Geotechnical earthquake engineering practice

Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards



NEW ZEALAND GEOTECHNICAL SOCIETY INC
www.nzgs.org
July 2010

New Zealand Geotechnical Society

Guidelines for Geotechnical Earthquake Engineering Practice in New Zealand

Rev 0

Issue Date July 2010

Important notice

This document is preliminary and the contents should be treated as draft guidelines. Submissions by the geotechnical community to the Society are encouraged for a period of one year from the issue of this document, after which a further review will be undertaken. The contents may be subject to further changes, additions, and deletions.

Disclaimer

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Preface

This document is the product of a working group of the New Zealand Geotechnical Society. The idea for the working group came from a panel discussion "Geotechnical Seismic Design Standards" which took place during the NZGS Biennial Symposium "Earthquakes and Urban Development" held in Nelson from 17–18 February 2006.

The main impetus for the panel discussion was the impending replacement of NZS 4203 by NZS 1170. While far from complete, NZS 4203 gave some useful guidance to geotechnical practitioners including a specific rule for "equivalent static method of analysis" of earth retaining structures. However, NZS 1170.5 specifically excludes design of soil retaining structures and civil structures including dams and bunds, the effects of slope instability, and soil liquefaction.

Even with the very limited guidance given in NZS 4203, there was perceived to be a significant and undesirable variability within geotechnical earthquake engineering practice in New Zealand. Ad hoc attempts are being made by individuals and organisations to interpret NZS 1170.5 for geotechnical design in ways that were perhaps never intended by the authors of that standard.

The Nelson meeting unanimously endorsed the motion that the Society should form a working group to prepare a set of guidelines to assist geotechnical practitioners with geotechnical seismic design. Some urgency in producing the guideline was seen as being necessary with a desire to avoid a long and protracted process. To this end, a two stage approach was suggested with the first stage being an interpretation of NZS 1170.5 with appropriate factors for geotechnical design, to be established in conjunction with the authors of NZS 1170.5, and identifying the relevant philosophy for geotechnical seismic design. Later, more specific guidelines for a range of geotechnical systems might be added.

The authors are well aware that standards for geotechnical seismic design are under development world wide, notably Eurocode and ISO. While there is no need to "reinvent the wheel" there is a need to adapt such initiatives to fit the New Zealand environment and philosophy.

The meeting also strongly endorsed the view that "guidelines" are far more desirable than "codes" or "standards" in this area. Flexibility in approach is a key part of geotechnical engineering and the technology in this area is rapidly advancing.

A working group was duly established with the following members:

Dr K J McManus – McManus Geotech Ltd and University of Canterbury (lead)
Associate Professor M Cubrinovski – University of Canterbury (major author)
Professor M J Pender – University of Auckland
Dr G McVerry – GNS Science
Mr T Sinclair – Tonkin & Taylor Ltd
Dr T Matuschka – Engineering Geology Ltd
Dr K Simpson – Beca Infrastructure
Mr P Clayton – Beca Infrastructure
Mr R Jury – Beca Infrastructure

A draft of the guidelines was presented as a discussion document at a workshop held in Auckland on 3 September 2008 immediately prior to the Society's biennial symposium. Feedback and discussion from the Workshop was included in this final draft document submitted to the Society's Management Committee in February 2009 for Internal Peer Review.

It is the intent of the working group that this initial document will be expanded and revised periodically in coming years. The science and practice of geotechnical earthquake engineering is far from mature and is advancing at a rapid rate. It is important that users of this document familiarise themselves with the latest advances and amend the recommendations herein appropriately.

Contributions from the working group and the workshop have addressed in part other aspects of geotechnical seismic design that will form the basis of future modules. This document was edited by Dr C Y Chin and A L Williams.

Financial support for this initiative has been provided by the Department of Building and Housing. The assistance of Graeme Lawrence and David Hopkins on behalf of the Department is gratefully acknowledged.

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Introduction

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New Zealand is a high earthquake hazard region and earthquake considerations are integral to the design of the built environment in New Zealand. The effect of earthquake shaking needs to be considered for every aspect of geotechnical engineering practice and frequently is found to govern design.

This document is not intended to be a detailed treatise of latest research in geotechnical earthquake engineering, which continues to advance rapidly, but to provide sound guidelines to support rational design approaches for everyday situations, which are informed by latest research.

Complex and unusual situations are not covered. In these cases special or site specific studies are considered more appropriate.

The guidance material has been developed primarily for use with NZS 1170 and as such should be regarded as being similarly restricted in the scope of applications. While every effort has been made to make the material useful in a wider range of applications, applicability of the material is a matter for the user to judge.

The main aim of this guidance document is to promote consistency of approach to everyday situations and, thus, improve geotechnical-earthquake aspects of the performance of the built environment.

The science and practice of geotechnical earthquake engineering is far from mature and is advancing at a rapid rate. It is important that users of this document should familiarise themselves with recent advances and interpret and apply the recommendations herein appropriately as time passes.

Scope

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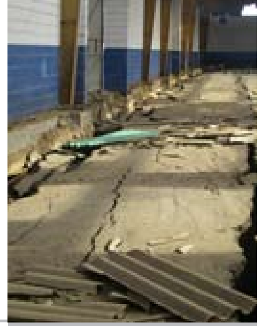
The material in this document relates specifically to earthquake hazards, and should not be assumed to have wider applicability. It is intended to provide general guidance for geotechnical earthquake engineering.

