BUILDING PERFORMANCE

Earthquake geotechnical engineering practice

Module 5. Ground improvement of soils prone to liquefaction

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HĪKINA WHAKATUTUKI

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Contents

Acknowledgements

NZGS/MBIE EDITORIAL PANEL FOR REVISION

- Campbell Keepa — WSP (lead author)
- Dr Alexei Murashev — WSP (lead author)
- Prof Misko Cubrinovski — University of Canterbury
- Dr Kevin J McManus — McManus Geotech Ltd
- Nick Traylen — Geotech Consulting Ltd
- Rick Wentz — Wentz Pacific Ltd
- Mike Stannard — Engineering New Zealand (project lead)
- Tony Fairclough — Tonkin & Taylor
- John Scott — EQC
- Kiran Saligame — Ministry of Business, Innovation & Employment
- Ananth Balachandra — NZGS

NZGS/MBIE EDITORIAL PANEL AND CONTRIBUTORS FOR REVISION 0 (PUBLISHED JUNE 20167)

- Prof R Orense — University of Auckland
- Prof Misko Cubrinovski
- Dr Kevin J McManus
- Charlie Price — Stantec, NZGS Chair
- John Scott — MBIE
- Dr Gilles Seve — MBIE
- Mike Stannard — MBIE
- Nick Traylen
- Rick Wentz
- Phil Clayton — Beca Infrastructure
- Tony Fairclough
- Nick Wharmby — March Construction Ltd
- Stuart Palmer — Tonkin & Taylor
- Prof Jonathon Bray — University of California, Berkeley
- Dr Geoffrey Martin — University of Southern California
- Prof Susumu Yasuda — Tokyo Denki University, Japan

MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT

- Jenni Tipler

NZGS MANAGEMENT

- Ross Roberts

ENGINEERING NEW ZEALAND

- Kaya Yamabe
- Eleanor Laban
- Tania Williams

EQC

- Jo Horrocks

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New Zealand Geotechnical Society (NZGS)
c/ Engineering New Zealand
PO Box 12–241
Wellington 6013

Ministry of Business Innovation & Employment (MBIE)
Building System Performance Branch
PO Box 1473
Wellington 6140

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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 4: Earthquake resistant foundation design
› Module 5A: Specification of ground improvement for residential properties in the Canterbury region
› Module 6: Earthquake resistant retaining wall design.

Online training material in support of the series is available on the MBIE and NZGS websites:

www.building.govt.nz and www.nzgs.org.

This module covers the design of ground improvement and supports the Canterbury Earthquakes Royal Commission recommendations to prepare national guidelines specifying design procedures for ground improvement, to provide more uniformity in approach and outcomes.

This ground improvement module is supported by Module 5A of the series, a specification dedicated to ground improvement for residential properties in the Canterbury region. Ground improvement options and design for residential properties have also been addressed in Section 15.3 and Appendix C of the MBIE document Repairing and rebuilding houses affected by the Canterbury earthquakes. Although these two latter documents were written with the Canterbury recovery in mind, their usefulness as guides for other liquefaction prone areas within New Zealand is recognised, with appropriate modifications being made to suit local conditions. Module 5 addresses this issue.

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

Eleni Gkeli
Chair
New Zealand Geotechnical Society

Jenni Tipler
Manager Building Performance and Engineering
Ministry of Business, Innovation & Employment

1 Introduction

1.1 Objective

The objective of this document is to provide guidance on the design of ground improvement to mitigate the effects of liquefaction and to improve design consistency in New Zealand. The guideline is aimed at engineers involved in the design of ground improvement but some parts could prove useful to consenting authorities and owners.

Situated on the subduction boundary between the Pacific and the Australian Plates, New Zealand is exposed to seismic hazard. The effects of earthquakes are a key consideration for the assessment and design of buildings. In areas underlain by young alluvial deposits, earthquake shaking can trigger liquefaction, the process where pore water pressures increase and soils soften, often having a profound effect on the built environment.

International experience has shown that buildings founded on sites that would otherwise be liquefiable can perform well, where well-engineered, robust ground improvement has been carried out. The experience in Christchurch during the Canterbury earthquake sequence was more varied, noting that the ground shaking, in some areas, was more intense than that allowed for in design. The Canterbury Earthquake Royal Commission (CERC) recommended consideration be given to the preparation of national guidelines to improve uniformity in the design approach and outcomes.

It should be noted that ground improvement techniques are the subject of ongoing research and development. New ground improvement techniques and design methods will evolve with time, and therefore geotechnical designers should keep abreast of the latest developments.

1.2 Scope

This document identifies the key issues that need to be addressed in the design and construction of ground improvement to mitigate the effects of liquefaction, cyclic softening and lateral spreading effects on buildings and provides a framework for resolving these issues through design and construction. The objective is to provide concise, practical advice and simplified procedures for the design of ground improvement by qualified, experienced engineers based on the latest research and observations of the performance of ground improvement in earthquakes in New Zealand and internationally.

A wide range of ground improvement techniques are available to mitigate the effects of liquefaction and many of these are briefly described including techniques that have not been used extensively in New Zealand to date. There is no attempt to provide a comprehensive discussion of all available liquefaction countermeasures in this guideline; rather, only commonly used methods in New Zealand are outlined in detail. A bibliography is provided that gives greater depth on specific topics and aspects of ground improvement and practitioners and constructors are encouraged to read these where relevant. Useful general references for the assessment and design of ground improvement to mitigate liquefaction include:

- Schaefer et al 2017, Ground modification methods, Reference manual, Vol. 1 FHWA-NHI-16-027
- Schaefer et al 2017, Ground modification methods, Reference manual, Vol. 2 FHWA-NHI-16-028
- Han, 2015, Principles and practices of ground improvement
- Japanese Geotechnical Society 1998, Remedial measures against liquefaction
- Kirsch and Bell, 2012, Ground improvement

Because ground improvement technologies change rapidly and as new techniques are developed and existing techniques are refined and tested by actual earthquakes, the relevant geotechnical literature should be periodically reviewed.

The setting of seismic performance criteria for the building, the investigation and characterisation of a site, the evaluation of the liquefaction and lateral spreading hazard and design of foundation systems are discussed briefly here. More detailed discussion on these topics is presented in Modules 1 to 4. These modules contain advice that is important to the successful design and construction of any ground improvement system and should be read in conjunction with this module. Module 5a provides specifications for ground improvement for residential developments.

A number of ground improvement solutions have been developed for the rebuild of the housing stock in Canterbury following the Canterbury earthquake sequence. These solutions and how these can be applied to residential construction on liquefiable sites across the remainder of New Zealand is discussed in Section 11.

Ground improvement is part of a larger system that includes the buildings foundation elements, the superstructure and the surrounding environment. Understanding and making due consideration of the interaction of all of these components is essential to obtaining the desired overall performance outcomes. This implies close collaboration between developers, architects, structural engineers and geotechnical engineers.

2 Site and liquefaction considerations

2.1 Site characterisation

One of the initial steps in the design of ground improvement is to develop a geotechnical model for the site which fits within the wider regional geology and geomorphology. This starts from review of available literature and site investigation information from previous studies and may be followed by site investigations to fill gaps in information to the extent needed to develop an appropriate ground model.

The topic of planning and undertaking site investigations for the purpose of characterising site geotechnical conditions and for the evaluation of liquefaction is discussed in Module 2 and further in Module 3. Module 4 gives guidance on the development of ground models and the selection of engineering soil properties for the design of foundations.

The effectiveness of many ground improvement techniques is highly dependent on the fines content of the soils and the variability of the ground conditions to be treated. A comprehensive investigation should be undertaken to assess soil conditions and, in particular, the fines content, location and extent of silt and clay layers at a site.

Penetration testing undertaken as part of the site investigation also forms the basis for assessing the degree of treatment achieved. As discussed in Module 3, there is a high degree of uncertainty in the relationship between fines content and the soil behaviour index (I_c), I_c calculated from Cone penetration tests (CPT) and fines content calculated from Ic should be calibrated against laboratory measured fines content and field descriptions of soils.

2.2 Liquefaction considerations

LIQUEFACTION EVALUATION

Liquefaction is associated with significant loss of soil stiffness and strength. The associated softening can result in large cyclic ground movements during shaking followed by subsidence and lateral spreads. These effects, either individually or as a combination, can be particularly damaging to the built environment.

Evaluation of the liquefaction hazard at a site involves three steps:

1 Assessment of the susceptibility of the site soils to liquefaction
2 For soils that are susceptible to liquefaction, assessing the level of shaking that would trigger liquefaction or the development of significant excess pore water pressure
3 Evaluating the effects liquefaction will have on the building if liquefaction is triggered.

Detailed recommendations on site investigations for assessment of liquefaction are given in Module 2. Guidance on the identification and assessment of liquefaction, and liquefaction induced ground deformation is provided in Module 3.

EFFECTS OF LIQUEFACTION ON BUILDINGS

The seismic behaviour of a building on liquefiable ground is affected by the depth and stiffness of the structural foundation, magnitude of contact pressure, seismic response of the structure and soil, the thickness and properties of liquefiable soil layers and the non-liquefiable crust, the intensity of ground motion and many other factors.

There are a number of ways liquefaction can affect a building and its connecting infrastructure, including:

› Reduced bearing capacity due to the associated reduction in soil strength
› Subsidence associated with shear deformation, cyclic ratcheting, lateral spreading and ground re-levelling, and reconsolidation
› Surface ejection of soil and water (sand boils) from beneath or around foundations
› Heave of ground bearing floor slabs and buoyancy of buried pipes, tanks, chambers and basements
› Horizontal displacement and stretching of the footprint and foundation with lateral spreading.
› Kinematic bending of piles with horizontal ground displacements and
› Pile down-drag (negative skin friction) caused by ground subsidence.

The degree to which these effects relate to a particular site and structure, depends on the site specific ground conditions and the details of the structural system. Detailed discussion on the effects of liquefaction on buildings is given in Module 4.

3 Ground improvement principles

The objective of ground improvement, in this context, is to mitigate the effects of liquefaction and lateral spreading to the extent needed to meet the design performance criteria for the structure. Performance requirements for ground improvement are discussed further in Section 7. Guidance on performance criteria for foundations is given in Module 4.

3.1 Methods of ground improvement

There are generally five principle methods employed to improve the ground and increase its resistance to liquefaction, these are:

- Replacement
- Densification
- Solidification
- Reinforcement
- Drainage.

Ground improvement methods utilise one or a combination of these mechanisms to improve the ground's resistance to liquefaction and improve seismic performance. Ground improvement mechanisms are briefly described here and summarised in Table 3.1 Design issues pertaining to the most common techniques used in New Zealand are discussed further in Sections 6 to 10.

Note

A secondary mechanism of some techniques is the potential improvement of the soil's resistance to liquefaction triggering by an increase in the lateral stress within the soil and thus changing its initial state. This mechanism cannot be easily verified in the field and may not greatly reduce the effects of liquefaction should it be triggered. Until further research gives a better understanding of its effectiveness at mitigating liquefaction and ways to confidently verify that the increase in lateral stress is achieved in the field, this mechanism should not be depended on in design.

REPLACEMENT

The replacement method involves the removal of the in situ liquefiable soil, and replacement with a non-liquefiable material. It is useful for treatment of shallow liquefiable layers or creating a mat of dense uniform ground to support lightweight structures. The engineered replacement fill can be cement treated soil from the excavation or well graded dense gravel.

DENSIFICATION

Densification is the most common mechanism of ground improvement and involves rearranging the soil particles into a tighter configuration, resulting in increased density. This increases the shear strength, stiffness and liquefaction resistance of the soil.

There are a variety of techniques available (refer to Table 3.1). Compaction techniques are most suited to sandy soils with low fines and can treat soils to depths of 4–12 m and deeper depending on the ground conditions, technique and plant. One of the major disadvantages is the noise and vibration produced during construction.

SOLIDIFICATION

Solidification involves either in situ mixing of cementitious or other additives into the soil or filling the voids with a reagent resulting in the soil particles being bound together. This will prevent the development of excess pore water pressure, preventing the occurrence of liquefaction.

Solidification techniques are typically expensive compared to other methods. Solidification techniques can be used to treat the full range of soils susceptible to liquefaction, including low plasticity silts to depths of 30 m or more although there are some limitations with specific techniques. The advantages are: high confidence in the end product when the entire depth of liquefiable soil is treated, low vibration and noise during construction and the ability to treat beneath existing structures.

REINFORCEMENT

When saturated sand deposits are sheared during seismic loading, excess pore water pressure is generated reducing the stiffness and strength of the soil and increasing strains. The aim of reinforcement is to reduce shear deformation in the ground during an earthquake to mitigate the development of excess porewater pressures. The increased composite strength of the reinforced ground also mitigates ground deformation and subsidence of the structure if liquefaction were to occur.

Reinforcement typically involves the construction of underground walls which usually intersect to form a lattice. The subterranean walls can be formed using ground solidification techniques or contiguous concrete piles. The advantages and disadvantages are similar to those for solidification except that it is less expensive and there is not the same level of confidence in prevention of development of excess porewater pressures in the soil contained within the lattice walls.

Grids of stiff isolated piles have been used to improve liquefiable soils by reinforcement. Open grid systems are relatively flexible and do not offer the same degree of confinement as a lattice. They are less reliable than other methods of improvement and generally only applicable for lightweight structures and where the piles extend to a competent non-liquefiable stratum.

DRAINAGE

Drainage to mitigate liquefaction potential typically requires either:

- installation of vertical drains typically installed at 1–2 m intervals to allow the rapid dissipation of excess pore pressures generated during earthquakes to prevent liquefaction development, or
- desaturating potentially liquefiable soil, by permanently lowering groundwater or gas entrainment.

Drainage methods are not widely used in New Zealand. Vertical drains can be installed with relatively low vibration and noise compared to compaction methods and are typically cheaper than solidification. However, the required drain spacing is sensitive to the soil permeability which is difficult to measure, their effectiveness cannot be verified and if liquefaction is triggered, they do not constrain ground movement.

Permanent dewatering can be a useful means of treatment when pumping is not involved and the water can easily be disposed of. If continuous pumping is necessary, there can be substantial ongoing running and maintenance costs and there is a risk of failure in aftershocks if the dewatering system is damaged in the initial earthquake.

3.2 Seismic response of buildings supported on improved ground

EFFECTS ON STRUCTURAL RESPONSE

Ground improvement can greatly increase the stiffness of the soil profile. It is well understood that the stiffness of the soil has a marked effect on seismic ground motions at the surface. Stiffening the soil can amplify accelerations at the surface but decrease displacements. Due to increased stiffness, the improved ground may also have an effect on seismic wave propagation and seismic response of the ground at neighbouring sites. Therefore, potential effect of ground improvement on seismic response of neighbouring sites should be considered in the design process.

