eg CU triaxial or Simple Shear test) and then applying an empirical correction factor.

Boulanger and Idriss (2006, 2007) proposed a procedure for evaluation of cyclic softening in 'clay-like' fine-grained soils during earthquakes. The procedure follows a format similar to that used in the simplified procedure for 'sand-like' soils and allows estimating the factor of safety against cyclic failure in 'clay-like' fine-grained soils (using a failure criterion of three percent peak shear strain). Several approaches are provided for estimating the cyclic resistance ratio (CRR) based on the undrained shear strength.

Chen et al. (2006) provide recommendations regarding correction factors to adjust the static undrained shear strength of a clay soil to represent its peak dynamic strength. Loading rate effects increase the peak dynamic undrained shear strength of clays relative to static strength whereas cyclic degradation effects reduce it. Progressive failure effects also influence the value of peak dynamic undrained shear strength that should be used in limit-equilibrium analyses. Importantly, the potential for a post-peak drop in strength should be evaluated for sensitive clay soils, and the shear strength used in the analyses must be compatible with the calculated level of deformation.

Pending further research in this area, designers should make assessments of stability and deformation using the above procedures.

7.2 Mitigation of clay soils

The possibility of damaging ground deformations in 'clay-like' soils should be evaluated, including the effects on foundation capacity and overall stability of a building.

Options for mitigating clay soils are more limited than for granular soils and may include pre-loading with or without additional drainage. The typical approach to mitigation will often be structural modification, including the use of deep foundations or stiff raft foundations, or ground improvement by soil cement walls, for example.

8 Reclaimed land and constructed fills

Uncompacted or poorly compacted fills, tailings and land reclamations comprised of susceptible soils have high liquefaction potential. They commonly involve large masses of soils and pose high-risk/ high-consequence vulnerabilities, and therefore require special attention in seismic assessment and design.

> Much of the reclaimed land in New Zealand (as well as worldwide) was constructed during the 20th century. The construction practices typically involved deposition of large volumes of soils into water, and then sedimentation of soils and formation of fill deposits through gravity action alone, with no additional compaction effort. As liquefiable soils were commonly used for the reclamations, these large-scale construction methods resulted in thick reclamations with high liquefaction potential. Reclaimed land more often than not is highly vulnerable to liquefaction and seismic instability.

In the 2016 Kaikoura earthquake, widespread liquefaction occurred in the reclamations of the port of Wellington (CentrePort) causing substantial damage to port structures and interruption of operations. Both gravelly fills constructed by end-damping of quarry soils and hydraulic fills sourced from the nearby seabed soils were affected by varying levels of liquefaction severity (Cubrinovski et al., 2017; Dhakal et al., 2020a). Severe liquefaction occurred in 10 – 20 m thick gravelly reclamations consisting of 50 –75 percent fine-to-medium gravels and 25 –50 percent finer sand-silt fractions. Despite the dominant gravel content, the reclamations exhibited behaviour typical for sandy soils with high liquefaction potential; large volumes of soil ejecta and settlements of 300 –400 mm were observed throughout the gravelly reclamation.

CPTs were successfully performed in the gravelly reclamation (Cubrinovski et al., 2018; Dhakal et al., 2020a) yielding consistently low cone tip resistance of q_c = 6 – 8 MPa, and soil behaviour type index values predominantly I_c = 2.0 – 2.2 which is typical for sand-silt mixtures. Despite the high-gravel content, the matrix of the gravel-sand-silt mixture was effectively controlled by the finer sand-silt fractions, and hence the behaviour of the gravelly reclamation was similar to that of sandy soils (Cubrinovski et al., 2018; Dhakal et al., 2020a). Cubrinovski et al. (2018) and Dhakal et al. (2020a; 2020b; 2021) have shown that, in this case, CPT-based liquefaction evaluation procedures could be applied to gravel-sand-silt fills due to a governing role of the sand-silt fractions in the soil matrix of the gravelly fill.

Gravel-sand-silt mixtures may have high liquefaction potential and may exhibit deformational behaviour typical for sandy soils. The key property in engineering assessment of soil mixtures is to identify the controlling fraction(s) in the soil mixture, as it governs the behaviour of the mixture. Approximately 25 – 30 percent sand-silt content in a gravel-sand-silt mixture is sufficient for these finer fractions to control the behaviour of the mixture (Cubrinovski, 2019).