It is intended that, when properly used in conjunction with NZS 1170 and relevant materials standards, the resulting design would comply with the New Zealand Building Code, and through that compliance, achieve the purpose stated in the *Building Act 2004* of ensuring that people who use buildings can do so safely and without endangering their health.

Other documents 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*, *New Zealand Society for Earthquake Engineering Guidelines for Tanks*, *Transit New Zealand Bridge Design Manual*, and *Transpower New Zealand Transmission Structure Foundation Manual*.

Where significant discrepancies are indentified among different guidelines and design manuals it is the responsibility of the designer to resolve such discrepancies as far as practicable so that the design meets the requirements of the *Building Code and Building Act*.



Geotechnical considerations for the built environment

SECTION 3

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

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

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

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

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

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

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

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

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

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

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

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

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

Geotechnical earthquake hazards

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4.1 General

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

The principal geotechnical hazards associated with earthquakes are:

1. fault rupture
2. ground shaking
3. liquefaction and lateral spreading
4. landsliding.

Each of these hazards is described in more detail below.

4.2 Fault rupture

For shallow earthquakes, the fault rupture surface may extend to the ground surface generating scarps and lateral offsets of up to several metres. The extent of surface deformation is dependent on the type of fault and the depth and nature of surface soils. These deformations may be very damaging to buried services, roads, and railways. Light structures may be literally torn apart if the rupture surface dissects the building footprint. For heavier structures (eg concrete buildings of more than three storeys), the fault rupture surface tends to deviate around the building footprint because of the effect of the additional soil confining pressure.

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

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

Wherever doubt exists, trenching or other means (geophysics and boreholes) should be used to establish the exact location (or locations) of an active fault trace near to or on a site.

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

4.3 Ground shaking

Ground shaking is one of the principal seismic hazards that causes 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 and duration of ground shaking. Similarly, the level of ground deformation, damage to earth structures and ground failures are closely related to the severity of ground shaking.

In engineering evaluations, three characteristics of ground shaking are typically considered:

- the amplitude
- frequency content
- significant duration of shaking (time over which the ground motion has relatively significant amplitudes).

These characteristics of the ground motion at a given site are affected by numerous complex factors such as the source mechanism, earthquake magnitude, rupture directivity, propagation path of seismic waves, source distance and effects of local soil conditions. 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.

4.4 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 is associated with significant loss of stiffness and strength in the liquefied soil and consequent large ground deformation. Particularly damaging for engineering structures are cyclic ground movements during the period of shaking and excessive residual deformations such as settlements of the ground and lateral spreads.

Ground surface disruption including surface cracking, dislocation, ground distortion, slumping and permanent deformations such as large settlements and lateral spreads are commonly observed at liquefied sites. Sand boils including ejected water and fine-grained fractions of liquefied soils are also typical manifestations of liquefaction at the ground surface. In 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 that sand boils are a definite sign of liquefaction, however they do not always occur at liquefied sites.

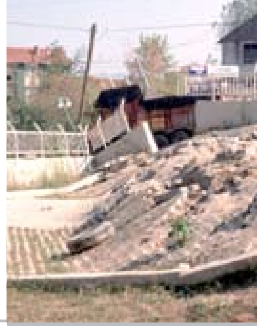
In sloping ground and backfills behind retaining structures in waterfront areas, liquefaction often results in large permanent ground displacements in the down-slope direction or towards waterways (lateral spreads). In the case of very loose soils, liquefaction may affect the overall stability of the 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). Recent papers by Boulanger and Idriss (2006, 2007) are helpful in clarifying issues surrounding the liquefaction and strain softening of different soil types during strong ground shaking.

For intermediate soils, the transition from "sand like" to "clay-like" behaviour depends primarily on whether the soil is a matrix of coarse grains with fines contained within the pores or a matrix of plastic fines with coarse grained "filler". The fines content of the soil is of lesser importance than clay mineralogy as characterised by plasticity. Engineering judgment based on good quality investigations and data interpretation should be used for classifying such soils as liquefiable or non-liquefiable.

4.5 Landslides

Landslides are a familiar geotechnical hazard in many parts of New Zealand. The rate of incidence of landslides is at its highest during or following high rainfall intensity events, but earthquakes also trigger many landslides, including very large, dangerous rock slides. Horizontal acceleration of the ground caused by earthquake shaking can significantly reduce the stability of inclined masses of soil and rock. Even though the acceleration pulses may be of short duration, they may be sufficient to initiate an incipient failure, especially where the soil or rock is susceptible to strain softening.



Estimating ground motion parameters

5.1 General

Most earthquakes occur on existing faults with a recurrence interval that depends on the rate of strain-energy accumulation and varies from hundreds of years 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.

Because there is so much uncertainty in predicting earthquake events a statistical approach is usually adopted to assess the seismic hazard at any location. The level of hazard varies significantly across New Zealand with very high levels near to the Australia/Pacific plate boundary where high rates of crustal strain occur. Seismic hazard generally decreases with distance from this zone.

For geotechnical earthquake engineering, the amplitude (the largest value of acceleration recorded during the earthquake, represented by peak ground acceleration, PGA) and duration of shaking (related to earthquake magnitude) are the key input parameters to most common design procedures, with no direct consideration of the frequency (represented by the response spectrum). Traditionally, seismic hazard is presented in codes as a response spectrum intended for building design where resonance effects are important. Resonance effects are not usually considered in the design of most geotechnical systems (exceptions include large earth dams).