DEFORMATION MODES

Well-engineered improved ground has proven to perform well in previous earthquakes (Mitchell & Wentz, 1991). The following paragraphs discuss deformation mechanisms and behavioural characteristics that need to be considered in the well engineered design of ground improvement to mitigate liquefaction.

In many cases, ground improvement will not fully eliminate the effects of liquefaction. Settlement of buildings with shallow foundations supported on improved ground will result from shear and volumetric changes within the improved zone and in the soils surrounding or underlying the improved zone. Module 4 discusses foundation performance in detail.

The prevalent mode of deformation depends on the ground improvement method adopted, size of the improved zone and its stiffness; the size, weight and stiffness of the structure (and the distribution of weight and stiffness) and the extent of the liquefiable soil beneath the improved zone.

Except for methods that completely solidify or replace the liquefiable soils with stiff (cemented) low permeability materials, subsidence, can develop from shear deformation in the improved ground under loading from the building. This is often more pronounced at the perimeter of structures, particularly for tall and heavy structures that can exert large loads on perimeter foundations. The magnitude of subsidence can be exacerbated by softening of the improved soils with cyclic shearing, the associated development of excess porewater pressure and the migration of excess porewater pressures from the surrounding liquefied soil into the improved zone. Reconsolidation of soils in the improved zone as excess pore-pressures dissipate will cause additional subsidence.

Lattice and columnar reinforcement elements can be subjected to considerable bending, shear and axial stress.

With partial depth of improvement, settlement and tilting of the improved ground overall can develop from shear induced deformation in the liquefied soil beneath the improved zone, reconsolidation of the liquefied soils as porewater pressures dissipate and ratcheting effects during earthquakes, similar to the mechanisms of settlement for shallow foundations on liquefaction prone sites as described in Module 4.

Ground improvement in areas of lateral spreading can experience large compression and tension stresses from dynamic and kinematic forces imposed on it by the surrounding spreading ground. This can cause horizontal displacement, stretching and shear deformation of the zone of ground improvement.

Table 3.1: Ground improvement techniques

TECHNIQUE	DESCRIPTION	SOIL CONDITIONS	TREATABLE DEPTH (M)	ADVANTAGES	LIMITATIONS	RELATIVE COSTS
REPLACEMENT						
Dense gravel replacement	Excavation of liquefiable soils and replacement with dense gravel	All soils	2–6 m	– Uses conventional construction equipment and methods – High confidence in level of treatment	– Dewatering and temporary shoring may be necessary – Subsidence of neighbouring properties associated with dewatering – Moderate vibration and noise with compaction of replacement materials	Low
Stabilised soil replacement	Excavation of liquefiable soils and replacement with stabilised soil	All soils	2–6 m	– Can treat the excavated soil and return to excavation (no cut to waste or fill import) – High confidence in level of treatment		Low to moderate
DENSIFICATION METHODS						
Dynamic compaction	Compaction of soils by repeated dropping of a 5–20 T tamper from a crane in a 2–6 m grid	Gravels, sand and silty sand	4–7 m	– Fast and economic – Moderate experience in NZ, extensive experience overseas. Proven effectiveness in earthquakes – Easily verifiable	– High vibration and noise, not suitable in built up areas – Clearance for crane – Full scale trial typically required to confirm effectiveness and refine the design	Low
Dynamic replacement	Construction of 2–3 m diameter gravel piers in a 6–12 m grid with dynamic compaction equipment	Sands, silty sands and silt	4–7 m			
Impact roller compaction	Compaction of near surface soils with a square sided high energy roller pulled behind a tractor	Gravels sands and silty sand	2–4 m	– Fast and economic – Easily verifiable	– Specialist equipment required – Limited depth of improvement, especially for sites with interbedded layers of silt – High vibration, not suitable in built up areas	Low
Vibro-compaction	Densification by vibration with a vibroflot hung from a crane in a 1.8–3.0 m square or triangular grid	Gravelly sand, sand and sand with minor silt	6–25 m +	– Secondary benefits of increased lateral stress – High level of construction quality control available – Can treat to large depths – Easily verifiable, proven effectiveness in earthquakes	– Requires specialist equipment – Moderate vibration, not suitable near existing structures – Containment and treatment of sediment produced during construction – Clearance for crane	Moderate
Vibro-replacement	Construction of dense granular columns using a vibroflot in a 1.8–3.0 m square or triangular grid	Gravelly sand, sands, silty sand, silt	6–25 m +	– Secondary benefits of reinforcement, drainage and increased lateral stress – High level of construction quality control available – Extensive experience in NZ and overseas – Proven effectiveness in earthquakes – Can treat to large depths – Easily verifiable	– Requires specialist equipment – Moderate vibration, not suitable near existing structures – Containment and treatment of sediment produced during construction – Clearance for crane – Not suitable for soils containing cobbles, boulders or other large inclusions	Moderate
Granular compaction piles	Densification by vibration and displacement with gravel to form columns in a 1.5–2.5 m grid	Sands, silty sand, silt	Up to 16 m	– Secondary benefits of reinforcement, drainage and increased lateral stress – Extensive experience in NZ and overseas. Proven effectiveness in earthquakes – Can be constructed using conventional equipment – Dry method, less sediment to manage compared to wet vibro-replacement	– Moderate vibration, not suitable near existing structures – Clearance for equipment	Moderate
Displacement auger piles	Construction of granular or concrete columns in a 1.5-2.5 m grid with a displacement auger	Sands, silty sand, silt	Up to 16 m	– Secondary benefits of reinforcement, drainage (for granular columns) and increased lateral stress – Low vibration construction – Can be used near existing structures when allowance is made for heave around columns	– Requires specialist equipment – Not as effective at compacting sands as compaction piles – Clearance for equipment	Moderate
Driven compaction piles	Densification by displacement and vibration with driven (timber or precast concrete) piles in a 1.2–1.6 m grid	Sands, sand with some silt	Up to 16 m	– Secondary benefits of increased lateral stress. Some reinforcement possible but typically low friction between piles and soil limits reinforcement effects	– Heave of ground near improvement piles – Moderate vibration and noise, not suitable immediately adjacent existing structures	Moderate
Compaction grouting	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure	Sands and silty sand	25 m	– Low vibration, compact plant, can be used to treat soil beneath existing structures	– Not suitable for treatment at shallow depths where there are low confining pressures.	Moderate
Resin injection	Densification from injection of rapidly expanding resin	Sands and silty sands	10 m	– Low vibration, compact plant, can be used to treat soil beneath existing structures	– Limited experience and capability in New Zealand	Moderate

TECHNIQUE	DESCRIPTION	SOIL CONDITIONS	TREATABLE DEPTH (M)	ADVANTAGES	LIMITATIONS	RELATIVE COSTS
Surcharging	Consolidation under the weight of the surcharge fill	All soils	Dependent on ground conditions and width of surcharge	– Secondary benefit of increased lateral stresses when soils are over consolidated – Low vibration	– Space for surcharge batters – Settlement of area near surcharge	Moderate
Blasting	Charges installed in a triangular grid with 3–8 m spacing at multiple depths. Shock waves and vibrations cause limited liquefaction, displacement, remoulding and settlement to higher density	Saturated gravelly sand and sands	25 m +	– Simple technology – Can treat large areas at great depths	– Limited to deep depths and green field sites away from the built environment due to vibration and noise during treatment	Moderate to high
SOLIDIFICATION METHODS						
Mass stabilisation	Lime, cement or bitumen introduced through rotating in-place mixer	Sands, silty sands, silt	2–6 m	– Low vibration and noise compared to other methods – Suitable for sites with interbedded cohesionless and cohesive soils or soils with higher fines that do not respond to tamping or vibration	– Specialist equipment required. Results depend on degree of mixing and compaction achieved. – Not suitable for soils with boulders, cobbles, interbedded dense gravel layers. May not be suitable for soils with organics	Moderate to high
Deep soil mixing	Lime, cement or bitumen introduced through vertical rotating augers or proprietary mixers to form stabilised columns	Sands, silty sands, silt	2–20 m	– Low vibration and noise compared to other methods – Suitable for sites with interbedded cohesionless and cohesive soils or soils with higher fines that do not respond to tamping or vibration	– Specialist equipment required – Brittle elements (individually) – Not suitable for soils with boulders, cobbles, interbedded dense gravel layers. May not be suitable for soils with organics	High
Jet grouting	High-speed jets at depth excavate, inject and mix a stabiliser with soil to form columns or panels	Sands, silty sands, silt	2–25 m+	– Low vibration, compact plant, can be used to treat soil beneath existing structures – Suitable for sites with interbedded cohesionless and cohesive soils or soils with higher fines that do not respond to tamping or vibration	– Specialist equipment required – Brittle elements (individually) – Not suitable for soils with boulders, cobbles or other inclusions that could mask jets. May not be suitable for soils with organics	High
Permeation grouting	Low viscosity cement or chemical grout pumped into the ground in a grid pattern. The grout permeates through the soil filling the pores with cement, and/or other reagents	Medium silts and coarser	20 m +	– No excess porewater pressures generated. Can localise treatment to selected layers. – Low vibration, compact plant, can be used to treat soil beneath existing structures – Produces no spoil	– Interbedded fine soils can hamper dispersion of grout. Most suited to homogeneous permeable sands – Can be difficult to contain in high permeability layers, risk of contamination of nearby waterways	High
REINFORCEMENT METHOD						
Lattice walls	Formation of a grid of intersecting walls with a 5–7 m grid spacing using either contiguous piles, jet grout or deep soil mixing (DSM). Shear strain in the soil between the walls is reduced to prevent liquefaction	Depends on construction technique	4–25 m +	– Lattice contains soils even if they liquefy – Can be more cost effective than complete stabilisation – If constructed with jet grout, can be used to treat soils beneath existing structures – Treatment zone may not need to extend beyond the structure footprint.	– Depends on construction technique (see above) – Unreinforced walls are susceptible to brittle failure but less than individual columnar elements	Moderate to high
Open grid of stiff columns	Formation of a grid of individual columns with a 1.5-2.5 m grid spacing using either timber or concrete piles, jet grout or DSM	Depends on construction technique	4–25 m +	– Provides some mitigation to differential subsidence even if the soils do liquefy assuming the tips of the columns are in a non-liquefiable competent layer	– Depends on construction technique (see above) – Unreinforced columns are susceptible to brittle failure – Treatment zone needs to extend beyond the perimeter	Moderate to high
DRAINAGE METHODS						
Permanent dewatering	Lowering of the water table by gravity drainage or pumping	Gravelly sand, sands and silty sand	2–8 m	– Can be a simple and low cost method to treat large areas if permanent dewatering can be achieved by gravity drainage	– Cost of running and maintaining pumps – Risk of pump failure – Subsidence associated with the increase in effective stress	High
Vertical prefabricated drains	Relief of excess pore water pressure to prevent liquefaction. Drains can be prefabricated or constructed from gravel/sand	Gravelly sand and sand	5–25 m +	– Simple, low vibration construction techniques, can be a relatively cheap to construct	– Effective design requires very good knowledge of the ground conditions and permeability – If triggered, the effects of liquefaction are not greatly reduced	Moderate to high

4 Performance requirements

Before selecting and designing a ground improvement system to mitigate liquefaction effects at a site, it is necessary to understand the performance requirements of the improved ground and the structural system.

This section briefly discusses the minimum regulatory performance requirements for building work in New Zealand, the elements and interactions between elements that affect the performance of structures on sites with ground improvement to mitigate liquefaction and the performance criteria for ground improvement.

4.1 Regulatory requirements

The New Zealand building Code specifies the minimum requirements for performance of new buildings in New Zealand. New buildings are typically designed for two limit (or damage) states, the serviceability limit state (SLS) and the Ultimate limit state (ULS). More important buildings are also designed for a third limit state, SLS 2.

The ULS is concerned with avoiding instability and collapse in rare events throughout the life of the building. The SLSs are concerned with maintaining amenity and restricting damage in relatively more frequent and smaller events in the life of a structure. Module 4 gives more detailed discussion of the legislative requirements.

Currently in New Zealand differences arise in the performance requirements between new and existing structures. For existing structures, the minimum legal requirement, specified in the Seismic Assessment of Existing Buildings — Technical Guidelines for Engineering Assessment (MBIE, 2017) is less than the new building standard.

4.2 Ground improvement performance objectives

The general philosophy for the design of ground improvement is to eliminate liquefaction and lateral spreading or mitigate their effects to the extent needed to meet the design performance criteria for the structure.

In this context, the effectiveness of ground improvement should be assessed within the performance-based design framework by estimating the reduction of effects of liquefaction in relation to a no-improvement case, and by assessing the seismic response in relation to specific performance objectives for earthquake loads associated with different return periods. Qualitative effects of ground improvement on the dynamic response of foundation soils, structure and soil-structure system should be also considered in this evaluation. Such relatively rigorous performance requirements imply the need for adequate standards for design, construction control and verification of the effectiveness of ground improvement.

Performance criteria for the acceptable damage, settlement and differential settlement for each damage state should be developed collaboratively between the owner/developer, structural engineer and geotechnical specialist to get an overall system that meets regulatory (minimum) requirements and the expectations of the owner/developer.

It is often not economic, nor required in a regulatory sense, to completely eliminate liquefaction beneath buildings with ground improvement. Apart from methods that completely solidify or replace all liquefiable soils with non-liquefiable material, excess pore water pressures can develop within the zone of improvement. The frequency of earthquake at which these aspects start to have a significant effect on the amenity of the structure should be discussed and agreed with the owner/developer.

It is important to discuss and agree the level of ground improvement and seismic performance of the building with the owner/developer who may prefer ground improvement options that are more robust than those required to comply with the New Zealand Building Code to ensure that the foundation system or both the foundation system and the superstructure will be repairable after earthquakes larger than SLS or even ULS earthquakes. It should be noted that if the improved ground fails in a seismic event where the structure remains undamaged or reparable, in many cases it can be impossible to repair the improved ground beneath the existing buildings to the pre-earthquake level, unless the ground improvement is designed to remain undamaged or be reparable after the design earthquake.

Consideration should be given to the resilience of the ground treatment and the overall response should be ductile. The weight and stiffness of the structure and its foundations; the type, extent, and stiffness of the ground improvement; the ground conditions, characteristics of earthquake shaking and the extent of liquefaction triggered in an earthquake, all affect seismic performance. In assessing seismic performance and resilience, the uncertainties in these parameters and the interaction between the superstructure, connecting infrastructure, foundation, improved ground and native soil need to be considered holistically. The high degree of uncertainty in many of the parameters affecting seismic response implies the need to assess the sensitivity of the system response to each parameter and apply an appropriate level of redundancy in the design. Sensitivity assessment should be undertaken as part of any ground improvement design and discussed within the foundation options and design reports.

