

**BUILDING
PERFORMANCE**

Earthquake geotechnical engineering practice

**Module 4. Earthquake resistant
foundation design**

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MINISTRY OF BUSINESS,
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Contents

Acknowledgements.....	iv
Preface	vi
1 Introduction	1
1.1 Objectives	2
1.2 Scope.....	3
1.3 Intended audience.....	3
1.4 Professional collaboration.....	4
1.5 Organisation of this Module	4
2 Site geotechnical model	5
2.1 Selection of representative design parameters	6
3 Foundation performance objectives for earthquake loading.....	8
3.1 Introduction.....	8
3.2 Regulatory environment.....	9
3.3 Serviceability limit state (SLS).....	10
3.4 Ultimate limit state (ULS)	12
3.5 Intermediate limit states	13
3.6 LRFD design procedure	14
3.7 LRFD design with earthquake loading	16
3.8 Soil structure interaction	18
3.9 Performance based design — alternative solutions	19
4 Site assessment and foundation selection	20
4.1 Introduction.....	20
4.2 Soil liquefaction.....	23
4.3 Lateral spreading.....	24
4.4 Other site stability issues	26
4.5 Natural raft.....	27
4.6 Ground improvement.....	28
4.7 Sand-like soils with $FL > 1$	29

4.8	Clay-like soils	30
4.9	Settlement.....	30
4.9.1	Free field settlement	30
4.9.2	Building induced settlements	31
4.9.3	Other settlement sources	32
4.10	Shallow foundation requirements	32
4.11	Deep foundation requirements.....	33
5	Shallow foundation design.....	34
5.1	Introduction.....	34
5.2	Types of shallow foundations.....	35
5.3	Pad footings and strip footings	36
5.3.1	Gravity design procedure	38
5.3.2	Seismic design procedure	40
5.4	Foundation beams.....	41
5.5	Design of mats and rafts	41
5.5.1	Gravity design procedures	42
5.5.2	Seismic design procedure	42
6	Deep foundation design	45
6.1	Types of deep foundations	47
6.1.1	Driven piles.....	47
6.1.2	Bored piles.....	47
6.1.3	Belled piles.....	48
6.1.4	CFA piles.....	48
6.1.5	Screw piles.....	48
6.2	Gravity design procedures.....	49
6.3	Seismic design procedures	52
6.3.1	Axial load capacity and settlement	52
6.3.2	Lateral load resistance and kinematic effects.....	53
7	References.....	59
	Appendix A. Worked example 1.....	62



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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 earthquake sequence and Canterbury Earthquakes Royal Commission recommendations. It is aimed at experienced geotechnical professionals, bringing up to date international research and practice.

This document should be read in conjunction with the other modules published to date in the series:

- › Module 1: Overview of the guidelines
- › Module 2: Geotechnical investigations for earthquake engineering
- › Module 3: Identification, assessment and mitigation of liquefaction hazards
- › 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

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/.

Robust foundation design that takes into account both the on-site ground conditions and the structure being supported is critical to the performance of buildings in earthquakes.

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

Eleni Gkeli

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1 Introduction



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

The high seismic hazard in New Zealand and profound relevance of geotechnical earthquake engineering were demonstrated by 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.

Many buildings in the Christchurch CBD were on shallow foundations and some of these were badly affected by liquefaction including excessive settlements and tilting. Shallow foundations bearing onto the shallow, dense gravels present in some parts of the city showed mixed performance because of the high variability of these deposits and high seismic loads. Stiff raft foundations bearing onto these shallow gravels appear to have performed relatively well.

Shallow foundations on the few sites where ground improvement was carried out prior to construction showed mixed performance. While bearing failures were prevented by the ground improvement, in accordance with the design philosophy, there were excessive differential settlements and tilting in some cases.

Buildings on deep pile foundations generally fared better where the piles penetrated to competent soils at depth of adequate thickness and not underlain by liquefied soil. However, a significant number of piled buildings suffered differential settlement where the bearing layer has been too thin or underlain with liquefiable layers or where there has been a loss of side resistance with liquefaction and consequent load transfer to an end bearing mechanism.

Hybrid foundations, where part of the building was supported on deep piles and part on shallow foundations, performed poorly because of complex dynamic interactions resulting in differential movements between the two systems.

Other significant foundation damage included the failure of ground floor and basement slabs in uplift under the very high pore-water pressures associated with soil liquefaction and ground shaking.

While it is acknowledged that the level of shaking far exceeded even the ultimate limit state (ULS) design level of shaking, a better understanding of site geotechnical performance during earthquakes is desirable in future in urban centres in New Zealand. This should lead to better foundation performance in order to achieve greater resilience and reparability for the building stock.

1.1 Objectives

This document is intended to provide guidance for earthquake resistant design of foundations for buildings in New Zealand. It is not intended to provide a fully comprehensive treatment of all aspects of foundation design and construction in all situations and ground conditions for which well-known published handbooks should be consulted, for example:

- › Salgado (2007)
- › Tomlinson and Boorman (2001)
- › Bowles (1997)
- › FHWA (Bored Piles, Brown et al 2010)
- › FHWA (Driven Piles, Hannigan et.al. 2006)

Instead, the intention is to provide supplementary guidance on earthquake design aspects for foundations that are not well covered in these handbooks or elsewhere.

The main objective is to identify the key issues that need to be addressed by designers and provide a rational framework for resolving these issues. Simplified approaches for everyday design cases are provided and worked examples used to provide additional detail (located in the Appendices).

Complex and unusual situations are not covered. In such cases, special studies are considered more appropriate.

This document promotes consistency of approach to everyday engineering practice in New Zealand and, thus, improve earthquake-geotechnical aspects of the performance of the built environment.

This is not a book of rules — users of the document are assumed to be qualified, practicing geotechnical engineers with sufficient experience to apply professional judgement in interpreting and applying the recommendations contained herein.

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.

1.2 Scope

This module of the *Guidelines* is concerned with the geotechnical design of building foundations to resist earthquake loading.

The geotechnical performance of the site, including issues of soil liquefaction, cyclic softening, lateral spreading and stability during shaking may have a significant effect on the performance of building foundations (or even dominate performance in the case of soil liquefaction and lateral spreading) and must be carefully considered prior to selecting a suitable foundation type or commencing foundation design.

Module 2 of the *Guidelines* should be consulted for much additional information on planning and executing a suitable site investigation and Module 3 provides detailed guidance on assessing the liquefaction hazard.

The design approach used herein follows the New Zealand Building Code document B1/VM1, ie a limit state, load and resistance factor (LRFD) design process as detailed in NZS 1170.0:2002 with earthquake provisions from NZS 1170.5:2004. It is intended that, when properly used in conjunction with these standards 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.

1.3 Intended audience

The *Guidelines* have been prepared, generally, for the use of qualified, practising geotechnical engineers with a sound background in soil mechanics, geotechnical engineering, and earthquake engineering. This module is concerned with building foundations and so will also be of interest to practising structural engineers although it is intended that they should work in close collaboration with geotechnical engineers.

B1/VM1 is not the only means of establishing compliance with the New Zealand Building Code. Alternative methods of achieving compliance are possible, as explained on the MBIE Building Performance website (refer www.building.govt.nz). A general discussion of alternative, performance based approaches for earthquake resistant foundation design is given at the end of Section 3, but is mostly beyond the scope of this module of the *Guidelines*.

Other documents may provide more specific guidelines or rules for specialist structures and these may 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, New Zealand Transport Agency Bridge Design Manual, and Transpower New Zealand Transmission Structure Foundation Manual.

Where significant discrepancies are identified 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.

1.4 Professional collaboration

Geotechnical considerations are crucial to successful designs for any part of the built environment, especially in New Zealand’s high earthquake hazard environment. Successful outcomes require close collaboration among the key professionals (geotechnical engineers, engineering geologists, and structural engineers) to properly consider the site geology, earthquake hazards, site response, soil response, foundation behaviour, structural interactions, and soil-structure system response.

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

A full consideration of the site response and soil response to shaking together with a sound understanding of the structural response including soil-structure interaction, is essential to making appropriate selections of suitable foundation types or ground treatments. This requires close collaboration among the geotechnical and structural engineers.

Geotechnical and structural engineers may have different performance objectives in mind, or simply not clearly understand what each discipline contributes or is able to contribute to the design process, or what actually matters for design (Oliver et al, 2013). Good design solutions require that the geotechnical and structural engineers sit down together to share each professional’s perspective of the project and coming to a shared understanding of all of the issues and interactions required for a successful outcome. The result would ideally be a joint report outlining the expected performance of the site, ground, foundations, and structure including their critical interactions and design assumptions.

1.5 Organisation of this Module

A thorough understanding of the site soil conditions and earthquake hazard is a key requirement and starting point for foundation selection and design.

- › Section 2 of this module provides guidance on the development of a geotechnical model suitable for foundation selection and design, including the selection of representative soil parameters needed for design.
- › Module 2 of the *Guidelines* provides detailed guidance on planning and implementing appropriate geotechnical site investigations for earthquake geotechnical engineering purposes including earthquake resistant foundation design.
- › Module 3 of the *Guidelines* provides detailed guidance on the identification and assessment of liquefaction hazard at a site.
- › Section 3 of this module discusses foundation performance during earthquakes and describes the foundation design philosophy used in New Zealand practice.
- › Section 4 describes in detail a strategy for evaluating a site and selecting the most appropriate foundation system in the context of New Zealand’s high earthquake hazard environment. Selecting the most suitable foundation system for a building is key to achieving good performance and economy.
- › Section 5 outlines a simplified approach for practical design of shallow foundations with earthquake loading.
- › Section 6 does the same for deep foundations. Worked examples for both shallow and deep foundations are appended.

2 Site geotechnical model

A site geotechnical model is a simplified representation of the site geotechnical conditions including stratigraphy, ground water, and geotechnical parameters relevant to site performance and foundation design.

The site geotechnical model is usually presented as one or more graphical cross-sections, but for simple sites with uniform stratigraphy, a tabular format may suffice. The level of detail in the model (eg number of layers) should be optimised to facilitate practical analysis of site performance and foundation design.

An appropriately detailed geotechnical investigation of each building site leading to development of a site geotechnical model is a key requirement for achieving good foundation performance. The objective is not simply to describe the soil and rock encountered, but to gain a good understanding of the geology and geomorphology of the site and thus the likely presence of geotechnical hazards such as soil liquefaction. The extent of the investigations should be sufficient to give designers confidence in predicting performance of the site and the building foundations.

An individual site cannot be considered in isolation, but only in the context of adjacent sites and the geomorphology of the area. Context is especially important when considering the risk of soil liquefaction and damaging lateral ground movements during earthquakes and other geological hazards.

The necessary depth of the sub-surface exploration requires careful judgement by the geotechnical engineer or engineering geologist. Frequently, explorations are terminated at too shallow a depth, especially where deep foundations may need to be used. The depth of exploration should extend through all soil strata capable of affecting the performance of the site and the building foundations, and then continued for a sufficient additional depth to give confidence that all potential problem soils have been identified.

Where deep pile foundations are being considered, the exploration should continue well into the proposed bearing layer and at least five diameters below the intended founding depth. For pile groups, the additional depth may need to be equal to the width of the group or greater.

The limitations of the subsurface information and the uncertainties inherent within the model should be recognised and alternative interpretations of the data considered when preparing the site geotechnical model.

Detailed guidance on planning, implementing, and reporting on suitable site investigations is given in Module 2 of the *Guidelines*.

2.1 Selection of representative design parameters

The site geotechnical model should include representative soil and rock parameters that will be needed for analysis of site performance and foundation design.

Three approaches are possible:

- a Direct measurement of properties in the laboratory from samples collected from the site
- b Correlation of properties from in situ test data (eg CPT, SPT, etc.)
- c Direct correlation of foundation resistance and settlement from in situ test data

Each of these approaches has advantages and disadvantages. Direct measurement in the laboratory of key parameters mostly requires un-disturbed specimens that may be difficult to obtain in practice (eg clean sands). Laboratory test procedures may not accurately represent the field stress, boundary conditions, or drainage conditions. Usually, only a small number of specimens are tested and these may not have statistical significance or be truly representative of the whole site.

In situ test methods avoid the problem of recovering undisturbed samples and are usually able to be carried out economically in greater numbers than laboratory tests. However, correlations with the required soil parameters include uncertainties because the in situ test result (eg q_c , N) may be influenced by multiple parameters of the soil or rock simultaneously that are difficult to separate (eg the penetration resistance of the CPT is not only influenced by the shear strength of the soil but also by the soil gradation and stiffness). Site specific correlations with laboratory test data may be very beneficial in improving interpretation of the data and accuracy of the results.

Direct correlation of foundation resistance and settlement with in situ test data avoids the above mentioned difficulties of determining representative soil and rock parameters. At the simplest level, the in situ penetration test may be considered as a small scale model of the prototype foundation (eg CPT, SPT), with the penetration resistance of the in situ device considered analogous to foundation bearing resistance. In practice, empirical factors must be used to adjust for the differences in scale, method of installation, rate of loading, and displacement. The reliability of direct correlation procedures is improved if site specific correlations are developed based on full-scale load tests of prototype foundations.

A summary of field and laboratory methods for determining soil and rock characteristics used for foundation design is given in Table 2.1 [adapted from FHWA 2010]. Much detailed information on the evaluation of soil and rock properties for geotechnical design applications is provided in FHWA [2002].

The selection of representative design parameters for each unit within the site geotechnical model requires careful consideration and judgement by the geotechnical engineer. Whenever more than one data point is available for a unit, a judgement must be made whether to adopt an 'average', 'conservative', lower bound', or 'worst case' value. The decision process should consider a range of issues that will be different for each case including:

- > Amount and variability of data available
- > The design application
- > Extent of physical 'averaging'
- > Criticality of the application

Laboratory data will typically be sparse for each unit and therefore of low statistical significance. More confidence will be obtained by correlating laboratory data to adjacent in situ test data (eg CPT) and using the resulting enhanced correlation and available data to better characterise the unit.

The CPT test typically produces a large number of data points at close (vertical) spacing. It would usually be considered over-conservative to design for a lower-bound value that might represent only a 5 mm thick layer of soil. On the other hand, SPT data points are typically spaced at 1 m or 1.5 m depths and each reading averages a 300 mm thickness of soil. The intrinsic variability and scatter of SPT readings also needs to be considered and excessive reliance should not be placed on any single reading.

Strength parameters used for calculating capacity of critical load bearing foundations are usually chosen to be 'moderately conservative'. Soil stiffness parameters used for settlement calculations are difficult to measure and highly non-linear, and should generally be given as a range, better reflecting the uncertainty in these parameters.

The extent of ‘physical’ averaging of soil parameters for each situation should be considered. For example, the side resistance of a large bored pile will effectively ‘average’ the soil shear strength over its surface, with local variations in strength being of little significance to the total capacity. By comparison, the bearing capacity of a small footing may be significantly reduced by even a small pocket of weak soil within the influence zone of the footing.

Typically, where good numbers of data points are available, the design of a large pile foundation would

be based on using the lower quartile of CPT or SPT data from a nearby sounding. Where few soundings are available to demonstrate the spatial variability across the site, then the worst case sounding overall would be adopted for design.

For small shallow footings, the worst case data might be used unless grade beams are being used to bridge over weak spots and effectively ‘average’ the local soil properties (or, isolated weak spots identified by close spaced in situ testing).

Table 2.1: Summary of field and laboratory methods for soil and rock characteristics used for foundation design [adapted from FHWA 2010]

DESIGN PARAMETER OR INFORMATION NEEDED	SUBSURFACE MATERIAL		
	COHESIONLESS SOILS	COHESIVE SOILS	ROCK
Stratigraphy	Drilling-sampling; SPT, CPT, DMT; geophysics	Drilling-sampling; SPT, CPT, DMT; geophysics	Drilling-sampling; rock core logging
Groundwater	Well/piezometer	Well/piezometer	Well/piezometer
INDEX PROPERTIES			
Gradation	Sieve analysis	Sieve analysis; hydrometer analysis	–
Atterberg Limits	–	Liquid limit and plastic limit tests	–
Classification	USCS Group Index	USCS Group Index	Rock type
Moisture content	Wet and oven dried weights	Wet and oven dried weights	–
Unit Weight, γ	SPT, DMT, CPT	Weight-volume measurements on USS	Weight-volume measurements on rock core
RQD and GSI	–	–	Rock core logging and photos
Slake Durability	–	–	Lab slake durability test
ENGINEERING PROPERTIES			
Effective stress friction angle, ϕ'	SPT, CPT, DMT	CD or CUpp triaxial on USS	Correlate to GSI
Undrained shear strength, S_u	–	CPT, VST, CU triaxial on USS	–
Preconsolidation stress, σ'_p	SPT, CPT, DMT	Oedometer test on USS; DMT, CPT	–
Soil modulus, E_s	PMT, DMT, SPT, CPT; correlate with index properties	Triaxial test on USS; PMT, DMT; correlate with index properties	–
Subgrade reaction modulus, k_s	SPT, CPT, PLT	SPT, CPT, PLT	–
Uniaxial compressive strength, q_u	–	–	Lab compression test on rock core
Modulus of intact rock, E_r	–	–	Lab compression test on rock core
Rock mass modulus, E_m	–	–	Correlate to GSI and either q_u or E_r ; PMT, PLT

Key:

CD consolidated drained triaxial compression test
 CU consolidated undrained triaxial compression test (CUpp — with pore pressures)

CPT cone penetrometer test (also CPTu — with pore pressure measurement)
 SPT standard penetration test
 DMT dilatometer test
 PLT plate load test

PMT pressuremeter test
 VST vane shear test
 USS undisturbed soil sample
 GSI geological strength index
 USCS unified soil classification system

3 Foundation performance objectives for earthquake loading



3.1 Introduction

This section considers the key performance requirements for foundations before, during, and after earthquake shaking in the context of the New Zealand building regulatory environment.

Foundation performance is critical to satisfactory performance of buildings during earthquakes, including site stability and control of settlements and distortion of buildings and other structures.

Earthquake actions differ from other structural actions in several important respects:

- a Earthquake actions are caused by ground motions with characteristics that vary greatly from one earthquake to another and that are impossible to accurately predict. Instead, ground accelerations based on probabilistic analysis are used for design. There is always a residual risk that the earthquake actions will be greater than the code specified design actions and buildings (including foundations) should be made sufficiently robust to accommodate such 'overloading' in a progressive manner, so as to avoid sudden collapse (a requirement of NZS 1170.5).
- b The ground motions must be transmitted into the building by the foundations. Compliance and yielding of the foundations may reduce the dynamic response of the building by lengthening the natural period and increasing damping (so called soil-structure-interaction effects). However, the resulting relative displacements may damage the foundations and building service connections, and the foundation rotations may increase the building displacements and place additional demands on the superstructure (see Section 3.7 and 3.8 for further discussion of *soil-structure-interaction effects* and *performance based design* in general).

- c Earthquake shaking may reduce the strength and stiffness of the founding soils and the load bearing capacity of the foundations. Certain soils may lose almost all of their strength and stiffness (ie liquefy). The resulting degradation in foundation performance may jeopardize the stability and integrity of the building structure and must be carefully considered (ie within the site assessment and foundation selection process, covered in detail in Section 4).
- d Earthquake shaking causes shear deformations within the ground below the surface that induce bending strains in buried foundation elements, especially deep piles, including both time dependant and permanent strains (ie kinematic loads in deep foundations). These strains are in addition to those caused by inertial loads from the building and may damage the piles such that they can no longer safely carry the weight of the building. Kinematic effects are most pronounced where deep piles pass through liquefied soil layers. (Kinematic effects on piles are discussed in detail in Section 6.)
- e The inertial response of the building induces dynamic, cyclic loading of the foundations that increase settlements by a process of 'ratcheting'.
- f Earthquake actions include lateral loads from building inertia applied at the foundation level (and sometimes moment loads), simultaneous with vertical load. The lateral and moment loads may reduce the vertical bearing capacity of shallow foundations and cause structural damage to deep foundations. These effects are considered in more detail in Sections 5 and 6. The overturning forces may result in a net uplift load being applied to individual foundation elements. Deep foundations may be used to resist these uplift loads with details given in Section 6. The possibility of not resisting these uplift forces, and thus permitting rocking of the building, is included in the discussion of *soil structure interaction and performance based design* in Sections 3.8 and 3.9.

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

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

3.2 Regulatory environment

All building work in New Zealand must comply with the New Zealand Building Code. For most building work, compliance with the Building Code is established by conformance to *Verification Method B1/VM1* for structural design published by the Ministry of Building, Innovation and Employment.

However, New Zealand's Building Code is performance based and alternative methods of demonstrating compliance are possible (termed *alternative solutions* in the Building Regulations) and these are discussed in Section 3.9.

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

Earthquake structural design actions for New Zealand are specified in detail by NZS 1170.5:2004 according to location, subsoil conditions, building period, and earthquake return period. NZS 1170.5:2004 also includes more specific requirements for methods of structural analysis for earthquake actions. (Note: these *structural* design actions are not intended to be used for *geotechnical* assessment or design, see Module 1 for more information.)

NZS 1170.5:2004 includes a requirement that ultimate limit state deformations be limited so that the structural system continues to safely perform its load bearing function, contact with neighbouring buildings is avoided, parts continue to be supported, and non-structural systems necessary for emergency evacuation of the building continue to function. Foundation movements will contribute to the building deformations and need to be considered.

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

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

3.3 Serviceability limit state (SLS)

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

SLS design actions and combinations of actions are considered likely (ie with probability of 86 percent) to occur during a 50-year lifetime of the building. Two serviceability limit states are considered by NZS 1170.0:2002: SLS1 and SLS2. SLS1 is a requirement for all buildings of Importance Level IL2 or above. SLS2 is a requirement only for buildings of Importance Level 4 (ie structures with special post-disaster functions). The annual probability of exceedance for each is given as:

- a SLS1 1/25 (except for buildings of low importance, IL 1, which have no SLS requirement)
- b SLS2 1/500 (for buildings with normal, 50 year design life, see NZS 1170.0:2002 for other cases).

The specified service criteria for earthquake shaking for both SLS1 and SLS2 are described in NZS 1170.5:2004 as follows:

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

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

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

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

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

For difficult sites or for buildings containing sensitive equipment, it may be acceptable to include special details so that re-leveling may be quickly carried out after an earthquake. The building would still need to meet all other SLS performance requirements.

Comment

The selection of tolerable settlements for the foundations of buildings at the serviceability limit state is a complex topic beyond the intended scope of these *Guidelines*. The entire building may settle vertically or rotate as a plane rigid body by significant amounts without causing any structural or architectural distress (although there are limits beyond which aesthetic and public confidence considerations would be of concern). It is *differential* settlements, below the tilt line (see Figure 3.1), inducing structural deformation of the building, that will be the cause of distress to the building fabric [Bowles, 1997].

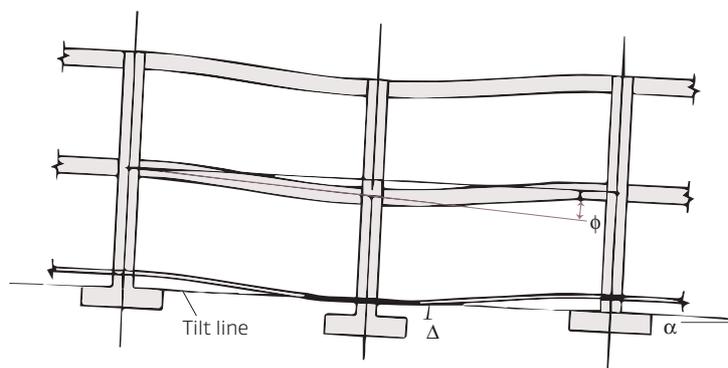
Traditionally, the assumption has been made [Terzaghi & Peck, 1967] 'that most ordinary structures such as office buildings, apartment houses, or factories, can withstand a differential settlement between columns of three quarters of an inch' (20 mm) and that such a differential settlement would not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

For certain buildings and for certain uses, 20 mm differential settlement might cause significant loss of amenity, while for other cases much larger movements would be tolerable. The linkage between loss of amenity, as intended by the NZ Building Code, *serviceability*, as defined by NZS 1170.0:2002, and *allowable bearing pressure*, as used in common practice to design shallow footings, needs careful consideration.