As incoming seismic waves travel from relatively stiff bedrock into much softer soils at a site they slow down and the amplitude of shaking increases. There may be resonance effects that amplify specific frequencies depending on the stiffness, thickness, density, and geometry of the soil deposit at the site and the amplitude of shaking. For very strong shaking there may be attenuation caused by yielding of weak soils and filtering of certain frequencies because of the non-linear, strain-dependent stiffness of soil.

Fault rupture for large earthquakes may involve surfaces of many kilometres in extent. Rupture typically initiates at a “point” and then propagates across the fault surface at a velocity similar to that of seismic wave propagation. At distances far away, the earthquake appears to be from a single point source but near to the fault the situation is much more complex with bunching up of seismic waves in certain directions.

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

- amplitude of shaking at bedrock beneath the site
- thickness and properties of soil strata beneath the site and overlying the bedrock
- proximity of the site to active faults
- three dimensional relief both of the surface contours and sub-strata.

The ground motion parameters at a site may be evaluated using one of three methods:

Method 1: Based on the earthquake hazard estimates presented in NZS 1170.5.

Method 2: Location specific probabilistic seismic hazard analysis (PSHA).

Method 3: Site-specific site-response analysis.

Method 1 is appropriate for everyday engineering design projects. Methods 2 and 3 are preferred for more significant projects, more difficult sites, or other cases where more analysis can be justified.

5.2 Method 1: NZS 1170.5:2004

Elastic site spectra for New Zealand are published in *Structural design actions – Earthquake actions – New Zealand*, derived from results of a probabilistic seismic hazard model developed by the Institute of Geological and Nuclear Sciences [Stirling et. al., 2000, 2002]. The model incorporates 305 known active faults, a distributed seismicity model to account for the hazard from unknown faults, and expressions to account for attenuation of shaking between earthquake source and site. From this document, it is possible to obtain values of design PGA for any site in New Zealand by using the procedure given below.

Amplitude of shaking

Peak horizontal ground acceleration (PGA) may be calculated as:

$$a_h = Z\,R\,C \qquad (5.1.1)$$

in which:

Z = base PGA called "Hazard factor" and is given by NZS 1170.5:2004 Table 3.3 and Figures 3.3 and 3.4

R = "Return period factor" and is given by 1170.5:2004 Table 3.5

C = Site response factor called "Spectral shape factor" in NZS 1170.5:2004 and is summarised here:

Class A, B	*Rock sites*	*C = 1.0*
Class C	*Shallow soil*	*C = 1.33*
Class D, E	*Deep, soft soil*	*C = 1.12*

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

Descriptions of the different site soil "classes" are given in NZS 1170.5:2004 clauses 3.1.3.2 to 3.1.3.6 and in Table 3.2. Selection of the appropriate site soil "class" should, preferably, be based on knowledge of the site subsoil stratification to bedrock.

Duration of shaking (related to earthquake magnitude) is accounted for implicitly within the design PGA values given in NZS 1170.5:2004 by a procedure of *magnitude weighting*: earthquake sources of magnitude M less than 7.5 were given lower weighting in computing the site spectra because the duration of shaking and thus damage potential is reduced for lower magnitude earthquakes. Where design procedures require specific input values for magnitude M (eg all common liquefaction assessment procedures), then the value M = 7.5 should be entered if values of PGA from NZS 1170.5:2004 are being used.

For buildings with natural periods of vibration greater than 1.5 seconds, NZS 1170.5:2004 requires a "near fault" factor to be applied to the design acceleration where the sites are located within 20 km of eleven named faults.

> Near fault effects need to be considered for major geotechnical structures with a natural period of vibration > 2 seconds located within 20 km of a major active fault. Suitable factors are given in NZS 1170.5:2004 Section 3.1.6 and Table 3.7.

In selecting accelerograms for response-history analysis, approximately one-in-three records (eg one in a set of three, or two to three in a set of seven) should possess strong forward-directivity characteristics for sites located within 20 km of any of the eleven named faults, irrespective of the natural period of the structure.

5.3 Method 2: Site specific probabilistic seismic hazard analysis (PSHA)

Method 2 is preferred to Method 1 for important structures. Method 2 allows peak ground accelerations and/or spectra to be developed for the location of interest for the appropriate NZS 1170 site class, rather than scaling these from the NZS 1170 hazard factor, Z. It also allows for updating of the 2000 seismic hazard study on which NZS 1170 was based, such as by using new fault parameters from Stirling et al. (2008).

The justification for performing a Method 2 analysis is that NZS 1170 spectra and PGAs for a given site class are usually conservative representations of the values that will be calculated specifically for a given location.

Method 2 site specific probabilistic seismic hazard analysis should only be carried out by experienced specialists.

5.4 Method 3: Site specific site-response analysis

Method 3 involves evaluation of site-specific scale (amplification) factors through detailed site-response analyses and hence potentially provides more realistic values for site effects than Methods 1 or 2, which both use generic site-response factors according to the NZS 1170 site class. Method 3 is appropriate for more significant projects, more difficult sites, or other cases where more analysis can be justified.

This method entails specific modelling of the soil profile in the vicinity of the site requiring much more geotechnical information than Methods 1 or 2 as stress-strain characteristics need to be specified for each of the modelled soil units.