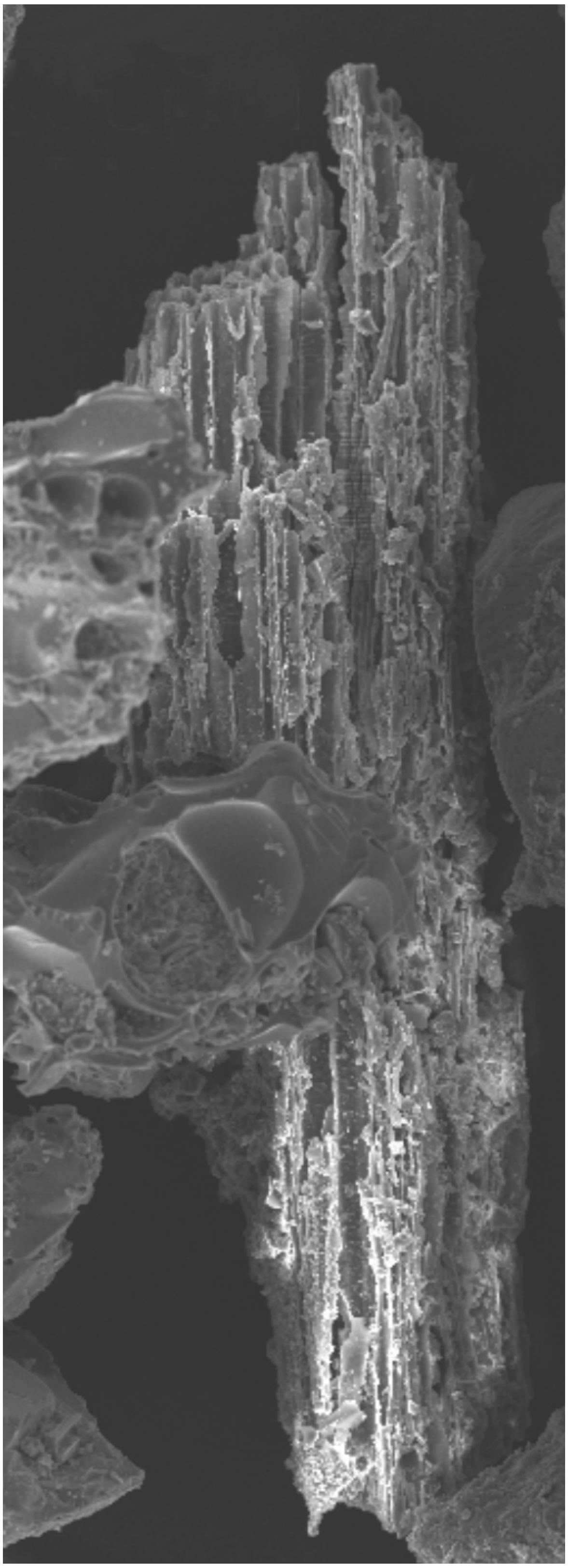
Current empirical methods for liquefaction evaluation have been largely developed based on case histories of sands and to a lesser extent sands with silts. In addition, empirical relationships in the laboratory, such as those used in the assessment of volumetric strains and settlement calculations, have been obtained from tests on a limited number of clean sands. Finally, these simplified empirical procedures either implicitly or explicitly involve conversion between the relative density of the soil (D_r) and its penetration resistance (q_{c1N}), which again has been established for clean sands.

Thus, when applying conventional liquefaction evaluation procedures to silty/clayey sands, silts and gravel-sand-silt mixtures, careful considerations and evaluation of various factors are needed in the assessment. Cubrinovski (2019) and Dhakal et al. (2021) identify key factors that require attention in the assessment and provide guidance on the use of simplified liquefaction evaluation procedures for assessment of gravel-sand-silt mixtures.

Hydraulic fills often include fine-grained soils of low, moderate, and high plasticity, but also mixtures of sand-silt and even medium gravels and shell content. They may exhibit varying degrees of spatial variability reflecting different soil sources used in the construction of the fills. In addition to CPT investigations for soil profile characterisation and liquefaction evaluation, sampling and testing of soils in the laboratory is recommended especially for soils near the I_c = 2.6 threshold (say for I_c = 2.4 – 2.8). It is also recommended that laboratory testing is performed on the immediate underlying marine sediments as they are often soft and may significantly influence the seismic performance and stability of the fill.

Liquefaction is one of the main design issues for tailings. Both static (flow) liquefaction and seismic liquefaction are of concern, as they often result in catastrophic failures. In addition to triggering issues, the focus in the evaluation is on the undrained (residual strength) of soils required in the stability assessment. Evaluation of tailings should only be carried out by experienced specialists.