The selection of settlement limit criteria for building foundations at the SLS should not be a decision of the geotechnical engineer in isolation, but should be agreed and documented with the structural engineer, architect, and owner, as appropriate depending on the structural form, building fabric, and intended use.

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

Figure 3.1: Frame building subject to differential settlement (Δ) and tilt (α). Building damage is mostly related to angular distortion (ϕ).



3.4 Ultimate limit state (ULS)

ULS design actions and combinations of actions are considered much less likely to occur during the lifetime of the building but are required to be resisted with a very low risk of structural collapse or failure of parts relevant to life safety. For buildings of normal importance (Importance Level 2) with a normal (50 year) design life, earthquake shaking with a 500 year return period is considered. Return periods for buildings of other importance levels and design life duration are defined in NZS 1170.0:2002.

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

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

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

On the other hand, resistance to **lateral** seismic loading is not necessarily critical to the safe performance of buildings in all cases. Lateral deformations are 'self-limiting' in the sense that lateral acceleration pulses are of short duration and act in both (all) directions. (But caution is required for buildings located on slopes or which are retaining soil because these will be subject to a ratcheting effect and possibly large, permanent offsets.)

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

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

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

Comment

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

Under NZS 1170.5:2004, *deformations* of the foundations under the ULS loads (including over-strength loads) should be considered as well as *ultimate resistance*. While there is no requirement to achieve the same low level of deformation as for the SLS case, the foundation deformations should be accounted for in the structural design, and should not be so great that they add significantly to the ductility demand of the structure or prevent the intended structural response from developing.

Comment

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

At this time, there is limited guidance available to be able to predict foundation settlements with earthquake loading at the ULS, especially where liquefaction or cyclic softening of the founding

soils is expected to occur (refer to Bray & Dashti 2014 and Bray & Macedo 2017 for more information and for a simplified procedure for estimating liquefaction-induced settlement). For important projects, advanced numerical procedures show promise and may be justified but for most everyday situations a pragmatic, conservative approach of limiting plastic deformations of foundations under the calculated loads and avoiding situations where liquefaction and cyclic softening effects may be significant is recommended.

3.5 Intermediate limit states

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

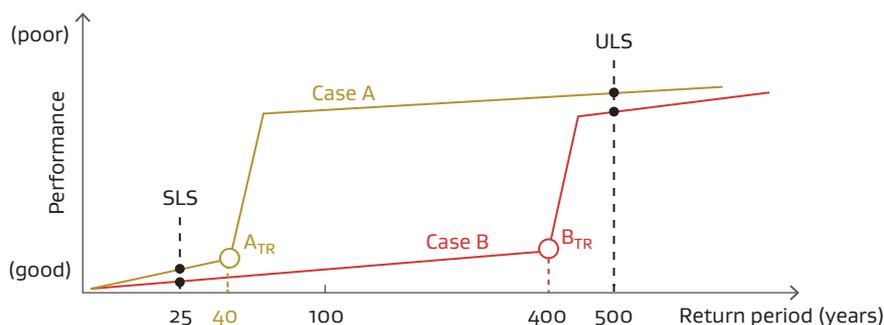
However, the behaviour of soils and geotechnical systems under earthquake shaking may be highly non-linear and even exhibit a pronounced 'step change' in performance with increasing intensity of shaking. Typical examples include sites affected by liquefaction of susceptible soils or slope instability.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

Consider two hypothetical cases as depicted in Figure 3.2. Case A shows a system for which large ground response (eg liquefaction, lateral spreading, or slope instability) is triggered for a ground motion intensity corresponding to 40-year return period, whereas in Case B the triggering occurs for a 400-year return period motion.

Importantly, both cases show small response and acceptable performance for SLS level of shaking (25-year return period), and large, damaging, but tolerable response for ULS level of shaking (500-year return period). Hence, if one evaluates these two systems by discrete calculations for SLS and ULS (shown with solid symbols), then the conclusion would be that both systems will exhibit similar and tolerable seismic performance.

However, Figure 3.2 clearly shows a significant difference in performance between Case A and Case B: For Case A, triggering is associated with 40-year return period, which implies for a 50-year lifespan of a structure, that there is 71 percent probability that triggering will occur during the life of the structure. The respective probability for Case B, with 400-year return period motion for triggering, is only 12 percent.

Figure 3.2: Step change in performance between SLS and ULS



Where triggering of such degraded performance is likely at a modest, intermediate return period (eg less than a 100 year return period for a building of normal importance) the resulting level of damage may be excessive and inappropriate

for such a high likelihood of occurrence. Tolerable impact limits for such intermediate cases will depend on the return period for triggering and the level of resulting damage to the facility.

Comment

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

3.6 LRFD design procedure

Under verification method B1/VM1 the design procedure for buildings in New Zealand given in AS/NZS 1170.0-2002 is a strength based, *Load and Resistance Factor Design* procedure (LRFD).

In this procedure, the uncertainty and variability in the loads and design actions on foundation elements are considered separately from the uncertainty and variability in the resistance of the foundation elements, according to the design inequality:

$$\phi R \geq \sum \psi_i E_i \quad (3-1)$$

in which:

ψ_i = combination factor action effect, i
(eg for the ULS under NZS 1170.0:2002, $\psi_i = 1.2$ for permanent actions, 1.5 for imposed actions, 1.0 for earthquake actions)

E_i = nominal value of action effect ('load') i

ϕ = resistance factor for foundation element

R = nominal value of resistance for foundation element

Comment

In this context *nominal* means the value calculated using an accepted design methodology (eg the *nominal* resistance of a footing at the ULS would be the calculated *ultimate bearing capacity*, and the resistance factor would be the appropriate value of the *geotechnical strength reduction factor*.)

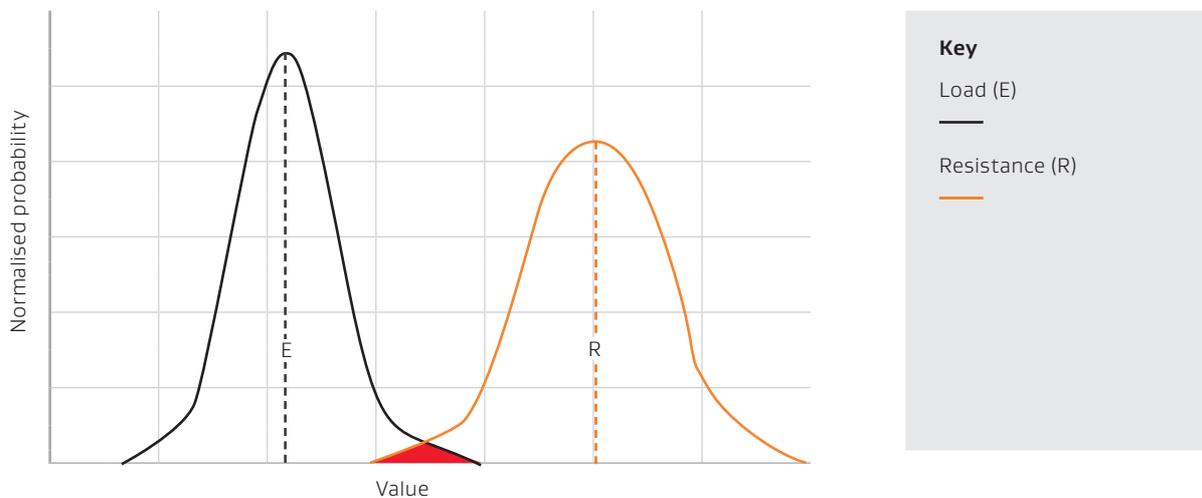
The above inequality must be satisfied for the ULS for various combinations of loading which are defined in NZS 1170.0-2002.

Figure 3.3 illustrates how the design inequality of Equation 3-1 operates: Both the nominal value of the action effect (ie 'load') (E) and the nominal resistance (R) have a certain variability illustrated as bell-shaped distribution curves.

Generally, the variability in foundation resistance will be much greater than for the load, and so the 'bell' is shown to be 'wider' and 'flatter'. The probability of failure (ie load being greater than resistance) may be computed as the area of overlap of the two bell curves, shown shaded. To achieve the required low probability of failure for the ULS case it is necessary to ensure that the nominal resistance is greater than the nominal loads ($R > E$) by applying a load factor greater than 1 and a resistance factor less than 1. The appropriate value for each of these factors depends on the uncertainty for the parameter (ie the width of the respective bell curve) and the required level of reliability.

For the *ultimate limit state* (ULS) in AS/NZS 1170.0:2002, load and resistance factors are chosen to provide a high level of reliability against rupture or excessive deformation of the foundation element. For the ULS the key issue for the foundations is stability and the nominal resistance is normally taken as the ultimate or limiting value of resistance as determined by calculation or load test.

Figure 3.3: Idealised probability distributions of load and resistance for a foundation element
 [Source: FHWA, 1999]



Load factors range typically from 1.2 for building self-weight or *permanent actions* (which can be calculated with reasonable reliability) to 1.5 for 'live' loads or imposed actions (which are much less certain). For the earthquake load case, earthquake action effects have a load factor of 1.0 applied in NZS 1170.0:2002 because the probabilistic assessment of earthquake hazard already accounts for the high level of uncertainty in earthquake actions.

The definition of resistance for foundation elements can be problematic when applying LRFD methodology because it is difficult to separate an *ultimate limit state* value for resistance from considerations of deformation. Foundation elements load-tested to 'failure' or *ultimate capacity* often do not reach a well-defined peak capacity and, instead, an *interpreted capacity* is reported, generally based on a limiting displacement (eg 25 mm or 5 percent of pile diameter, 5 percent of footing width). Semi-empirical calculation procedures for deep foundation resistance are linked to load test results and so they also include a deformation limit, intrinsically. On the other hand, the calculated bearing capacity for shallow foundations (often termed *ultimate capacity* in text books) is typically calculated using *limiting equilibrium* procedures that require large displacements to become fully mobilised (typically 5 to 10 percent of foundation width).

The resistance factor, Φ in Equation 3-1 is applied to account for the variability and uncertainty in actual resistance of individual foundation elements about the nominal (calculated) value to achieve the required level of reliability.

The resistance factor is selected taking account of:

- › Reliability target for performance criteria.
- › Reliability of the method of calculation (or load test).
- › Uncertainty and variability of the geotechnical site conditions.
- › Uncertainty of the geotechnical parameters critical to foundation performance.
- › Degree of redundancy (eg single piles would have a lower resistance factor than a pile group).
- › Reliability of construction and installation processes of the foundation.

For ULS foundation design, the resistance factor is usually called the *geotechnical strength reduction factor*, Φ_g . Guidance on selecting appropriate values for Φ_g is provided in various detailed design manuals (eg AS 2159-2009 *Piling-Design and Installation*). Recommended values for shallow foundations are given in Section 5 and for deep foundations in Section 6.

3.7 LRFD design with earthquake loading

LRFD design has been adapted to geotechnical engineering from structural engineering practice where the main concern at the ULS is to avoid structural ‘failure’ or ‘rupture’ of critical structural elements. Deformation of structural elements at the ULS prior to rupture is seldom a significant consideration, or may even be considered desirable as a visual warning of an overload condition under gravity loading or as a means of dissipating energy with earthquake loading.

For the geotechnical behaviour of foundation elements, though, deformation (ie settlement) may become very large and unpredictable as loads approach ultimate or limiting values, especially for shallow footings. Resistance factors are chosen to ensure that there is a low risk of foundation loads approaching ultimate values after accounting for the high variability and uncertainty of soil parameters and calculated foundation response in general.

In normal practice, considering gravity load cases, settlements are checked only at the SLS with the assumption being made that if a footing meets the SLS settlement criteria and the ULS strength criteria then the settlement at the ULS would not be so excessive as to endanger the stability of the structure.

LRFD design under NZS 1170.0:2002 is intrinsically a strength based, quasi-static procedure whereby factored, static loads are compared with factored static resistance capacities. Self-weight, imposed (live) loads, snow loads, etc., are all essentially static loads and even wind loading may reasonably be considered as quasi-static in nature. Earthquake loading on the other hand is truly dynamic in nature and highly dependent on the dynamic response behaviour of the building as well as the signature of the particular earthquake and site response.

For LRFD design with earthquake loading, the factored design loads considered at the foundations are still treated essentially as quasi-static but the factored resistance capacities must be adjusted to account for changes in soil response

caused by the shaking and from the dynamic nature of the applied loading. The details of these adjustments are given in Section 5 for shallow foundations and in Section 6 for deep foundations.

The key assumption underlying the LRFD design procedure is still implicit, however, ie provided the factored, calculated foundation resistance is greater than the factored loads then the settlement under the ULS earthquake will not be excessive or impair the required structural performance. The appropriateness of this assumption will be affected by at least two counter-acting dynamic effects:

- a the short duration of each cycle of dynamic loading limiting the mobilisation of settlement, but
- b the repeated cycles of dynamic loading causing a ‘ratcheting effect’ that tends to increase settlement.

Given the uncertainty in predicting either of these effects it is advisable to limit the possibility of significant plastic deformations by applying resistance factors.

Satisfactory foundation performance using the LRFD design procedure also assumes that the foundations will not be significantly affected by soil liquefaction, cyclic softening, lateral spreading or other site hazards. A careful assessment of each site and the site soil response to earthquake shaking is critical, with detailed guidance provided in Section 4.

The successful application of LRF design for earthquake loading requires the geotechnical engineer and the structural engineer to have a common understanding of the source and appropriateness of the calculated design actions on the foundations so that these can be clearly communicated. Three methods of seismic analysis of buildings are considered by NZS 1170.5:2004:

- 1 **Equivalent static analysis method** — where a set of ‘equivalent static loads’, derived from the site hazard spectra and a simple single mode assumption of the building dynamic response, are applied simultaneously at each level in the building allowing ‘equivalent static actions’ on the foundations to be calculated from static equilibrium. The intention is that a building designed to resist these equivalent static loads, and meet other important detailing requirements, would perform safely under a ULS level earthquake. The use of the ‘equivalent static’ load set to size the foundations using LRF procedure is implied.
- 2 **Modal response spectrum method** — where an envelope of actions is calculated for each structural element by carrying out a modal analysis and combining the actions from the multiple response modes of the building. An envelope of actions applied to the foundations can be calculated, however, such an envelope may not result in a consistent set of actions at the foundations (eg the maximum vertical action may not occur simultaneously with the maximum horizontal action.) NZS 1170.5:2004 supports the use of a simplified approach for cases of capacity design of reinforced concrete and structural steel frames whereby the ‘equivalent static forces’ of Method 1 above are replaced by the first mode actions of the building, resulting in a consistent set of actions that would be used for foundation design. The use of LRF procedure for sizing the foundations is, again, implied.

- 3 **Numerical integration time history method** — where a 2D or 3D finite element or finite difference model of the structure is subjected to a suite of selected earthquake records scaled to match the ULS design level earthquake hazard for the site. The result would be an envelope of actions for each structural element and including the foundation elements. The worst-case actions should be chosen for LRF design of foundations (but selecting a consistent set of simultaneous vertical, horizontal, and moment actions).

Comment

In International codes (eg ASCE 7-16), where 10 or more time-history records are modelled it is permissible to carry out probabilistic assessment and eliminate extreme results and choose ‘reasonably conservative’ values. However, NZS 1170.5 requires the maximum response to be used regardless of the number of records modelled with a minimum of three records required.

Alternatively, a performance based approach may be adopted where full numerical integration time history modelling of the foundations and site soils are carried out in a rigorous way. A more detailed discussion of performance based design of foundations is provided in Section 3.9.

In all cases, the performance objective remains the same, to ensure that the foundations are able to safely carry the weight of the building before, during, and after the earthquake without excessive deformation that would endanger the safety and stability of the building.

3.8 Soil structure interaction

The underlying idea behind soil structure interaction modelling is a desire to treat the foundation and supported structure as a single system.

There are several ways of treating soil structure interaction (SSI), the simplest being to consider that the soil supporting the foundation remains elastic and this is the usual context of SSI usage.

The inclusion of more realistic, non-linear interaction between the soil and foundation elements requires more sophisticated analysis and is usually termed soil-foundation-structure-interaction (SFSI).

In general, the introduction of elastic SSI foundation springs will ‘soften’ the building response and lengthen the building period, usually resulting in a reduction of the spectral acceleration but possibly at the cost of an increase in displacement. The inclusion of more realistic, non-linear interaction effects and rigorous analysis (SFSI) may indicate much more significant reductions in building response from lengthening of the building period and increased damping. The clearest evidence of these potential benefits has been obtained using sophisticated nonlinear three-dimensional dynamic finite element analysis (eg Gazetas, 2015) which has shown that reducing foundation size (capacity) and permitting soil yielding during strong shaking may significantly reduce building response in some cases. Similar results have been confirmed using centrifuge modelling (eg Kutter and Wilson, 2006).

The use of such sophisticated modelling is not practical as an everyday design tool and simpler approaches have been developed. The so-called macro-element (eg Chatzigogos et al, 2007, Deng et al, 2012) provides a single computational entity that represents nonlinear foundation stiffness along with the ultimate capacity of the foundation and enables useful modelling to be done in a manner that is much simpler than full dynamic three-dimensional nonlinear finite element

analysis. Other simplified design methods have been developed from macro-element computations (Paolucci et al 2013, Loli et al 2015) which do not require the use of specialised software.

The behaviour of foundation elements are often approximated using springs, either linear or non-linear, and dashpots. When selecting spring parameters to use in these studies, it is important to consider the changes in soil properties caused by shaking (as described in Section 4), and the timing of these changes. For example, the foundation response may change from stiff at the start of the earthquake to be much softer by the end. Also, the considerable uncertainty in soil stiffness properties, in general, must be accounted for, usually by a parametric study. The spatial variability in soil properties from one foundation element to another also needs to be considered.

Extending the logic of soil-foundation-structure interaction, researchers (eg Gazetas, 2015) are increasingly advocating the deliberate reduction in foundation capacity (and thus stiffness) to further reduce building dynamic response. While these approaches show promise, they would not necessarily comply with B1/VM1 (which discusses the possibility, for instance, of permitting foundation uplift, but requires a ‘special study’, ie it would be considered an *alternative solution*).

The proposed ‘tailoring’ of foundation capacity and stiffness to reduce building dynamic response to earthquakes is an example of *performance based design*, which is discussed further in Section 3.9.

For further information on soil structure interaction, refer to FEMA (2020), P-209, A Practical Guide to Soil-Structure Interaction.

3.9 Performance based design — alternative solutions

The New Zealand Building Code is performance based and permits the use of alternative design procedures (*alternative solutions*) rather than following LFRD procedures discussed in Section 3.7 to demonstrate compliance with the Building Code. An alternative design procedure replacing codified strength based design (eg B1/VM1) with a more holistic appraisal of the building performance under various loading scenarios can be used to directly demonstrate that the design meets Building Code performance requirements.

This requires more sophisticated modelling of building response to loading including dynamic modelling of earthquake loading. Modelling of the foundation system and soil response needs to be included in a rigorous way, including the effects of soil non-linearity, otherwise the results may be misleading and inaccurate. Structural and geotechnical engineers need to work together closely on such studies to achieve realistic results.

Gazetas (2015) demonstrates a number of possible significant benefits of using this alternative solution design procedure for building–foundation systems. It can reduce the cost of the foundations while improving building safety overall. He adds some cautions, though, including the important caution that the approach is not a ‘panacea’ and is not appropriate for all buildings and all soils, and that differential settlements (eg from variable soil conditions) may inflict additional distress in the superstructure. Its main limitation is the inability to reliably predict performance, ie deformation of the building, especially the foundations, and to properly assess the uncertainty and variability in foundation performance. Uncertainties include the ability of practitioners to be able to make the necessary complex analyses, uncertainty in the models used to make the analyses, uncertainty in the soil properties required as inputs, and spatial variability in site soil conditions between one foundation element and another.

One approach showing promise, is to replace the site soils beneath shallow footings with engineered soils with more uniform and predictable characteristics of strength and stiffness (Gazetas, 2015, Anastopoulos, 2015). In this way, the foundation performance and building dynamic response could be analysed more readily and with greater reliability.

While the New Zealand Building Code prescribes minimum performance requirements including safety and reliability of building systems which need to be addressed explicitly when using alternative solutions, there are key principles from the design philosophy of NZS 1170 that should be followed, including:

- › Uncertainty in the earthquake loading must be accounted for. For methodologies based on response spectra, the hazard spectra derived from NZS1170.5 should be the basis for design. For dynamic time history modelling, uncertainty is considered by using a suite of relevant earthquake records, selected and scaled to match the hazard spectra derived from NZS 1170.5
- › Uncertainty in foundation performance and soil response should be accounted for (usually by means of a parametric study including a wide range of key soil strength and stiffness parameters).

Note

NZS 1170.5 requires a suite of at least three earthquake records, but in international practice it is more common to require 7 to 10 or more scaled earthquake records for time history modelling. For detailed guidance in the selection and scaling of suitable earthquake records refer to NIST GCR 11-917-15 (2011).

Toh et al (2011) report on a SFSI study comparing the effects of soil variability and the effect of different earthquakes and showed that the earthquake to earthquake variability was more significant than variability in soil properties.

4 Site assessment and foundation selection



4.1 Introduction

Good foundation performance with earthquake shaking depends critically on the response of the site soils to shaking and the response of the site itself. Most observations of poor foundation performance during earthquakes have been associated with ground failure including liquefaction or cyclic softening of the site soils and lateral spreading effects (eg Bray and Dashti 2014).

Foundation selection and design must be carried out in the context of a good understanding of the site soil response to earthquake shaking and the overall performance of the site including settlement and stability. The following matters require careful consideration and are discussed in this section:

- 1 **Soil response** — including liquefaction, cyclic softening, and other changes in soil properties caused by shaking
- 2 **Site performance** — including liquefaction severity, lateral spreading, settlement, and instability and the impact of these on different types of foundations
- 3 **Building Interaction effects** — the presence of a building may significantly alter the response of the site and exacerbate the effects of ground failure and settlement
- 4 **Foundation suitability** — including specific requirements and issues to be considered for different types of foundations taking account of the above effects.

Site soils may experience increase in pore water pressure with earthquake shaking causing reductions in effective stress, shear strength, and stiffness as well as time dependant reductions in volume resulting in settlement. In some cases, soil liquefaction may occur. Reductions in strength and stiffness of the founding soils may greatly affect the foundation performance and influence the selection of foundation type.

Significant settlements of the free field ground surface may occur because of volume changes following dissipation of excess pore water pressures. Surface ejection of water and soil following liquefaction may significantly increase settlements overall and differential settlements in particular.

The presence of a building changes the flow of water drainage within the ground and may exacerbate the ejecta and associated settlements. The weight of the building (heavier/taller buildings in particular) may induce shear deformations within the weakened soil layers to increase building settlement relative to the free field. Our ability to analyse and predict the settlement of buildings in these cases is very limited and a cautious approach is necessary.

Uniform (ie poorly graded), clean, dry, loose cohesionless soils may also undergo densification and settlement.

A thorough site investigation is required in all cases to identify all problem soils and to be able to quantify the effects of shaking on the soils (see Module 2 of the *Guidelines* for detailed guidance on planning, implementing, and reporting on suitable site investigations, and Module 3 for guidance on liquefaction assessment).