Improved structural measures that can be incorporated to reduce damage susceptibility due to liquefaction, improve resilience and reduce or eliminate the need for ground improvement. These can comprise:

› Use of robust matts or a stiff grid of intersecting ground beams instead of standalone footings.
› Making above ground structural elements or connections between structures flexible and ductile to cope with total and differential settlements or lateral spread.
› Constructing foundation systems that seismically isolate the building from the ground and allow it to be relevelled.
› Pile foundations to competent ground that is not underlain by liquefiable soils to prevent bearing failure and mitigate settlement and uplift (buoyancy).
› Control of ground deformation and structural performance by structural measures (rigidity of the structure, rigid rafts, sheet piles to confine liquefiable material, geogrids, base isolation of structures).

Consideration also needs to be given to the performance requirements for auxiliary facilities, emergency egress facilities and connecting utilities. Utilities are susceptible to damage at the margins of ground improved zones due to the discontinuity in soil properties and stiffness.

5 Ground improvement design

5.1 Design process

Engineering assessment, consideration and design process for ground improvement can be summarised as follows:

- Determine performance requirements for the building and its foundation system based on the NZ Building Code, NZS1170, Module 1 and Module 4 and the Seismic Assessment of Existing Buildings — Technical Guidelines for Engineering Assessment (MBIE, 2017).
- Assess site conditions, ground conditions and geohazards (geologic hazards) including seismicity and susceptibility to liquefaction and lateral spreading (refer to Modules 1, 2 and 3). Where existing geotechnical information is insufficient, a geotechnical investigation should be carried out (refer to Module 2).
- Assess if liquefaction will be triggered, severity of liquefaction and the free field effects of liquefaction at the site (refer to Module 3).
- Assess the lateral spreading hazard at the site and the potential for differential lateral displacement across the building footprint.
- Assess the effects of liquefaction on the structure (with shallow or pile foundations and no ground improvement) and compare with the performance criteria. Consider whether there are readily available structural options to reduce susceptibility to damage from liquefaction. Where reasonable structural options alone are not sufficient to satisfy the performance requirements, consider ground improvement options.
- Select suitable methods for ground improvement (refer to Section 5.2).
- Design the extent (depth and size in plan) of improvement needed to meet design objectives. Consider soil-ground improvement-structure interaction. Early engagement between the structural and geotechnical engineers, and where practicable contractors, will enable a more efficient and holistic assessment of ground improvement and foundation options

(also refer to Module 4). A few iterations may be required to optimise ground improvement design and achieve a cost-effective solution.

- Design the size and arrangement of the ground improvement; determine material requirements, eg unconfined compressive strength of soil-cement mixture.

  The usual goal of ground improvement is to eliminate liquefaction. However ground improvement does not necessarily need to eliminate liquefaction within the improved zone but should control and mitigate the effects of liquefaction, and meet the performance criteria.

- Determine quality control (QC) and quality assurance (QA) requirements. In many cases a ground improvement trial will be required to confirm design assumptions and QA methods and optimise the design.

5.2 Selection of ground improvement method

The following factors should be considered when selecting an appropriate remediation technique:

- The required performance of the ground improvement system, its durability, reliability and resilience within the context of the overall structure-foundation-ground improvement-ground system.
- The effectiveness of each method to treat the site soil conditions and meet the performance requirements. Further guidance on the suitability of techniques to treat different soil types is presented in Table 3.1 and discussed in Sections 6–10.
- Site constraints (space to boundary, etc).
- Construction constraints (noise, vibration, contamination, resources and specialist plant and labour availability).
- Field verifiability.
- Environmental impact eg settlement from permanent drainage and effects on neighbouring infrastructure.
- Cost.
- Safety in design.

5.3 Extent of ground improvement below buildings with shallow footings

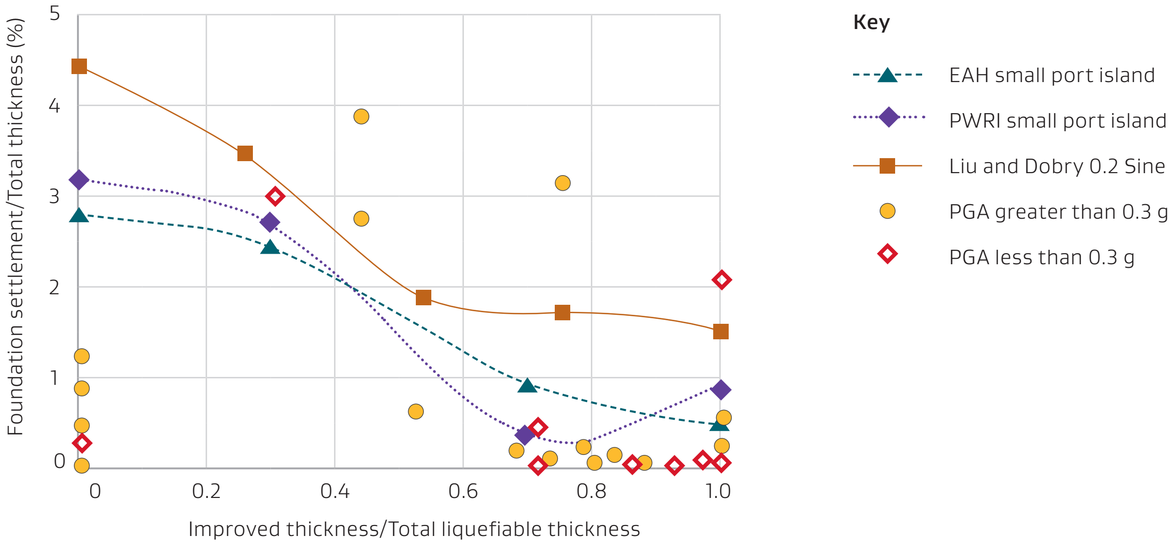
Generally, even when the soil undergoes liquefaction over a wide area and considerable depth, the region requiring improvement is limited to the zone which controls the stability and structural performance of the structure.

In principle, the minimum depth and lateral extent of improvement required is dependent on many factors, such as ground conditions, type of ground improvement, purpose of ground improvement (ie to mitigate lateral spreading or settlement, or both), performance requirements, foundation type, depth of liquefaction, and interaction between structure, improved ground and natural ground at the perimeter of the improved section. Moreover, the extent of improvement is dependent on the stiffness and strength of the improved ground, generally set to meet performance requirements.

This section presents simple approaches for determining the extents of ground improvement for normal importance, small to medium buildings and as a first stage of design for higher importance, heavy or complex structures.

Effective stress dynamic numerical analysis of the structure, its foundations, the improved ground and the surrounding natural soils is a useful and sometimes necessary tool for design from the perspective of understanding the complex system interactions and its capacity to predict strains and displacements. However, dynamic effective stress numerical analyses techniques are not appropriate for all situations and are typically only viable for large complex projects. Modules 3 and 4 discuss numerical analysis in more detail.

Figure 5.1: Normalised improvement depth vs normalised building settlement (Liu and Dorby 1997, Hausler 2001, 2002)



DEPTH OF TREATMENT

Ideally, the full depth of liquefiable soils should be treated beneath a structure. This eliminates subsidence from cyclic ratcheting, shear deformation and reconsolidation of liquefied soils that otherwise underlie the improved zone. It also eliminates softening of the improved zone with the upward dissipation of excess porewater pressures from the liquefied soils beneath and the potential for seepage erosion of soil under the improvement zone.

Full depth improvement is unlikely to be economic for sites underlain by deep liquefiable deposits and partial depth improvement can often give acceptable performance by reducing the magnitude of settlement. Assessment of nearly 60,000 lightweight single family dwellings in Christchurch following the Canterbury Earthquake Sequence clearly showed that less structural damage occurred in liquefaction prone areas containing an intact relatively stiff non-liquefying crust that was at least 3 m thick (Wansbone and van Ballegooy, 2015)

From case history studies, Hausler and Sitar (2001) noted that one of the reasons why unacceptable performance was noted in the majority of ground improvement cases they investigated was due to inadequate remediation zone depth. Centrifuge studies on this topic (Liu and Dorby, 1997; Hausler 2002) came to a similar conclusion.

Figure 5.1 summarises measured settlement (normalised against the thickness of the liquefiable layer) vs portion of depth of liquefiable soils treated from case studies and centrifuge tests. The case studies and centrifuge tests indicate a marked increase in settlement for treatment depths that are less than 50 percent of the thickness of the liquefiable layer.

> **Note**
> For the two outlier cases, the magnitude of settlement at these sites were compounded by lateral spreading.

The case histories and centrifuge testing highlights the importance of taking a cautious approach and due account of the increased performance uncertainty when designing solutions with partial depth of treatment.

In a simplified approach, the bearing capacity of the improved ground (considering it to be a rigid body) should be assessed using conventional bearing capacity theory (see Module 4) with reduced strengths and stiffness for liquefiable soils to establish a minimum depth of improvement.

The improved zone needs to be stiff enough to bridge liquefiable soils. The overall stiffness of the improved zone is a function of both the modulus of the improved zone and its depth. Psuedo-static numerical analysis can be used to give some insight into the deformation characteristics and adequacy of the depth of improvement to mitigate differential settlement of the structure. In this analysis, the stiffness of the improved zone may need to be reduced for the effects of excess porewater pressure developed from cyclic loading during earthquake shaking and migration from adjacent and underlying liquefied soils.

LATERAL EXTENT OF TREATMENT

The required lateral distance or width of soil improvement outside the perimeter of the structure is determined by the size of the zone that controls the stability and deformation of the structure, even if liquefaction occurs over a wide area (PHRI, 1997). However, the zone that controls the stability of the structure is complex. Factors that need to be considered when determining the lateral extent of improvement include the following:

› stresses applied to the improved ground by the building during earthquake shaking. Compressional and shear stresses near the edge of structures can fluctuate greatly and may be higher compared to static stresses.

› strength and stiffness of the improved ground and the potential for a reduction in strength and stiffness due to excess pore pressures generated in the surrounding liquefied soil migrating laterally into the improved zone during and after shaking.

Referring to Figure 5.2, model tests and analysis of ground improved by densification over the full depth of liquefiable soils (Iai et al, 1991) indicate that in the soils bounded by the square ABCD, the pore pressure ratio, r_U is often greater than 0.5. The triangular area ACD exhibits particularly unstable behaviour and hence, this part should be treated as liquefied in the design of ground improvement that utilises densification techniques. Figure 5.2 demonstrates indicative effects of excess pore pressure migration only and should not be misinterpreted.

Within the foundation footprint, the improved zone should extend upwards to the underside of the foundation or to the drainage blanket beneath the foundation. For densification methods, the improved zone beyond the foundation footprint should extend upwards to the ground surface level or to the drainage blanket located immediately beneath the ground surface.

It is recommended that ground improvement should extend laterally under the full footprint of the building and a distance equal to two thirds of the thickness of the liquefiable layers (including any non-liquefiable layers between the liquefiable layers) beyond the edge of the building footprint.

For light structures, lateral extent of ground improvement beyond the building footprint can be reduced to a half of the thickness of the liquefiable layers (including any non-liquefiable layers between the liquefiable layers).

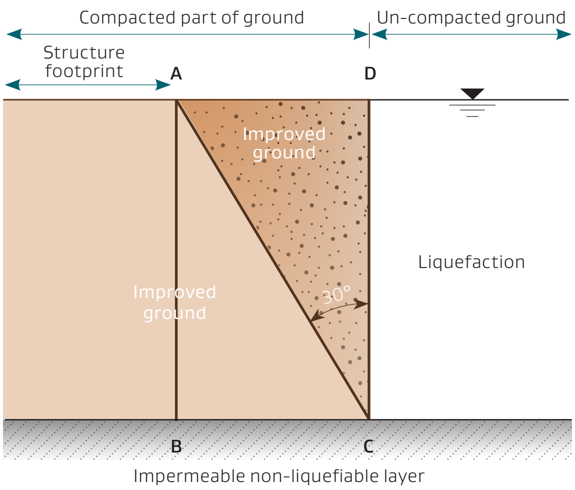
Extending ground improvement beyond the boundary of the main ground improvement zone can be considered to create gradual transition from the improved zone to unimproved ground that would reduce the effects of total and differential settlement on underground services. The depth of ground improvement in the transition zone should be gradually reduced from the full depth at the edge of the main ground improvement zone to zero at some distance from the main ground improvement zone.

There is no need to consider the effects of pore pressure migration when an impermeable barrier such as a diaphragm wall has been installed at the perimeter of the improved zone to shutout the inflow of pore water from the liquefied perimeter soils.

It is sometimes not possible to extend improvement the recommended distance beyond a structure because of the presence of other structures, property boundaries, or utilities. In these cases, it may be possible to cantilever the foundation over the area of ground improvement affected by lateral migration of porewater pressure.

Lattice ground improvement structures and other ground improvement methods that solidify or constrain the lateral deformation of soil beneath the foundation typically do not need to extend far beyond the foot print of the building.

Figure 5.2: Area of softening in ground improved by densification (ACD) due to porewater pressure migration (after Iai et al, 1991)



MITIGATION OF LATERAL SPREADING EFFECTS ON BUILDINGS

Damage to structures may be especially severe where they are subjected to lateral spreading in conjunction with liquefaction. Lateral spreading is potentially significant for sites when a free face such as a river channel or the coastline is within a few hundred meters of the site. However, lateral spreading also occurs on sloping sites, or sites underlain with variable and sloping zones of liquefiable soils, eg infilled river channels. Methods for assessing the lateral spreading hazard at a site are discussed in Module 3. Refer to Module 4 for more discussion on the effects of lateral spreading on buildings.

There is a high level of uncertainty in lateral spread predictions and this uncertainty needs to be managed in the design of lateral spreading mitigation measures. The consequences of lateral spreading on a building site are horizontal displacement, stretching of the ground (differential lateral displacement) and subsidence.

Strategies to mitigate lateral spreading and its effects at building sites include:

- Construction of structural walls separate from the building. These could be soldier pile walls tied back to anchor piles that cantilever from non-liquefiable soils or caissons founded on non-liquefiable ground.
- Using a buttress of ground improvement on the down slope side of the building but separate from the building foundations. This may be desirable for piled structures in laterally spreading zones as the greater stiffness and strength of the improved soils could place larger kinematic loads on the piles and increase structural inertia.
- Improving the ground under the structure to mitigate lateral spreading as well as provide a suitable platform for the building.
- A combination of these treatments except that ground improvement should extend under the entire footprint of the building or not at all to avoid high contrasts in stiffness beneath the building that could cause differential subsidence and increase torsional response.