9 Volcanic soils



Many volcanic soils have different properties compared to the more common sedimentary soils that comprise the majority of the case histories and research studies of liquefaction. The lack of studies and comprehensive empirical evidence of liquefaction in volcanic soils limits the availability of data to enable specific recommendations. However, the following properties and behavioural characteristics are important to consider in the engineering evaluation of their potential for liquefaction and consequential effects.

Volcanic soils include the following:

› Airfall and pyroclastic flow deposits including ash, tuff, scoria, and ignimbrite soils
› Residual soils and completely weathered volcanic rocks
› Transported materials, including alluvium and lahar deposits
› Other interbedded soils including paleosoils, loess, colluvium and diatomaceous silt.

Ignimbrite soils are typically pumice dominant granular soils and are common in the Bay of Plenty/Central North Island area. These can be locally homogeneous and thick, however, they are often interbedded with ash (eg ashfall between flow events) and paleosoils. Pumice grains are commonly described as being lightweight, vesicular, highly crushable and with very rough surfaces. The relatively low crushing strength of pumice grains makes pumiceous soils problematic from a characterisation viewpoint, as conventional penetration methods (eg SPT and CPT) are unreliable for field characterisation of pumice-rich deposits. Wesley et al. (1999) showed that cone penetration resistance was completely insensitive to the relative density of pumiceous sand. Gens et al (2016) showed significant effects of the crushability of particles on cone penetration resistance, and that effects of particle crushing strongly depend on the relative density.

> Current empirical methods for liquefaction evaluation based on penetration tests (eg CPT and SPT) are not applicable to pumiceous soils. Orense et al. (2020) and Clayton et al. (2019) have found that empirical CPT-based procedures for liquefaction evaluation produced inconsistent results with liquefaction observations from New Zealand case histories of pumiceous deposits.

Alluvial soil in regions of New Zealand with pumice-rich deposits generally have a highly variable proportion of pumice material in the soil mixture. These soils are often interbedded with other materials. Given that pumiceous and silica soils have quite different characteristics and also different requirements regarding their in situ state characterisation, a key property in the engineering evaluation is the percentage of pumice in the soil mixture. Stringer (2019) proposed a method for determining the amount of pumiceous sand and gravel in the soil mixture using gravity separation. The method makes use of the fact that pumiceous materials have a specific gravity of $G_s = 2.35$ or less, which is distinctly lower than the typical $G_s = 2.65 - 2.7$ of silica soils, to determine the percentage of pumice by mass in the soil mixture.

Note: measurement of G_s is difficult in these soils (Wesley, 2001), and care is required around the interpretation of particle size distributions (PSD) by mass since it can be significantly different from the PSD by volume.

> When evaluating pumice-rich deposits, it is important to estimate the pumice content in the mixture and then use this property in the evaluation and interpretation of the effects of pumice on behaviour of the soil.

As penetration-based methods are not applicable to pumiceous soils, V_s — based procedures are potentially an alternative field-based approach for liquefaction evaluation of pumice-rich deposits (eg Clayton et al., 2019). However, there is clear evidence that current empirical CRR – V_{s1} relationships are not applicable to pumiceous deposits (Orense et al., 2020), and that the relationships amongst V_s, D_r and CRR for pumice-rich sand are quite different from those for hard-grained sands (Asadi et al., 2018a).

> Current empirical criteria for liquefaction evaluation based on shear wave velocity (V_s) are not applicable to pumiceous soils.
>
> Note: this does not disqualify the use of V_s-based methods for soil and site characterisation of pumice-rich deposits, but rather only emphasises that current empirical **criteria** for hard-grained (silica-based) soils cannot be directly applied to evaluate the liquefaction resistance of pumiceous soils.

Pumiceous soils exhibit different pore pressure response and strain development under cyclic loading, as compared to hard-grained sand (eg Asadi et al., 2018b; Stringer et al., 2019). Particle crushing causes a pronounced initial increase in excess pore water pressures and also a relatively large and steady increase in the deformation during cyclic loading, as shown by a laboratory study by Asadi et al. (2018b). This behaviour is distinctly different from the pore pressure and strain development patterns observed for 'conventional' hard-grained (Toyoura) sand. Even though pumiceous specimens showed this very contractive behaviour, they exhibited higher liquefaction resistance than Toyoura sand, which is attributed to the irregular and complex texture of pumice particles. Interestingly, the pore pressure and strain development patterns were quite different between loose and dense pumice specimens which likely reflects the different contributions of particle crushing to the behaviour, as a function of the density of packing.