Stability of the site is also a critical issue and must be evaluated taking account of changes in soil properties caused by the shaking. In some cases, lateral spreading may occur that is known to be especially damaging to structures and challenging to prevent. Other stability issues include the presence of steep slopes and retaining structures.

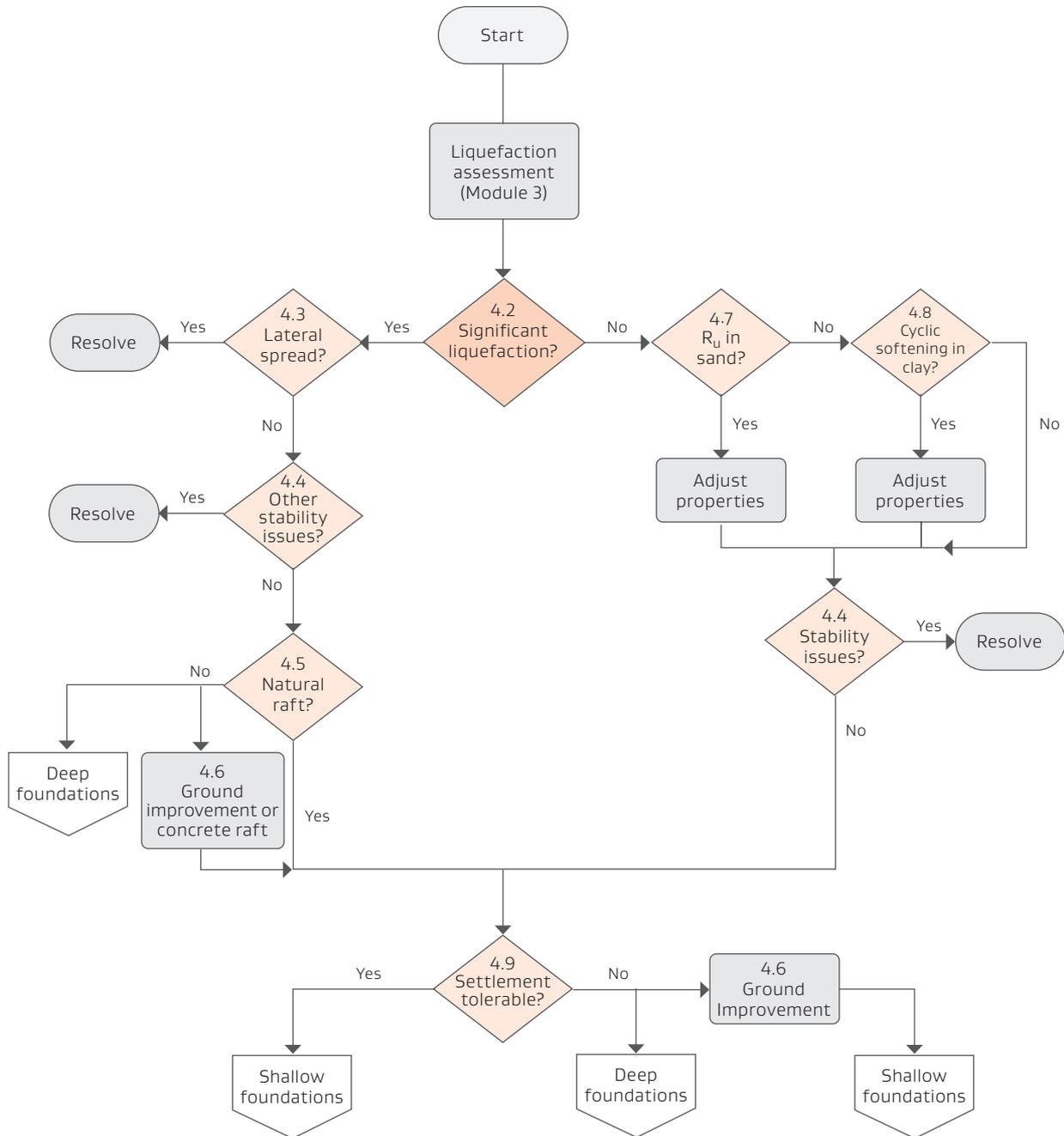
For sites with severe issues of soil liquefaction and lateral spreading, the recommendation of finding an alternative site should be discussed with the Client given the likely high cost of remediation in such cases.

Many different types of building foundations are in use and these are usually classified as being either *shallow* or *deep*, each class having quite different requirements, advantages, and limitations for earthquake resistant design. Shallow foundations require a suitable bearing layer of firm ground at shallow depth and may be affected by underlying liquefiable layers. Deep foundations may be useful where the above requirements for shallow foundations are not met, where loads are too heavy for spread or mat footings, where scour may be a problem, or where uplift resistance is needed. Deep foundations, though, are vulnerable to relative lateral movements of the various soil layers during shaking (kinematic loading), lateral spreading, loss of support and down-drag from liquefying intermediate soil strata, and buckling within thick layers of liquefied soil.

An alternative to deep foundations at sites with problem soils is to carry out well-engineered ground improvement. Experience indicates that strong, well-engineered shallow foundations founded onto dense, strong soil or well-engineered fill or improved ground can perform well during earthquake shaking. Even for larger and more heavily loaded buildings, well-engineered ground improvement and shallow foundations may prove to be a more economical solution than deep foundations at sites where piling conditions are unfavourable or where significant liquefaction induced settlement or lateral spreading is expected.

A general strategy for site assessment and foundation selection is presented as a flow-chart in Figure 4.1. While every situation is different, the flow-chart is intended to summarise a logical approach to site assessment and to act as a checklist for the key issues to be resolved to finally arrive at a safe and economical foundation solution. The remainder of this section provides detailed explanations for the process summarised in the flow-chart.

Figure 4.1: General strategy for site assessment and foundation selection
Note: refer to section numbers indicated in the boxes for detail



4.2 Soil liquefaction

The first and key issue in the site assessment and foundation selection process is to determine whether the site soils will be significantly affected by liquefaction during earthquake shaking.

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. Detailed guidance and procedures for assessing liquefaction hazard at a site are given in Module 3 of the *Guidelines*.

Separate assessments should be made for both the SLS and ULS level earthquakes. For cases where significant triggering of liquefaction is identified for the ULS level earthquake but not for the SLS level, then the intermediate return period for the level of earthquake shaking required to trigger significant liquefaction should also be determined. Liquefaction triggering may cause a degradation in site performance that may be intolerable for return periods close to the SLS. Judgement needs to be applied.

As far as possible, it is preferable to avoid building on sites that are at risk of damaging liquefaction. Where avoidance is not possible, the severity of the liquefaction and the resulting site performance should be assessed. Predictions of site performance need to be interpreted in the context of what will be tolerable for the building planned for the site, and whether the liquefaction will be triggered at the SLS level event, ULS level event, or at a level intermediate between the SLS and ULS.

Site performance during and immediately after an earthquake will be affected by the severity of liquefaction manifestations including sand boils, ground oscillations, ground cracking, settlement, lateral spreading, and slope instability. These effects are difficult to predict or quantify but may have severe consequences for a building and its foundations. Significant judgement is required to estimate the resulting site performance.

Site performance after liquefaction triggering can vary widely from imperceptible to devastating. The key characteristics affecting site performance include the thickness, depth, grading, and continuity of the liquefiable layer(s), thickness, stiffness, and strength of any overlying crust, as well as the intensity and duration of shaking.

Surface ejecta (sand boils) may be very damaging to buildings on shallow foundations. The likelihood of surface ejecta and other associated damaging surface manifestations of liquefaction is related to the thickness and properties of the surface crust and the thickness and properties of the liquefiable layer. Some guidance in assessing the adequacy of the surface crust for consideration as a 'natural raft' for shallow foundations is given in Section 4.5.

Shallow foundations are most at risk where liquefaction is triggered in a soil layer within the influence zone of a footing, usually taken to be within a depth equal to twice the footing width. The possibility of a 'punch through' failure of a footing into a liquefying layer beneath should be considered. (Guidance on assessing 'punch through' failure is given in Section 5).

The performance of highly layered sites, where liquefiable layers are comparatively thin and separated by multiple layers of non-liquefiable dense or cohesive soils, are more difficult to assess, but seem to be less damaging than sites with thick layers of liquefiable soils, in general. Triggering of liquefaction in thinner soil layers is less likely to result in severe effects such as void redistribution and generation of water films leading to surface ejecta, although these may still occur from layers close to the ground surface. Early triggering of liquefaction in lower soil layers may isolate, and protect upper soil layers from subsequent liquefaction. Pending further research, such sites should still be treated with caution.

Table 5.1 in Module 3 of the *Guidelines* provides some general guidance for correlating site performance to key characteristics of the site and shaking, at least for simpler cases. For the SLS, site performance levels LO and L1 (*insignificant or mild*) may be tolerable for most buildings of normal usage, L2 (*moderate*) may be tolerable for some buildings, while performance levels L3 (*high*) and higher would be intolerable for most buildings.

For cases where the predicted site performance is intolerable for the building on shallow footings,

4.3 Lateral spreading

For cases where significant liquefaction is expected, the risk of lateral spreading should be assessed, especially where a 'free face' is present (eg proximity to waterfront). Lateral spreading, where it occurs, may seriously compromise site performance and be very damaging to buildings. Guidance for assessing the risk of lateral spreading and potential for lateral movements is given in Module 3 of the *Guidelines*.

Comment

The recent paper by Cubrinovski and Robinson 2015 provides many useful insights into the features and characteristics of the numerous lateral spreading failures that occurred during the Christchurch earthquakes.

Lateral spreading is almost always observed wherever significant liquefaction has occurred and where a 'free face' is present although the scale of such movement varies widely. The most extreme and damaging lateral spreading is observed close to waterfront features such as harbours, lakes, rivers, and streams because of the presence of a 'free face' and because these locations are often associated with liquefaction susceptible soils. Less obvious lateral movements may be widespread far from such obvious features even where the ground slope may be very gentle, although such movements are usually less damaging to buildings.

then it will be necessary to consider the options of either ground improvement, use of a robust raft foundation, or deep foundations. In some cases, it may be preferable to consider a different structural form for the building or a different location.

For cases where significant liquefaction is predicted then the risk of lateral spreading should also be considered and assessed (ie left branch of the flow chart in Figure 4.1).

The different components and features of lateral spreading are illustrated in Figure 4.2 including large scale block movements (which may not damage buildings on shallow foundations eg Building A in Figure 4.2) and more intense, localised 'stretch' of the ground surface across a building footprint (which may cause severe damage from differential movements and tilting, eg Building B in Figure 4.2).

Severe lateral stretch is most often associated with proximity to waterfront features or sloping ground but may also occur elsewhere including locations where there is a sharp change in sub-surface soil conditions (eg buried former river channel). It is not possible to predict in advance where ground cracks will occur and so a cautious, defensive approach is required where potential lateral spreading is identified.

Comment

Our ability to predict the location, scale, and intensity of lateral spreading movements is very limited and unreliable and is the subject of ongoing research. Until more reliable methods for predicting lateral spreading hazard become available, a cautious approach to sites with identified risk factors is recommended.

Shallow foundations alone, without ground treatment, are unlikely to be suitable for sites where there is a risk of significant lateral stretch across the footprint of the building unless specifically designed to accommodate such demands (eg heavy reinforced concrete mat or raft). Such foundations must have sufficient strength to prevent the building being 'pulled apart' considering all sources of friction and passive resistance of embedded foundations.

Deep pile foundations may increase the 'pull apart' forces in the building and the resulting loads may damage the pile head connections (see Figure 4.3). Kinematic loading of the piles is also likely to be severe because of the lateral movement of the soil blocks.

Where these effects cannot be resolved, it may be necessary to increase the setback from the waterfront feature or improve the site performance using engineering interventions such as ground improvement, infilling of the feature, buttressing, or use of retaining structures.

Where such engineering measures are used to remove or reduce the lateral spreading hazard in an effort to make the site suitable for shallow foundations, then the measures effectively become part of the building foundation and should be engineered to the same high level of confidence and reliability.

Figure 4.2: Components of lateral spreading

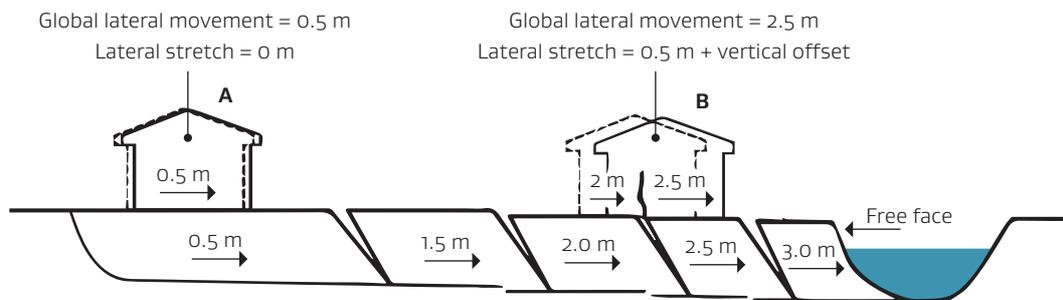
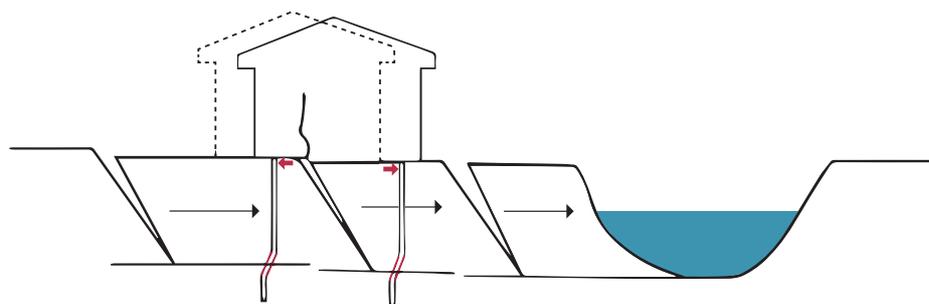


Figure 4.3: 'Pull-apart' of building with lateral spreading



4.4 Other site stability issues

Lateral spreading is an extreme example of site instability particular to cases of liquefaction adjacent to waterfront features. However, earthquake shaking may significantly increase the risk of other types of instability at sites with the following features:

- › Sloping ground
- › Cut batters
- › Fill batters
- › Retaining structures.

Where failure or excessive deformation of any of these features might compromise the required performance criteria for building foundations, including serviceability criteria at the SLS and safety, resilience, and egress criteria at the ULS, then site stability should be carefully evaluated.

The potential for reduction in strength and stiffness of the site soils with shaking needs to be taken into account (including liquefaction, increase in pore water pressure, and cyclic softening in clay soils) when making the stability evaluation as well as the effect of earthquake induced inertial loads.

Where ULS level earthquake shaking might trigger a significant slippage from up-slope, including rock fall, that might block egress, collapse the building, or otherwise endanger the occupants, then engineering works should be carried out to mitigate the hazard (ie removal, re-shaping, reinforcing, or retaining).

Comment

Assessment of potential hazards needs to include a wider perspective than just the immediate boundaries of the site. For example, large numbers of boulders travelled several hundreds of metres in some cases in the Port Hills following the Christchurch earthquake of 2011.

Where earthquake shaking might trigger a significant slippage downslope that potentially undermines the building foundations, then engineering works should be carried out to mitigate the risk (ie buttressing, re-shaping, reinforcing, or retaining).

For less-steep slopes where slippage (rupture) is considered unlikely, less extreme deformation of the slope may still occur and ground deformations should be estimated for both the SLS and ULS earthquakes and compared with the relevant performance criteria for the building (eg Jibson 2007).

Shallow foundations may not be suitable on sloping sites where there is a significant risk of instability or where ground deformations are likely to be excessive. It may be necessary to increase the setback from the slope or remove the instability hazard using engineered interventions such as ground improvement, infilling, reinforcing, buttressing, or retaining structures.

Deep foundations may be subject to lateral deformation resulting from slope movement and the resulting foundation strains need to be assessed to ensure that they remain within safe limits (see Section 6 for detailed guidance).

Where engineered interventions are used to make the site suitable for building, they should be designed to the same level of confidence and reliability as other components of the building foundations. Where retaining walls are used to provide stability and support to a building foundation or to protect the building and its occupants from damage, they should be designed to resist earthquake shaking. Module 6 of the *Guidelines* provide detailed guidance for the design of retaining structures to resist earthquake shaking.

Waterfront sites need careful assessment for both stability and deformation, even where liquefaction is not an issue.

4.5 Natural raft

For sites with liquefiable soils but where the lateral spreading hazard has been deemed to be insignificant or within tolerable limits for the building proposed, shallow foundations may be used provided the surface crust is thick and competent enough to act as a 'natural raft'.

The surface crust needs to be able to receive the foundation loads and also to protect the foundations from the effects of any underlying liquefaction or other weak strata. The following requirements need to be confirmed:

- a The surface crust must be thick enough and strong enough to be able to bridge over any underlying liquefiable or weak soils. The necessary thickness is relative to the weight of the building and building form as well as the properties of the crust layer and underlying soils. Conceptual guidance is provided by Ishihara [1985], but is based on very limited data and not intended for design purposes (refer to Module 3 for more information). The possibility of 'punch through' failure of shallow footings into underlying liquefiable or weak layers should be checked (see Section 5 for more guidance).
- b The properties of the surface crust must be proven to be continuous across the site so as to provide uniform support to the entire footprint of the building. Alluvial soils are often variable over short distances and the potential for lenses of loose or liquefiable soils within the layer, or changes in thickness or density should be considered. No building should be supported part on shallow foundations and part on deep piles without structural separation of the two parts.

In all cases where the bearing layer overlies liquefiable soils the following additional design requirements for shallow foundations should be followed:

- a The foundation system should be well tied together and capable of spanning pockets of lost support from pore water penetration into the layer. Multi-storey buildings should have raft foundations (or deep pile foundations).

- b Concrete mat or raft foundations (including basements) should be capable of resisting the uplift forces from the high pore-water pressures caused by soil liquefaction at depth. Non-structural floor slabs are likely to be badly damaged and need replacing after the earthquake (but may be considered as a serviceability issue).

Comment

For large warehouse and low-rise industrial buildings, the cost of either ground improvement across the entire footprint, substantial concrete raft, or deep foundations to support the entire floor slab may prove uneconomic. For such buildings, and for certain uses, it may be acceptable to consider the floor slab as a non-structural, replaceable fit-out item poured on grade with only the structure itself (walls, roof, suspended floors) supported on deep foundations or shallow foundations with ground improvement (ie the floor slab may need replacing after an SLS or larger earthquake on vulnerable soils).

Where the requirements for a 'natural raft' cannot be confirmed, it may be possible to manufacture an 'artificial raft' by carrying out well-engineered ground improvement (or excavation and replacement with reinforced, engineered fill). Alternatively, a robust concrete raft may be used or deep foundations considered, (ie following the left hand branch in the flow-chart of Figure 4.1).

4.6 Ground improvement

The objective of ground improvement is to treat loose, weak soils at a site to prevent liquefaction or reduce its effects and to create a 'raft' of soil with improved soil strength and stiffness so that shallow foundations may be used.

A very wide range of ground improvement techniques are available and are subject to on-going innovation and development. The wide range of techniques and the many parameters associated with each, result in a wide range of outcomes both in terms of level of improvement and subsequent performance during shaking. The greater the level of improvement the greater the cost, in general, and there needs to be a level of sophistication in specifying and implementing ground improvement to achieve the necessary level of performance.

Module 5 of the *Guidelines* provides detailed guidance on the design of ground improvement schemes, the many different techniques available, together with a discussion of construction and verification considerations.

Where ground improvement is being relied on to prevent soil liquefaction and permit use of shallow foundations to support a building, then the ground improvement effectively forms part of the foundation system of the building and should be considered as an essential component of the design.

The considerable uncertainty in predicting the performance of the ground after treatment should be considered during design by use of appropriate safety factors (similar to the strength reduction factors used in pile and footing design) to ensure a reliable outcome. Appropriate verification strategies such as pre- and post-treatment measurement of soil properties are a key part of the overall improvement strategy.

A significant limitation of most ground improvement techniques is the need to extend treatment to beyond the building footprint because of edge effects and lack of confinement. For many inner city properties, there is a strong commercial necessity to build up to the property line and an impossibility of extending ground improvement over the property line. Instead, consideration may be given to cantilevering a concrete foundation raft over the zone of ineffective ground improvement which is assumed to lose support with earthquake shaking.

4.7 Sand-like soils with $F_L > 1$

Saturated sandy and silty soils below the water table and assessed as having $F_L > 1$ are not expected to liquefy (ie to have increases in pore water pressure to the extreme where $\Delta U = \sigma'_v$, or, $R_u = 1$). However, these soils may experience significant increases in pore water pressure resulting in loss of effective strength and stiffness and settlement as the excess pore water pressures dissipate.

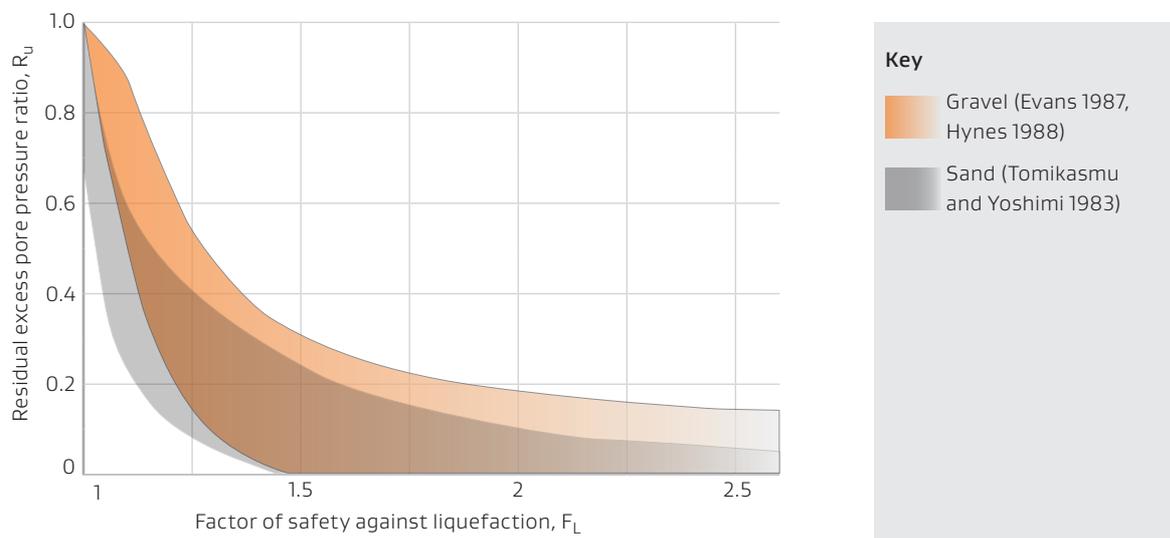
At level-ground free-field sites the excess pore water pressure ratio, R_u , may be estimated from Figure 4.4, as a function of F_L . The resulting reduction in effective strength of the soil should be included in capacity calculations for foundations and also for slope stability analysis (eg Marcuson et al 1990).

For shallow foundations, soil-structure interaction effects may significantly increase the generation of pore water pressures above these free-field values, especially near to the foundation edges, by increasing the cyclic stresses in the adjacent soil from the building dynamic response with no corresponding increase in vertical effective

stress (eg Travasorou et. al 2006 showed that the reduction in F_L could be by as much as 50 percent near the edge of shallow mat foundations).

Caution is required for cases where the liquefaction triggering analysis for the free-field condition indicates marginally liquefiable soils at shallow depths underneath a building foundation (eg $F_L < 1.5$). The possibility of liquefaction induced settlements should be considered in such cases. (These are discussed further in Section 4.9. Additional information is provided by Travasorou et al 2006, Bray and Dashti 2014, and Bray and Macedo 2017).