For some past projects, partial ground improvement was carried out, eg beneath parts of buildings that are closer to free surfaces (river banks, seawalls etc.). Recent assessments of seismic damage to buildings indicated that partial ground improvement had adverse effect of seismic response of the ground, foundations and superstructure (including torsional response) and resulted in increased structural damage. (Siddiqui, 2019). It is therefore recommended that partial ground improvement be avoided where possible.

Currently, field case histories and research to support guidelines on the extent of the treatment zone to guard against lateral spreading are scarce, and, if available, they are not comprehensive. It is known that the area that controls the stability and deformation of the structure when subjected to lateral spreading is complex and the size of zone that is necessary to protect the structure from significant lateral deformation and subsidence associated with lateral spreading requires careful consideration.

A simplified approach to determine the extent of the treatment zone is by calculating the extent of improvement needed to get a factor of safety of 1.1 with post-earthquake strengths for the native and improved ground. Satisfaction of this criteria should mean that the ground does not spread substantially after the earthquake has passed but will not entirely prevent horizontal displacement and deformation beneath the structure.

Lateral deformation of the improved zone can be estimated by applying horizontal pressure to the upslope side of the improved zone and frictional loads along the sides of the improved zone parallel to the direction of spreading, using an average shear modulus for the improved ground, reduced for stress strain non-linearity and any anticipated excess pore pressure. Some recommendations for calculation of the applied horizontal loads is given in PHRI (1997) and JGS (1998).

The front of the improved zone will practically be unsupported by the spreading ground and the associated drag friction on the sides of the zone will vary spatially, reducing with depth and increasing distance from the front of the improved block inducing tension in the front of the improved zone.

At sites where there is potential for lateral spreading, foundation elements should be well tied together to reduce the risk of elongation between supports. A slip layer beneath shallow foundations, constructed from two layers of HDPE sheet for example, can also be used to isolate the structure from stretching. When selecting the method of improvement to mitigate lateral spreading, consideration needs to be given to differential lateral displacement and stretching within the improved zone.

5.4 Drainage blankets

Apart from methods that solidify the ground or replace it with cemented non-liquefiable materials, some development of excess porewater pressure is almost inevitable within the improved zone in strong earthquake shaking. Potential migration of excess porewater pressures from liquefiable soil below or around the improved zone may further exacerbate porewater pressures in the improved ground beneath structures.

Except for ground treatment involving replacement with clean granular fill or full depth solidification, a filtered drainage blanket should be installed over the improved zone for all new builds where there is suitable access. The drainage blanket should be designed to allow relief of excess pore pressures without ejecting soil on the surface or causing uplift on the base of ground bearing floors or shallow foundations. If some liquefied material is left in place, volume strains of liquefiable material located beneath the improved zone associated with soil densification should be used to assess volumes of expected groundwater upward flow through stone columns, and the drainage blanket should be designed accordingly. With volume strains in the order of 1 percent, flows may not be substantial but for strains in the order of 5 percent the flows may be large. The flow velocities should also be considered to make sure that excess pore pressure can dissipate quickly.

Gravel drainage blankets also improve the distribution of loads from shallow foundations across the stiff inclusions (piles for example) within the improved zone. BS8006 describes load transfer mechanisms and gives design recommendations.

Where only partial depth of liquefied soils are solidified or replaced with stabilised soil, the drainage blanket protects against ejecta penetrating through cracks and alleviates the effects of abrupt differential movement at cracks. A perimeter subsoil drain installed around the outside of the improved area to relieve water pressure and prevent soil seepage erosion at the edges of solidified zones or lattice structures is prudent where the improved zone does not extend beyond the perimeter of the structure.

Drainage blankets should be a minimum of 300 mm thick and consist of clean aggregate either placed on a filter fabric or with a grading designed to filter the subgrade.

5.5 Other considerations

QUALITY CONTROL AND QUALITY ASSURANCE

The effectiveness of ground improvement is highly dependent on the skill of the constructors and the construction equipment used. The importance of post improvement testing cannot be over emphasised in order to verify, where possible, that the required level of treatment has been achieved.

A range of construction quality control methods have been developed and continue to be developed. These include, for example, automatic measurement of probe depth and compaction time between lifts for vibro-compaction or the quantity of stone placed per metre depth of stone column. Construction quality control is essential for the production of a consistent product. It will also aid in understanding any issues that may arise from quality assurance testing undertaken to verify the effectiveness of the ground treatment. Both quality control and quality assurance testing are required and construction quality control should not be seen as a substitute for post treatment quality assurance verification.

All construction quality control and quality assurance records should be supplied to the consenting authority together with the relevant producer statements on completion of the ground treatment.

ENVIRONMENTAL CONSTRAINTS

The following environmental constraints need to be considered in the design of ground improvement:

- the space available for construction
- noise and vibration effects on adjacent properties during construction
- the potential for temporary and permanent changes to the groundwater regime
- whether there is a ground or groundwater contamination hazard at the site
- the archaeological significance of the site.

Most ground improvement techniques use relatively inert materials and, in themselves, do not contaminate the ground. The exceptions are some non-cementitious grouts and, to a lesser extent, some treated timber piles.

On the other hand, ground improvement can increase the dispersion of pre-existing ground contaminants either through the construction process (eg with the excavation of contaminated soil) or while in service (eg cross contamination of aquifers) and can be a health and safety hazard. A ground contamination hazard assessment may be carried out during the design phase. Even if site investigations and assessment indicate a low contamination hazard, protocols should be put in place for the management of contaminated soils if they are encountered during construction.

It is important to note that it will be necessary to comply with various requirements relating to hours of work, erosion and sediment control, contamination of groundwater, rivers, lakes and the sea, construction noise and vibration.

Ground improvement can damage tree roots and underground services. This should also be taken into account while considering the ground improvement options and footprint.

For geothermal sites, the effect of ground improvement on the geothermal regime of the site and potential hazards (geothermal chemicals in groundwater, stream and other gases under pressure, potential for hydrothermal eruption, geothermally altered ground etc.) should be considered in the design process.

6 Replacement methods

6.1 Outline

The replacement method involves the removal of the in situ liquefiable material, and replacement with a non-liquefiable material. The replacement material may be non-liquefiable by composition or by density/stress state. Well compacted, well graded gravel or soil mixed with cement or other additives are commonly used for replacement in liquefaction remediation.

Where ground conditions are suitable it may be possible to remove and recompact the same material to a higher density.

Replacement with dense granular fill has been a common method of ground improvement in the rebuild of Christchurch following the 2011 Christchurch earthquake. The method was proven effective at mitigating differential subsidence for lightweight structures in the ground improvement trials undertaken by EQC in 2013.

There is a high degree of confidence in the ability of the replacement soil to resist liquefaction and it uses construction equipment and practices that are widely available and easily tested.

6.2 Site conditions suitable for replacement

The replacement method is most suited for areas with a shallow liquefiable layer but replacement can also be used to form a uniform stiff platform for new structures where acceptable structural performance can be achieved by only partial replacement of the depth of liquefiable soils. The replacement method can be used to treat both sands and silts.

The depth of treatment is typically limited by the feasibility of excavating and dewatering for placement and compaction of materials below the water table and, where the site is near existing structures, the cost of temporary excavation support to protect neighbouring structures from damage.

6.3 Design considerations

Module 5a includes specifications for the construction of dense gravel mats for lightweight residential structures. These can be adapted for use with larger structures.

Placement of a limited depth of clean, open graded granular fill or tremied stabilised flowable fill could be used for construction of replacement fill below water level. Where compaction of backfill below water level is required design should consider the risk and potential consequences of the required density not being achieved.

Dewatering can affect a wide area beyond the site and the associated increase in effective stress can cause subsidence at the site and in neighbouring buildings. The risk of subsidence is greatest when there are organics and soft soils. Powers et al (2007) gives guidance on practical solutions and design methods for dewatering.

It is good practice to place a layer of filter fabric and geogrid below granular replacement fill. These facilitate compaction of the initial fill layers, mitigate migration of fines from underlying layers with dissipation of excess porewater pressures and provide some protection against lateral stretch. Because of their low axial stiffness, a single layer of geogrid typically does little to increase the overall flexural stiffness of a granular raft.

Cement stabilised soils are brittle and have low strength in tension. The replacement dimensions and modulus should be designed to avoid concentration of strains at large widely spaced cracks that could cause abrupt differential settlement of the structure. This is especially important where only partial depth of soils prone to liquefaction are treated. A granular layer placed over the cemented fill can smooth out abrupt changes in level or grade beneath shallow foundations.

The compaction of replacement materials can involve moderate levels of noise and vibration that could be a nuisance or damaging to neighbouring properties. NZS 6803 provides guideline noise limits and management practices for construction works. The State Highway construction and maintenance noise and vibration guide provides practical information and advice on prediction, management and mitigation measures.

7 Densification methods

7.1 Outline

Densification or compaction methods involve rearranging the soil particles into tighter configuration, resulting in increased density. This increases the shear strength and liquefaction resistance of the soil, and encourages a dilative instead of a contractive dynamic soil response. Densifying loose sandy deposits with vibration and/or impact has been used extensively, making it the most popular liquefaction countermeasure.

An increase in soil density can be achieved through a variety of means. These include:

› Compaction by displacement (penetration of granular material, eg stone columns or piles into the liquefiable deposit will laterally compress the surrounding soil and result in reduced void ratio, and therefore increase the soils resistance to liquefaction).

› Compaction by vibration (subjecting the loose sandy deposit to vibration energy will compact the soil and increase its strength).

› Compaction by surface impact energy (impact energy can densify loose granular deposits).

Densification is a common method of ground improvement with well developed methods that are proven to be successful in mitigating the effects of liquefaction when properly designed and constructed. Advantages of densification are that the degree of treatment can be easily verified and if liquefaction is triggered, displacements are reduced.

Disadvantages include high levels of noise and vibration associated with many densification methods, the lateral extent of improvement needs to be wider than for solidification or lattice reinforcement techniques, and it may take several weeks to verify the treatment. With the exception of compaction grouting and resin injection, densification methods are typically not suitable for treating ground below existing structures.

Densification is a key improvement method in:

- › Rapid impact compaction
- › Dynamic compaction and dynamic replacement
- › Deep vibro-compaction
- › Stone columns
- › Compaction piling
- › Compaction grouting
- › Resin injection.

7.2 Site conditions suitable for densification

Densification methods, with the exception of compaction grouting and resin injection, are most suited to free field sites that are not in close proximity to other buildings, infrastructure or amenities that are sensitive to vibration or noise.

Densification techniques are most suited to treating soils with less than 15 percent fines and less than 3 percent clay with a corresponding CPT soil behaviour index, $I_c < 1.8$. Some techniques can be used to treat silty soils but densification methods are generally less effective in treating silty soils. The inclusion of wick drains between treatment points can be used to aid in the densification of silty soils (Shenthan et al 2004; Theranayagam et al, 2006). Ground condition constraints specific to each technique are discussed in more detail in the following sections.

Densification methods, except injection methods like resin, can involve moderate to high levels of noise and vibration that could be a nuisance or damage to neighbouring properties. NZS 6803 provides guideline noise limits and management practices for construction works. The NZTA State Highway construction and maintenance noise and vibration guide also provides practical information and advice on prediction, management and mitigation measures for both noise and vibration.

Because of the variable nature of the ground, construction vibration levels are difficult to predict accurately. Figures 7.1 and 7.2 can be used to get an initial gauge on vibration levels for dynamic and vibro compaction techniques. Threshold vibration levels for annoyance and cosmetic damage to buildings are generally about 1 mm/s and 10 mm/s (Figure 7.1) respectively but this depends on the nature of adjacent land use, building types and condition. The German Standard DIN 4150-3 (1999) provides guideline vibration level thresholds for buildings. Careful assessment is required when the soils to be treated are underlain or interbedded with dense layers which tend to transmit vibrations for larger distances with relatively little attenuation and when particularly sensitive structures (hospitals and schools for example) are in the potential zone of influence.

Normally, the ground vibrations are measured with a seismograph at the time of construction. The readings should be taken on the ground adjacent to nearby structures and underground services. Before starting construction operations, it is necessary to predict the particle velocity of ground vibrations, because this may affect the level of energy application in close proximity to existing facilities.

Figure 7.1: Scaled energy factor versus particle velocity (FHWA, 2017)

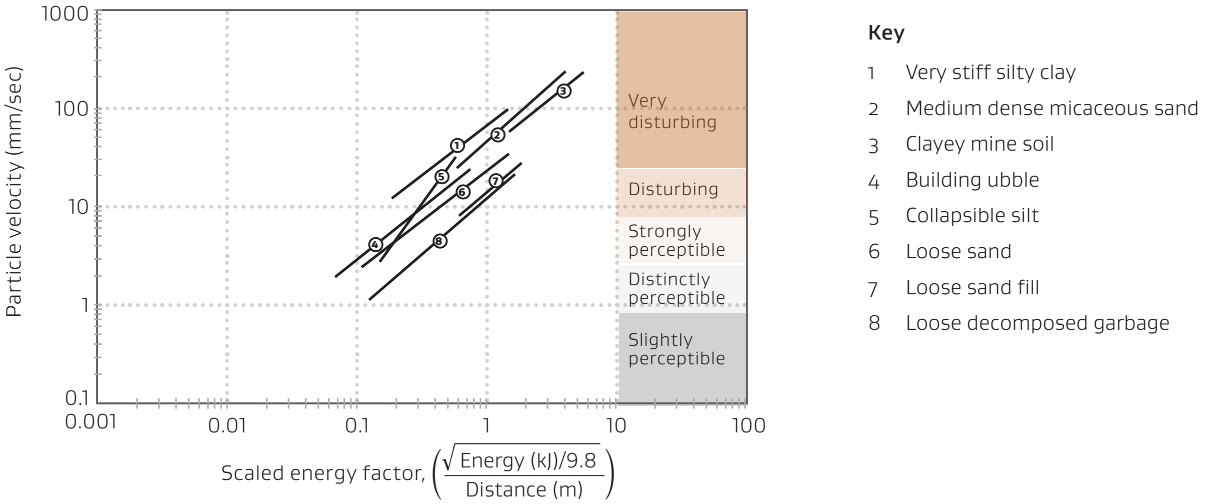
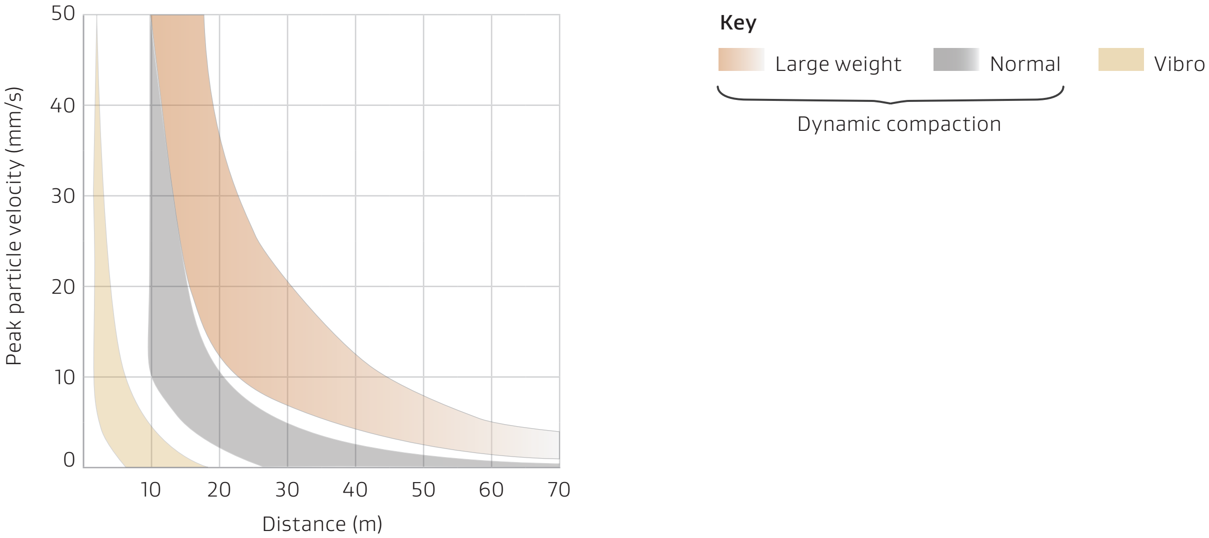