Stringer et al. (2018) have successfully recovered high-quality or 'undisturbed' soil samples with low, medium and high proportion of pumice using Gel-Push and Dames & Moore samplers, from a site at Whakatane. Despite the wide-range of pumice content and different grain-size composition of the soils, they found similar liquefaction resistance for all investigated soils. Importantly, all specimens generated large excess pore water pressures early in the cyclic loading and showed relatively large and steady increase in strains, a behaviour similar to that observed by Asadi et al. (2018b).

Pumice-rich soils exhibit fundamentally different behaviour from conventional hard-grained soils. Cyclic laboratory tests are therefore recommended for liquefaction assessment of pumice-rich deposits for high risk/high consequence projects. Both reconstituted and high-quality ('undisturbed') specimens of pumiceous soils should be tested, as results from such tests will provide basis for geotechnical evaluation and design, including reference in relation to established empirical criteria and behaviour of 'conventional' soils.

For low-risk/low-consequence projects, it is recommended to adopt an engineering approach that is appropriate for poorly understood non-standard soils, without ready-to-use empirical methods. Performing careful site characterisation using conventional field investigations (penetration tests, V_s profiling) and soil characterisation using laboratory index tests will provide data on the composition of soils (proportion of pumice in the soil; eg Stringer, 2019) and in situ state (density, ageing effects) of soils. Depending on the soil composition, the significance of the pumice content in the soil structure and consequent effects on soil behaviour should be considered.

Following this evaluation, the field data should be interpreted. In the evaluation, data from similar soils by composition and especially data from regional soils should be used to facilitate the interpretation and provide useful reference behaviour. Published data (field and laboratory) on pumiceous soils as well as well-known empirical models on hard-grained soils can be also used as comparative references in the evaluation. Scenario analyses (described in Section 10.3) can then be used to examine alternative assumptions for the behaviour of pumiceous soils and evaluate consequent effects on the seismic performance. Such process and careful considerations of key factors contributing to the seismic performance (soil composition, state of the soil in the field, behavioural characteristics of pumiceous soils, etc) including uncertainties in their effects will provide basis for engineering judgement and decision-making. If the above is deemed unsatisfactory, then laboratory testing of reconstituted soils or 'undisturbed' soils should be performed.

Volcanic ash refers to fine grained volcanic soils, with particle size from 0.001 mm to 2 mm). These include airfall deposits, sometimes reworked by climate and weathering effects. The properties of ash vary widely and many deposits are heterogeneous. Ash soils can be sensitive and their behaviour can change significantly when subject to large strains. The methods of assessing granular or cohesive behaviour using the plasticity index are useful. If undisturbed samples can be taken, then cyclic loading in triaxial cells or cyclic simple shear can provide useful results.

All of the above, as well as previous studies on liquefaction resistance of calcareous sands (which are also highly crushable), suggest that conventional liquefaction evaluation procedures based on empirical charts for sedimentary soils of common sand minerals cannot be applied to volcanic soils, and that special considerations and assessment is required for such cases. Laboratory testing of undisturbed and reconstituted soil samples may provide basis for quantifying the liquefaction resistance and developing experimental evidence for establishment of liquefaction criteria for such soils.

Recovering high-quality samples of liquefiable (non-plastic or low plasticity) soils is challenging. Recently, significant advances have been made in developing practical methods for recovering high-quality samples of such soils. Despite these advancements, it is prudent to assume that high-quality specimens are always disturbed at some level. For this reason, it is always useful and recommended to include testing of reconstituted soils (in addition to the high-quality specimens) to provide an independent reference for the cyclic behaviour of the tested soil, even though clearly reconstituted specimens will have different fabric and structure (and hence liquefaction resistance) compared to the 'undisturbed' specimens (field deposits).

10 Best practice considerations

Engineering evaluation of liquefaction problems is a complex task involving geological assessment, site investigations, laboratory testing of soils, interpretation and analysis of liquefaction processes and their effects on land and structures.

Simplified liquefaction evaluation procedures provide practical means to address many important issues in the liquefaction assessment but must be applied with engineering judgement and additional considerations for best practice, as explained in this section.

10.1 Holistic evaluation of performance

Soil liquefaction represents an extreme soil response or ground failure.