Figure 4.4: Excess pore water pressure ratio ($R_u = \Delta U / \sigma'_v$) versus factor of safety against liquefaction triggering under level ground conditions [Source: Marcuson et al 1990]



4.8 Clay-like soils

Clay-like soils are non-liquefiable (see Module 3) because they do not exhibit typical liquefaction features. However, they may soften significantly with shaking and the potential strength loss should be considered when assessing site stability and foundation bearing capacity.

Boulanger and Idriss (2007) provide a methodology for assessing the cyclic strength of clay-like soils using an approach similar to the simplified procedure for assessing liquefaction triggering in sands. The Cyclic Resistance Ratio (CRR) or 'cyclic strength' of the clay-like soil being defined as the normalised

cyclic load causing 3 percent peak-to-peak shear strain after 15 cycles (considered to be 'failure'). The CRR is shown to depend primarily on the static undrained shear strength of the soil and on the initial static shear loading such as from an embankment or footing.

4.9 Settlement

Ground shaking may cause significant settlement of buildings especially where liquefaction is triggered at a site or where other weak soils are present.

Settlements result from several contributing mechanisms and may be broadly classified as either:

- a 'Free Field' settlement, ie settlements that would occur at the site without a building being present, and
- b Building induced settlements, ie caused by the presence of the building and building platform and from interactions between the building and the ground.

4.9.1 FREE FIELD SETTLEMENT

'Free field' settlement of the ground surface is frequently observed after significant earthquakes at sites with loose, weak soils. Causes include:

- a Contraction and consolidation of loose, cohesionless soils (saturated or dry)
- b Ejection of waterborne sediment after liquefaction (sand boils)
- c Lateral spreading
- d Shear deformation of weak strata
- e Tectonic subsidence (can be uplift).

Liquefaction is caused by a tendency of loose, cohesionless soils to contract with shaking. Consolidation and volume reduction follows liquefaction causing settlement of the ground surface.

The potential volumetric strain depends on the value of F_L . A certain amount of contraction and settlement can occur even for cases where $F_L > 1$ although liquefaction triggering is not expected. Detailed procedures for estimating free-field ground surface settlement from the effects of contraction and consolidation are given in Module 3 of the *Guidelines*.

Where surface ejection (sand boils) occur much additional settlement of the ground surface may result from the lost volume of soil. The extent and significance of these additional settlements depends on the integrity and thickness of overlying non-liquefiable crust, although our ability to predict ejection and resulting settlement is very limited.

Liquefaction near to the ground surface has the greatest potential to cause non-uniform differential settlements that are most damaging to buildings, especially where surface ejection (ie sand boils) occur. Settlement from consolidation of liquefied soil at depth below thick, competent surface layers is more uniform and causes much less damage, in general. Lateral spreading causes additional slumping and settlement of the ground surface and may be very severe near to waterfront features (see Figure 4.2 and 4.3).

Contraction may also occur in dry, loose, cohesionless soils especially poorly compacted fill.

Comment

Experience in Christchurch has shown that settlements at sites where soil liquefaction has occurred tend to be highly non-uniform and correlate very poorly with settlement analysis procedures based on CPT and SPT data. If liquefiable soils are identified at a site, then it should be assumed that the foundations will be subject to ground settlement, including non-uniform settlement.

The calculated re-consolidation settlements from the liquefaction analysis consider only one single mechanism of liquefaction induced settlement, and do not consider other mechanisms such as ejection (sand boils), and the building induced effects which may dominate the response.

The calculated 1D settlements should be considered only as a crude 'index' of the scale of liquefaction beneath the site, not a reliable predictor of actual settlements.

Shear deformation at sites with non-uniform topography, including placed fills, may undergo 'flattening' of high ground relative to low ground from shear deformation resulting from cyclic softening of the site soils and high shear stresses induced by the earthquake. (Including non-liquefiable cohesive soils, eg Carrefour site in the 1999 Kocaeli Earthquake, Boulanger and Idriss, 2007.)

Tectonic movements of the ground surface frequently accompany major earthquakes and can be a significant component of uniform ground surface 'settlement.' Note: tectonic movements can be up or down. Such movements will be mostly uniform (except near to the surface fault rupture) and thus not damaging to buildings, in general, but may cause problems with drainage and flooding.

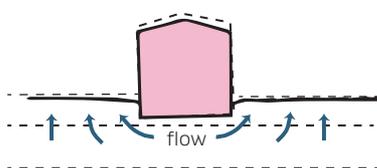
4.9.2 BUILDING INDUCED SETTLEMENTS

Buildings at a site will generally be subject to the 'free field' settlements listed above as well as additional settlements resulting from the presence of the building and building platform from mechanisms identified in Figure 4.5 as follows (Bray and Dashti 2014):

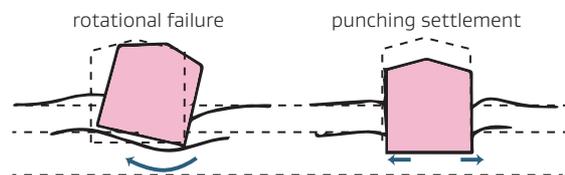
- a Localised volumetric strains during partially drained cyclic loading controlled by 3D transient hydraulic gradients, followed by downward displacement due to sedimentation and re-solidification after liquefaction followed by consolidation as excess pore water pressures dissipate (Figure 4.5a)
- b Shear-induced deformations including punching settlement and tilting from partial bearing failure under the weight of the structure following softening of the soil, including displacement of liquefied soil from beneath the building (Figure 4.5b)
- c Cumulative ratcheting settlement of the foundations into the softened ground from building dynamic cyclic loading (Figure 4.5c).

Figure 4.5: Liquefaction induced displacement mechanisms: (Source: Bray and Dashti 2014)

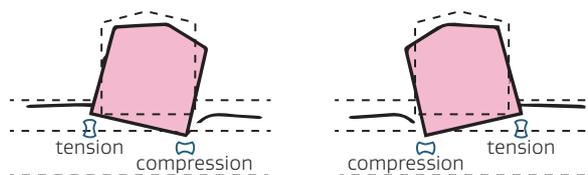
a Volumetric strains caused by water flow in response to transient gradients



b Partial bearing failure due to soil softening



c Soil-structure-interaction induced building ratcheting during earthquake loading



If there is a sufficient thickness of liquefiable soil present underneath the building and close to the foundations (ie within the influence zone of the foundations), then significant liquefaction-induced settlements can occur, governed mostly by shear deformation mechanisms.

Most damaging are non-uniform, differential settlements of individual footings because these are most likely to cause excessive angular distortion within the building. Care is required to ensure that individual foundation elements will not be affected by layers or pockets of softened or liquefied soil by ensuring a surface 'raft' of competent material of sufficient thickness.

Where significant pore water pressures are expected to be generated in soils within the influence zone of a shallow foundation (eg $F_L < 1.5$) then the possibility of liquefaction induced building settlements should be considered (as discussed in Section 4.7). These settlements may be severe and difficult to quantify, and a cautious approach to such cases of either avoidance or remediation is warranted (see Bray and Dashti 2014 for some useful case studies and more information, and Bray and Macedo 2017 for a simplified procedure for estimating building settlements with liquefaction).

4.10 Shallow foundation requirements

Having completed the site assessment process of Figure 4.1 and concluded that shallow foundations may be suitable for a site, the following checklist summarises the key requirements:

- a There should be a clearly identified bearing layer at shallow depth capable of providing adequate support for the building loads. Alternatively, well-engineered ground improvement should be carried out.
- b The bearing layer should be thick enough and strong enough to be capable of bridging over any underlying liquefiable or weak soils without 'punch through'. The necessary thickness is relative to the weight of the building and building form as well as the properties of the bearing layer and underlying soils.
- c The bearing layer should be continuous across the site to provide uniform support to the entire footprint of the building. Buildings should not be supported part on shallow foundations and part on deep piles without structural separation between the two parts.
- d Where the bearing layer overlies liquefiable soils, the foundation system should be capable of spanning pockets of lost support from pore water penetration into the layer. Multi-storey buildings should have raft foundations (or deep pile foundations).
- e Where the bearing layer overlies liquefiable soils concrete mat or raft foundations (including basements) should be capable of resisting the high pore-water pressures resulting from soil liquefaction at depth.
- f All shallow foundations should be well tied together (unless supported on a continuous layer of dense gravel or rock).
- g In all cases, it is important to keep in mind the limitations of the site testing and sampling and the potential for variations in layer thickness, density or inclusions of loose or soft lenses within any unit between the site investigation locations.

4.9.3 OTHER SETTLEMENT SOURCES

The earthquake induced mechanisms of settlement described above will occur in addition to other non-earthquake related settlements arising from gravity loading including compression and consolidation of the founding soils.

Estimation of settlement under gravity loading is beyond the scope of these *Guidelines* and instead, standard references should be consulted (see Section 1.2 for recommended references). Pre-existing settlements under gravity loading will be cumulative with earthquake induced settlements. Excessive pre-existing differential settlements may damage the structure and compromise the seismic resistance mechanisms even before the earthquake occurs.

Sites with liquefaction susceptible soils often have other problem soils present such as normally consolidated clay and peat soils and care is required to ensure that the total settlements are not excessive.

Where the total settlements are expected to be intolerable for the building, then it will be necessary to consider using deep foundations or well-engineered ground improvement to a depth sufficient to reduce the settlements to a tolerable level.

4.11 Deep foundation requirements

Having completed the site assessment process of Figure 4.1 and concluded that deep foundations may be the most suitable option, the following checklist summarises the key requirements for deep foundations:

- › There should be a clearly identified bearing layer (or layers) capable of providing support for the pile type and the building loads. Piles should be installed (driven, bored, screwed) to a target depth (or depth range) within the bearing layer, as determined by the site investigation, and not simply driven to refusal or to a set.
- › The bearing layer should be sufficiently deep to be below any layers of liquefiable or weak soils or be thick enough to bridge over any underlying liquefiable or weak soils.
- › The bearing layer should be continuous across the site to provide uniform support to the entire footprint of the building. Site testing of the bearing layer should be sufficient for the designer to have confidence that it is consistently thick and dense enough with a low probability of looser inclusions within it which could adversely impact on piles bearing above them.
- › If the piles are founded into a bearing layer at depth, underlain with liquefiable soil, redundancy in piles is desirable to lessen impact of individual piles not performing as well as expected.
- › Piles should not be founded within different bearing strata at different depths beneath a single building.
- › Piles should be capable of transferring the vertical loads from the building into the bearing layer, reliably, and meet settlement requirements, even with liquefaction and cyclic softening of overlying soils, including the effects of loss of side resistance, load re-distribution, and down-drag.
- › Piles should be designed to withstand relative lateral movements of intermediate soil layers (kinematic effects) including both transient and permanent lateral movement of the ground surface (lateral spread) without excessive damage which might compromise their ability to carry the building vertical loads reliably.
- › Piles, pile connections, and building sub-structure should be designed to resist 'tear-apart' loads resulting from lateral stretching of the site (see Figure 4.3).
- › Piles should be designed to resist loads from the dynamic response of the building (ie base shear) without excessive damage which might compromise their ability to carry the building vertical loads reliably.
- › Piles should be designed and detailed with sufficient ductility to survive additional demand from possible earthquake shaking greater than the design ULS event without excessive damage which might compromise their ability to carry the building vertical loads reliably.
- › Heavily loaded, slender piles penetrating through thick layers of liquefied soil may fail by buckling. The possibility of pile instability with liquefaction should be considered by designers.
- › In all cases, it is important to keep in mind the limitations of the site testing and sampling and the potential for variations in layer thickness, density or inclusions of loose or soft lenses within any unit between the site investigation locations.

5 Shallow foundation design

5.1 Introduction

This section outlines a simplified approach for practical design of shallow foundations with earthquake loading. The approach taken is to adapt familiar design procedures for gravity design and apply them within a rational framework for design of shallow foundations to resist earthquake loading that is compatible with NZS 1170.0:2002 and NZS 1170.5:2004.

This section is not intended to provide a fully comprehensive treatment of the design of shallow foundations under gravity loading in all situations and soil conditions. Instead, well known published handbooks should be consulted for much additional guidance on the detailed design and construction of shallow foundations (see Section 1 for recommendations). The intention in this section is to provide supplementary guidance on earthquake design aspects that are not well covered elsewhere.

The main problems with shallow foundation performance during earthquakes have been related to poor site performance and poor soil conditions especially liquefaction, cyclic softening, lateral spreading and other instability. These issues need to be addressed before proceeding with shallow foundation design. A full site assessment should be carried out, as detailed in Section 4, to decide whether the site will be suitable for shallow foundations, or whether ground improvement is first required, or whether deep foundations would be more appropriate. A list of the key requirements for shallow foundations is given in Section 4.10.

The main causes of earthquake induced settlement of buildings are discussed in Section 4.9.

The following situations may result in severe differential settlements that would be intolerable for buildings in most cases:

- a Liquefaction and especially ejection of soil from shallow depth beneath foundations
- b Soils within the influence zone of a foundation that are expected to generate significant pore water pressures with shaking (eg $F_L < 1.5$)
- c Soils within the influence zone of a foundation that are expected to suffer significant cyclic softening.

These situations are exacerbated by soil-structure interaction effects that may greatly increase generation of pore water pressures near to shallow foundations and are difficult to quantify. However, these situations are readily identifiable with a thorough investigation and should be avoided or mitigated. In any case, a simplified approach to foundation design is not possible for these situations.



Other earthquake effects may increase the settlement of shallow foundations but are considered less severe and should be tolerable for most buildings founded on good soils which are unlikely to generate significant pore water pressures. These effects include:

- a Re-consolidation of liquefied soils in deeper layers beneath a competent surface crust of sufficient thickness
- b Building dynamic response to shaking increasing the foundation loads (generally) and including lateral and moment loading
- c Vertical accelerations of building (momentary)
- d Shear failure of founding soil from inertia of the soil mass (momentary).

Liquefaction of deeper soil layers, below the foundation influence zone, and provided the building is founded on a competent surface crust of sufficient thickness, may cause additional settlement following re-consolidation of the liquefied soil, but is unlikely to cause damaging differential settlements. (See Module 3 of the *Guidelines* for guidance in estimating the 1D re-consolidation settlements and thickness of crust for suppressing damage to buildings at the ground surface.)

Building dynamic loads applied to foundations are cyclic loads of relatively short duration, depending on the natural period of the building. The high rate of loading and short duration may reduce the resulting settlement to less than the equivalent static values, but, on the other hand, the multiple cycles may cause a 'ratchetting' effect from the inelastic behaviour of soil in general.

Building dynamic response includes components of lateral and moment loading of the foundations that may reduce the vertical bearing capacity of shallow footings exacerbating the 'ratchetting' effect described above.

Vertical accelerations of the earthquake are of very brief duration and are usually neglected when assessing foundation design because any increment in settlement will be small.

5.2 Types of shallow foundations

Earthquake resistant design procedures for the most commonly used types of shallow foundation are discussed in this section, including:

- › Pad footings used to support individual columns; (Note that pad footings should always be well tied-together in New Zealand so as to improve the robustness of the structural system as a whole to resist earthquake shaking)
- › Strip footings used to support walls
- › Foundation beams used to support rows of columns
- › Mat and raft foundations used to support multiple columns and/or walls either in groups or across an entire building footprint.

High ground accelerations during shaking induce high shear stresses in the soil beneath shallow foundations, that, in combination with shear stresses from the foundation loading, may cause a momentary shear failure in the soil and additional increments of settlement. So called 'seismic bearing capacity factors' have been proposed to include these effects (eg Richards et al, 1990, 1993, Shi and Richards, 1995). However, such shear failures will be of very short duration and the resulting increment in settlement will be small in most cases.

Observations from previous earthquakes (eg Christchurch CBD, Cubrinovski and McCahon 2011) indicate that buildings on shallow foundations on good soils or improved soils, not subject to significant pore water pressure generation, have performed satisfactorily. The foundations for the buildings observed were (presumably) designed using normal, familiar, LRFD procedures for (quasi) static loads and normal resistance factors, the same approach that is recommended for use in these guidelines.

Building dynamic loads may be much greater than gravity loads, depending on the aspect ratio and structural configuration of the building, and footings should be sized to resist these. The structural engineer would normally calculate load sets for each foundation for the critical load combinations specified in NZS 1170.0:2002 following analysis of the building response (see Section 3.7 for more discussion of LRFD design for earthquake loading).

The foundation load sets for the earthquake load case may be treated as 'equivalent static' loads and the footings sized using normal static design procedures. Provided 'normal' resistance factors are applied to the calculated ultimate bearing capacities (see recommendations below), then 'ratchetting' of foundation settlements under cyclic loading during an earthquake should remain modest.

For a discussion of alternative design approaches using principles of performance based design refer to Section 3.9.

5.3 Pad footings and strip footings

Strip footings are a special case of pad footings but with a high ratio of breadth to width. Both may be referred to as *shallow spread footings* or *shallow footings*. Strip footings are normally used to support continuous loads from walls.

When used to support a row of columns, strip footings should properly be called foundation *beams* because they are subject to significant longitudinal bending and require additional analysis (see Section 5.4).

Typical load-settlement curves for shallow footings are shown in Figure 5.1, in which Curves 1 and 2 reach a defined maximum value while Curve 3, typical for loose or soft soils, does not reach a defined maximum value. The *ultimate bearing capacity* is interpreted as being the maximum load achieved for Curve 1 and Curve 2 and as the point on Curve 3 where a constant rate of penetration was achieved (often not clearly defined).

The *ultimate bearing capacity* is usually taken to represent an 'ultimate limit state' in terms of NZS 1170.0:2002, but in some circumstances should properly be considered as a 'collapse state' depending on the structural configuration (eg see Figure 5.2).

The load-settlement behaviour of shallow footings under large applied loads is unpredictable and difficult to interpret even from full-scale load tests. Large and mostly intolerable settlements are required to mobilise the ultimate bearing capacity (typically 5 percent of the footing width) for all but the narrowest of footings (less than about 0.5 m).

Accordingly, large factors of safety have traditionally been applied to ensure that there is a low risk of loads approaching the ultimate bearing capacity throughout the life of a structure (eg $FS > 3$ for gravity case and $FS > 2$ for seismic case). With the adoption of LRFD design procedures in

New Zealand practice, these traditional factors of safety for shallow footings have been preserved (essentially) by adopting equivalent resistance factors (ie *geotechnical strength reduction factors*) of $\Phi_g = 0.45 - 0.6$ in combination with the prescribed load combination factors of NZS 1170.0:2002.

The need for such low values for resistance factors may be understood by reference to Figure 5.3, which shows the load settlement relationship for a strip footing on clay soil (simulated using finite element analysis for a typical clay). The blue curve in the Figure represents the expected performance for an assumed design soil strength of $S_u = 50 \text{ kN/m}^2$. The yellow curve represents the performance if the soil strength were 30 percent lower than expected, and the grey curve if the soil strength were 30 percent higher than expected (eg Eurocode 7 uses a partial safety factor on undrained shear strength of 0.7).

For a strength reduction factor of $\Phi_g = 0.5$, settlement under the full ULS load would be expected to range from 6 to 25 mm in this example (red arrow in Figure 5.3), depending on the actual soil strength, and quite tolerable for most buildings at the ULS. For higher values of Φ_g though, the range of expected settlement increases rapidly, to be from 9 to 50 mm for $\Phi_g = 0.6$, and from 12 to $\gg 100$ mm (ie 'failure') for $\Phi_g = 0.8$.

Figure 5.1: Load settlement relationships for shallow footings [Source: Vesic, 1975]

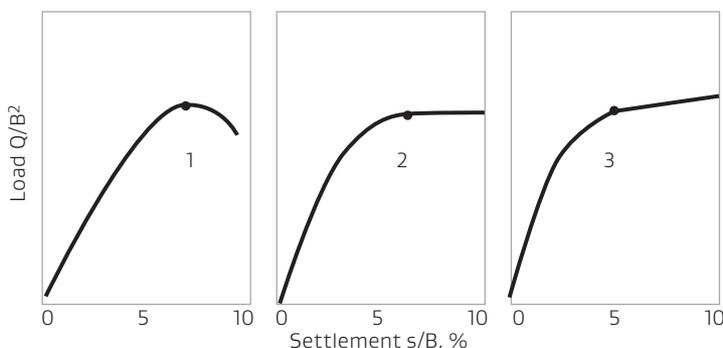
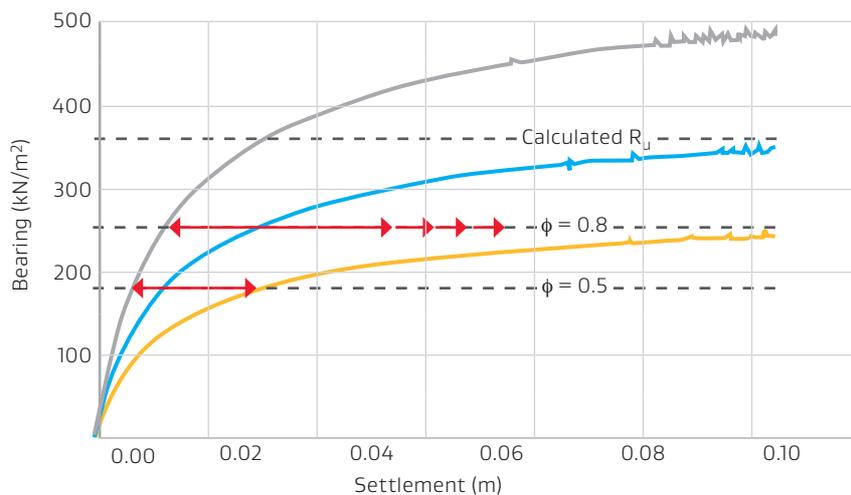


Figure 5.2: Bearing capacity failure of a silo foundation [Source: Tschebotarioff, 1951]



Figure 5.3: Load-settlement response for a 2 m wide shallow footing on clay soil with $S_u = 50 \text{ kN/m}^2$



The example of Figure 5.3 is somewhat idealised because it only considers the uncertainty of a single soil parameter, S_u , and does not include other sources of uncertainty in predicting the performance of shallow footings including:

- › Uncertainty in soil strength parameters and strength mobilisation (including uncertainty in interpreting strength test results)
- › Spatial variability in soil parameters from one footing to another across a building
- › Computational uncertainty in the bearing capacity equation (ie Equation 5-1 is not an exact solution, and solutions from different analysts including Meyerhof, Brinch-Hansen, and Vesic all give different values because of different approximations made)
- › Physical uncertainty (for certain soils, including loose sands and soft clays, the well-defined failure surface assumed for Equation 5-1 does not develop, instead a punching shear failure may occur at a lower load)
- › Scale effects for large footings (eg Vesic, 1975).

In considering all of these sources of uncertainty, the *geotechnical strength reduction factor*, Φ_g should be selected from the range given in Table 5.1.

Table 5.1: Geotechnical strength reduction factor Φ_g for shallow footings

LOAD COMBINATION	Φ_g
All ULS load combinations	0.45 – 0.6 ¹

¹ The value of the strength reduction factor used in design will depend on the designer's knowledge of the site and the investigations undertaken. As a guide the lower end of the range will generally be appropriate when a limited site investigation is undertaken, average geotechnical properties are used, published correlations are used to obtain design parameters or there will be minimal construction control. The upper end of the range will generally be appropriate when a comprehensive site investigation and laboratory testing is undertaken, geotechnical properties are chosen conservatively, site specific correlations are used for design parameters and there will be careful construction control.