Figure 7.2: Construction vibrations (Mosely and Kirsch 2004)



7.3 Design considerations

The effectiveness of densification techniques is highly dependent on the fines content of the soils and the variability of the ground conditions to be treated. A comprehensive investigation should be undertaken to assess soil conditions and, in particular, the fines content, location and extent of silt and clay layers at a site. The CPT should not be relied upon as the sole method for assessing the fines content of the soil.

Once the site soil conditions have been evaluated, a target post treatment penetration resistance profile is calculated for each layer to be treated. The target penetration resistance is calculated from either the target relative density or the cyclic resistance ratio (CRR) required to meet the performance criteria, taking account of the fines content of the soils using a suitable empirical method (eg Boulanger and Idriss, 2014).

Typically, the target penetration resistance for soils in the improved zone is selected to get a liquefaction factor of safety of 0.8–1.2 for the ultimate limit state and 1.5 for the serviceability limit state, noting that excess pore pressure build up and settlement will become increasingly significant when FoS get below 1.2. Values at the lower end of the range are sometimes selected for lightweight lower importance ductile structures and foundation systems that are unlikely to collapse with moderate ground deformation and strength loss. The potential for concentration of stresses at the edges of foundations, especially for tall heavy structures, and the associated reduction in resistance to triggering of liquefaction needs careful consideration when selecting target penetration resistances.

For improvement techniques that involve installing stiff, continuous, closely spaced columnar inclusions, some discount can be made to the target penetration resistance for reinforcement effects where sufficient improvement through densification alone is not practical. The flexural stiffness of the columns, and the potential for slip or gapping at the interface need to be considered. The evaluation of reinforcement effects are discussed in more detail in Section 9.

An example of the calculated target CPT cone resistance q_c for a site treated with stone columns is presented in Figure 7.3. The stone columns are to be constructed by compaction of gravel delivered through a steel mandrel (bottom feed method) to form dense, continuous columns. Both the target penetration resistance for densification alone and the beneficial effects of including reinforcement are shown. Reinforcement benefit have been calculated using the method by Rayamajhi et al (2014).

The initial treatment layouts are developed from experience on other projects with similar ground conditions or using published charts.

For large projects or where there is little experience with a particular technique or equipment or its application to the site soil conditions, a pilot study may be carried out to refine the design and construction methodology. This frequently involves testing of different treatment spacing and refining the installation method, compaction vibration frequency and lift height for example.

Figure 7.4 shows the layout of a trial used to refine the spacing for vibro-compaction. Here three different spacings are tested with pre and post treatment testing between and adjacent to the treatment points.

Figure 7.3: Example of a stone column field trial

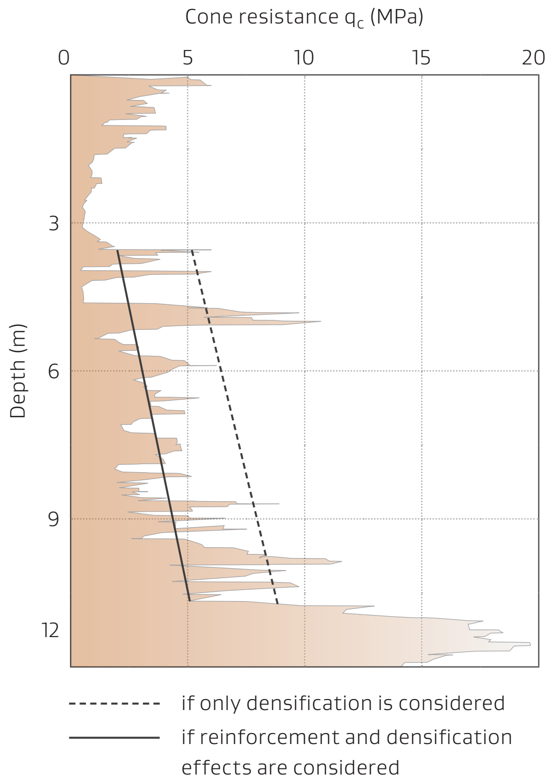
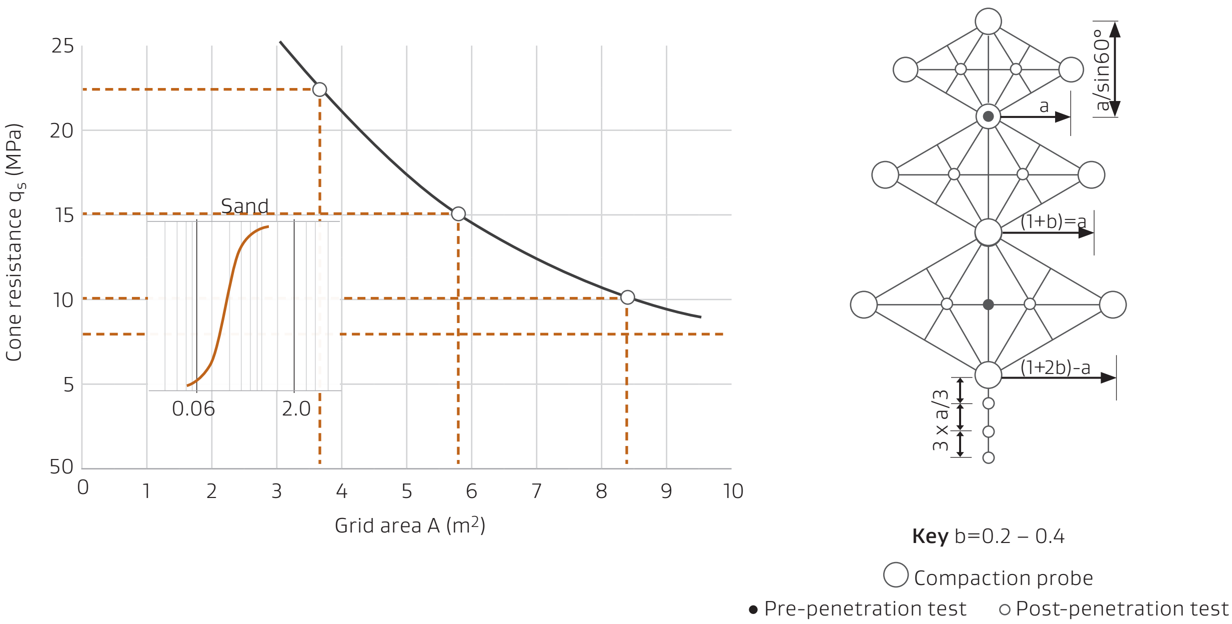


Figure 7.4: Example of a stone column field trial



7.4 Design verification

Verification testing involves carrying out penetration testing of the soil equidistant between treatment points and comparing the results with those taken before treatment to test if target penetration values have been achieved. The factor of safety against liquefaction may be calculated directly using the measured post treatment penetration resistance.

The importance of the overall stiffness of the ground improvement system was evident in the Christchurch Ground Improvement trials (EQC, in press). In these trials, cross-hole shearwave velocity measurements were taken to assess the improved stiffness from ground improvement. One of the advantages with cross-hole shearwave velocity measurement is that it can measure the overall stiffness of the improved zone accounting for both the stiffness of the inclusions and the natural ground. Cross-hole shearwave velocity measurement is a specialist skill and needs further development before it can be used as routine verification of ground improvement but is a promising method for verification of the reinforcing effects of ground improvement.

There can be some delay between treatment and the dissipation of excess pore pressures generated during construction, so verification of effectiveness may not be confirmed for some weeks after treatment. Experience in Christchurch has shown that the full degree of improvement is often not realised for at least a week after treatment and sometimes up to three weeks after treatment.

The compaction process can affect the ratio of CPT sleeve friction to cone resistance. Where the CPT is used to verify treatment, pre-treatment values of the soil behaviour index should be used in the evaluation of the liquefaction factor of safety for the improved ground.

7.5 Dynamic compaction

Dynamic compaction (DC) involves repetitively dropping a large weight from a significant height onto the ground causing the soil grains to rearrange and form a denser arrangement.

Figure 7.5 illustrates the application of dynamic compaction. Additionally, the impact of the dropped weight on the ground surface produces dynamic stress waves, which can be large enough to generate significant excess porewater pressure in the soils beneath the point of impact (Idriss and Boulanger 2008). Dissipation of the excess porewater pressures results in densification, accompanied by surface settlement. The drop height, weight and spacing vary depending on ground and groundwater conditions.

Tampers are typically concrete or steel with a weight of 5 to 35 tonnes and dropped using crawler cranes from heights of 10–40 m. (Moseley, 2004; Schaefer, 1997; Lukas, 1995). Drop locations are organised in a grid pattern with a spacing of 4–15 m. Treatment is carried out in a series of passes of different energy levels to treat different layers within the depth of treatment. The first pass targets the deeper layers with high energy tamping in a relatively widely spaced grid pattern. Successive passes use lower energy levels and closer grid spacing to treat the intermediate and surface layers, this compaction technique is also called ironing compaction.

Dynamic compaction is known to be fast and economic, especially in treating large areas. However, it has obvious disadvantages due to the noise and vibration that are produced.

Figure 7.6 shows, from experience on previous DC projects, soils most amenable to improvement by dynamic compaction categorised by grading, plasticity index and permeability (Lukas, 1995). Pervious soils in Zone 1 are most treatable by DC. Intermediate deposits in Zone 2 can be treated to a limited extent with dissipation of excess porewater pressures. Silty sand deposits (Zone 2) may be made more readily treatable when supplemented with wick drains installed between treatment locations before DC to aid with the dissipation of excess porewater pressure (Dise et al, 1994; Andrews, 1998; Thevanayagam, 2006).

The design of dynamic compaction should consider the following influence factors:

- type of soils
- groundwater table
- depth and area of improvement

- tamper geometry and weight
- drop height and energy
- pattern and spacing of drops
- depth of crater
- number of drops and passes
- degree of improvement required
- induced settlement or heave
- environmental impact (vibration, noise, and lateral ground movement)
- elapsed time effect (Han, 2015).

Treatment is usually effective only in the upper 5–7 m of the deposit for 8–15 t weights, but treatment to greater depth is possible with heavier weights. It is less effective for soils with fines content greater than about 15 percent or granular deposits interbedded with layers of silt and may be ineffective for soils with more than 25 percent fines Grading limits for soils suitable, marginally suitable and unsuitable are given in Figure 7.6.

If the predicted particle velocity is higher than desired (refer to Section 7.2), it will be necessary to either reduce the energy or increase the distance between the point of impact and the adjacent structure. Either would reduce the scaled energy factor (Figure 7.1). At some sites, trenches have been dug along the property line to reduce the particle velocity. This was found to be partially helpful in reducing the surface waves that travel off site. The effectiveness of the trenches can be established at the time of construction from vibration readings taken on the near and far side of the trench following impact of the tamper (FHWA, 2017).

The effective depth of treatment is related to the ground conditions and the energy input and is often expressed as

$D = \alpha\ (WH)^{0.5}$,
where D is the effective depth of treatment (m), W is the weight of the tamper (tonne), H is the drop height (m) and α is an efficiency factor that typically ranges between 0.4 and 0.6. It should be pointed out that the formula is units dependent. The specific units as noted in the definitions should be used.

There are no detailed analytical procedures available to analyse the effects of field dynamic compaction operational parameters and soil conditions to determine the densification and the degree of improvement achievable in the field. Current practice relies mainly on field pilot tests, and past experience based on case histories. Initial estimates of fall height, spacing between drop locations, number of drops and wait times between drops can be estimated using the methods described in Elias et al (2006), Thevanayagam et al (2006), JGS (1998), and Lukas (1995).

It is common practice to provide a layer of free draining granular material on the ground surface, 600–2000 mm thick. This layer acts as an ‘anvil’ to help transfer the high stresses imparted by the drop weight into the in situ soils. In weak saturated soils, the granular material can be driven to depths of up to about four metres to form large diameter columns of stone. Strictly speaking, this is termed dynamic replacement but the principle and equipment used for construction are similar to DC.