The occurrence of liquefaction generally implies a substantial increase in damage and sharp deterioration of the seismic performance of the site or soil-structure system. This feature is schematically illustrated in Figure 10.1 in which ground damage is plotted against the intensity of the ground motion (earthquake load), for a hypothetical site or soil-structure system. The relationship shows a pronounced 'step change' in the performance or sharp increase in the damage once the earthquake loading is sufficient to trigger soil liquefaction (ground failure). Therefore, from an engineering assessment viewpoint, it is important to estimate the level of:

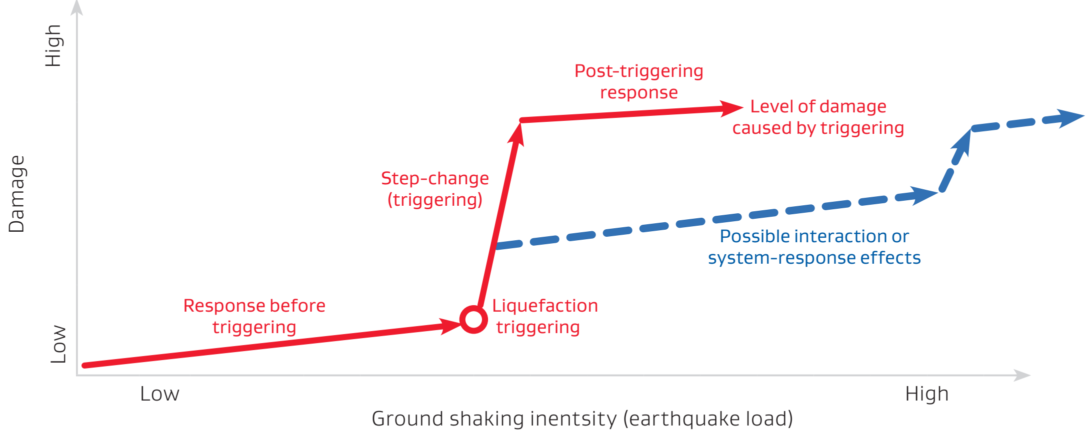
1. the earthquake load (or ground motion intensity) that is required to trigger liquefaction and,
2. damage and severity of effects that will result as a consequence of liquefaction.

Complex systems (stratified soil deposits or soil-structure systems) may exhibit additional complexities in the deformation-load relationship. For example, as illustrated with the dashed line in the figure, complex interactions and system-response effects which are demand dependent, may be activated at higher levels of shaking thus producing another step change or additional deterioration in the performance. While such effects may introduce additional challenges in the evaluation, they should not change the principal objectives in the assessment, ie to identify the triggering thresholds and consequent level of damage, as they are critical for a holistic evaluation of the seismic performance described below.

A key objective in the engineering assessment is to estimate the performance of the site/structure for earthquakes of various intensities and understand how its response and damage might evolve with increasing earthquake loading. In this process, it is important to identify threshold loads at which step-changes in performance may occur, and to quantify the severity of damage (effects of liquefaction). Various performance criteria could be employed in such assessment (eg serviceability requirements, onset of damage, onset of significant but repairable damage, onset of irreparable damage, loss of service, life safety requirements, and other system- or owner-specific requirements).

Such engineering evaluation when combined with seismic hazard information will provide a basis for a holistic assessment of the performance (damage) of the site/structure over the entire range of relevant earthquake loads including estimates for the likelihood of particular performance (damage) states occurring during the life of the structure.

Figure 10.1: Schematic load-damage relationships for a hypothetical site or soil-structure system affected by liquefaction illustrating important response characteristics and key objectives in the engineering evaluation



National guidelines and standards stipulate minimum requirements that are often insufficient for a holistic engineering evaluation of the seismic performance. Instead they focus on isolated checks for particular limit states (eg SLS and ULS). For example, there is no Code requirement to consider intermediate limit states or to understand how damage is going to evolve between SLS and ULS (eg at the 100-year level of shaking), or how the system would respond to a rare event exceeding ULS (eg the 1,000-year level of shaking).

Such limited assessment may miss important aspects and consequences of the seismic response (performance) of the system. Therefore, a holistic engineering evaluation of the performance, as described above, should always be the aim of the seismic assessment. The degree of rigour and detail in the assessment should be appropriate for the importance of the facilities planned for the site, and the level of risk to people and property arising from structural/ground performance.

10.2 Appropriate use of methodology

The simplified liquefaction evaluation procedures described in this Module are empirical in nature and have been developed predominantly from case histories on sandy soils and observations from laboratory tests on clean sands.