Site-specific analysis can be carried out to varying levels of complexity:

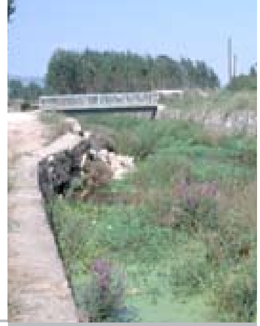
- **1-D analysis**: Various software programs are available to perform this analysis but require good judgement and a good knowledge of the soil properties and profile to bedrock for the result to be meaningful. Non-linear soil response may be modelled either through an equivalent-linear analysis or a fully non-linear analysis.
- **2-D analysis**: Useful for sites with strong 2-D geometry effects where focussing of incoming seismic waves and resonance effects (such as at the edge of a basin, eg Kobe 1995) may occur. These effects may be analysed by time history analysis using available commercial finite element or finite difference software. The direction of incoming seismic waves may significantly affect the result and care is required.

- **3-D analysis**: Directivity and resonance effects are known to be significant for many sites and are a subject of ongoing research. 3-D analysis is highly complex and requires research level software and expertise. 3-D analysis may be justified for projects of special significance or for regional micro-zoning studies.

Method 3 (site specific site-response analysis) should only be carried out by experienced specialists.

5.5 Site investigation

All three methods for estimating ground motion parameters at a site require a good knowledge of the thickness and properties of soil strata beneath the site down to bedrock. For some sites, the thickness and general nature of the sub-soil strata may be known with sufficient confidence from local geological modelling and previous borings. For small scale developments it may be appropriate to use a conservative assumption of the thickness and general nature of the sub-soil strata based on interpretation of the regional geomorphology by an experienced professional. Otherwise, new borings and/or geophysics should be used to establish the site parameters. An "engineering bedrock" may be defined as a rock mass with a strength of at least 1 MPa and a shear wave velocity of greater than 360 m/s.



Identification, assessment and mitigation of liquefaction hazards

SECTION
6

6.1 Introduction

Cyclic behaviour of saturated soils during strong earthquakes is characterized by development of excess pore water pressures and consequent reduction in the effective stress. In the extreme case, the effective stress may drop to zero (100 % excess pore pressure rise) and the soil will liquefy. In these guidelines, liquefaction refers to the sudden loss in the shear stiffness and strength of soils associated with the reduction in the effective stress due to cyclic loading caused by an earthquake.

The mechanism of pore pressure build-up is governed by a tendency of soils to contract under cyclic loading. When saturated soils are subjected to rapid earthquake loading, such contraction is prevented by the presence of pore water and the contractive tendency instead results in a build-up of excess pore water pressure and eventual liquefaction. In this context, loose granular soils are particularly susceptible to liquefaction because they are very compressible and contractive.

Identification of liquefaction hazard at a site firstly requires a thorough investigation and sound understanding of the site geomorphology. 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. Remedial techniques for mitigation of liquefaction and its consequences are briefly addressed.

6.2 Site investigation and hazard identification

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

Most cases of soil liquefaction have occurred in relatively young deposits of poorly consolidated alluvial soils with a high water table (saturated soils). Typically, these are fluvial deposits laid down in a low energy environment. 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 high density and not prone to liquefaction but may overlie backwater deposits), abandoned river channels, former pond, marsh or swamp, and reclamation fills. 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.

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.

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.

Liquefaction can occur within strata at great depths, and this possibility is addressed in the liquefaction evaluation procedure through the overburden stress factor, $K\sigma$. Current state-of-practice considers that the likelihood of surface damage decreases with increasing depth, and therefore liquefaction-related investigations are commonly limited to depths of up to 20 m except for thick reclaimed fills or earth dams.

6.2.1 Investigation plan

The main objective of the site investigation is to identify susceptible soil strata and to determine the in-situ state of susceptible soils. A suitable investigation should include the following features:

- continuous profile of the subsoil to firm basement (usually by borehole and/or CPT)
- measurement of depth to water table
- in-situ testing of all susceptible strata (usually by CPT or SPT)
- sampling of all susceptible strata
- grading curves for susceptible soils
- atterberg limit tests for all soils.

Greater depths of profiling may be required for some projects, for example where deep foundations are being considered. Determination of in-situ soil state will typically be carried out by penetration soundings for “sand like” soils and by measurement of undrained shear strength and sensitivity for “clay-like” soils.

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

Variation of subsoil stratification spatially is often severe 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 should be sufficient for liquefaction assessment and will vary with the size, importance of the structure and spatial variability of the soil profiles at the site. Sampling and lab testing should be carried out for each continuous stratum.

6.2.2 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. 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 thin strata of liquefiable soil may pose considerable hazard. Procedures relying on sample recovery may fail because of the difficulty of retaining loose, cohesionless soils within a sampler unless in a highly disturbed state.

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 (CPT, also CPTU or piezocone where pore water pressure is measured) is the preferred test procedure because of its sensitivity, repeatability, and ability to provide continuous profiling and to detect very thin strata. Typically, 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 cobbly soils that refuse penetration by the CPT and it will be necessary to pre-drill through these.

The Standard Penetration Test (SPT) is performed within a borehole at set intervals, typically 1 m to 2 m. It has the advantage that a disturbed soil sample is recovered after each test but has the disadvantage that test intervals are widely spaced and susceptible soil strata may be overlooked. A one-metre interval in measuring SPT resistance is recommended for liquefaction assessment. The SPT procedure has other technical limitations including relatively poor repeatability and operator dependence. If SPT is to be relied upon for an investigation, then the results should be carefully interpreted and corrected according to the recommendations of Youd et. al. (2001).