5.3.1 GRAVITY DESIGN PROCEDURE

NZS 1170.0:2002 requires designers to carry out two design checks for the gravity load case:

- Strength check under prescribed combinations of factored ULS loads. This requirement is stated as:

$$R_d \geq E_d$$
 (Equation 7.2 from NZS 1170.0:2002)

$$R_d = \text{Design Capacity}, \Phi R$$
 (R = ultimate bearing capacity for footing)
 (Φ = resistance factor, 'geotechnical strength reduction factor')

$$E_d = \text{Design Action Effect}$$
 (ULS combination of factored loads on footing)
- Serviceability check using combinations of non-factored serviceability loads.

The following general approach is usually followed:

- Calculation of ultimate bearing capacity** — the *ultimate bearing capacity* is equal to the *ultimate bearing pressure* multiplied by the *effective area* of the footing.
 The ultimate bearing pressure is calculated using the familiar Terzaghi-Buisman equation based on considerations of limiting equilibrium:

$$q_u = cN_c + qN_q + \frac{1}{2} B\gamma N_\gamma \quad 5-1$$
 Equation 5-1 is for an infinite strip footing of width, B, on level ground on stiff soils. Various significant adjustments and modifying factors are applied to account for the influence of footing shape, depth, load inclination (lateral load), eccentricity of load (moment), soil stiffness, base tilt, and ground slope. In particular, the effects of simultaneous lateral and moment loading may be very significant in reducing the bearing capacity for vertical loads (eg Pender, 2017).

The key inputs required for Equation 5-1 are the appropriate soil strength parameters c and ϕ for drained conditions or S_u for undrained conditions.

- Select value for geotechnical strength reduction factor** — a value for Φ_g is selected from Table 5.1 and used to calculate the *design capacity* for the footing $R_d = \Phi_g R$
- Estimation of settlement** — foundation settlements are generally divided into three components, each considered separately according to:

$$S = S_i + S_c + S_s \quad 5-2$$

in which:

- S = total settlement
- S_i = immediate or 'elastic' settlement
- S_c = consolidation settlement
- S_s = secondary or creep settlement.

For cohesionless soils and unsaturated cohesive soils, the immediate settlement predominates and may be calculated using published elastic theory solutions. The soil modulus of elasticity, E, and Poisson's ratio, ν , are required as inputs and must be determined either by correlation with in situ tests or from laboratory testing of undisturbed soil samples.

Alternatively, semi-empirical procedures are available for calculating settlements directly from SPT or CPT soundings [eg Schmertmann et al, 1978, Burland and Burbridge, 1985] (Note: the comments below on the use of semi-empirical correlations with in situ test data).

For saturated cohesive soils, consolidation settlement, S_c , may be significant and can be calculated using consolidation theory. The necessary soil parameters are usually determined by laboratory testing of undisturbed soil samples (ie oedometer test).

Secondary or creep settlement may be significant and even predominant for soils with a high organic content and for certain silts. Assessment

of creep settlement for susceptible soils is difficult and further guidance is given by Mesri and Castro [1987] and Leoni et al [2008].

Given the considerable uncertainties in predicting settlement of shallow footings, estimates should be presented as a range based on a parametric study considering the likely range of soil parameters and using different calculation methods.

Comment

Semi-empirical correlations with in situ tests

Charts providing correlations of allowable bearing pressure with SPT test results have long been used to design footings on sand [eg Terzaghi and Peck, 1967]. *Allowable bearing pressure* is usually defined as being the maximum soil pressure for which the foundation settlement should not exceed 1 inch (25 mm) and the differential settlement $\frac{3}{4}$ inch (20 mm).

Care is required in the use of these charts because they do not easily account for all of the many factors affecting the performance of shallow footings including the influence of adjacent footings or soil layering [eg Bowles, 1997].

The design of narrow footings on sand and most footings on clay may be governed by the ultimate bearing capacity (ie ULS geotechnical strength case) and it is not possible to determine the ultimate bearing capacity easily from these charts.

For sites where soil properties are likely to degrade with earthquake shaking, including significant increases in pore water pressure, liquefaction, or cyclic softening, direct correlation of foundation performance with penetrometer data will only be meaningful for the gravity load case. These design charts should be limited in use to preliminary sizing of footings for the gravity load case.

Scala penetrometer test

The design chart of Stockwell [1977] is widely used in New Zealand to design shallow footings for small structures by correlation of *allowable bearing pressure* to Scala penetrometer blow counts. Adjustments are made for depth to water table, depth of embedment, width, soil type, and 'vibrational effects.'

The Scala penetrometer cannot be used to assess the potential of soils for generation of pore water pressures during shaking, liquefaction, or cyclic softening. Therefore, the Scala penetrometer cannot be used for design of shallow foundations to resist earthquake loading.

The Scala penetrometer may, however, be useful as an ancillary tool for checking consistency of shallow surface layers and detecting local shallow 'soft spots' between other more widely spaced CPT and SPT soundings, etc. The Scala is also useful for other purposes such as quality control of engineered fills.

5.3.2 SEISMIC DESIGN PROCEDURE

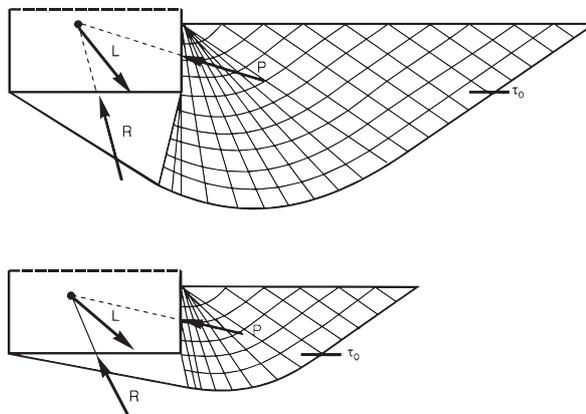
The seismic design procedure is adapted from the gravity design procedure of NZS 1170.0:2002 as follows:

- 1 **Soil and site effects** — Care must be taken to identify any shallow soils, within the influence zone of the footings that may liquefy, generate significant pore water pressures, or suffer cyclic softening. The simplified approach to shallow foundation design is not appropriate for these situations.
- 2 **SLS settlement check** — Settlement under the SLS earthquake loads should be estimated including the pre-earthquake gravity settlements, site settlements caused by the shaking, and additional settlements from the building earthquake load case.
- 3 **Lateral load path** — The load path and available resistance for the building lateral loads (base shear) needs to be considered, including the consequences of permitting lateral sliding if insufficient resistance is available. The passive lateral resistance of all buried foundation elements and other down-stands with sufficient structural capacity may be included in the calculation. Where the available passive resistance is less than the base shear, then friction will be mobilised on the underside of the footings requiring load inclination to be considered in the bearing capacity calculations.

Comment

The interaction between passive soil pressure acting against the sides of embedded shallow footings and the bearing capacity of such footings was considered in detail by Brinch-Hansen [1970]. He demonstrated (see Figure 5.4.) that the passive earth pressure acting against the side of a footing and the bearing capacity of the same footing under combined vertical and lateral loads are one and the same mechanism of limiting equilibrium. He recommended that the passive force against the side of the footing (P in the figure) could be simply subtracted from the total force applied to the footing (L) prior to calculating the bearing capacity in the normal way (R).

Figure 5.4: Rupture surfaces for inclined footing loads [Source: Brinch-Hansen, 1970]



Comment

Resistance to lateral seismic loading is not necessarily critical to the safe performance of buildings in all cases. Lateral deformations are 'self-limiting' in the sense that lateral acceleration pulses are of short duration and act in both (all) directions. (But be aware that buildings located on slopes or which are retaining soil will be subject to a down-slope ratchetting effect and much larger permanent offsets.)

- 4 **ULS bearing capacity check** — The ULS load combinations applied to the footings for the earthquake load case may be significantly larger than for the gravity load case and govern the sizing of the footing. The *design capacity*, E_d , is calculated as $\Phi_g R$ and may differ from the gravity load case because of the effects of load inclination, (where lateral loads exceed the available passive resistance) and moment loading (eccentricity).

The effect of soil layering on the bearing capacity of shallow footings needs careful consideration, especially where underlying layers are likely to soften or liquefy with earthquake shaking. The possibility of a 'punch through' failure into an underlying layer of softened or liquefied soil should be considered.

In cases where significant moments are applied to footings in addition to vertical and lateral loads, the above approach may be un-conservative. With application of increasingly large moments to a footing (ie load eccentricity), the effective width of the footing decreases rapidly to a point where a small increase in moment may cause a disproportionate

decrease in vertical bearing capacity. The true factor of safety against a bearing failure in such cases may be much less than intended or would be acceptable. Pender [2017] provides a more realistic approach for checking the ultimate bearing capacity of footings with large applied moments.

5.4 Foundation beams

Foundation beams are differentiated from strip footings by having significant bending strength and stiffness and will usually be doubly reinforced for seismic conditions.

Foundation beams are used to span between individual building columns along a grid and may contribute to frame action of the building structure. The design of foundation beams is controlled, mostly, by structural capacity (ie bending and shear). Foundation beams may be analysed by

treating the soil as an elastic half-space or as a series of Winkler springs (so called 'beam on elastic foundation' analysis).

The analysis and design of foundation beams generally follows the same procedure as for mat foundations, as described in the following section.

5.5 Design of mats and rafts

Mat foundations are useful for cases where individual footings would cover a large proportion of the building footprint, where the subsurface soils are variable or contain pockets of compressible, weak, or even liquefiable soils, or if resistance to hydrostatic uplift is required (eg basements).

Mat foundations are useful for distributing concentrated column and wall loads and equalizing differential settlements on variable ground conditions.

Raft foundations are a special case of mat foundations that are sufficiently stiff and strong to distribute the entire superstructure load uniformly across the base and to behave as a rigid unit (rare, in practice).

Mat and raft foundations must satisfy the same requirements as other foundation types, ie have structural integrity, not settle excessively, and have an adequate factor of safety against instability from shear failure of the soil (ie bearing failure). Of these requirements, structural considerations of strength and stiffness will most often govern design. Bearing failure is unlikely unless the mat or raft is very heavily loaded, of narrow width, is built on

weak cohesive soil (eg Figure 5.2), is subject to high overturning moments, or is built on a shallow crust overlying a liquefiable soil. Where heavy loads are applied close to the edge of the mat (eg heavy wall loads) a bearing failure underneath the edge of the mat and consequent structural failure are possible.

Care is required for cases where mats are founded on soils susceptible to pore water pressure generation during earthquake shaking or cyclic softening effects. Soil structure interaction effects near to the edges of mats have been shown to significantly increase pore water pressure generation and to greatly reduce the factor of safety against liquefaction, F_L . These effects may be very severe for tall/heavy buildings even leading to instability (eg Figure 5.5). (See Section 4.9.2 for more discussion of building induced settlements, also Bray and Dashti, 2014 and Bray and Macedo 2017).

5.5.1 GRAVITY DESIGN PROCEDURES

Gravity design of mats is usually carried out using elastic analysis assuming that the mat behaves as an elastic plate and the soil behaves as an array of individual springs (*Winkler spring analysis*). Analysis using computer methods or published elastic solutions (*beam on elastic foundation*) provides the required bending moments and shear forces to design the mat. The required input to Winkler spring analysis is the *coefficient of subgrade reaction* (ie the spring stiffness) for the soil, usually obtained from semi-empirical correlations with in situ test data. More sophisticated analysis using non-linear springs or finite element methods with non-linear soil models may also be used but requires additional soil parameters as inputs.

Gravity design of *rafts* may be simplified by making the assumption that the raft acts as a rigid body. The soil contact pressure may then be assumed to have a planar distribution and the raft design carried out using statics. The centroid of the contact pressure is assumed to coincide with the line of action of all loads acting on the raft.

Selection of appropriate values of stiffness for the soil (k_s or E_s) is the main source of uncertainty for these analyses and should be determined from direct on-site measurement (eg plate load test), correlations with in situ test data, and laboratory data as appropriate. See Section 2.1 and Module 2 for more information. The soil spring stiffness needs to include the effects of short term and long term soil compliance (ie $S_i + S_c + S_s$).

Under NZS 1170.0:2002, the following design checks should be carried out for the gravity load case:

- › Settlement check under the SLS load combination
 - › Strength design of the mat (bending and shear) under the ULS load combinations
 - › Stability check of the whole mat against deep seated bearing failure of the soil under the ULS load combinations (unlikely to occur unless the mat is of narrow width, founded on weak cohesive soil, founded above a thick layer of liquefied soil, heavily loaded, or subject to high overturning moment).
- 1 **Estimation of settlement** — The settlement check under the SLS load combination should be carried out using the same procedures as for pad and strip footings (Section 5.3).
 - 2 **Structural design of the mat** — Usually carried out by means of a beam on elastic foundation analysis using Winkler springs.
 - 3 **ULS Bearing capacity check** — Usually a bearing capacity calculation would not be carried out unless one or more of the risk factors listed above were identified. If necessary, the calculation would be made using Equation 5-1 with a resistance factor selected from Table 5.1.
- For mats with heavily loaded walls located close to an edge, the possibility of a structural failure of the edge of the mat and bearing capacity failure of the underlying soil should be checked.

5.5.2 SEISMIC DESIGN PROCEDURE

The seismic design of mat and raft foundations should be carried out in the context of the above gravity design procedures while also considering the various issues affecting the performance of shallow foundations with earthquake shaking. The following general approach is recommended:

- 1 **Soil and site effects** — Care must be taken to identify any shallow soils that may liquefy, generate significant pore water pressures, or suffer cyclic softening. This simplified approach to shallow foundation design is not appropriate for these situations and assumes that there is a 'raft' of adequate strength and stiffness to support the mat and to mitigate the effects of any underlying liquefiable soils.
- 2 **SLS settlement check** — Settlement under the SLS earthquake loads should be estimated including the pre-earthquake gravity settlements, site settlements caused by the shaking (see Section 3.8), and additional settlements from the earthquake induced foundation loads.
- 3 **Lateral Load Path** — The load path and available resistance for the building lateral loads (base shear) needs to be considered, including the consequences of permitting lateral sliding if insufficient resistance is available. The passive lateral resistance of all buried foundation elements and other down-stands with sufficient structural capacity may be included in the calculation.

Comment

It may not be practical to prevent buildings on mat foundations from sliding at the ULS level of shaking (and may not be critical to the safe performance of the building, as discussed in Section 5.4.2). Sliding at the SLS would be less tolerable because of damage to building service connections and access ways. Sliding is likely to increase the risk of damaging differential settlements and should be avoided for the SLS level earthquake. Where considered necessary, base sliding resistance may be increased by providing additional down-stands such as ribs to increase the total passive soil resistance.

- 4 **ULS strength check** — The required bending and shear strength of the mat for the ULS earthquake load case should be assessed using the same method of analysis as the gravity load case (ie elastic analysis), but using soil stiffness parameters adjusted for the effects of shaking.

The mat must have sufficient bending strength to be able to bridge over (or cantilever over) pockets of shallow liquefaction where such a risk has been identified for the site.

Comment

The MBIE Guidelines for repairing and rebuilding houses affected by the Canterbury earthquakes [2013] stipulate that all mat foundations at sites with a risk of significant liquefaction should be capable of spanning 4 m internally or cantilever 2 m at an edge.

Where the mat or raft is constructed above a liquefiable layer at depth below the founding stratum there is a risk of high fluid pressure being injected against the underside of the mat or raft. Such pressures may be very high, especially where the mat or raft is below grade (eg basement) and approximately equal to the total vertical stress at the depth of the liquefaction. Drainage measures should not be relied on because they will typically become permanently clogged after a liquefaction event. The mat should have sufficient strength to resist such pressures.

- 5 **ULS bearing capacity check** — A bearing capacity calculation need not be carried out unless one or more risk factors are identified. If necessary, the calculation would be made using Equation 5-1, with a resistance factor selected from Table 5.1.

For mats with heavily loaded walls located close to an edge, the possibility of a structural failure of the edge of the mat and bearing capacity of the underlying soil should be checked.

Cases with a high overturning moment (ie relative to the width of the mat or raft) need to be considered carefully. With application of increasingly large moments (ie load eccentricity), the effective width of the mat decreases rapidly to a point where a small increase in overturning moment may cause a disproportionate decrease in vertical bearing capacity and increase in the risk of instability (eg Figure 5.5). The true factor of safety against a bearing failure in such cases may be much less than intended. Pender [2017] provides a more realistic approach for checking the ultimate bearing capacity of shallow footings (including mats and rafts) with large applied moments.

Care must be taken on layered soils when considering the ultimate bearing capacity of mat and raft foundations because the limiting equilibrium failure surface extends to a significant depth below the surface approximately equal in width to the mat or raft. The soil conditions may vary considerably over such depth and include layers of weak soil or even liquefiable soil. Where a layer of liquefiable soil is located beneath the mat and supporting 'raft' of non-liquefied crust, the possibility of a 'punch through' failure into the liquefied soil should be considered.

The effect of soil-structure interaction effects near to the edges of mat foundations increasing the pore water pressure generation or even liquefying the adjacent soil needs careful consideration (see Section 4.9.2 and Bray and Dashti 2014). These effects may significantly increase the risk of instability, especially for tall/heavy buildings. In such situations, tall/heavy buildings should be founded on deep foundations.

Figure 5.5: Instability of a tall building on a raft foundation during the 1999 Kocaeli earthquake



6 Deep foundation design

This section outlines a simplified approach for practical design of deep foundations with earthquake loading. The intention is to provide guidance that is compatible with AS/NZS 1170.0:2002 including strength based, ultimate limit state, design procedures and estimation of settlement for the serviceability limit state.

This section is not intended to provide a fully comprehensive treatment of the design of deep foundations under gravity loading in all situations and soil conditions. Instead, well known published handbooks should be consulted for much additional guidance on the detailed design and construction of deep foundations (see Section 1 for recommendations). The intention in this section is to provide supplementary guidance on earthquake design aspects that are not well covered elsewhere.

Deep foundations may provide a good foundation for buildings at sites with poor soil conditions near the ground surface that are considered unsuitable for shallow foundations. This is because they can transfer loads to deeper soil layers that are usually stronger, denser, older, and more resistant to liquefaction. They can also resist vertical uplift loads where required.

Observation from previous earthquakes (eg Christchurch, Cubrinovski and McCahon, 2011) has shown that, even in areas of severe liquefaction, deep pile foundations have performed well where these were founded into competent soil layers at depth. However, there have been many cases where deep pile foundations were seriously damaged by lateral movements of the ground surface, especially from lateral spreading near to waterfront features. Excessive settlement

from 'down drag' with liquefaction has also been observed (eg Turner et al 2014).

Deep foundations are susceptible to lateral movements of the ground during earthquakes and other effects related to liquefaction of site soils. The following issues affecting deep foundations need careful assessment:

- › Loss of side resistance from increased pore water pressure, including liquefaction, in susceptible soil layers
- › Down-drag caused by post-liquefaction reconsolidation within liquefied layers and from settlement of competent layers located above liquefying layers
- › 'Punch through' into liquefied or softened soil beneath the pile base where the bearing layer is of inadequate thickness
- › Increased axial loading from the dynamic response of the building
- › Lateral and moment loading from the dynamic response of the building
- › Kinematic interaction between embedded piles and deformations of the soil mass, especially from lateral movement of crust overlying liquefied soil layers
- › Buckling of slender piles within thick layers of liquefied soil



The side resistance of deep foundations may be reduced during and immediately after an earthquake because of increases in pore water pressure in susceptible soil layers. Reduction in side resistance capacity (at the same time that axial load demand is increased because of building dynamic response) may result in transfer of axial loads to the end bearing resistance mechanism of the pile and probably represents the main mechanism of earthquake induced settlement for deep pile foundations. It is critical that the bearing layer itself in which the pile base is embedded is not susceptible to significant pore water pressure increase or cyclic softening and that there is no risk of 'punch through' into underlying liquefied or softened layers.

Building dynamic response to earthquake shaking may significantly increase the axial loads applied to the foundations, and may result in uplift loads, and will usually be the critical load case. Deep foundations need to be sized to resist these loads so as to prevent excessive deformations that would be intolerable for the building.

Building dynamic response includes lateral loads and moments applied to the head of deep pile foundations. These may damage piles and jeopardise their ability to safely carry the building vertical loads.

Where a deep pile passes through liquefying soil layers above the pile base, then contraction, re-consolidation, and settlement of the liquefying layer may result in 'down-drag' from side resistance acting in a downwards direction, especially where competent soil layers overlie the liquefying layers.

Kinematic effects are caused by lateral ground deformations from vertically propagating shear waves and surface waves. Embedded piles must either conform to these deformations (if sufficiently flexible) or resist deformation and react to the resulting soil pressures. With soil liquefaction, lateral ground deformations are more extreme and potentially damaging to embedded piles. Large shear deformations may occur in liquefied layers and the high contrast in stiffness between liquefied and non-liquefied layers causes high localised curvatures and bending strains in piles.

The most damaging kinematic effects are where a non-liquefied surface crust is subject to cyclic oscillations and lateral spreading relative to the underlying bearing stratum. With significant soil liquefaction, permanent lateral movements of the surface crust may be widespread and vary

from extreme (severe lateral spreading near to water courses) to subtle but potentially damaging widespread movements of the surface crust caused by minor surface gradients [eg Cubrinovski et al 2009].

Even where soil liquefaction does not occur, kinematic effects can still be significant adjacent to steep slopes such as waterfronts, bridge abutments, etc. where permanent lateral deformations are likely after strong earthquake shaking.

Buckling of piles has been reported where slender, heavily loaded piles penetrated through thick layers of liquefied soil [eg Battacharya and Bolton, 2004].

The simplified approach adopted in these guidelines, to design of piles to resist vertical (axial) loads, is based on observations from previous earthquakes. The observations showed that deep pile foundations have generally performed satisfactorily where they were founded into competent soil layers at depth, even where severe liquefaction occurred at the site (except for cases where piles were damaged by large lateral movements and kinematic effects). The foundations of the buildings observed to perform well were (presumably) designed using normal, familiar LRFD procedures for (quasi) static loads and normal resistance factors, the same approach that is recommended for use here.

Building dynamic loads may be much greater than gravity loads, depending on the aspect ratio and structural configuration of the building, and deep foundations should be sized accordingly. The structural engineer would normally calculate load sets for each foundation for the critical load combinations specified in NZS 1170.0:2002 following analysis of the building response (see Section 3.7 for more discussion of LRFD design for earthquake loading). The foundation load sets for the earthquake load case may be treated as 'equivalent static' loads and the deep foundations sized using normal static design procedures.

The possibly significant effects of lateral and moment loading and kinematic loading need careful consideration in all cases where deep foundations penetrate through layers of weak soils (ie almost all situations where deep foundations are used). A simplified, quasi-static approach is provided herein for assessing the resulting strains.

For a discussion of alternative design approaches using principles of performance based design refer to Section 3.9.

6.1 Types of deep foundations

Deep foundations are available in a wide range of types and are the subject of continual innovation. Each type has different advantages and disadvantages making them more or less suitable for use in earthquake resistant design. The types most commonly used in New Zealand practice are discussed:

6.1.1 DRIVEN PILES

Driven piles (treated timber, precast concrete, steel tubes, steel H-piles) have a significant advantage over other pile types for seismic design because the driving process pre-loads the base of the pile within the target bearing layer, potentially reducing the effects of down-drag and load re-distribution if there is liquefaction within soil layers above the bearing layer.

Driven displacement piles have the additional benefit of densifying loose soils where driven at reasonably close spacing, potentially reducing the risk of liquefaction.

Where jetting, pre-drilling, and vibrating hammers are used as aids to the installation of piles through intermediate stiff strata and to reduce noise and vibration, these procedures should not be used to penetrate into the target bearing layer. Otherwise the full bearing capacity will not be achieved. Penetration of the pile into the bearing layer should be achieved using a suitable drop hammer (or hydraulic drop hammer).