Figure 7.5: Procedure for densifying soil through dynamic compaction

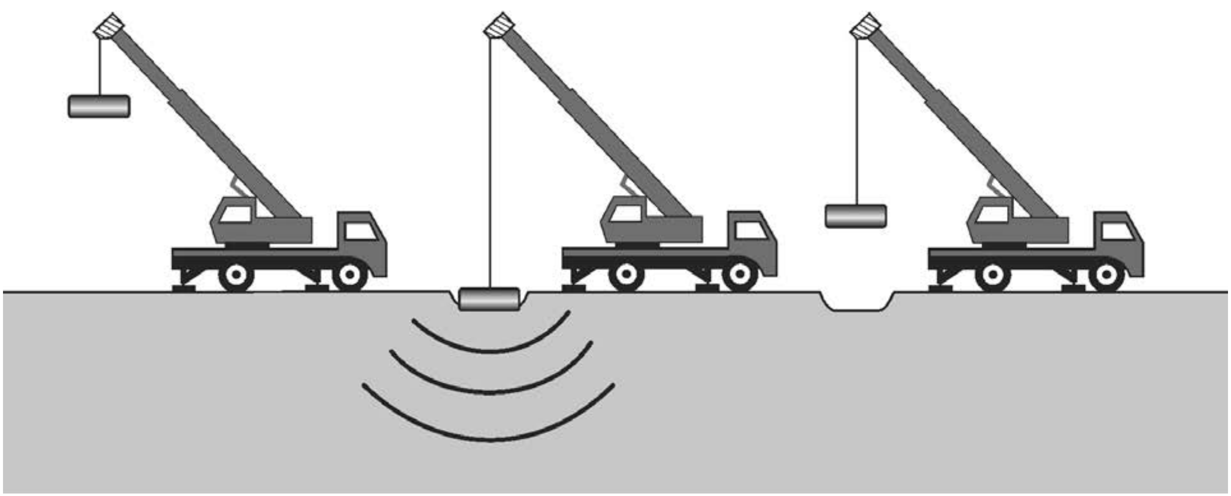
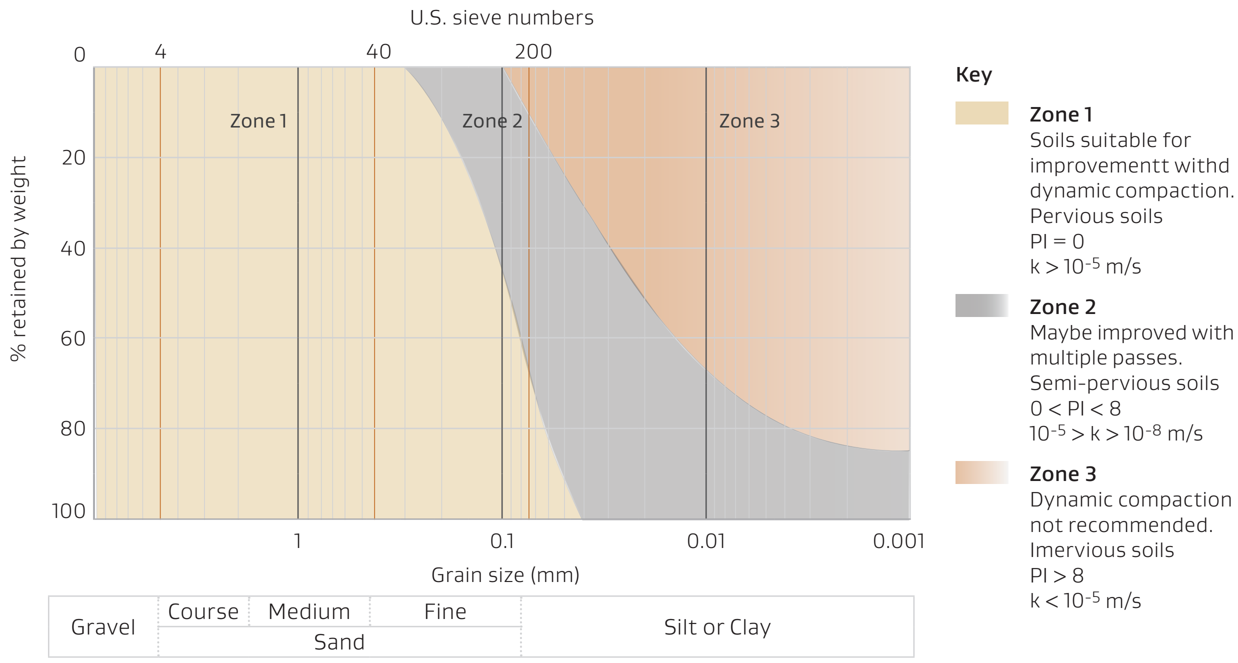


Figure 7.6: Soils suitable for dynamic compaction



When a hard layer with thickness of 1–2 m exists near the ground surface, it distributes the applied energy over a wide area so that the energy transmitted to the depth is greatly reduced. As a result, the depth and degree of improvement are reduced. Under such a condition, the hard layer should be removed or loosened. When a hard layer is thin, however, a tamper may penetrate this layer and deliver proper energy to the underlying layer. It is a general requirement for dynamic compaction that the groundwater table should at least 2 m below the ground surface. When the groundwater table is within 2 m, dynamic compaction likely encounters some difficulties. Typically, a crater depth ranges from 1.0 to 1.5 m. Dynamic compaction generates excess pore water pressure so that the groundwater rises and enters the craters. The geomaterial and water can be intermixed during compaction. To avoid such a problem, the groundwater table should be lowered by dewatering or additional fill should be added to increase the distance from the ground surface to the groundwater table (Han, 2015).

For dynamic compaction, measurement of the energy being delivered to the ground, the sequence and timing of drops, as well as ground response in the form of crater depth and heave of the surrounding ground are important quality control parameters. Similarly, the location of the water table and presence of surface 'hard pans' could greatly affect the quality and outcome of the densification process. Groundwater pressures should be monitored throughout the process and compared to baseline data.

Dynamic compaction induces noise, vibration, and lateral movement, which may cause problems to nearby sites and structures, substructures, and utility lines. This method often requires instrumentation to monitor vibration and noise levels. It is common to start dynamic compaction at the perimeter of the site and then gradually move towards the centre to reduce heave and lateral movement at the neighbouring sites.

Han (2015) provided a summary of available methodologies to estimate peak particle velocity (PPV) in terms of applied single-drop energy and distance to the drop point as well as scaled energy factor.

Dynamic compaction was used to treat potentially liquefiable soils beneath the Te Papa Museum, Hutt Valley Wastewater Treatment Plant and Mobil oil tank farm in Wellington and for the improvement of liquefiable soil at bridge sites along SH1 between McKays crossing and Peka-Peka north of Wellington.

A design example for dynamic compaction ground improvement is given in the Appendix (Example 4).

7.6 Vibro-compaction

Treatment by deep vibro-compaction involves inserting a probe into the ground to primarily apply horizontal vibrations in a square or triangular grid across the site. The vibratory energy reduces the inter-granular forces between the soil particles, allowing them to move into a denser configuration, typically achieving a relative density of 70 to 85 percent. The treated soils have increased density, uniformity, friction angle and stiffness.

Various methods and plant have been developed for vibro compaction to suit different site conditions. The most common type of probe is the vibroflot, a cylindrical steel tube with a diameter typically between 300 and 500 mm containing an eccentric weight linked to a motor. The length and weight of vibroflots typically vary between 3 and 4.5 m and 1500–4500 kg respectively.

Figure 7.8 shows the vibro-compaction process. The vibrator is typically suspended from a crawler crane and lowered vertically into the soil under its own weight. Penetration is usually aided by water jets (wet method) and compressed air. After reaching the bottom of the treatment zone, the soils are densified in lifts as the probe is extracted. The probe is installed and then retracted in a square or triangular grid pattern with a grid spacing typically between 1.2 and 2.5 m. Conical depressions that form at the surface as the ground is densified are filled with imported aggregate that is added around the probe at the surface during treatment. Typical treatment depths range between 5–15 m, but vibro compaction has been performed to depths as great as 35 m.

Densification by vibro-compaction relies on the soil particles rearranging under vibraton and gravity into a more dense state. The degree of compaction attainable depends on the grain shape, soil grading and the probes vibration intensity. Vibro compaction is most suited for the treatment of sands with low fines content, typically less than 12 percent fines and less than 2 percent clay. Figure 7.7 shows the particle size distribution limits that are most suitable to vibro-compaction. Very hard or cemented layers within the soil profile may need to be pre-bored to allow penetration of the vibrator to treat loose layers.

Vibrations may be a nuisance to neighbouring properties but are generally less than those from impact methods like dynamic compaction. Turbid water from the wet vibro-compaction method of installation needs to be contained and sediment removed before being disposed of. Vibro compaction can disperse ground or groundwater contaminants and alternative methods of treatment should be found if there is a contamination hazard at the site.

Figure 7.7: Ground conditions suitable vibro-compaction (Elias et al 2006)

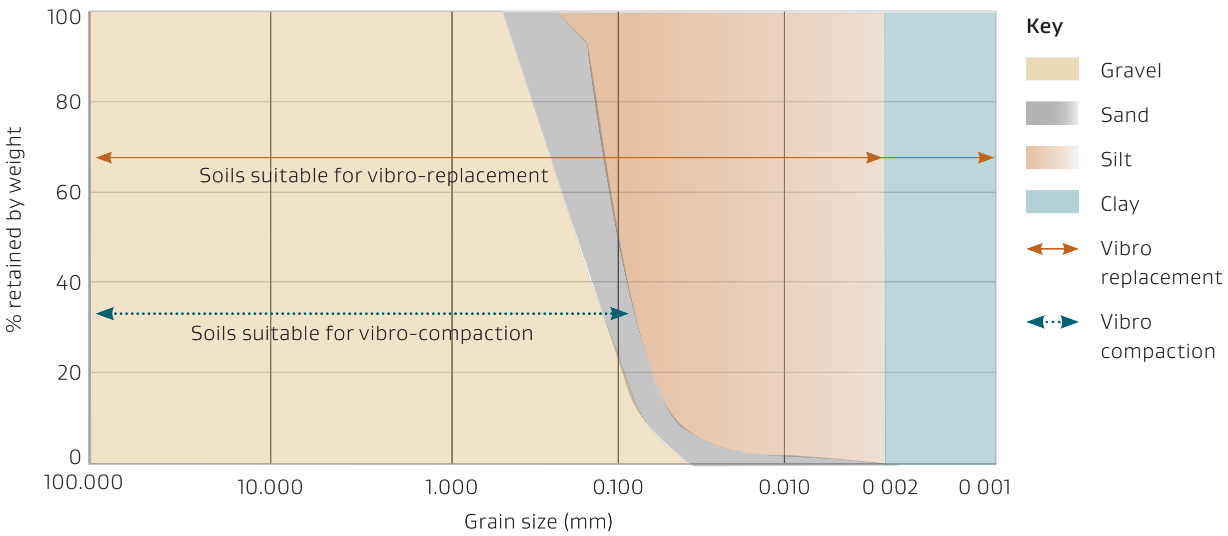
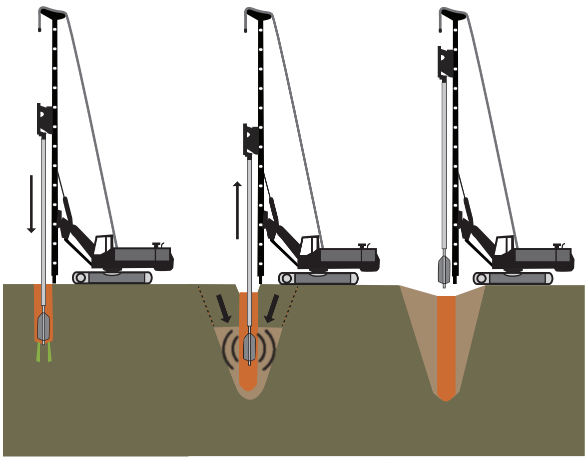


Figure 7.8: Vibro-compaction using a vibroflot



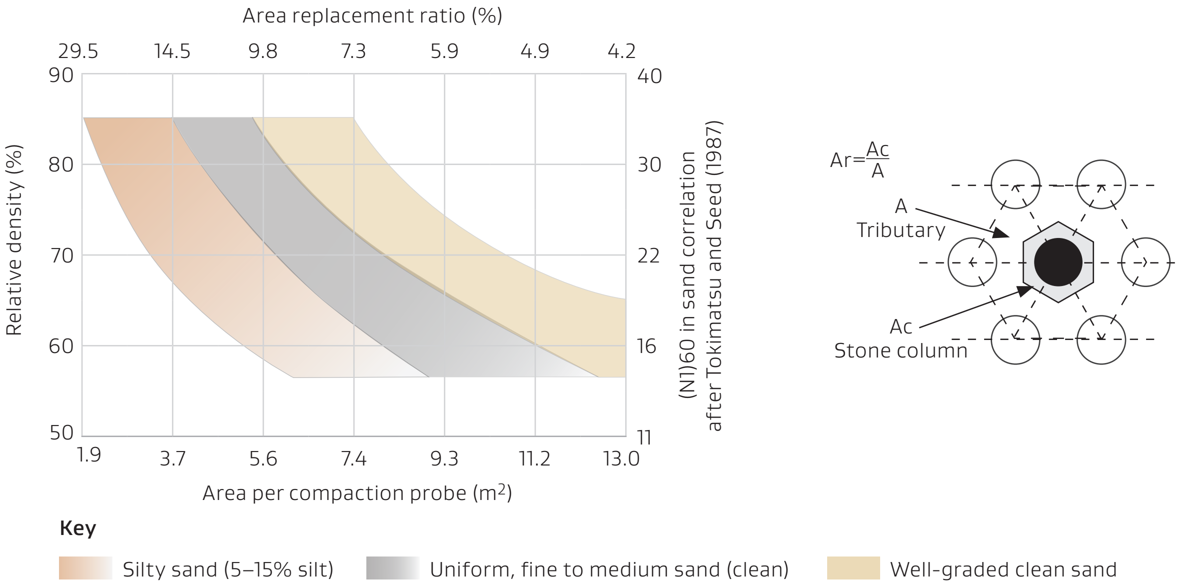
The degree of compaction generally decreases with increasing distance from the probe. Establishment of the treatment spacing, frequency of the vibrator and advancement and withdrawal rates is based on past experience or a field pilot study. Spacing of the treatment grid can initially be estimated from Figure 7.9. Other useful charts for determining a treatment spacing are in JGS 1998.

The supplementary aggregate added to fill the cavity and depression formed at each treatment point needs to fall freely in water to the base of the probe. Particle size recommendations based on settling rate and experience are given by Elias et al (2006).

Vibro-replacement rigs can be fully instrumented with an on-board computer to monitor specific parameters. Monitoring these parameters allows the operator to correct any deviations in real-time during the construction process to keep the stone column within project specifications. Data from the Data Acquisition (DAQ) system such as amperage and lift rate are recorded and displayed in real-time alongside specified target values on an in-cab monitor. The 'free hanging' amperage as well as the amperage developed during construction are strong indicators of the likely success of the densification effort. On some rigs it is possible to monitor the pressure and quantity of the flushing media with time for each treatment location.

The imported aggregate should be sampled randomly and the particle size distribution measured and compared to the specified envelope. The quantity of aggregate used at each treatment point should also be recorded.