Evaluation procedures using profiles of shear wave velocity versus depth are becoming widely accepted. Typically, shear wave velocity profiles are obtained using a seismic CPT (a CPT probe with an in-built geophone) and performed in conjunction with a CPT sounding. Shear wave velocity measurements are taken at set intervals (typically 1 m) and so do not provide a continuous profile with depth. However, the shear wave velocity may be correlated with CPT penetration resistance to give a pseudo-continuous profile. Other techniques, both destructive and non-destructive, 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 simple shear) may be considered for large projects. Pending additional research into pumice soils, shear wave velocity profiling may prove to be the most appropriate investigation tool but there is no database of proven case studies for these soils.

6.3 Assessment of liquefaction and lateral spreading

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

1. liquefaction susceptibility
2. triggering of liquefaction
3. consequences of liquefaction.

These steps address the following concerns respectively:

- Are the soils at the site susceptible to liquefaction?
- 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 ground deformation and effects on engineering structures?

This section discusses susceptibility criteria and analysis procedures for assessment of triggering and consequences of liquefaction.

6.3.1 Liquefaction susceptibility

Determination 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.

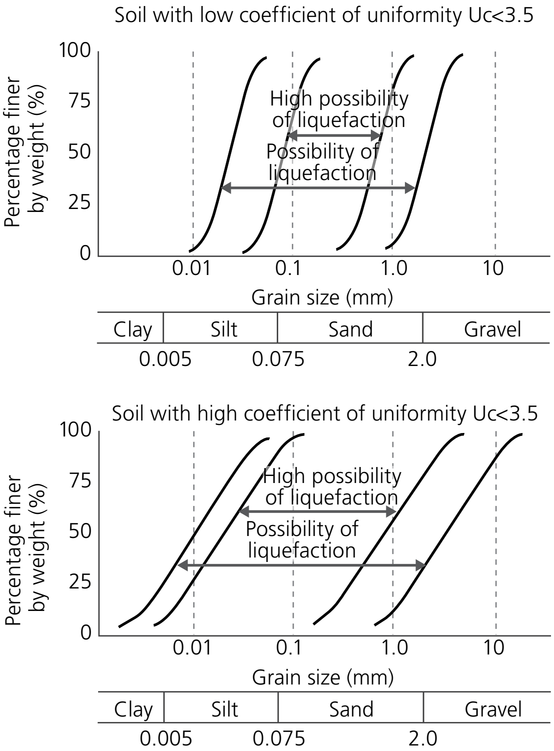
Geological criteria

The age of the deposit is an important factor to consider when assessing liquefaction susceptibility. Young Holocene sediments and man-made fills in particular are susceptible to liquefaction (Youd and Hoose, 1977; Youd and Perkins, 1978). In fact, most liquefaction failures and nearly all case history data compiled in empirical charts for liquefaction evaluation come from Holocene deposits or man made fills (Seed and Idriss, 1971; Seed et al., 1985). It has been generally accepted that aging improves liquefaction resistance of soils, however, these effects are difficult to quantify and are usually not directly addressed in design procedures. Note that liquefaction has been reported in late Pleistocene sediments (Youd et al., 2003), though such episodes are rare and comprise a very small part in the total body of liquefaction case histories.

Compositional criteria

Classification of soils based on soil type and grain-size composition has been commonly used for preliminary evaluation of liquefaction susceptibility. For example, Figure 6.1 shows grain size distribution zones of soils susceptible to liquefaction according to the criteria of the Ministry of Transport, Japan (MTJ, 1999). One should acknowledge though that these criteria are of an approximate nature and that there are no generally accepted compositional criteria for liquefaction susceptibility.

Figure 6.1 **Grain-size distribution of liquefiable soils (Ministry of Transport, Japan, 1999)**



Most cases of liquefaction have occurred in saturated cohesionless soils within the coarse silt to fine sand range. However, there is abundant evidence of liquefaction occurring in non-plastic and low-plasticity soils outside this range. In the 1995 Kobe (Japan) earthquake, for example, massive liquefaction occurred in well-graded reclaimed fills containing 30 % to 60 % gravels. In the 1999 Chi-Chi (Taiwan) and 2000 Tottori (Japan) earthquakes, extensive liquefaction occurred in sands containing a significant amount of fines. Current knowledge thus suggests that grading criteria alone are not a reliable indicator of liquefaction susceptibility.

There is general agreement that sands, non-plastic silts, 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 coarse-grained soils and non-liquefiable clays, such as sands containing low-plasticity silts and/or some amount of clays.

There are numerous subtle differences between the undrained responses of sands and clayey soils. The key difference in the context of liquefaction examined here is the fact that the pore pressure rise in clayey soils is typically limited to 60 % to 80 % of the overburden stress whereas sands, gravels and non-plastic silts generate 100 % rise in the excess pore pressure. 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 that behave more fundamentally like clays (clay-like behaviour) and those that behave more fundamentally like sands (sand-like behaviour).

The guidelines for treatment of fine-grained soils herein are based on the knowledge and recommendations from recent studies by Boulanger and Idriss (2004, 2006), Bray et al. (2004), Bray and Sancio (2006), and Seed et al. (2003). Liquefaction susceptibility of fine-grained soils ($F_C > 30$ %, where F_C = percent passing through an 0.075 mm sieve) in the transition zone is simply characterized 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 (30 % < F_C < 50 %) Moderately 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 (F_C > 30 %), 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 6.4.