More specific comments on some common types of driven piles are as follows:

- › **Timber piles** — Suitably treated timber poles are economical, easily handled on site and are resilient to driving stresses and to lateral ground movements. Timber poles are available in a limited range of lengths (up to about 15 m) and diameters but are able to be spliced where greater lengths are required (Note: splicing may not be appropriate where significant kinematic effects are expected). They have limited ability to penetrate intermediate dense layers and may require jetting or pre-drilling in such cases.
- › **Pre-cast concrete piles** — Can be manufactured economically to a wide range of desired lengths (up to about 40 m when pre-stressed). Concrete piles need to be specially detailed for ductility where significant lateral ground movements are expected (kinematic effects).

- › **Steel tube piles** — Available in a wide range of diameters and weights and can be fabricated in any length or extended on site by welding. They can be driven either open ended (to ease penetration through hard layers) or closed ended. They are highly ductile (especially when concrete filled) and resistant to lateral ground movements (kinematic effects).
- › **Steel H-piles** — Are more expensive than other pile types but are readily available in a range of sizes and stock lengths (9 to 18 m). They have the advantage of being relatively easy to drive through intermediate hard layers compared to other pile types. They have less side resistance than other pile types meaning that they will pick up less down-drag from the overlying soil crust. However, they have less end bearing resistance than other pile types and are more suited to sites with a very dense or thick bearing layer (or rock). They may be more susceptible to buckling within liquefied soil layers than other pile types.

6.1.2 BORED PILES

Bored piles have certain advantages over driven piles including a greater ability to penetrate difficult intermediate layers to achieve a desired target depth, provide the ability to observe and confirm the properties of the soil layers during construction, and can be made in large diameters.

Bored piles have some disadvantages over driven piles for sites where soil liquefaction is likely in intermediate layers above the pile base. Bored piles obtain most of their initial axial load capacity from side resistance, which is a much stiffer load transfer mechanism than end bearing, and at the end of construction most of the building weight will be carried by the side resistance mechanism. With liquefaction of intermediate layers, much of the side resistance may be lost resulting in a significant transfer of load to the base of the pile. Mobilisation of the end bearing mechanism requires significant settlement to take place, typically from 5 to 10 percent of pile diameter. Additional settlements may occur where the base of the pile was excessively disturbed during construction or poorly cleaned out.

To counteract these effects, special construction techniques may be used at additional cost. It is possible to pre-load the base of bored piles using pressure grouting techniques or special devices. Excavating bored piles using bentonite slurry is known to reduce side resistance and permanent sleeves may be installed to reduce load transfer to side resistance.

Installation of deep bored piles may be complicated by the presence of artesian ground water pressures within a target gravel bearing layer (eg Christchurch City, Hutt Valley).

6.1.3 BELLED PILES

Belled piles are bored piles with an enlarged diameter base, used originally to improve bearing capacity in dense soil layers. In local practice (notably Wellington) belled piles have been used to improve the uplift capacity of bored piles. The upper surface of the bell is considered to act as an 'upside down footing' and treated as such for the calculation of capacity using bearing capacity formulae. However, the mobilisation of the bearing mechanism in soil, upwards or downwards, requires significant movement of the pile (5 to 10 percent diameter in each direction) and may result in a very soft load-displacement response, especially if gapping develops. The resulting structure response may be more like foundation rocking and possibly quite different to that intended by the designer. Foundation displacement, both upwards and downwards is likely to govern design in such cases.

For belled piles, an 'upside-down' punching shear failure is likely where weak or liquefied soil overlies the founding stratum in which the bell is embedded. Significant penetration into the bearing layer may be necessary to develop the maximum uplift capacity of the bell.

In overseas practice, belled piles are used infrequently because, with modern drilling equipment, it is considered preferable to instead use larger diameter piles drilled deeper, with unit side resistance increasing rapidly with depth. Such piles should also provide a stiffer response under seismic loading.

6.1.4 CFA PILES

CFA (continuous flight auger) piles are bored piles installed using a hollow stemmed auger which eliminates the need for ground support in caving conditions. They have most of the same advantages and disadvantages as bored piles but are more limited in diameter and depth range and have more limited ability to penetrate difficult intermediate layers. CFA piles pre-load the pile base to a limited extent by the injection of concrete under pressure during installation, and may simplify control of artesian ground water pressures.

6.1.5 SCREW PILES

Screw piles consist typically of one or more steel plate helixes welded to a steel tube. The pile is screwed into the ground and then the tube may be filled with concrete. Torque measurements are used to identify penetration into the target bearing layer. These piles have an advantage for seismic loading in some situations because almost all of the load is transferred to end bearing on the steel helixes embedded into a target bearing layer, with minimal side resistance along the shaft to pick up down-drag loads.

Screw piles also have the advantage that the typical concrete filled steel tube forming the pile stem is highly ductile and may be able to tolerate significant lateral ground deformations. Buckling of the stem may be a risk where screw piles with slender stems penetrate through thick liquefiable layers.

Screw piles share many of the same issues as belled piles because they also rely on a bearing mechanism both in compression and uplift. The load-displacement response under compression-uplift cycling during an earthquake is likely to be soft and may govern design.

6.2 Gravity design procedures

Gravity design procedures for deep foundations include:

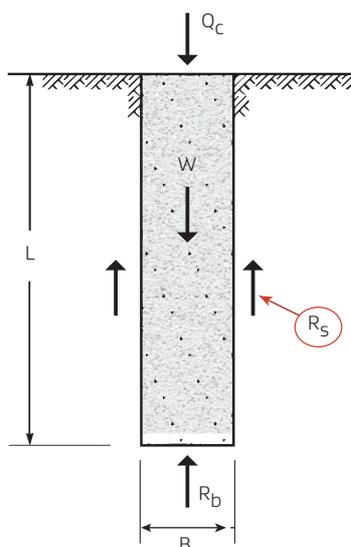
- › Calculation of pile capacity and settlement using soil parameters derived from laboratory or in situ test data
- › Direct correlation of pile capacity with SPT and CPT test data
- › Wave equation analysis (for driven piles)
- › Static load testing
- › Dynamic load testing (for driven piles)

The resistance of deep foundations to axial load in compression is illustrated in Figure 6.1. The total resistance Q_c is made up of two separate components:

- › Side resistance, R_s , and
- › Base resistance, R_b .

Each of these components has a distinct and contrasting load-displacement response as illustrated in Figure 6.2. The side resistance typically shows a peak and is mobilised at a relatively small displacement (approximately 12 mm, FHWA 2010), while the base resistance typically does not show a peak and is mobilised at a much larger displacement, typically 5 percent of diameter B for cohesive soils and 10 percent of diameter B in cohesionless soils.

Figure 6.1: Axial load resistance mechanisms of a deep foundation [Source FHWA 2010]

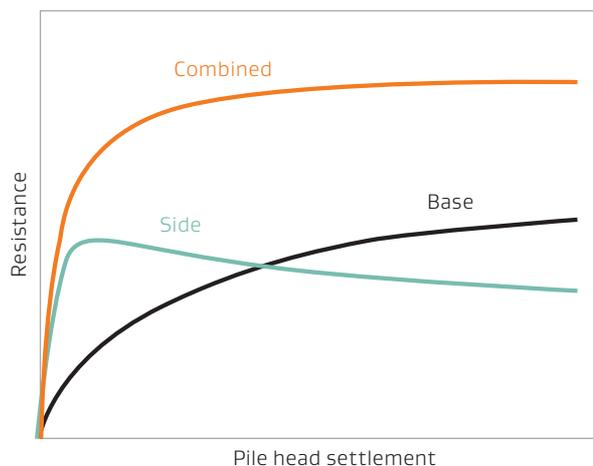


In uplift, only the side resistance mechanism is considered (side resistance in uplift is usually assumed to be the same as side resistance in compression). Any contribution from tension acting under the base (ie suction, negative pore water pressure) is usually neglected as being transient.

Unlike shallow foundations, the gravity design of deep foundations is usually controlled by the ULS case (ie geotechnical capacity). In practice, the geotechnical capacity is usually defined in terms of a limiting settlement (eg AS2159-2009, capacity is defined at $s = 10 \text{ mm} + 0.05 \text{ diameter} + \text{elastic shortening of pile stem}$) and so settlement is seldom found to be an issue even at the SLS. In many cases SLS loads will not exceed the side resistance of the pile and so the settlement would not be expected to exceed about 12 mm, well within the serviceability criteria for most buildings. The main exceptions are long, slender piles in soft soils, large diameter belled piles and screw piles that rely mainly on end bearing.

Many different design methods are available in the literature and tend to be specific to particular types of piles, especially for proprietary designs such as screw piles.

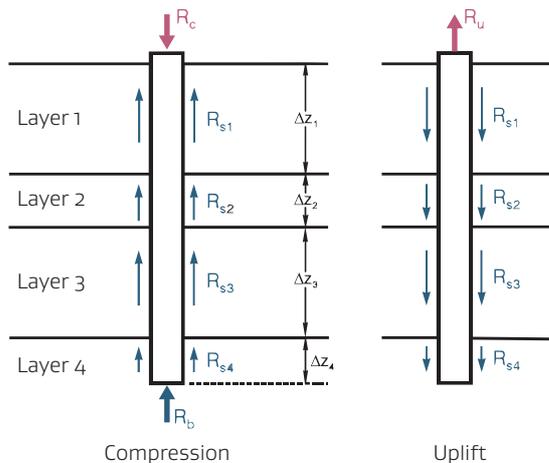
Figure 6.2: Generalised load-displacement response of a deep foundation [Adapted from FHWA 2010]



The following is a generalised approach that is suitable for adaptation to seismic design:

- 1 **Geotechnical soil model** — The geotechnical soil model for pile design is derived from the geotechnical site investigation and site model (see Section 2 and also Module 2 of the *Guidelines*). The soil profile through which the pile is to be installed is divided into layers (see Figure 6.3) to aid calculation of side resistance and end bearing. Layering may be simplified to a level of practical detail with some grouping of similar soil types and averaging of properties. The soil model should extend to a depth of at least five pile diameters below the base of the pile, otherwise there is a risk that an underlying weak layer may reduce the end bearing capacity of the pile. (For pile groups, the depth should be increased to be at least the width of the pile group.)

Figure 6.3: Geotechnical soil model and calculation of ultimate compression and uplift capacity for a pile [adapted from FHWA, 2010]



- 2 **Calculation of ULS axial capacity** — The ultimate axial compression capacity of the pile, R_c , is calculated as the sum of the base resistance plus the side resistance of each layer:

$$R_c = R_b + \sum R_{si} \quad (6-1)$$

The design capacity, R_d , is then calculated as:

$$R_d = \Phi_g R_c \quad (6-2)$$

where Φ_g = geotechnical strength reduction factor (resistance factor) from AS2159-2009.

Comment

The self-weight of the pile, W , is included as part of the building self-weight and included with the factored loads.

The selection of values of unit side resistance and unit base resistance depends significantly on the soil properties, pile type, and method of pile installation. Well established, published methods should be used for making these calculations (see Section 1.2 for recommended handbooks) using soil strength parameters derived from laboratory testing, in situ test results, or by direct correlation with SPT and CPT results.

Comment

For cases of layered soils, piles should be embedded at least two pile diameters into the bearing layer and should be terminated at least three pile diameters above weaker underlying layers or the full base resistance will not develop. The possibility of 'punch through' failure should be considered where weaker layers underlie the bearing layer.

The ultimate axial uplift capacity for the pile, R_u , is calculated as:

$$R_u = \sum R_{si} \quad (6-3)$$

the sum of the side resistance of each layer without considering any end resistance, and

$$R_d = \Phi_g R_u \quad (6-4)$$

Comment

The value of Φ_g adopted for the uplift case may be different from that used for the compression load case because the risk factors may be different. Refer to AS 2159-2009.

- 3 **SLS settlement check** — For relatively short, stiff piles a simple estimate of settlement may be made with reference to Figure 6.2. The full side resistance may be assumed to be fully mobilised at 12 mm displacement. For longer, more flexible piles, it is necessary to account for elastic shortening of the pile under the applied load. A more detailed analysis may be carried out by dividing the pile into segments (eg 't-z' method, as implemented in various commercial software packages) and considering separately the elastic properties of the pile, the pile/soil interface, and the pile base response.
- 4 **Driveability check** — For driven piles, it is necessary to consider whether it will be practical to drive the pile to the target design depth using available equipment, especially where there are strong intermediate layers to penetrate. Driveability is usually checked using wave equation analysis and published software. (For simple projects pile driving formulae may be used but these are considered much less reliable.)
- 5 **Load tests** — Load testing of test piles or production piles is often carried out for more significant projects to refine and economise the preliminary pile design. Dynamic load testing combined with wave equation analysis (eg CAPWAP) is routinely used to prove the capacity of driven piles. Load testing of bored piles is less routine because dynamic load testing is impractical in most cases and static load tests may be time consuming and expensive. The objective in all cases is to gain greater confidence in the calculated pile capacity so that a higher value of Φ_g can be used resulting in smaller and/or fewer piles.

Comment

- › For cases where the earthquake load combination governs design of a pile, load test results will not be relevant, or will need careful adjustment to account for layers where pore water pressure increase, cyclic softening, or 'down-drag' effects are expected. AS 2159-2009 provides useful guidance for assessing pile capacities with down-drag.
- › Where two or more piles are placed close to each other in a group, interaction effects between the piles need to be considered. Groups of piles placed relatively closely together may fail together as a 'block' at a combined load that is less than the sum of the individual pile capacities, either in uplift or compression. The possibility of 'block' failure needs to be considered in such cases (see AS 2159-2009). For further guidance refer to the handbooks recommended in Section 1.2).

Note: even in the case of two piles located close together, interaction effects will increase the settlement of each individual pile.

6.3 Seismic design procedures

As for gravity design, seismic design of deep foundations under NZS 1170.0:2002 must consider both ultimate bearing capacity (strength) for the ULS case and settlement (serviceability) for the SLS case.

The axial loads from the earthquake load case will usually be significantly higher than the gravity loads because of the building dynamic response, while the capacity may be reduced because of the effects of pore water pressure generation in susceptible layers including liquefaction and possible down-drag. Additional settlements are difficult to avoid in such cases and need to be considered.

With earthquake loading, deep piles may be required to resist lateral loads from the building dynamic response (base shear) and from the kinematic effects of ground deformations. Axial load resistance of deep piles is generally not affected by simultaneous lateral loading and the two cases are considered separately in the following procedure.

6.3.1 AXIAL LOAD CAPACITY AND SETTLEMENT

The following approach has been adapted from the generalised approach to gravity design of deep foundations of Section 6.3:

- 1 **Soil and site effects** — The effects of pore water pressure generation, liquefaction, and cyclic softening may significantly reduce the side resistance of deep piles and the presence of these soils needs to be carefully identified in the geotechnical model for the site.

Sites with significant issues of lateral spreading or other types of serious instability may not be suitable for deep foundations (see Section 4 for guidance on site assessment and foundation selection).

- 2 **Calculation of ULS axial capacity** —

When evaluating the axial *compression* capacity of the pile using Equation 6-1, values for unit side resistance and unit end bearing should be reduced for layers where pore water pressure generation or cyclic softening is expected. For cohesionless soils the effective stress and thus friction may be reduced by a factor of $1/(1 - R_u)$ (Boulanger and Brandenburg, 2004). R_u may be evaluated from Figure 4.4. For clay soils, a reduction in undrained shear strength S_u may be estimated using the procedure of Boulanger and Idriss (2007).

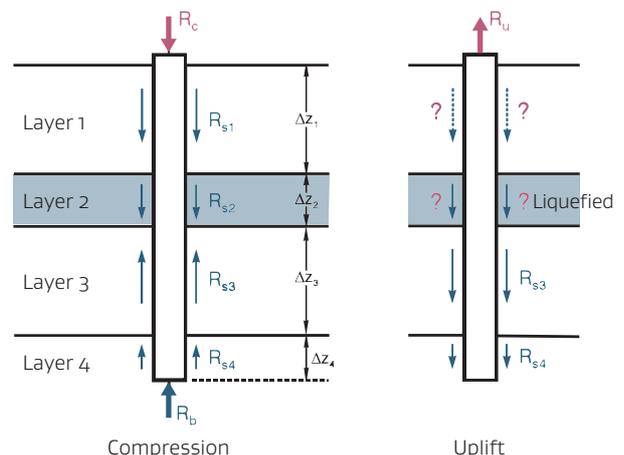
Down drag (ie side resistance acting in a downwards direction) is likely from post-liquefaction consolidation of liquefied layers and from settlement of non-liquefied layers above a liquefied layer (see Figure 6.4) and should be subtracted from the calculation of axial compression capacity (eg $R_C = R_{s3} + R_{s4} + R_b - R_{s1} - R_{s2}$ in Figure 6.4). The liquefied soil is considered to provide side resistance equivalent to its *post-liquefaction residual undrained strength*, $S_{u,r}$, multiplied by an adhesion factor, α , of 0.5 to 1.0 [Turner et al 2014].

Uplift capacity is also calculated with reference to Figure 6.4, with the side resistance of all layers acting downwards. The side resistance from the liquefied soil layer and from soil layers above the liquefied layer should be assumed to contribute zero resistance because of the risk of high pressure fluids migrating along the pile/soil interface to the ground surface (eg $R_U = R_{s3} + R_{s4}$ in Figure 6.4).

Comment

The treatment of Layers 1 and 2 in the example of Figure 6.4 is apparently contradictory between the compression and uplift cases, but appropriately conservative given the large uncertainty in the effects of liquefaction in such cases.

Figure 6.4: Calculation of ultimate compression capacity and uplift capacity for a pile with liquefaction above the base



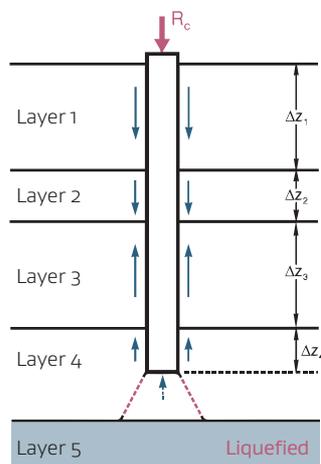
For cases where there is a liquefiable soil layer located below the base of the pile, the end bearing capacity of the pile may be reduced by 'punch through' failure (see Figure 6.5).

For cases where the direction of loading reverses from compression to uplift and back to compression with repeated cycles during an earthquake, there may be a significant degradation in the side resistance component of capacity. Further information including guidance for assessment of cyclic degradation is given in McManus 2003.

- 3 **SLS settlement check** — For cases where the SLS axial load is able to be resisted entirely by the mechanism of side resistance, settlements should remain small (ie less than 12 mm, FHWA 2010). For cases where the mechanism of side resistance is likely to be fully mobilised, and where down-drag is likely, then some transfer of load to the pile base will occur causing partial mobilisation of the end bearing mechanism of the pile. Additional pile settlement is required for mobilisation and might be significant, especially for bored piles. A crude estimate of the additional settlement may be made by assuming that the calculated end bearing capacity of the pile will be fully mobilised at settlements of between 5 and 10 percent of pile diameter, depending on the assumptions of the calculation procedure used to determine capacity.

Additional settlements may occur because of densification and contraction of loose, cohesionless soils underneath the bearing layer (including liquefied layers) and these should be estimated and added to the total settlement.

Figure 6.5: Calculation of ultimate compression capacity for a pile with 'punch through' into a liquefied layer below the bearing layer



- 4 **ULS settlement check** — A similar crude check of settlement may also be made for the ULS load case, especially where significant liquefaction and down-drag are expected. However, where the pile is sized using the recommended procedure and resistance factors from AS 2159-2009 are applied, then settlements should remain within tolerable limits.
- 5 **Pile buckling** — The strength and stiffness of liquefied layers may be reduced to very low values and the soil may provide inadequate stabilising support to slender piles through thick liquefied layers resulting in pile buckling. Further information is given by Battacharya and Bolton [2004].

6.3.2 LATERAL LOAD RESISTANCE AND KINEMATIC EFFECTS

Building dynamic response to earthquake shaking causes inertial lateral loads (base shear) and moments to be applied to the foundations. Lateral loads and moments are resisted by passive soil pressure acting against embedded foundation elements including piles, pile heads, foundation beams, and other structural features such as basement walls and lift pits etc.

Deep foundations are also subject to lateral loading from surrounding soils or kinematic interaction from induced ground deformations. Site response to earthquake shaking may cause amplification of the arriving shear waves and surface waves near the ground surface resulting in shear deformations of the ground which lead to interactions between embedded piles and surrounding soil (eg see Figure 6.6). Generally, such ground deformations are not damaging for well-designed piles except in the case of soft soils, liquefied soils, localised severe ground deformation, and/or large relative displacements between soil and stiff piles caused by severe earthquake excitations.

Liquefaction is an extreme case of soil softening and kinematic effects will often be important for piles which pass through a thick liquefying layer as large oscillations of the surface non-liquefied crust may develop and, in some cases, lateral spreading may also occur (see Figure 6.6). In such cases the kinematic effects may be very significant and cause structural damage to the piles. In the extreme, a pile may be so severely damaged that it is unable to continue to carry the weight of the building. Such damage is typically concentrated at interfaces between a thick liquefied layer and non-liquefied layers above and beneath it.

Liquefaction does not occur instantaneously at the start of earthquake shaking but develops progressively during shaking as pore water pressures are generated in susceptible soils. Three phases are typically recognised in the evolution of the liquefaction response, and each phase requires separate consideration (all phases will not necessarily develop at every site or for every earthquake excitation):

› **Phase A: Prior to liquefaction triggering (or liquefaction does not occur)**

In this phase, the building is responding dynamically to the earthquake shaking and the resulting inertial loads (base shear) are applied to the foundations. The available resistance to lateral load should be calculated including passive soil pressure acting against the embedded structural elements and the lateral load capacity of the piles. If there is insufficient resistance to the inertial loads, then excessive lateral movement of the building may occur with damaging consequences for the piles.

› **Phase B: After liquefaction triggering**

In this phase, liquefaction has triggered in susceptible layers with resulting significant reduction in soil stiffness and strength. Other soil layers may also be softened by elevated pore water pressures or cyclic softening effects. Cyclic oscillations of the ground surface may develop and become large where there are thick layers of liquefied soil, resulting in significant kinematic interactions with deep piles. The building dynamic response may also be significantly altered by softening of the foundation stiffness and changes in the ground motion.

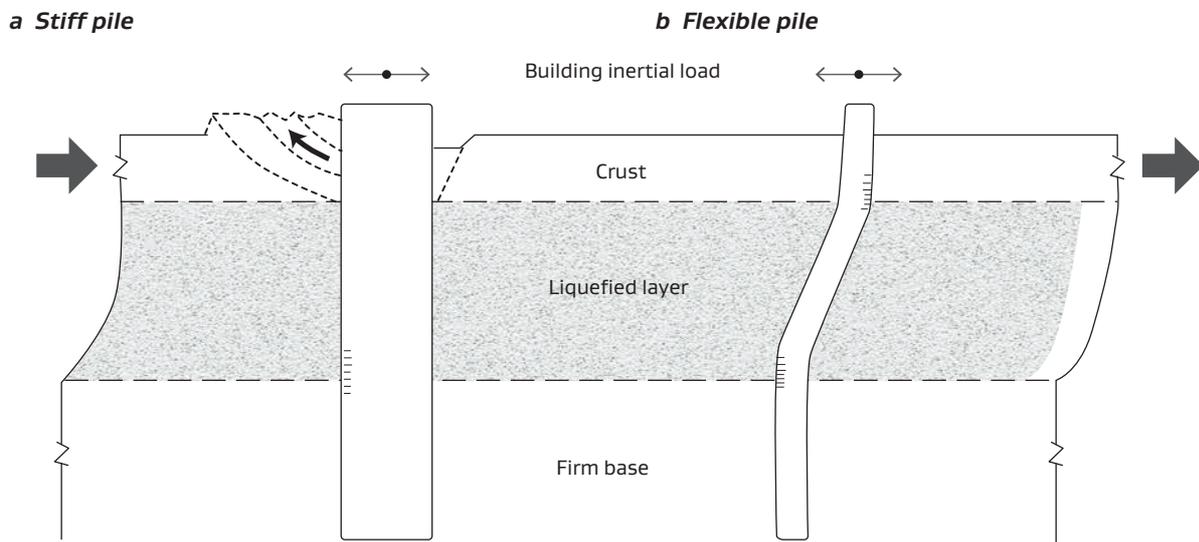
› **Phase C: Lateral spreading**

In some cases of liquefaction, lateral spreading may develop, especially near river banks, in waterfront areas and other 'free faces' (refer to Module 3 for more information). Generally, lateral spreading develops after the onset of liquefaction, either during shaking or after strong shaking has diminished. In extreme cases, displacements of the surface crust may become very large — in the order of metres — and the kinematic demands on piles very significant.