Figure 7.9: Level of improvement vs area replacement ratio (Barksdale and Bachus 1983)



7.7 Stone columns

With stone column ground improvement, columns of dense stone are compacted into the ground in either a triangular or square grid across the site. The columns are typically spaced 1.5–4 m apart and have a diameter of 0.6–1.2 m. The depth of improvement is typically 4–15 m but soils as deep as 30 m have been treated using this method. Generally the fill material consists of crushed coarse aggregates of various sizes, with the particle size distribution prescribed. Crushed recycled concrete can also be used to construct the columns.

A variety of granular column construction methods have been developed out of the need to adapt the method to different site and ground conditions and to make use of locally available plant. In New Zealand, stone columns have been constructed using the vibro-replacement method (using a vibroflot), by driving a casing and compacting gravels out of the base of the casing and with displacement augers modified to compact aggregate delivered to the bottom of the column through the casing. Construction of stone columns using the driven casing method is depicted in Figure 7.10. Construction of stone columns using displacement rammed aggregate pier technique is shown on Figure 7.11. The stone within the rammed aggregate piers (RAP) compacted by impact ramming typically exhibits a higher effective friction angle and higher stiffness (modulus) compared to conventional stone columns.

The results of EQC ground improvement trials (EQC, 2015) indicate that soil densification may be considered to be the primary liquefaction mitigation mechanism in soils with a soil behaviour type index, $I_c < 1.8$, and that composite dynamic stiffness of the stone column treated soil likely dominates the liquefaction resistance mechanism in soils with $I_c > 1.8$. Large strain T-Rex testing during the EQC trials showed that the composite reinforced ground improved by RAP within both the clean sand and silty soil horizons exhibited shear stiffness values greater than the unimproved soil by a factor of 3 to 5, confirming the effectiveness of reinforcing non-densifiable soil with RAP elements (Wissmann et al, 2015).

The EQC trials also indicated that generally a lower level of in situ soil densification (in between the RAP piers) was achieved compared to the soils improved by conventional stone columns. This is attributed to lesser vibration levels during RAP construction compared to conventional stone column installation.

Stone columns are most effective at treating sands with less than 20 percent fines but can be used effectively to treat silty sands and sandy silts. Wick drains pre-installed between the stone columns improve the densification of silty soils (Thevanayagam et al. 2006, Rollins et al 2009).

The primary mechanism for improvement is densification of the soil between the columns by displacement and compaction. Depending on the construction method, installation of a grid of granular columns can also improve liquefiable soil deposits by increasing the in situ lateral stress, replacing the liquefiable in situ soil with non-liquefiable material, reinforcing the original ground with stiffer columns of fill material and providing drainage paths for the relief of excess porewater pressure (Munfakh et al, 1987; Sondermann and Wehr, 2004).

The maximum spacing between columns to obtain the required improvement depends strongly on the method of installation. Spacing of columns are based on past experience with similar construction techniques and ground conditions or published relationships between degree of improvement and area of treatment per column, see Figure 7.9 and JGS (1998). Pilot studies can be used to optimise column spacing and construction method.

Because of the uncertainty regarding the effectiveness of drainage and increased lateral stress, these mechanisms are usually ignored in the design of stone columns. The benefit of improved overall stiffness of the improved zone was evident in the EQC ground improvement trials. For bottom feed methods that form continuous dense and stiff stone columns, account can be made for the benefit of reinforcement where it proves impractical to densify the ground to the extent needed to meet the performance requirements. From numerical simulations, Rayamajhi et al (2016) concluded that the reinforcement effects of stone columns can be greatly over estimated by methods based on the assumption of shear strain compatibility (eg Baez and Martin, 1992). The method proposed by Rayamajhi et al (2015) can be used to assess the benefits of reinforcement. Shear stress relief from reinforcement effects of stiff columnar inclusions are discussed further in Section 9.

To avoid the migration of fines into the columns with the dissipation of excess porewater pressure during or after an earthquake and the resulting subsidence and reduction in stiffness and permeability of the column, the grading of the aggregate should be designed to filter the surrounding soil. Criteria for filtration are provided in NYSDOT (2013). The stone should be well graded, angular sound stone. Although drainage is not depended on it, the fines content of the aggregate should be less than 8 percent.

Figure 7.10: Stone column construction using the driven casing method

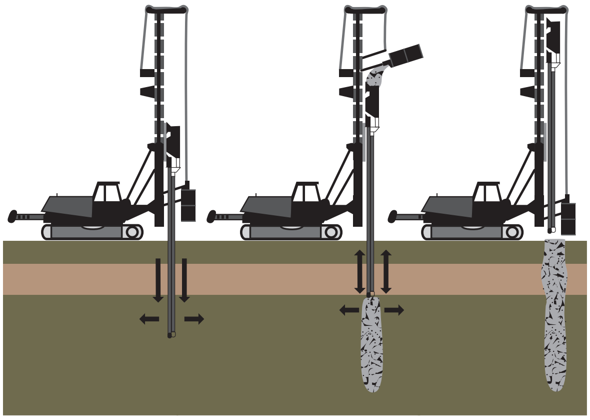
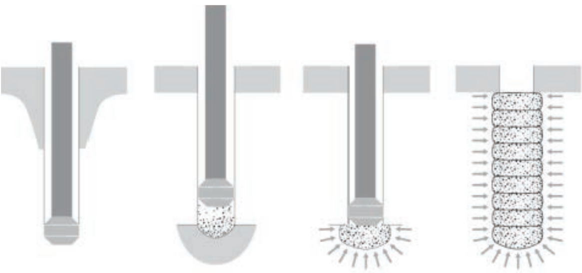


Figure 7.11: Rammed aggregate pier construction using displacement based method (after Wissmann et al, 2015)



Vibrations may be a nuisance to neighbouring properties but are generally less than those from impact methods like dynamic compaction. As with vibro-compaction, turbid water from installation of the stone columns needs to be managed during construction.

Stone columns can disperse ground or groundwater contaminants during construction and while in service. Alternative methods of treatment should be found if there is a contamination hazard at the site. Similarly, stone columns should be avoided where groundwater conditions could be significantly altered, for example from penetration of an aquiclude and relief of pressures in an artesian aquifer with continuous flow of groundwater to the surface; perched water tables may also be affected.

There are many factors affecting the price of stone column construction, including labour, the price and availability of stone, weather, environment, etc. Therefore, it is recommended that experienced contractors with a record of installing aggregate columns is involved early in the design process to verify both the cost and the technical feasibility of stone column installation.

Construction quality control should include records of depth of each column, the volume of stone installed in each column, preferably per metre depth and the compactive effort exerted per metre depth in the construction of each column. Where reinforcement effects are relied upon, quality assurance testing should include standard penetration testing (SPT) through the column to verify the level of compaction.

A design example for stone column ground improvement is given in the Appendix (Example 3).

7.8 Compaction piles

Installing permanent driven piles in a square or triangular grid is another method of ground improvement. The rows of piles are typically spaced at 3–4 pile diameters and designed to densify the soil between them by displacement and vibration.

The piles may also reduce shear strains in the soil between the piles to an extent, improving their resistance to liquefaction and may be relied upon when it is not practical to improve the ground through densification alone. Methods for the assessment of reinforcement effects are discussed in Section 9. When founded in a non-liquefiable layer, the stiffening effect of the piles reduces settlement.

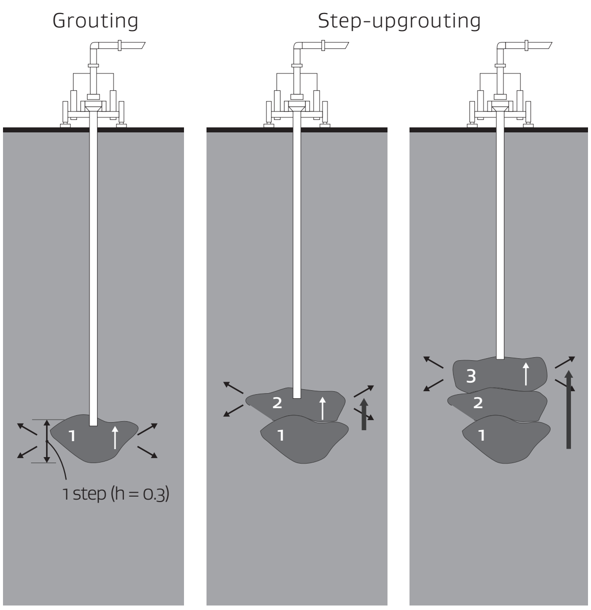
Compaction piles are usually made of prestressed concrete or timber and are generally installed in a square or triangular grid pattern to depths of up to 16 m. The durability and potential leaching of timber preservatives needs careful consideration when assessing the use of timber piles. Water jetting to aid installation may reduce the densification of soil around the piles but can be useful for penetrating interbedded dense or hard layers.

7.9 Compaction grouting

In compaction grouting, a very stiff grout is injected into the soil such that it does not permeate the native soil, but results in coordinated growth of the bulb-shaped grout that pushes and displaces the surrounding soil (see Figure 7.12). Typically the grout consists of a soil-cement-water mixture with sufficient silt sizes to provide plasticity, together with sand and gravel sizes to develop internal friction (Welsh, 1992).

Note
The strength of the grout is unimportant because the purpose of the technique is to densify the surrounding soil by displacement.

Figure 7.12: Compaction grouting



Since the technique involves the pressurized injection of grout into the soil deposit using small-scale, manoeuvrable and vibration-free equipment, the method is especially suited for improving the soil below existing structures; it also has a building releveling advantage.

However, it has some disadvantages; for example, stabilisation of near surface soils is generally ineffective due to the fact that the overlying restraint is small (ie low confining pressures) and the grouting pressures can heave the ground surface rather than densify the soil. Results from the EQC ground improvement trials indicated that shallow treatment using low mobility grout tended to dilate soils for ground profiles with interbedded sand and silty soils as the grout tended to spread through soft layers because of the low confining pressures, actually increasing the potential for liquefaction (EQC, 2015).

Because grout is typically injected in stages from the bottom up, at each stage a stopping criteria of grout volume, pressure, or heave is followed before proceeding with the next stage. Usage of grout casing with less than 50 mm in internal diameter should be avoided as it could cause the detection of high back pressures before sufficient grout is injected. Over injection of grout in a primary phase may lead to early ground heave and may diminish densification effectiveness. Spacing and sequence of the grout points may also affect the quality of densification or ground movement achieved.

Compaction grouting requires the verification of slump and consistency of the mix, as well as careful monitoring of grout volumes, injection pressures, and ground movement at the surface or next to sensitive structures. Critical projects also monitor porewater pressure and deep ground heave (borros points) that develop during the compaction grouting procedures.

7.10 Resin injection

Resin injection primarily provides liquefaction mitigation as a result of densification of the soil from an aggressively expanding polyurethane resin. Secondary mechanisms of improvement from increased composite stiffness and horizontal stress increases may also be present (Traylen et al, 2017).

Injection tubes are driven into the ground at regular intervals, through which low viscosity resin materials (which have been mixed at specific pressures and temperatures) are pumped at controlled pressures into the soil matrix. Either 'top down' or 'bottom up' methods can be employed. In a typical 'bottom up' installation the tube is withdrawn either in set stages with set volumes of material injected at each stage, or it is slowly withdrawn at a uniform rate, with set volumes of material being injected per unit length of withdrawal.

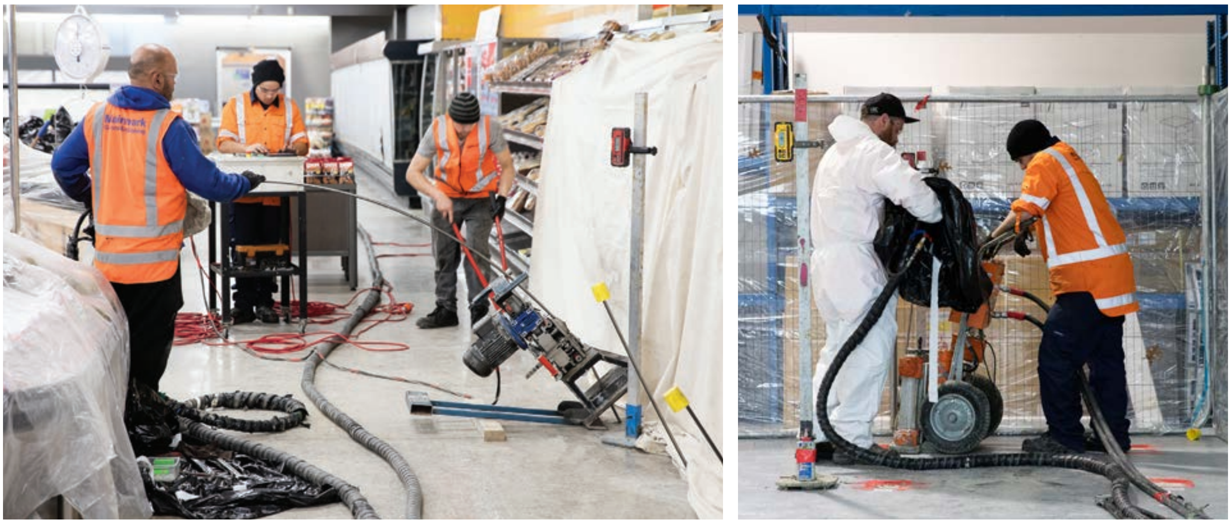
The resin penetrates the soil mass along pre-existing planes of weakness or through fracturing of the soil mass (it also permeates the soil mass to a limited extent; depending on the porosity of the soil). The resin mix chemically reacts soon after injection, rapidly expanding to many times its original volume, and changing from a fluid form to a solid one. This expansion of the injected material in the soil matrix results in densification of the adjacent soils.

Unlike compaction grouting (which uses a high viscosity medium), the low viscosity expanding resin injection process typically results in a 'veining' of expanded material distributed through the soil mass as dykes, sills or networks of sheets or plates, typically tens of millimetres thick (refer to Figure 7.13).

Research trials and also commercial application of this technology have shown increases in CPT cone resistance of 25–100 percent being achieved, depending on the soil type being treated. As with most densification methods, the best results are achieved in clean sands (ie $I_c < 1.8$) but good results are also achieved in silty sands up to an I_c of at least 2.0, and densification is still noted in soils with even higher silt contents.

Figure 7.13: Hand-exhumed resin veins (left) and hydro-exhumed resin veins (right)

Figure 7.14: Installing injection tubes, injecting resin inside a supermarket



Although applicable to cleared sites, the particular advantage of resin injection is its suitability for use beneath existing structures (see Figure 7.14). For existing structures, the degree of soil improvement that can be achieved may be limited by the magnitude of the heave that the structures can tolerate. On greenfield sites, some loosening of the top layer is possible due to low confinement.