Note that further use of the so-called "Chinese Criteria", which have been traditionally used to determine liquefaction susceptibility of fine-grained soils, has been discouraged in the US practice (Boulanger and Idriss, 2004, 2006). Current understanding of the seismic behaviour of fines-containing sands is limited and therefore in cases where characterization of such soils is difficult, the soils should be either conservatively treated (as liquefiable) or detailed laboratory testing should be conducted.

6.3.2 Triggering of liquefaction

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 simplified procedure based on the empirical method originally proposed by Seed and Idriss (1971) as summarized in the NCEER guidelines by Youd et al. (2001). Other simplified methods based on energy considerations are also available though these methods are not in common usage.

In the simplified approach 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 adopted design earthquake, and the Cyclic Resistance Ratio (CRR), which represents the capacity of the soil to resist liquefaction.

The liquefaction triggering factor (F_L) is computed using Equation (6.1):

$$F_L = \frac{CRR}{CSR} \qquad (6.1)$$

Liquefaction will be triggered if $F_L \le 1.0$. The triggering factor F_L is determined (for liquefiable soils) throughout the depth of the deposit up to about 20 m depth. Methods for calculation of CSR and CRR are given in Youd et. al. (2001) for data from SPT, CPT, and Vs profiles.

In the course of the ground shaking during an earthquake, the soil is subjected to cyclic shear stresses, τ_{cyc}. For the purpose of liquefaction evaluation, these cyclic shear stresses are expressed in terms of the Cyclic Stress Ratio (CSR):

$$CSR = \frac{\tau_{cyc}}{\sigma'_{vo}} \qquad (6.2)$$

where

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):

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \qquad (6.3)$$

continued over page

where

a_{max} = peak horizontal acceleration at the ground surface (*Note that a_{max} is an estimate for the peak ground acceleration at the site for a hypothetical response without effects of excess pore pressure or liquefaction*)

g = acceleration of gravity (in same units as amax)

σ_{vo} = total vertical stress

σ'_{vo} = effective vertical stress

r_d = stress reduction factor accounting for soil flexibility

(Refer to Youd et. al. 2001 for an expression for calculating r_d as a function of depth. Alternatively, an improved expression for r_d that takes into account the effects of earthquake magnitude on the distribution of shear stresses throughout the depth of the deposit is provided by Idriss and Boulanger, 2006 and could be used instead.)

The peak ground acceleration a_{max} required in Equation 6.3 is obtained using one of the three methods described in Section 5.

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 hence in effect indicates the liquefaction resistance of the soil.

In the simplified procedure, CRR is evaluated by means of empirical charts for a magnitude M = 7.5 event and defines the stress ratio that causes liquefaction in 15 cycles. A correction factor, so-called Magnitude Scaling Factor, MSF, is then used to estimate CRR for different magnitudes or number of cycles, as explained in detail in Youd et. al. (2001).

For each liquefiable layer consisting of sands, non-plastic silts or fine-grained soils with $PI < 7$ (*or* $PI \leq 12$), it is recommended that CRR be estimated using the NCEER criteria (Youd et al., 2001) based on penetration resistance (SPT or CPT). The corresponding NCEER criteria based on the shear wave velocity (V_S) are considered appropriate for assessment of gravelly soils.

Procedures for gravelly soils based on large 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).

6.3.3 Liquefaction-induced ground deformation

The significant reduction in stiffness and strength of liquefied soils results in development of large shear strains in the ground during intense ground shaking. The peak cyclic shear strains can easily reach several percent in liquefying soils, and typically range from 2 % in dense sands to 4 % in loose sands, resulting in large cyclic lateral displacements of the liquefied layer. In the 1995 Kobe earthquake (Ishihara and Cubrinovski, 2005) the peak lateral displacements within liquefied fills reached about 0.5 m. These large cyclic lateral movements are important to consider because they may generate significant kinematic loads on buried structures and deep foundations.

Post-liquefaction behaviour is characterized by a very complex process involving dissipation of excess pore water pressure, sedimentation, solidification and re-consolidation of the liquefied soil resulting in settlement of the ground. 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.

For level ground sites, the severity of ground damage caused by liquefaction is proportional to the thickness of the liquefied layer. Ground displacements and liquefaction-induced damage generally increase with the thickness of the liquefied layer. Surface manifestations of liquefaction (ground rupture and sand boils) are also influenced by the presence and thickness of an overlying non-liquefied crust. Effects of liquefaction on the ground surface decrease with increasing thickness of the crust (Ishihara, 1985).

In sloping ground as well as for soils beneath an overlying structure or a foundation, the residual strength is commonly used in the assessment of the risk against flow slide and bearing failure. The residual strength of liquefied deposits in the field (Seed and Harder, 1990; Olson and Stark, 2002) can be much lower than the undrained shear strength of the soil because of potential redistribution of voids ratio and loosening of near-surface soils due to upward water flow during dissipation of excess pore pressures (Idriss and Boulanger, 2008). In cases where a low permeability layer acts as a barrier and prevents the upward flow, significant loosening of the liquefied layer may occur at this interface (eg water film effects and void re-distribution, Kokusho 2003).

If at any depth of the investigated deposit the liquefaction triggering factor is $F_L \leq 1.2$ 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 of the liquefied layer, 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. Some procedures for estimating liquefaction-induced ground deformation are discussed below. These methods apply to level-ground free field conditions, if not specifically stated otherwise.