For Phases B and C, large displacements of the surface crust may develop with the potential to cause severe damage to embedded piles. The response of piles to such movements varies significantly, depending on the relative stiffness of the pile and soil profile, ranging from relatively stiff/strong piles (Figure 6.6a) which are able to resist movement of the soil crust after liquefaction, to relatively flexible/weak piles (Figure 6.6b) which are unable to resist movement of the crust and must therefore deform with the ground.

Stiff/strong piles (Figure 6.6a) are able to resist the movement of the crust, possibly mobilising the full passive load of the crust against the pile, by transferring the resulting lateral pile loads to deeper soil strata by cantilever action. The resulting internal bending moments and shears may be very large and need to be carefully checked. During Phase B, stiff piles may also continue to attract significant inertial loads from the building dynamic response in addition to the kinematic loads from the soil. Some reduction in building dynamic response is likely, depending on how

Figure 6.6: Kinematic effects for deep piles with liquefaction



much reduction in foundation stiffness occurs following liquefaction (but note that substantial reduction in foundation stiffness will not occur unless yielding and pile damage occurs).

Note

For situations like this, where the soil crust pushes against the pile, the passive soil load may be much greater than for a laterally loaded pile pushing into the soil. The passive load may be assumed equivalent to the passive soil pressure for a continuous wall times 4.5 pile diameters, compared to the usual 3 pile diameters used for calculating pile lateral load capacity.

The passive soil loads can often be reduced by isolating other parts of the structure (eg pile caps, foundation beams, etc) from the ground.

Flexible/weak piles (Figure 6.6b) are unable to generate a passive soil failure of the crust and will generally follow the ground movement. The resulting pile curvatures and material strains need to be carefully checked to ensure that the structural integrity of the piles, and especially their axial load carrying capacity, is maintained. High ductility will be important. During Phase B, the building dynamic response is likely to change significantly because of softening of the foundations:

- › the period will elongate
- › accelerations will decrease
- › but displacements may increase.

Building inertial loads will still be applied to the piles, but most often at a reduced level from Phase A.

Comment

For flexible piles, there may be an advantage to **maximising** the amount of structure embedded in the crust to **increase** the passive resistance to building inertial loads, otherwise additional pile head displacement might occur.

For cases where lateral spreading develops (Phase C), by the time the lateral spread has resulted in maximum ground displacements, it is likely that the building dynamic response will have largely dissipated.

The design of deep pile foundations in all cases where significant soil softening occurs, and especially where piles penetrate through thick layers of liquefaction, is challenging with complex interactions to resolve including significantly degrading soil properties, pile relative strength and stiffness, often complex site characteristics, and building dynamic response.

Three methods of analysis are suggested depending on the level of complexity and importance of the case:

› Method 1: Limiting equilibrium calculation with factor of safety

Method 1 is appropriate for simple cases where site soils are expected to remain firm and where the site is stable, ie only appropriate for Phase A.

The passive resistance of soil acting against all the embedded elements including piles, pile heads, foundation beams, and other structural features such as basement walls and lift pits etc. should be calculated and summed. The ultimate lateral resistance of the piles below the pile head may be calculated using the conventional limiting equilibrium procedures of Broms [1964] or Brinch-Hansen [1961]. Friction acting on the underside of ground slabs should not be included because settlement of the ground surface and loss of contact is likely with earthquake shaking. Friction along the sides of embedded elements also should not be included because of the likelihood of gapping developing.

The above calculations assume that large relative displacements take place, sufficient to mobilise the full passive soil resistance and, for longer piles, the full plastic moment capacity of the pile. The resulting pile strains are not evaluated using this simple procedure and may be excessive. Both shear and bending modes of pile failure are possible when the demand reaches pile capacity. Therefore, a low value for strength reduction factor (resistance factor) should be applied to the calculated lateral resistance ($\Phi_g = 0.3$ is recommended).

Comment

Eurocode 8 prescribes a reduction factor of 0.3 for the passive component of lateral resistance to earthquake loading.

› **Method 2. Quasi-static beam-spring analysis**

Method 2 is appropriate for most cases and can be adapted to analysing piles for all three phases (A, B, and C) of site response.

Beam spring analysis (eg 'p-y' analysis, Reese et al 1974, and subsequent commercial software packages) is used to calculate lateral displacements and pile bending strains under the ground deformations and building inertial loads (see Figure 6.7). Estimates of soil stiffness (Winkler springs) are necessary inputs in addition to the limiting soil passive pressures and pile moment-curvature relationships (relevant to the simultaneous pile axial load). The resistance of all embedded elements (foundation beams, down-stands, etc) needs to be included in the analysis as additional 'springs' attached to the pile head, and the relevant pile head fixity included in the model. Calculated pile bending strains should remain within the limits given in Table 6.1

Separate analyses should be carried out for each relevant phase of site response (ie phases A, B, and C). It is essential that appropriate values of soil stiffness (Winkler springs), limiting soil pressure, and building inertial loads are selected for each phase, recognising that these key inputs may vary considerably as the soil and site response evolves and changes during

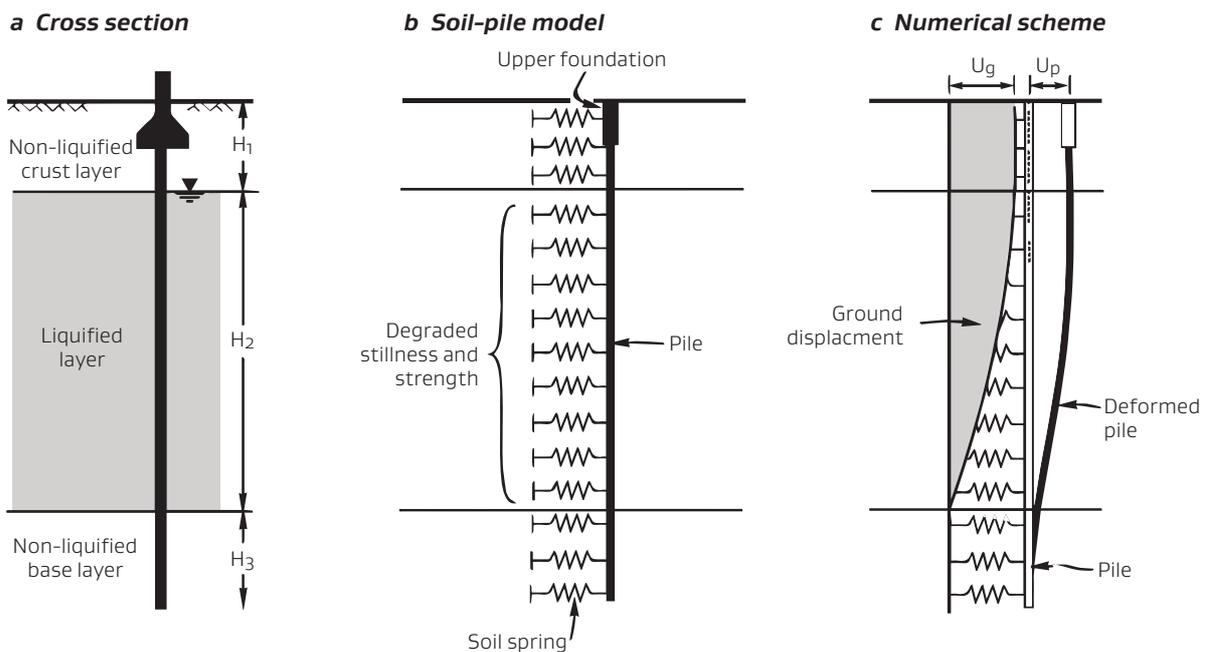
an earthquake shaking. It is important that, for each phase analysed, the estimates of soil spring, limiting soil pressure, and building dynamic response, are all compatible with each other for the response characteristics anticipated for the phase.

Comment

For Method 2, there is considerable uncertainty in estimating appropriate values for soil springs, limiting pressures, etc and these may vary greatly for each phase of the response. Therefore, it is necessary to carry out a parametric study considering a wide range of possible values for each key parameter to properly understand the realistic range of pile response and to ensure a safe design (see Cubrinovski et al 2009 and NZTA 2018, Section 3, p.14-32 for additional guidance).

The parametric study should also consider the likelihood that the response of individual piles may vary significantly from each other and that liquefaction may evolve at different times at different parts of the site. Such variances can cause a significant re-distribution of building inertial loads among individual piles.

Figure 6.7. Typical beam-spring model for pseudo-static analysis of pile [Source Cubrinovski et al, 2009]



For Phase A, for most cases the building dynamic response will be at its maximum while kinematic effects will not be dominant. The full design building inertial load (base shear) should be applied to the foundations and the resulting pile strains checked.

For sites where the piles do not penetrate through liquefiable soil layers it may be assumed that kinematic effects are unlikely to cause significant pile damage unless the piles are of high stiffness and low ductility and penetrate through layers of high stiffness contrast (ie very soft layers overlying very stiff layers) or soil layers with substantially different displacement profiles.

For Phase B, after liquefaction triggering, oscillations of the surface crust may cause significant kinematic interaction effects. The resulting pile bending strains need to be checked under the estimated free-field displacements (see Module 3 for detailed guidance on estimating the free-field displacements).

It is unlikely that the full dynamic response of the building (ie base shear) would be superimposed simultaneously with the maximum kinematic effects for the following reasons:

- a The building motion will not usually be in phase with the ground oscillation.
- b The softening of the liquefied soils will tend to isolate the building from the maximum intensity of shaking. In other words, the two maxima (inertial and kinematic effects) will not occur at the same time (and most often, not in the same phase of the response).
- c The natural period of the building/foundation/ground system will lengthen, especially for flexible piles.

Comment

Selecting an appropriate value of building inertial load to use for quasi-static analysis following liquefaction is a complex problem burdened by significant uncertainties. These uncertainties arise from the unpredictable nature of the ground motion characteristics (time history, and hence temporal evolution of the response) and also from the reduction of a complex dynamic problem to an equivalent static analysis. In principle, the uncertainties in the selection of the combination of inertial and kinematic forces cannot be reduced. Hence, parametric study is critical for the assessment. Some general guidance for the selection of combined forces is given in Tokimatsu et al (2005) and Boulanger et al (2007).

For Phase C, with lateral spreading, large ground surface displacements may accumulate inducing large kinematic effects on deep piles with potential for severe damage. (Methods for estimating displacements of the surface crust are provided in Module 3.) Usually, the lateral spread does not reach its maximum displacement until the end of ground shaking or even later, and so for most cases it would be safe to assume for analysis that simultaneous building inertial loads can be either ignored or significantly reduced, especially for flexible piles.

Note

Maximum ground displacement may not correspond to maximum kinematic load on the pile; it depends what is happening with the soil stiffness/strength during the process. Again, parametric scrutiny is the only option to address the effects of such variations in ground displacement and soil parameters.

› **Method 3: Numerical time-history analysis.**

For more complex cases (eg piled waterfront structures) or buildings of high importance (eg IL4 buildings) it may be more appropriate to carry out numerical time-history analyses. Such analyses should include both the soils, foundations, and building structure in a realistic way so that the complex interactions among them can be better understood. Such a holistic consideration of the soil-foundation-structure system while rigorously modelling the behaviour of its key components in time-history analyses provides unique insights to the designer, in addition to those obtained from equivalent static analyses. Multiple ground motion excitations reflecting the site-specific hazard are recommended to be used, to examine the system response under a range of relevant earthquake scenarios.

Comment

Method 3 is an example of performance-based design. See Section 3.9 for further discussion and guidance for performance-based design.

Comment

Bridge structures have important features that are different from other buildings, including very stiff pile foundations (often), abutments incorporating steep ground or retaining structures, and are often sited where lateral spreading is likely to occur (eg river banks). Accordingly, NZTA has adopted more specific guidance for analysis of bridge foundation piles within the NZTA Bridge Design Manual and these should take precedence over this document.

Table 6.1: In-ground plastic hinge strain limits for piles¹

PILE TYPE	STRAIN LIMITS
Pre-stressed solid concrete piles	$\epsilon_c \leq 0.008$ ($\epsilon_c \leq 0.005$)* $\epsilon_p \leq 0.015$
Steel pipe piles	$\epsilon_s \leq 0.010$
Steel pipe piles (concrete filled)	$\epsilon_s \leq 0.010$
Timber piles (normal and high density)	$\epsilon_t \leq 0.0034$

ϵ_c = extreme fibre concrete compressive strain

ϵ_p = pre-stressing strand tensile strain

ϵ_s = steel shell extreme fibre strain

ϵ_t = timber extreme fibre strain

*Note: peak curvature will always develop in the more competent crust or bearing layer where the surrounding soil confines the compression face of the concrete pile permitting a higher strain limit of $\epsilon_c \leq 0.008$. However, within the liquefying layer the degree of confinement will be minimal and a reduced strain limit of $\epsilon_c \leq 0.005$ is recommended.

7 References

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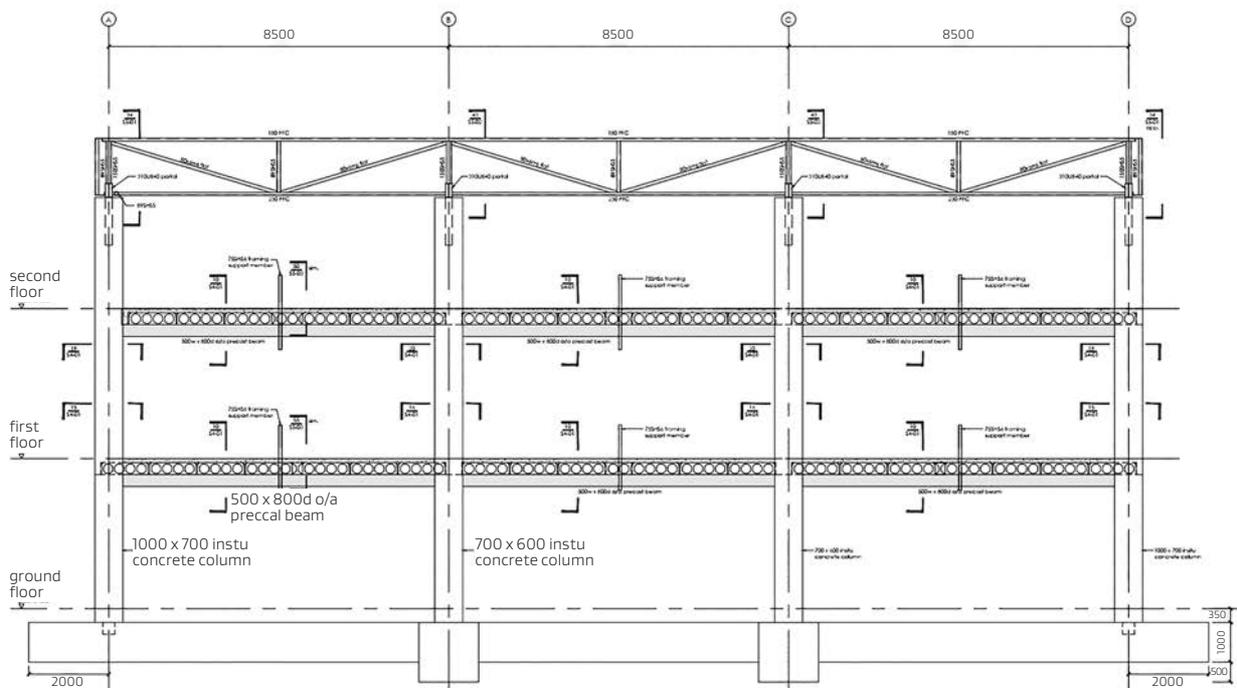
Appendix A. Worked example 1

Design of shallow foundations for a three-storey building

The example building is a three-storey commercial building designed with a frame mechanism in the EW direction and shear wall mechanism in the NS direction. Calculations are provided for the EW frame action only.

Two options are considered: Isolated pads with light tie beams and a continuous foundation beam. Basic dimensions of the frame are shown in Figure A1.

Figure A1: Cross-section showing frame acting in the EW direction



Load calculations for the building were provided by a structural engineer and the relevant foundation load cases are summarised as follows:

LOAD CASE (IL2)	INNER COLUMN (KN)	OUTER COLUMN (KN)
Gravity (SLS) (G,ΨQ)	1632	621
Gravity (ULS) (1.2G, 1.5Q)	1895	932
Earthquake (ULS) (G, E ₀ , Ψ _E Q)	1144	1031

FOUNDATION DESIGN STRATEGY

- 1 Develop site geotechnical model and relevant design parameters (Section 2).
- 2 Site assessment and foundation selection procedure from Section 4 — (and structural form selection working with the structural engineer).
- 3 Preliminary estimate of footing sizes assuming SLS gravity settlement governs design (many cases).
- 4 Calculate ultimate bearing capacity and check against ULS gravity loads.
- 5 Calculate lateral load capacity of preliminary foundation system and check against building ULS base shear demand.
- 6 Revise ultimate bearing capacity calculation as necessary and check against ULS EQ loads.

1 Site geotechnical model

An engineering geological assessment of the site identified an alluvial plain with relatively consistent sands and gravels expected to extend to significant depth. Four machine boreholes were made across the site to a depth of 15 m with SPT's at intervals of 1 m, and these confirmed the presence of medium-dense to dense SAND and GRAVEL to 15 m depth. The water table was measured at 3 m depth.

A representative borelog (BH3) is summarised in Figure A2 and was the 'worst case' for assessing shallow foundation options for the building.

A liquefaction assessment carried out following the procedures of Module 3 with the resulting values for FL indicated on Figure A2 together with estimates of R_u (excess pore pressure ratio).

Figure A2: Summary of borelog BH3

Depth (m)	Soil Description	N	F_L (ULS)	ϕ	R_u	ϕ_r
1						
2		N=20	-	33	-	33
3		N=20	-	33	-	33
4		N=28	1.6	36	0.2	30
5	SAND	N=29	1.5	36	0.2	30
6	to Gravelly	N=28	1.4	36	0.2	30
7	SAND	N=30	1.2	36	0.4	24
8		N=31	1.2	36	0.4	24
9		N=31	1.0			
10		N=33	1.3			
11		N=29	0.8			
12		N=35	1.3			
12		N=36	1.3			

2 Site assessment and foundation selection

Following the flow chart of Figure 4.1:

- 1 Liquefaction assessment indicated a possibility of limited liquefaction triggering under the design ULS earthquake shaking. Two layers were indicated as having $FL \leq 1.0$, at 9 m depth and 11 m depth. The upper 6 m of soil had $FL \geq 1.4$. The conclusion was that there is a low risk of surface manifestation of liquefaction at the site for the ULS earthquake.
- 2 Reasonable enquiry indicated that the risk of lateral spreading affecting the site was negligible.
- 3 Some soil layers were found to have $FL < 1.5$. Therefore, excess pore water pressures are likely to be generated during ULS earthquake shaking reducing the effective strength of those layers (ie need to adjust effective strength parameters in the calculations).
- 4 No other stability issues were identified (eg steep slopes, batters, or nearby retaining structures).
- 5 No liquefaction is indicated for the SLS earthquake, but liquefaction was found likely to occur in some layers for the ULS earthquake. Liquefaction triggering is likely for earthquake shaking with a return period of 200 years (ie 22 percent probability that triggering will occur within the 50 year life of the building). The resulting "step change" in performance was discussed with the Owner and considered an acceptable risk.
- 6 SLS gravity settlements (less than 25 mm total and 20 mm differential, see calculations below) were considered to be tolerable for the building structure and intended use after consultation with the Owner and Structural Engineer.
- 7 Therefore, shallow foundations were considered suitable for this building.

3 Footing size based on SLS gravity load case

Often, the footing size for shallow foundations will be governed by settlement limits for the SLS gravity load case. For the example building the footings will be founded onto granular soils (sands and gravels) and the method of Schmertmann (1970, 1978) is appropriate for estimating settlements.

The method requires estimates of soil modulus, E_s , and these may be obtained by correlation with the SPT N values provided in the site model. Normally, the maximum acceptable settlement under an individual column is taken as 25 mm for which the maximum differential settlement would usually be less than 20 mm (depending on the functional requirements of the building).

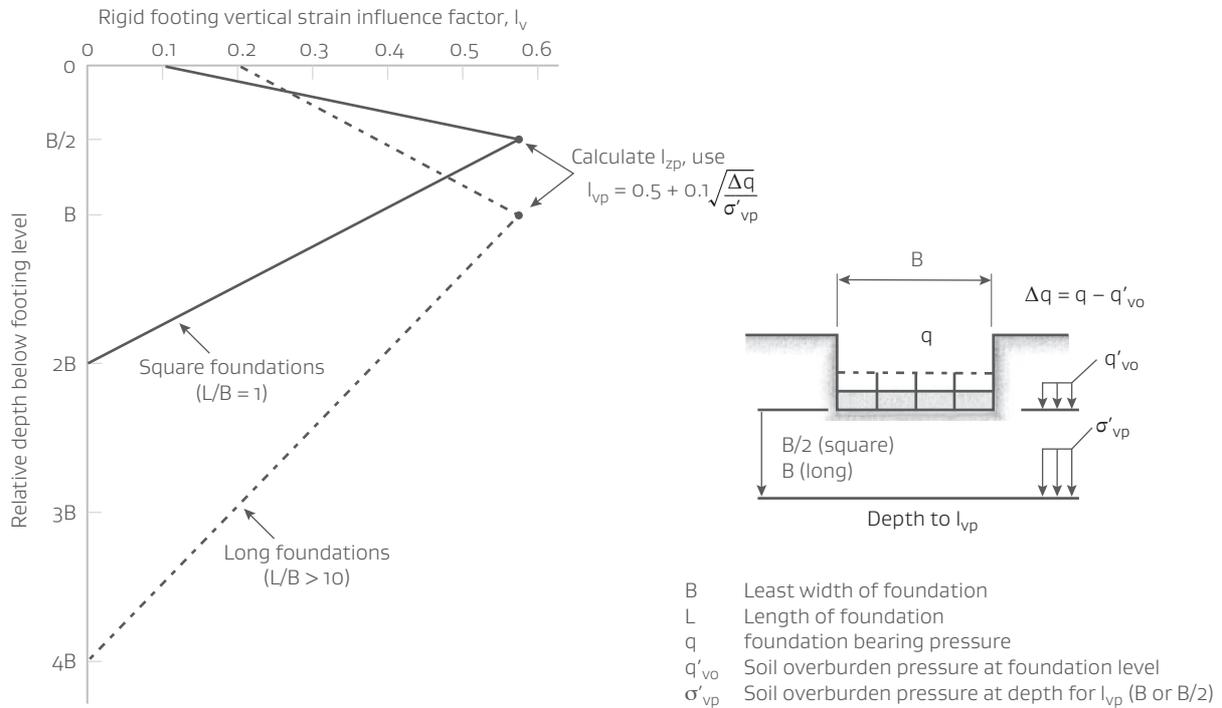
The method can conveniently be set up in a spreadsheet allowing rapid optimisation of footing size, depth, and maximum settlement. For the example building, two types of shallow footing are considered here: Square footings and continuous beam footings. (In practice, the real example building uses continuous heavy beam foundations to provide necessary frame action for the structure.)