For example, relationships for clean sand have been used in the conversion between penetration resistance and relative density of soil, and post-liquefaction volumetric strains measured in laboratory tests on clean sand have been used to establish the simplified procedure for evaluation of liquefaction-induced settlement. Consequently, empirical procedures for liquefaction evaluation are centred on clean sands, and use highly simplified material characterization of soils, eg fines content, FC or soil behaviour type index, I_c to make critical adjustments for sandy soils containing fines.

When evaluating soils that are not well represented in the empirical database (eg soils with moderate plasticity, gravel-sand-silt mixtures, etc.), a more cautious approach should be taken in the assessment. It is important to critically examine the applicability of simplified procedures to the site and soils of interest. Direct application and indiscriminate use of the empirical procedures to various 'non-standard' soils, could potentially result in erroneous outcomes and an incorrect evaluation of the seismic risk associated with liquefaction.

It is important to rigorously and correctly use empirical correlations in the liquefaction assessment. For example, use of non-standard penetration tests, conversion of their blow counts to SPT N-values, and then use in liquefaction triggering analysis is inappropriate.

Current empirical criteria for liquefaction evaluation based on penetration tests and shear wave velocity are not applicable to pumiceous soils. Laboratory tests are recommended for liquefaction assessment of problematic soils for which insufficient empirical evidence is available especially for high risk/high consequence projects.

For low-risk/low-consequence projects, careful site characterisation and soil characterisation using conventional field and laboratory methods is recommended. Factors such as soil composition, field parameters and behavioural characteristics of pumiceous soils should be considered in conjunction with regional data on similar soils and general data on similar soils to form an engineering judgement in the evaluation of pumice-reach deposits. Scenario analyses (described in Section 10.3) can then be performed to examine the seismic performance using different assumptions with regard to the liquefaction resistance or resulting deformations of pumiceous soils.

Predicting liquefaction-induced deformation (ie settlement, lateral displacements, etc) is challenging and burdened by significant uncertainties. While simplified procedures provide a means for an explicit calculation of liquefaction-induced settlements and displacements, it is important to understand the limitations of those calculations and to appropriately report, interpret and use such estimates in the engineering evaluation.

> Given the limitations of the methods and uncertainties in the assessment of liquefaction-induced deformation, it is prudent to estimate a range of values and use those as an indicator of the level of damage rather than as a precise estimate of deformation. Crudeness of the employed procedures in relation to the complexity of the problem addressed should be carefully considered and embodied in the engineering interpretation of liquefaction problems.

The simplified procedures described in this Module incorporate numerous important effects and factors in the liquefaction assessment. They often employ the principle of superposition in which individually evaluated effects are added to estimate their cumulative effect while mostly ignoring interactions. There is clear evidence that cross-layer interactions, system-response effects and soil-structure interaction can substantially influence and even govern the seismic performance of sites and soil-structure systems.

> Effects of interactions and overall response of the system should be considered in engineering evaluations.
>
> Note: some methods completely ignore interactions, and therefore an additional effort is needed in the engineering assessment to estimate such effects. The effort required in the assessment should be commensurate with the anticipated significance of these effects and importance of the structure.

10.3 Treatment of uncertainties

Liquefaction assessment is always burdened by significant uncertainties, and the appropriate treatment of these uncertainties is critical in engineering evaluations.

A methodical and rational approach is recommended in which:

1. Critical uncertainties are first identified (ie uncertainties most significantly affecting the outcomes of the assessment and performance)
2. For each critical uncertainty (parameter or relationship), an appropriate range of values are determined encompassing the estimated uncertainty and
3. A sensitivity study is performed (using lower-bound, upper-bound and best-estimate values) to estimate the range of the response (level of damage) and quantify the effects of each critical uncertainty.

Liquefaction damage indices (eg LPI and LSN) are not intended to be used as precise measures of liquefaction-induced ground damage, but rather as indicators of the general level of liquefaction severity. Given the aggregated uncertainties in these damage indices, they should be cautiously used and interpreted. As a given value of LPI or LSN may often encompass a wide range of potential liquefaction performances, a conservative approach adopting the higher level of damage in the range is recommended, except for cases where clear evidence exists justifying the lower level of ground damage.

Scenario analysis[4] is a useful and recommended approach to scrutinize the seismic response of the structure or its foundations including various combinations (and associated uncertainties) of load intensity and distribution, properties of liquefied soils, and ground response (rapid change in stiffness and strength of soils, water flow effects, instability effects, effects of liquefaction on ground motion and structural response, interactions, etc). Scenario analyses can be performed both within simplified and advanced assessment procedures.