Cyclic 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. The procedure is based on an empirical chart correlating the maximum cyclic shear strain in liquefied soil with the SPT blow count and CSR.

Liquefaction-induced settlements

Several simplified procedures 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 1 % for dense sands to 5 % for loose sands. Hence, thick deposits of poorly compacted sandy soils have especially high potential for large settlements. Additional settlements may be caused by shear stresses induced by overlying structures.

Lateral spreading displacements

—Empirical charts

There are several empirical methods available for evaluation of lateral spreading displacements (Youd et al., 2002; Tokimatsu and Asaka, 1998). Using field observations from case histories of lateral spreads caused by liquefaction in past earthquakes, Youd et al. (2002) developed equations for prediction of lateral ground displacements due to spreading. Factors such as site configuration, SPT resistance, earthquake magnitude and source distance are accounted for in this procedure.

—Newmark (rigid-block) method

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 either by integrating a given acceleration time history over the range of accelerations exceeding the yield level or by using approximate equations developed from analyses based on a suite of earthquake records.

Additional information is given by Jibson (2007) and Olson and Johnson (2008).

Performance levels

The magnitude of liquefaction-induced ground displacements is generally related to the liquefaction triggering factor F_L and to the overall thickness of the liquefied layer (Ishihara, 1985; Ishihara and Yoshimine, 1992). Based on general interpretation of these relations, Table 6.1 summarizes performance levels for liquefied soil deposits.

Table 6.1 **General performance levels for liquefied deposits**

Performance Level	Effects from excess pore water pressure and liquefaction	Characteristics of liquefaction and its consequences
L0	Insignificant	$F_L > 1.5$; No significant excess pore water pressures.
L1	Mild	$F_L > 1.2$; Limited excess pore water pressures without complete liquefaction; relatively small deformation of the ground with relatively small settlements (few tens of millimetres).
L2	Moderate	$F_L < 1.0$; Liquefaction occurs in layers of limited thickness (small proportion of the deposit); ground deformation results in differential settlements.
L3	High	$F_L < 1.0$; Liquefaction occurs in significant portion of the deposit resulting in differential movements, large settlements (few hundreds of millimetres) and lateral displacements.
L4	Severe	$F_L < 1.0$; Complete liquefaction develops in most of the deposit resulting in very large settlements (total and differential) and lateral displacements of the ground.
L5	Very severe	$F_L < 1.0$; Liquefaction resulting in lateral spreading (flow).

Table 6.1 should be used for general guidance only.

Other factors

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.

6.3.4 Residual strength of liquefied soils

Residual shear strength of liquefied soils can be used in the assessment of post-liquefaction stability, risk of bearing failures and liquefaction-induced lateral displacements. Experience from previous earthquakes and experimental studies on scaled-down models indicates that residual strength of liquefied soils can be much lower than the undrained strength of soils. Loosening of liquefied soils (void ratio redistribution and expansion) due to upward water flow is considered the primary reason for the low residual strength of liquefied soils. There are three sets of empirical relationships available for estimating the residual shear strength of liquefied soils proposed by Seed and Harder (1990), Olson and Stark (2002) and Idriss and Boulanger (2008).

The empirical correlation of Seed and Harder presents the residual strength S_r as a function of the equivalent sand SPT blow count, $(N1)_{60cs}$. Olson and Harder provide empirical correlations both based on normalized SPT blow count $(N1)_{60}$ 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 by the effective overburden stress. Idriss and Boulanger (2007) recommended relationships in terms of (S_r/σ'_{VO}) based on both $(N1)_{60cs}$ and $(q_{c1N})_{cs}$ for two separate conditions:

a) where void ratio redistribution effects are expected to be negligible

b) where void ratio redistribution effects could be significant.

It is important to note that all three sets of relationships are based on the same (similar) data, and they essentially differ in the interpretation of observations from a certain set of case histories. At a recent specialised session on residual strength (GEESDIV Conference, Sacramento, May 2008) there was general consensus that, for the time being, both normalized and non-normalized relationships should be used in parallel.

6.3.5 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, and lifelines). 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 these guidelines, 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 or not 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. 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. There are no widely accepted simplified procedures for prediction of structure-induced settlements. Prediction of differential settlements in liquefied soils is particularly difficult and therefore these settlements are simply assumed to be proportional to the total settlement (Martin et. al., 1999). Most differential settlements develop after strong shaking, and this should be accounted for when evaluating the capacity of the structure to accommodate such settlements.
- Large lateral movements from ground oscillation and spreading of liquefied soils in particular are damaging for pile foundations. Large passive soil pressures from a non-liquefied crust layer and effects from kinematic loads due to ground movement and inertial loads due to vibration of the superstructure, need to be considered in the assessment of pile foundations. Inertial and kinematic loads may or may not occur concurrently depending upon

characteristics of the ground motion, development of pore pressures 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. Care must be taken to account for uncertainties in loads and properties of liquefied soils when using these simplified methods.

- Lateral spreading displacements are 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 spread apart the foundation and adversely affect the vertical carrying system of the structure.
- 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 vertical carrying capacity of the foundation soil should be considered in the design.
- 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 ($F_L \geq 1.5$).
- 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 relative movements caused by spatial variability of ground displacements.

6.3.6 Advanced numerical procedures

Advanced numerical procedures may be appropriate for significant projects or may be justified where there are uncertainties about the likelihood of liquefaction or where the consequences of liquefaction may be significant.