Table 4.1: Approximate value for E_s related to soil type and SPT N (McCarthy, 2002)

SOIL TYPE	E_s (MN/M ²)	
	IN TERMS OF N	IN TERMS OF Q_c
Sand-silt mixture	0.4 N	0.15 q_c
Sand	0.7 N – 1.0 N	0.2 q_c – 0.3 q_c
Sand-gravel mixtures	1.2 N	0.4 q_c

Figure A3: Calculating settlement of shallow footings by method of Schmertmann

$$\Delta S_n = (\epsilon_{v_{avg}}) (\Delta z_n) = \Delta q (I_v / E_s) (\Delta z_n)$$



For a square footing supporting the more heavily loaded internal columns, 2.65 m square gives an estimated settlement of 24 mm at the intended founding depth of 1.35 m.

In practice, the building was founded on heavy beams with dimensions 1.0 m deep and 0.8 m wide. Working back from the maximum settlement of 25 mm, the allowable bearing pressure is found to be 370 kN/m² or 313 kN/m. Therefore, the column load of 1691 kN needs to be distributed over a length of at least 5.4 m along the foundation beam. A total span of 8.5 m is available.

The structural design of the beam (by the structural engineer) would need to be checked to ensure that it has sufficient strength and stiffness to achieve the required load distribution without excessive deformation. The beam bending moments and shears should be determined using a beam-on-elastic-foundation analysis with Winkler spring constants derived (by the geotechnical engineer) from the settlement analysis as follows:

$$ks = p/\delta = 370/0.024 = 15,420 \text{ kN/m}^3$$

Figure A4: Settlement calculations for square footing

D=	1.35	(m)		
B=	2.65	(m)		
Zw=	3	(m)		
Δq =	220	(kN/m ²)	Q =	1697 kN
γ (moist)	16	(kN/m ³)		
γ (buoyant)	8	(kN/m ³)		
sig'vp=	42.8	(kN/m ²)		
lvp=	0.72672			
C1=	0.95			

Layers (B)	z (m)	dz (m)	σ'_v (kPa)	lv	Es (MN/m ²)	ds (mm)
0	1.35	0	21.6	0.2		0
0.1	1.615	0.265	25.8	0.253	20	0.660
0.2	1.88	0.265	30.1	0.305	20	0.813
0.3	2.145	0.265	34.3	0.358	20	0.967
0.4	2.41	0.265	38.6	0.411	20	1.120
0.5	2.675	0.265	42.8	0.463	20	1.274
0.6	2.94	0.265	47.0	0.824	28	1.340
0.7	3.205	0.265	49.6	0.799	28	1.690
0.8	3.47	0.265	51.8	0.775	28	1.639
0.9	3.735	0.265	53.9	0.751	28	1.589
1	4	0.265	56.0	0.727	28	1.538
1.1	4.265	0.265	58.1	0.702	28	1.488
1.2	4.53	0.265	60.2	0.678	28	1.437
1.3	4.795	0.265	62.4	0.654	28	1.387
1.4	5.06	0.265	64.5	0.630	28	1.337
1.5	5.325	0.265	66.6	0.606	28	1.286
1.6	5.59	0.265	68.7	0.581	28	1.236
1.7	5.855	0.265	70.8	0.557	28	1.185
1.8	6.12	0.265	73.0	0.533	28	1.135
1.9	6.385	0.265	75.1	0.509	28	1.084
2	6.65	0.265	77.2	0.484	28	1.034
					Sum =	25.240
				S =	C1 x Sum	24

Figure A5: Settlement calculations for continuous beam footing

D=	1.35	(m)	
B=	0.8	(m)	
Zw=	3	(m)	
Δq =	370	(kN/m ²)	>> Q = 313 kN/m
γ (moist)	16	(kN/m ³)	
γ (buoyant)	8	(kN/m ³)	
sig'vp=	34.4	(kN/m ²)	
lvp=	0.82796		
C1=	0.97		

Layers (B)	z (m)	dz (m)	$\sigma'v$ (kPa)	lv	Es (MN/m ²)	ds (mm)
0	1.35	0	21.6	0.2		0
0.2	1.51	0.16	24.2	0.326	20	0.778
0.4	1.67	0.16	26.7	0.451	20	1.150
0.6	1.83	0.16	29.3	0.577	20	1.521
0.8	1.99	0.16	31.8	0.702	20	1.893
1	2.15	0.16	34.4	0.828	20	2.265
1.2	2.31	0.16	37.0	0.773	20	2.369
1.4	2.47	0.16	39.5	0.718	20	2.206
1.6	2.63	0.16	42.1	0.662	20	2.042
1.8	2.79	0.16	44.6	0.607	20	1.879
2	2.95	0.16	47.2	0.552	20	1.716
2.2	3.11	0.16	48.9	0.497	20	1.552
2.4	3.27	0.16	50.2	0.442	20	1.389
2.6	3.43	0.16	51.4	0.386	20	1.225
2.8	3.59	0.16	52.7	0.331	28	0.759
3	3.75	0.16	54.0	0.276	28	0.642
3.2	3.91	0.16	55.3	0.221	28	0.525
3.4	4.07	0.16	56.6	0.166	28	0.408
3.6	4.23	0.16	57.8	0.110	28	0.292
3.8	4.39	0.16	59.1	0.055	28	0.175
4	4.55	0.16	60.4	0.000	28	0.058
					Sum =	24.844
				S =	C1 x Sum	24

4 Calculate ultimate bearing capacity (static case)

The ultimate bearing capacity is calculated using familiar limiting equilibrium procedure. For the example, the equations derived by Vesic (1975) have been used as these are comprehensive and readily formulated in Mathcad for convenience.

For the example, calculations are given for both the square footing option and the continuous beam option. For the square footing, the limiting equilibrium yield surface is assumed to penetrate below the water table and so an average value for soil density was assumed. For the beam footing, the yield surface should remain above the water

table. The effective friction angle was estimated by correlation with the SPT N values.

A strength reduction factor of 0.5 was adopted from Table 5.1.

For the square footing the dependable ultimate bearing capacity was calculated as follows:

Drained bearing capacity shallow footing — vesic

$c := 1 \cdot \frac{kN}{m^2}$	$\phi := 33 \cdot deg$	$\gamma_q := 16 \cdot \frac{kN}{m^3}$	$\gamma_{eff} := 13 \cdot \frac{kN}{m^3}$	Soil parameters (drained)
$B := 2.65 \cdot m$	$L := 2.65 \cdot m$	$D := 1.35 \cdot m$		Footing dimensions (effective)
$\beta := 0 \cdot deg$				Ground slope in front of footing
$\eta := 0$				Tilt of footing (refer diagram)
$\Phi_{bc} := 0.5$				Bearing capacity resistance factor
$c_a := 1.0 \cdot c$				Adhesion (underside of footing)
$q := \gamma_q \cdot D \cdot \cos(\beta)$				Surcharge
$H_u := 0 \cdot kN$	$V_u := 0 \cdot kN$			Factored loads applied to footing
$N_q := e^{\pi \cdot \tan(\phi)} \cdot \left(\tan\left(\frac{\phi}{2} + \frac{\pi}{4}\right) \right)^2$	$N_c := (N_q - 1) \cdot \cot(\phi)$	$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi)$		
$N_q = 26.092$	$N_c = 38.638$	$N_\gamma = 35.188$		
Shape factors				
$\lambda_{cs} := 1 + \frac{B \cdot N_q}{L \cdot N_c}$	$\lambda_{\gamma s} := 1 - 0.4 \cdot \frac{B}{L}$	$\lambda_{qs} := 1 + \frac{B \cdot \tan(\phi)}{L}$		
$\lambda_{cs} = 1.675$	$\lambda_{\gamma s} = 0.6$	$\lambda_{qs} = 1.649$		
Depth factors (D < B) $D := B$ delete as required				
$\lambda_{qd} := 1 + 2 \cdot \tan(\phi) \cdot (1 - \sin(\phi))^2 \cdot \frac{D}{B}$	$\lambda_{cd} := \lambda_{qd} \cdot \frac{1 - \lambda_{qd}}{N_c \cdot \tan(\phi)}$			
$\lambda_{qd} = 1.28$	$\lambda_{cd} = 1.269$	$\lambda_{\gamma d} := 1$		
Ultimate bearing pressure				
$q_u := c \cdot \lambda_{cs} \cdot \lambda_{cd} \cdot \lambda_{ci} \cdot \lambda_{cs} \cdot \lambda_{ci} \cdot N_c + q \cdot \lambda_{qs} \cdot \lambda_{qd} \cdot \lambda_{q1} \cdot \lambda_{q2} \cdot \lambda_{q3} \cdot N_q + \frac{1}{2} \cdot \gamma_{eff} \cdot B \cdot \lambda_{\gamma s} \cdot \lambda_{\gamma d} \cdot \lambda_{\gamma 1} \cdot \lambda_{\gamma 2} \cdot \lambda_{\gamma 3} \cdot N_\gamma$				
$q_u = (1.626 \cdot 10^3) \frac{kN}{m^2}$				
$V_{ustar} := B \cdot L \cdot q_u \cdot \Phi_{bc}$	$V_{ustar} = (5.711 \cdot 10^3) kN$	$\frac{V_{ustar}}{L} = (2.155 \cdot 10^3) \frac{kN}{m}$		

Therefore, the dependable bearing capacity of the square footing (5711 kN) is much greater than the factored ultimate load (1895 kN).

For the beam footing:

Drained bearing capacity shallow footing — vesic

$c := 1 \cdot \frac{kN}{m^2}$	$\phi := 33 \cdot deg$	$\gamma := 16 \cdot \frac{kN}{m^3}$	Soil parameters (drained)
$B := 0.8 \cdot m$	$L := 30 \cdot m$	$D := 1.35 \cdot m$	Footing dimensions (effective)
$\beta := 0 \cdot deg$			Ground slope in front of footing
$\eta := 0$			Tilt of footing (refer diagram)
$\Phi_{bc} := 0.5$			Bearing capacity resistance factor
$c_a := 1.0 \cdot c$		+	Adhesion (underside of footing)
$q := \gamma \cdot D \cdot \cos(\beta)$			Surcharge
$H_u := 0 \cdot kN$	$V_u := 0 \cdot kN$		Factored loads applied to footing (for inclined loading)
<hr/>			
$N_q := e^{\pi \cdot \tan(\phi)} \cdot \left(\tan\left(\frac{\phi}{2} + \frac{\pi}{4}\right) \right)^2$	$N_c := (N_q - 1) \cdot \cot(\phi)$	$N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi)$	
$N_q = 26.092$	$N_c = 38.638$	$N_\gamma = 35.188$	
Shape factors			
$\lambda_{cs} := 1 + \frac{B \cdot N_q}{L \cdot N_c}$	$\lambda_{\gamma s} := 1 - 0.4 \cdot \frac{B}{L}$	$\lambda_{qs} := 1 + \frac{B \cdot \tan(\phi)}{L}$	
$\lambda_{cs} = 1.018$	$\lambda_{\gamma s} = 0.989$	$\lambda_{qs} = 1.017$	
Depth factors (D < B) D := B delete as required			
$\lambda_{qd} := 1 + 2 \cdot \tan(\phi) \cdot (1 - \sin(\phi))^2 \cdot \frac{D}{B}$	$\lambda_{cd} := \lambda_{qd} - \frac{1 - \lambda_{qd}}{N_c \cdot \tan(\phi)}$		
$\lambda_{qd} = 1.28$	$\lambda_{cd} = 1.269$	$\lambda_{\gamma d} := 1$	
Ultimate bearing pressure			
$q_u := c \cdot \lambda_{cs} \cdot \lambda_{cd} \cdot \lambda_{ci} \cdot \lambda_{cg} \cdot \lambda_{ct} \cdot N_c + q \cdot \lambda_{qs} \cdot \lambda_{qd} \cdot \lambda_{qi} \cdot \lambda_{qg} \cdot \lambda_{qt} \cdot N_q + \frac{1}{2} \cdot \gamma \cdot B \cdot \lambda_{\gamma s} \cdot \lambda_{\gamma d} \cdot \lambda_{\gamma i} \cdot \lambda_{\gamma g} \cdot \lambda_{\gamma t} \cdot N_\gamma$			
$q_u = (1.001 \cdot 10^3) \frac{kN}{m^2}$			
$V_{ustar} := B \cdot L \cdot q_u \cdot \Phi_{bc}$	$\frac{V_{ustar}}{L} = 400.362 \frac{kN}{m}$		

Therefore, the ULS gravity load from a centre column needs to be spread along a length of foundation beam = 1895/400 = 4.7 m. The structural capacity of the beam needs to be checked to confirm that it can safely distribute the column load over this length of beam.

5 Calculate lateral load capacity of foundation

Calculate the passive resistance against sliding of the building including the foundation elements and other embedded parts of the building (eg lift pits) and check the capacity against the ULS base shear demand.

If the available passive resistance to lateral loading is greater than the base shear demand, then the ULS EQ loads may be considered to act vertically. If not, they must be treated as inclined loads with a horizontal component of loading equal to the imbalance.

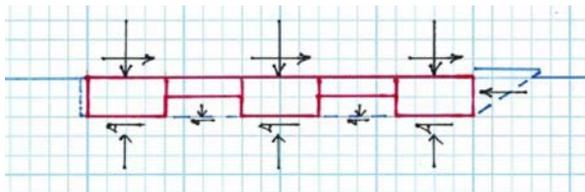
For the example building, the structural engineer has calculated a base shear demand in the NS direction (ie perpendicular to the frame being analysed here) as $V_{base} = 4500$ kN.

Three possible failure modes are possible as shown in Figure A6:

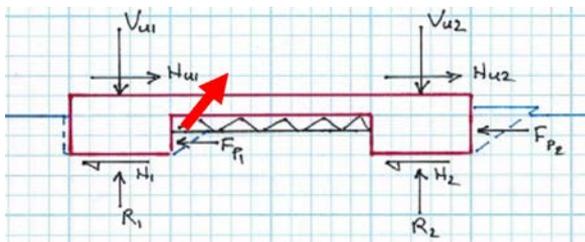
- a tip-to-tip sliding, where beams are close spaced and connected by a strong floor slab
- b sliding with tilting, where beams are further apart and connected by a strong floor slab and the resulting passive wedge forming under the floor will 'jack up' one side of the building;
- c sliding without tilting, where the passive wedge is able to lift the floor without lifting the building.

Figure A6:

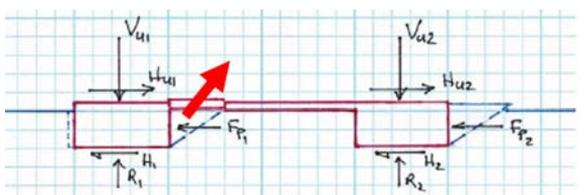
a Tip-to-tip sliding



b Sliding with tilting



c Sliding with weak floor slab



For the example building, failure mode (c) was found to be critical after also checking mode (a). The example building has a non-structural, slab-on-grade ground floor slab, and so mode (b) is not possible as it requires a strong floor to resist uplift force from the passive soil wedge.

Calculations of passive resistance acting against the side of the foundation beams may be made using static values of K_p such as those shown below from the NAVFAC chart (included at the end of the worked example). It is not necessary to use dynamic values of K_p (such as from the Mononabe-Okabe equations) because the building base shear is from the dynamic response of the building above ground and will normally be out of phase with the ground accelerations.

For the example building, a total length of 29.5 m of EW running foundation beams is providing resistance to NS sliding calculated as follows:

Soil Passive Resistance (NAVFAC)			
$\phi := 33 \text{ deg}$	$\delta := 0.5 \cdot \phi$	$\delta = 16.5 \text{ deg}$	$\beta := 0 \text{ deg}$
$\gamma := 16 \frac{\text{kN}}{\text{m}^3}$	$H := 1.35 \text{ m}$	$L := 59 \text{ m}$	
$K_p := 8.5$	From NAVFAC chart		
$R := \frac{3}{5} (0.746 - 0.674) + 0.746$	$R = 0.789$		
R = Reduction factor for interface friction			
$P_{ph} := 0.5 \gamma \cdot H^2 K_p \cdot R \cdot \cos(\delta)$	$P_{ph} = 93.778$	$\frac{\text{kN}}{\text{m}}$	
$P_{pv} := 0.5 \gamma \cdot H^2 K_p \cdot R \cdot \sin(\delta)$	$P_{pv} = 27.778$	$\frac{\text{kN}}{\text{m}}$	
$H_u := P_{ph} \cdot L$	$H_u = (5.533 \cdot 10^3)$	kN	

The available resistance is calculated as 5500 kN which is greater than the demand of 4500 kN. Therefore, the simultaneous vertical load applied to the foundation from the building can be considered to act vertically at the base of the footing.

No strength reduction factor is applied to the calculated passive resistance because base sliding is self-limiting and will not endanger the stability of the building in most cases (ie sites on level ground).

Note

Mobilisation of the passive resistance will also generate a vertical uplift component acting against the side of the footing (28 kN/m), and this may be either added to the calculated ultimate bearing capacity of the footing or conservatively ignored.

6 Calculate ultimate bearing capacity for ULS earthquake case

Before re-calculating the ultimate bearing capacity for the earthquake load case the following issues need to be considered:

- › Are there unresolved simultaneous horizontal loads applied at the base of the footings — these will reduce the vertical bearing capacity.
- › Is there an increase in pore water pressure in the founding soils (ie in granular soils, or possible cyclic softening in cohesive soils)?
- › Is there a risk of ‘punch through’ into a weakened or liquefied layer below the footings?

For the example, all the horizontal base shear was able to be resisted by passive resistance against the sides of the foundation beams, so the building load for the earthquake case can be treated as a vertical load applied at the base of the footing. (But note that the depth factor in the limiting equilibrium calculation in this case must be set equal to 1.0, because the soil shear strength above the base of the footing has already been considered fully mobilised in providing passive resistance against the side of the foundation beams).

For the example, the soil within the zone of potential limiting equilibrium bearing slip circle for the beam foundation option is located above the water table and so not subject to pore water pressures. However, for the square footing option (2.65 m x 2.65 m), the yield surface dips below the water table into a zone where $R_u = 0.2$ is predicted (ie pore water pressures may be elevated above hydrostatic). Therefore, for the square footing, the ultimate bearing capacity for the earthquake load case would be calculated using a reduced friction angle of 30 degrees. (See Section 4.7 for more information.)

‘Punch through’ into a liquefying layer is considered unlikely in either case given that the shallowest liquefiable layer is identified at 9 m depth.

For the example, therefore, the ultimate bearing capacity was re-calculated for the earthquake load case using the Vesic formulation with the depth factors set = 1 giving a reduced dependable ultimate bearing capacity of 334 kN/m (compared to 400 kN/m for the gravity load case).

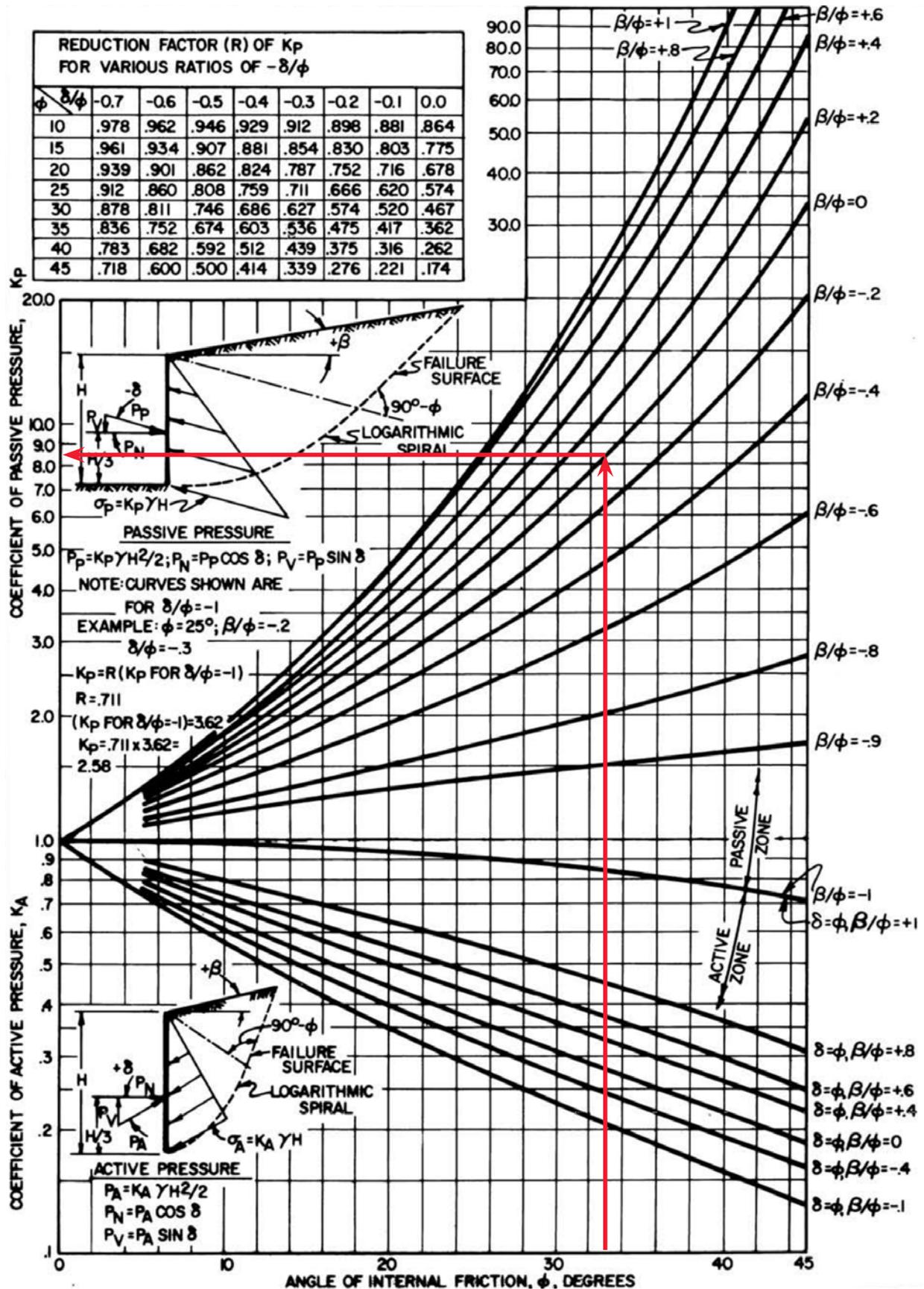
Conclusion

The building as designed with 0.8 m wide and 1 m deep foundation beams meets normal performance (SLS) and safety (ULS) geotechnical requirements. It also should perform well for intermediate limit state earthquakes up to a 200 year return period (22 percent probability of exceedance within the 50 year lifetime of the building) when liquefaction triggering is likely to initiate.

Structural design of the foundation beams under the design loadings will be important but is not covered here. Structural analysis of the foundation beams should be carried out using beam-on-elastic-foundation analysis and using Winkler spring values calculated above, together with the calculated limiting soil pressures (ultimate bearing pressures).

Square pad foundations may also be feasible, but these would require a major structural re-design. Pad foundations would need to be well tied together at this site because of the risk of soil liquefaction at depth.

Figure A7: NAVFAC (1986) chart for active and passive soil pressure



References for Appendix A

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