4 Scenario analysis is a generic term for an analysis approach in which the seismic performance of a structure is investigated assuming different potential responses (‘scenarios') for the ground or soil-structure system. For example, in the case of a building on shallow foundations with variable subsurface conditions over the footprint of the building, different scenarios for the lateral extent of liquefaction and consequent differential settlements can be adopted in two or three analyses to quantify the effects of differential settlement on the building superstructure. In another example, scenario analyses can be used to examine the effects of timing of occurrence of differential settlement on the building superstructure. Different proportions of settlement occurring during the strong shaking and post shaking could be used in scenario analyses to examine the performance of the superstructure under different combinations of inertial loads, cyclic deformations and settlement-induced deformations.

There are similarities between scenario and sensitivity analyses. In a sensitivity analysis, the sensitivity of the seismic response to changes of specific analysis parameters is investigated. In a scenario analysis, the seismic performance of the system is evaluated using different (but realistic) assumptions for possible alternative responses and interactions developing within the system. In this context, scenario analysis could be seen as a higher-level sensitivity analysis, in which alternative response scenarios are investigated. Scenario analysis is an excellent tool for the designer to scrutinize key uncertainties and assumptions in the assessment, and develop better understanding of the seismic performance of the soil-structure system.

Significant uncertainties in the seismic demand (hazard) will always be present in engineering evaluations. However, it is important to consider this significant uncertainty in seismic demand uncertainties in the context of the engineering evaluation. Uncertainties in the seismic demand (earthquake load) will not influence in any way the estimate of the seismic performance of the site/structure, for a given earthquake load. They will only affect the risk of a specific level of performance (damage) occurring during the life of the structure. This separation of the engineering evaluation of the performance and quantification of its associated risk via the seismic hazard should be recognized.

The seismic demand (hazard) uncertainties should not be used as an excuse for avoiding holistic performance assessment, inappropriate (non-rigorous) use of methodology or inadequate treatment of other uncertainties in the engineering evaluation. In other words, seismic demand uncertainties should not be used as a barrier to best practice.

When employing the scenario analysis approach within simplified evaluation procedures (ie equivalent static methods), it is important to recognize the highly dynamic nature of liquefaction problems, and consider temporal and spatial evolution of the earthquake loading, ground response and response of the soil-structure system. Such considerations are critically important to ensure that the adopted loads, soil properties and boundary conditions are compatible and realistic for the phase of the dynamic response considered. Failing to do so may lead to erroneous estimates of performance including possible omission of key mechanisms of deformation and modes of interaction that may either govern or strongly influence the performance of the system.

Scenario analyses, when executed rigorously, also provide a practical approach for evaluation of the effects of uncertainties on the seismic performance.

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Appendix A. Important differences between Boulanger and Idriss (2014) and Idriss and Boulanger (2008) methods

The most recent method of Boulanger and Idriss (B&I 2014), as an updated and improved version of the Idriss and Boulanger (I&B 2008) method, has received particular attention in the profession.

This commentary outlines some of the key features of this method. There are several important details in which the B&I 2014 method differs from the I&B 2008 triggering evaluation method. The key additions and modifications provided in B&I 2014 are listed first and then briefly discussed.

- In B&I 2014, the CPT database has been updated adding data from recent earthquakes, and also some of the older case studies have been re-examined. Importantly, a significant number of liquefaction case histories from the Canterbury earthquake sequence (50 in total) have been added to the dataset.
- New magnitude scaling factor (MSF) relationship has been proposed. This MSF relationship is fundamentally different from the MSF relationships used in all other liquefaction triggering evaluation methods because it is density and soil type dependent.
- A simplified procedure for estimating fines content for use with the CPT-based liquefaction evaluation has been recommended. The fines content estimation is based on a newly established relationship between the fines content (FC) and CPT-derived Soil Behaviour Type Index (I_c). Again, data from Christchurch soils have been used in the development of this relationship, and the proposed relationship in B&I 2014 is similar to the FC – I_c relationship developed by Robinson et al. (2013) using data on liquefiable soils along the Avon River in Christchurch.
- A probabilistic version of the CPT-based liquefaction triggering procedure has been developed.