Advanced numerical procedures for liquefaction assessment include total-stress and effective stress dynamic analyses. The latter is specifically tailored to analysis of soil deposits 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. The effective stress analysis addresses triggering of liquefaction, consequent ground deformation and effects of liquefaction on structures in an integrated manner and hence provides very realistic simulation of the complex ground response and soil-structure interaction in liquefying soils. 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 and post-shaking dissipation of excess pore pressure. It provides realistic simulation of earthquake loads throughout the depth of the foundation soil by considering responses of individual layers and cross interaction amongst them (base-isolation effects and progressive liquefaction due to upward flow of water).
- Spatial and temporal variation of ground deformation develops in accordance with changes in stiffness and earthquake loads. Thus, both inertial effects due to vibration of the structure and kinematic effects due to ground movements are concurrently considered while accounting for soil nonlinearity and influence of excess pore pressure on soil behaviour.

- Effects of soil-structure interaction are easily included in the analysis, in which sophisticated nonlinear models can be used both for soils and for structural members.
- The analysis allows assessment of the effectiveness of countermeasures against liquefaction (ground improvement and/or structural modification) including their effects on ground deformation and seismic performance of structures.

Key disadvantages of the effective stress analysis are that it requires/imposes:

- Selection of appropriate earthquake records to be used as input motion in the analysis by considering the seismic hazard for the site.
- High-quality and specific data on the in-situ conditions, physical properties and mechanical behaviour of soils (field investigations and specific laboratory tests on soil samples), particularly if the analysis is used for quantifying the seismic performance of important structures.
- High demands on the user with respect to knowledge and understanding of the phenomena considered and particular features of the adopted numerical procedures.

The total stress analysis is an alternative procedure for assessment of the seismic response of ground and SSI systems. This analysis, however, does not include directly effects of excess pore water pressures, and hence requires additional interpretation of non-linear soil behaviour and its simplification for modelling.

6.4 Ground failure of clay soils

Assessment of the cyclic strength ("cyclic softening") of "clay-like" soils should be made using quite different procedures, either by:

a) cyclic laboratory testing of an undisturbed sample, or,

b) measuring monotonic undrained shear strength using standard in situ or laboratory procedures and then applying an empirical reduction factor.

Boulanger and Idriss (2006, 2007) proposed a procedure for evaluation of cyclic softening in silts and clays ("clay-like fine-grained soils") during earthquakes. The procedure follows a format similar to that used in the semi-empirical liquefaction evaluation procedure and allows estimating the factor of safety against cyclic failure in clay-like fine-grained soils (using a failure criterion of 3 % peak shear strain). Boulanger and Idriss provide several approaches for estimating the cyclic resistance ratio (CRR) based on the undrained shear strength. Pending further research in this area, designers should make assessments of stability and deformation using the above recommendations for soil shear strength.

6.5 Mitigation of liquefaction and lateral spreading

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

> Ground deformation hazard arising from earthquake shaking (including liquefaction and lateral spreading) should be considered:
>
> 1. where failure or excessive deformation of the ground might contribute to loss of life within or loss of amenity of a building of Importance Level 2* or higher
> 2. where failure or excessive deformation of the ground is a risk to services to or access to buildings of Importance Level 3* or higher.
>
> *Refer to NZS 1170.0 for definition of building Importance Levels.

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

- soil remediation
- structural modification.

6.5.1 Soil remediation

Soil remediation methods effectively reduce ground deformation and effects of liquefaction either by preventing/limiting/slowing-down the development of excess pore water pressure and/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 effects:

- **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 and soil mixing 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 (1997), 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.

6.5.2 Structural modification

Potential effects of liquefaction can be taken into account and accommodated in the design of the structure in order to reduce differential settlements and lateral movement within the foundation. This is commonly achieved by using stiff raft, rigid foundation beams/walls or pile foundations with sufficient lateral capacity to resist both inertial loads due to vibration of the superstructure and kinematic loads due to ground movement. Reduction of differential settlements and lateral deformation can also be achieved through structural modification that provides control of the damage of structural systems and load distribution to the foundations.

6.5.3 Mitigation of clay soils

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

Options for mitigating clay soils are more limited then for granular soils and might include pre-loading with or without additional drainage. The main approach to mitigation will usually be by structural modification including the use of deep foundations, stiff raft foundations etc.

Identification, assessment and mitigation of earthquake induced landslides

SECTION 7

7.1 General

Guidelines for assessing landslide hazard have been prepared by GNS and the Australian Geomechanics Society. These are currently being assessed by a working group of the NZGS with a view to making a recommendation for their adoption for New Zealand practice.

7.2 Mitigation

Earthquake induced landslide is a major hazard for the built environment in New Zealand. Assessment of landslide hazard is outside the scope of these guidelines but, where identified:

> Earthquake induced landslide hazard arising from earth slopes (including natural slopes, cut batters, and fill batters) should be considered:
>
> 1. where failure or excessive deformation of the slope might contribute to loss of life within or loss of amenity of a building of Importance Level 2* or higher
> 2. where failure or excessive deformation of the slope or foundations is a risk to services to or access to buildings of Importance Level 3* or higher.
>
> *Refer to NZS 1170.0 for definition of building Importance Levels.

Options for mitigating earthquake induced landslide hazard include:

- re-grading of the slope
- reinforcement, or,
- use of retaining structures.

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SECTION
8

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