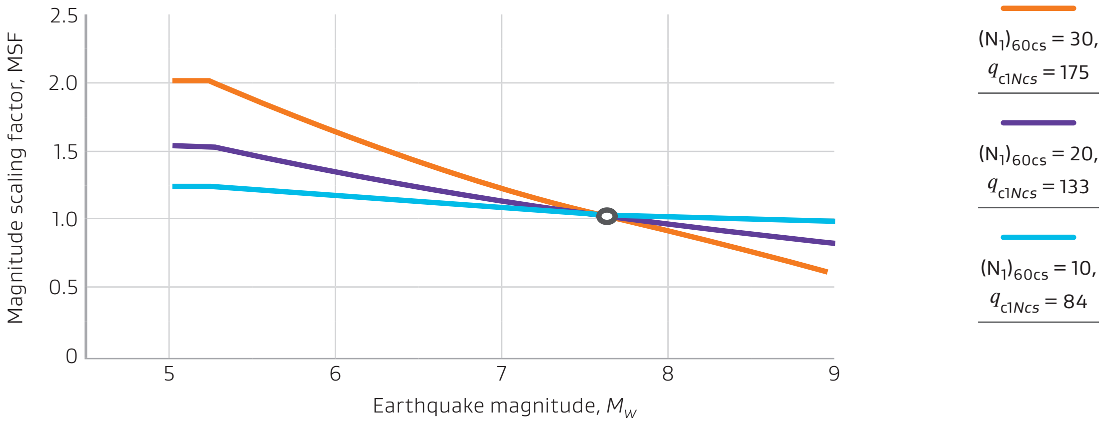
The most significant change in relation to all previous liquefaction triggering evaluation methods is the use of a density and soil type dependent MSF relationship. As illustrated in Figure A.1, the proposed MSF relationship by B&I 2014 accounts for differences in the penetration resistance (or soil characteristics). It implies that the MSF varies significantly in dense sands (high penetration resistance), while the variation of MSF with M_w is much smaller for loose sands (low penetration resistance). All other currently available methods provide a single MSF – M_w relationship for all cohesionless soils and soil densities.

When evaluating the proposed density and soil type dependent MSF relationship of B&I 2014, it is important to recognize that in the context of the simplified liquefaction evaluation procedure or calculation of factor of safety against liquefaction triggering, the MSF essentially combines two relationships:

- number of equivalent shear stress cycles and earthquake magnitude relationship, and
- the relationship between the amplitude of cyclic shear stress and number of cycles required to trigger liquefaction.

The former relationship defines the earthquake load (seismic demand) in terms of the number of cycles with significant amplitudes, while the latter determines the liquefaction resistance or capacity of the soil (‘cyclic strength’ or ‘liquefaction resistance’) in terms of shear stress amplitude — number of cycle combinations that cause the soil to liquefy.

Figure A.1: Variation in MSF relationship with q_{c1Ncs} and $(N_1)_{60cs}$ for cohesionless soils (Boulanger and Idriss, 2014)



The procedure of B&I 2014 is considered to more accurately depict the shape of the liquefaction resistance curve as it depends on the soil type and density, as observed in laboratory soil tests. In this context, it is important to understand that MSF should not be interpreted as only a correction of the seismic demand (earthquake loading). This detail is particularly important to be recognized when using the B&I 2014 method since it uses soil density dependent MSF – M_W relationships. The above discussion is still applicable to other methods, but it is of no practical significance as they use a single MSF – M_W relationship, which is independent of soil density or soil type.

The addition of Christchurch data in the B&I 2014 procedure, and the similarity of the proposed FC – I_c relationship with the specific FC – I_c relationship derived for Avon River soils (Robinson et al., 2013), improve the assessment of liquefaction for alluvial soils of similar origin, composition and depositional environment as the Canterbury soils.

The proposed FC – I_c relationship by Boulanger and Idriss (2014) and their commentary on its use also deserves some attention. When site-specific sampling and testing (the preferred approach) are not available, and one refers to the use of the FC – I_c relationship for estimating FC from CPT data, Boulanger and Idriss recommend to explicitly consider the uncertainties in FC and soil classification estimates, and to evaluate their effects on the engineering evaluation using parametric analyses. Their recommendation is adopted in this guideline. It reflects that, on one hand, a significant correction of the liquefaction resistance is made based on the fines content, and that, on the other hand, significant variability and uncertainty are associated with the FC – I_c relationship, which in turn is directly used for the fines content estimation.

Note: the FC – I_c correlation is particularly weak and unreliable for low fines content of less than 10 percent to 20 percent, and that uncertainties exist regarding the I_c threshold value (I_c = 2.6) separating between liquefiable and non-liquefiable soils, as discussed in Section 5.2.2.

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