8 Solidification methods

8.1 Outline

Solidification involves either in situ mixing of cementitious or other additives into the soil or filling the voids with a reagent resulting in the soil particles being bound together. This will prevent the development of excess porewater pressure, preventing the occurrence of liquefaction. The strength and stiffness of the soil are increased by the stabiliser, and thus the solidified ground can mitigate differential subsidence.

Solidification methods are advantageous because installation is relatively quiet, and the techniques induce relatively small vibrations as compared to compaction methods. These are important considerations for the improvement of sites with adjacent infrastructure or inhabitants that could be affected by noise and vibration from densification techniques. Their disadvantage is the relatively high cost as compared to compaction methods.

There is a high degree of confidence that liquefaction will be prevented within the zone of solidification and when the full depth of liquefiable soil is treated, liquefaction effects can be eliminated entirely. Another advantage of solidification is that the soils usually do not need to be treated outside the perimeter of the building although this is not easy to control for permeation grouting. This can be an important issue for buildings near to the section legal boundary.

8.2 Techniques for solidification

Typical methods include:

- Soil mixing
- Jet grouting
- Permeation grouting.

With jet grouting and columnar deep soil mixing techniques, either the entire footprint can be solidified by overlapping the columns or the improved area can be partially solidified in a grid of individual columns or to form a lattice of intersecting walls that reinforce the ground. Partial solidification to mitigate liquefaction by reinforcement is discussed in Section 9.

8.3 Site conditions suitable for solidification

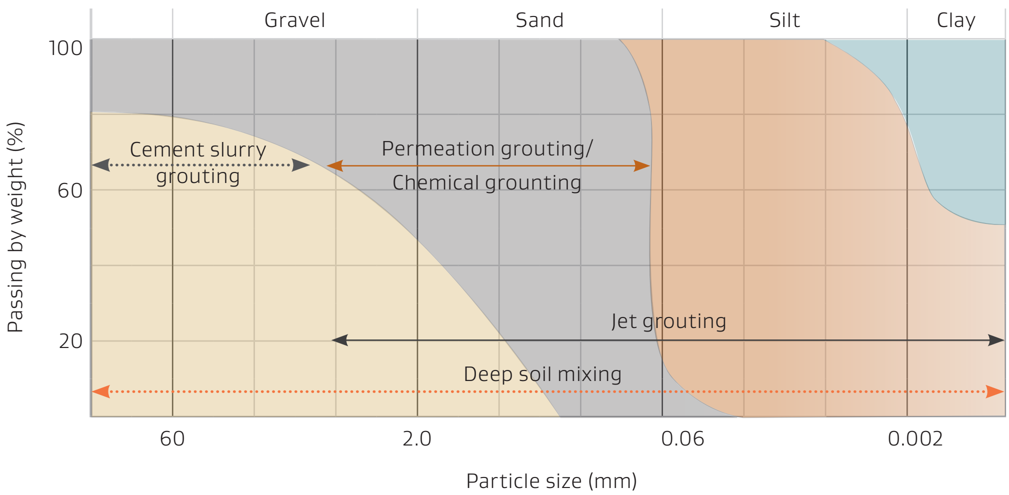
Solidification techniques, in general, can be used to treat a wide range of soil types. Some organic soils may not gain appreciable strength from mixing with cement and soils containing large inclusions such as gravels or even large shell are not readily treatable with jet grouting. Figure 8.1 shows the soils suitable for different solidification techniques.

Jet grouting and permeation grouting use comparatively compact and low vibration equipment and are suitable methods for treating the ground below existing structures or on sites with limited space where remediation is difficult using other methods. Deep soil mixing is limited to use on open sites with access for drilling machinery.

Permeation grouting is more suited to moderately permeable soil and relatively homogeneous ground profiles. With layered soil profiles there can be a tendency for the grout to spread through more permeable or weaker layers although this can be combated with multiple grouting phases. It may be possible to treat some silty soils by permeation grouting with expensive silicate grouts.

Near waterways there is a risk of contamination with permeation grouting and of a loss of lateral confinement with jet grouting. This risk can be mitigated with sheet piles or other measures to protect waterways.

Figure 8.1: Range of applicability of soil grouting techniques



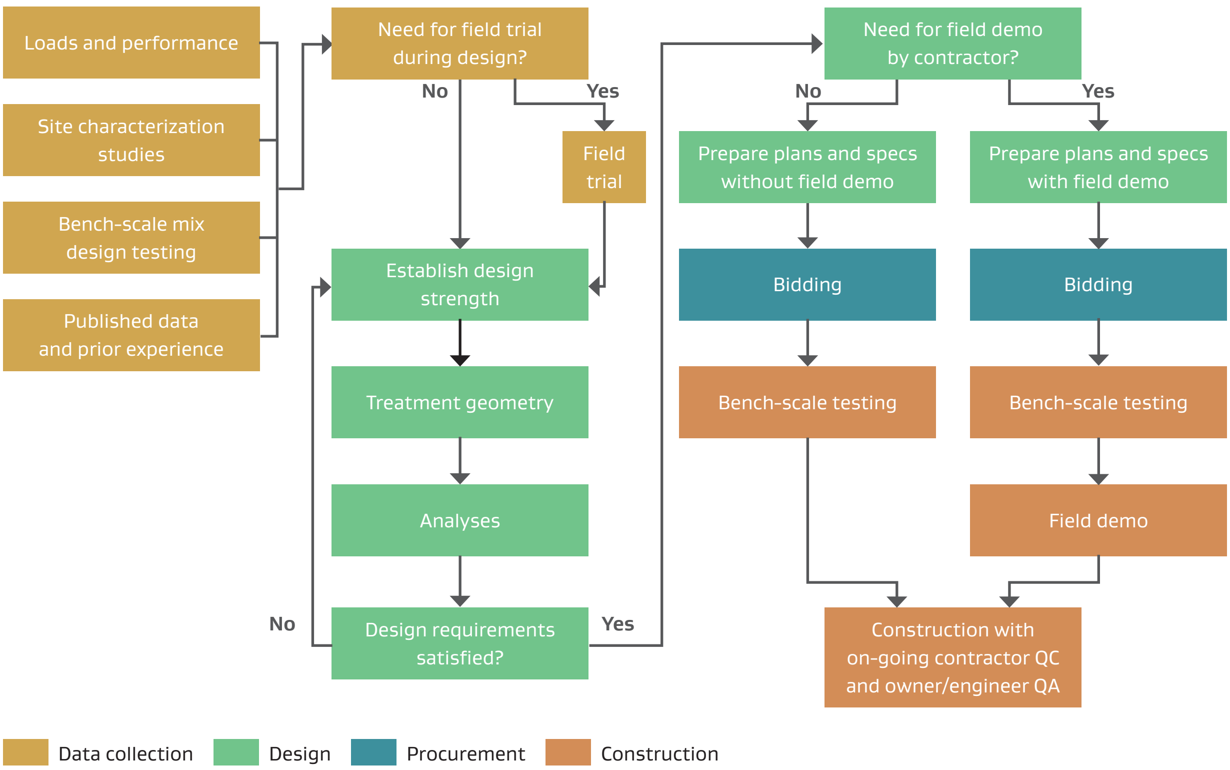
8.4 Design considerations

The soil-cement materials can have a wide range of unconfined compressive strengths, depending on the homogeneity of the mixing or grouting process, the degree of compaction imparted by the solidification technique, the amount of cementitious material used and the in situ soil characteristics (Kitazume and Terashi, 2013; Porhaba, 2000). The unconfined compressive strength (UCS) of the overall block needs to be characterised using past experience and laboratory testing and a design value adopted.

In the early stages of the project, laboratory tests are undertaken to ascertain the constraints that the soil characteristics may have on the ground treatment and to characterise the level of treatment that can be expected from application of different binders or different binder rates. For soil mixing and jet grouting, each layer to be treated can be mixed with a range of binders and dosages in the laboratory to ascertain the soils reactivity and strength gain with different volumes of additive.

Laboratory scale tests (bench-scale tests) do not always reflect field experience as the nature and efficiency of mixing in the field will affect the stiffness and strength of the solidified material. Porhaba (2000) discusses selection of a UCS for design. Practically achievable strengths can be estimated from ground improvement trials in the field (field demonstrations) and experience on prior projects in similar materials. In addition to establishing trial values of stabilised material strength, strength variability should also be considered. A flowchart for design and construction of deep mixing projects is provided in Figure 18 (FHWA, 2017).

Figure 8.2: Flowchart for design and construction of deep mixing projects (FHWA, 2017)



Consistency of column diameter, tolerances on drilling position and verticality and the strength gain between installation of successive jet grout or DSM columns needs to be considered when selecting a column spacing and planning the timing of installation of each column. This is especially important for jet grouting as the columns effectively cannot be formed if the drill head penetrates into an adjacent hardened column.

For permeation grouting the permeability of the soils in the horizontal and vertical directions needs to be assessed in detail as this has the greatest impact on the effectiveness of permeation grouting. Permeability of the ground profile as a whole can be investigated with pumping tests and individual layers can be tested with down hole testing or in the laboratory on undisturbed samples.

Jet grouting and soil mixing loosen the ground or temporarily turn it into a slurry during construction. This can destabilise, or cause subsidence of existing building foundations if they are near to the area of treatment. The location of columns and the timing between different stages of the construction works need to be planned to avoid instability or unacceptable subsidence of existing foundations. Permeation grouting is done under high pressure and can heave the ground and foundations above.

Underground utilities may need to be relocated or protected prior to treatment. Permeation grouting and jet grout can fill sewer and stormwater pipes through any open joints or cracks. There can be large differential movements at the interface between the solidified ground and surrounding liquefied soils that can severely damage underground services

Cement stabilised soils are brittle and have low strength in tension. The solidified ground should be designed to avoid concentration of strains at large widely spaced cracks that could cause abrupt differential settlement of the structure. This is especially important where only partial depth of soils prone to liquefaction are treated. A granular layer placed over the cemented fill can smooth out abrupt changes in level or grade beneath shallow foundations.

8.5 Design verification and quality control

Verification of solidification methods involves coring of the treated area and undertaking unconfined compression tests on samples of the core to confirm the extent and homogeneity of improvement and the strength and stiffness of the solidified soil. Areas of overlap and the zone equidistant between columns as well the centre of columns should be sampled.

The binders are controlled for quality by checking consistency as measured by specific gravity. This is generally checked with mud balance or hydrometer devices. Pumping pressures and rates are designed to achieve production and strength requirements of the product.

8.6 Soil mixing

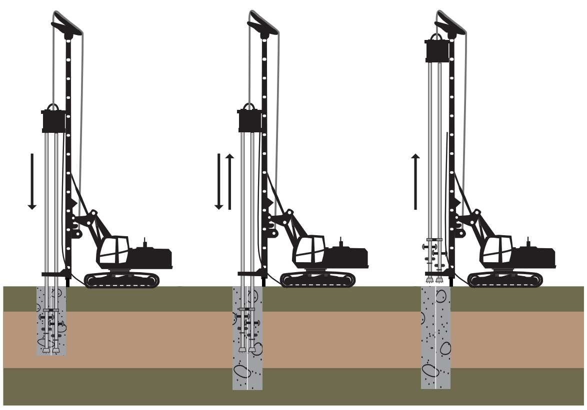
Soil mixing involves agitating and mixing stabilizing material such as cement in sandy soil and solidifying the soil. A variety of plant has been developed for soil mixing.

For mass stabilisation, a rotating drum cutter attached to an excavator arm can be used to mix the soil with the stabilising agent. This method is generally limited to treatment depths of about 6 m.

Other methods use rotating augers or blades attached to rods to mix soils in vertical columns up to depths of 30 m or more. This technique is commonly referred to as deep soil mixing (DSM) or deep mixing method (see Figure 8.3).

Cutting heads have also been attached to directional drilling plant to mix soils in horizontal beams below existing structures (Wansbone and Van Ballegooy, 2015).

Figure 8.3: Deep soil mixing process



8.7 Jet grouting

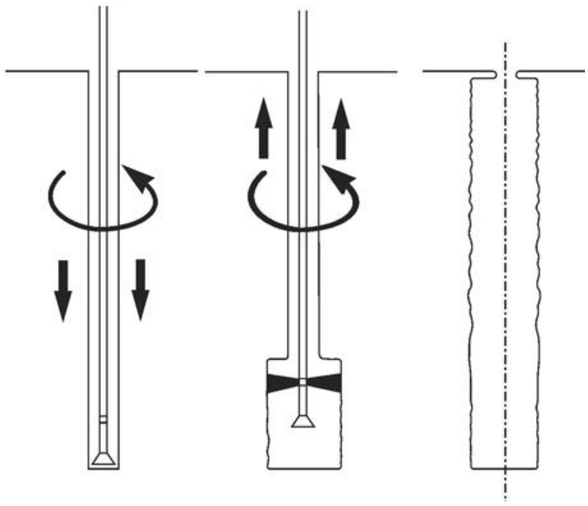
In jet grouting, high-pressure jets of air and/or water and grout are injected into the native soil in order to break up and loosen the ground and mix it with thin slurry of cementitious materials.

In essence, it is not truly grouting but rather a mix-in-place technique to produce a soil-cement material. Depending on the application and soils to be improved, different kinds of jet are combined by using single fluid system (slurry grout jet), double fluid system (slurry grout jet surrounded by an air jet) and triple fluid system (water jet surrounded by an air jet, with a lower grout jet). The process can construct grout panels, full columns or anything in between (partial columns) with a specified strength and permeability.

Construction of jet grout columns involves drilling to the base of the column then mixing a cement slurry into the soil in situ with rotating high pressure jets that are located just above the drill head as the drill string is brought to the surface. The process is illustrated in Figure 8.4. The double and triple fluid processes are capable of producing larger diameter but generally weaker columns compared to the single fluid process. Column diameters of up to 8 m are possible in dispersive soils with specialist equipment.

Other advantages of jet grouting are that treatment can be for targeted layers only and the ability to treat multiple depths at any location.

Figure 8.4: Jet grout construction process



8.8 Permeation grouting

Permeation grouting, sometimes called chemical grouting, is a technique that transforms clean gravel and sands into hardened soil mass by injecting cement or other grouting materials that permeate and fill the pore space. The hardened grout improves the native soil by cementing the soil particles together and filling the voids in between (minimising the tendency of the soil to contract during shearing). The treated soil has increased stiffness and strength, and decreased permeability.

Because of its minimal disturbance to the in situ soil, it is an effective method in treating liquefiable deposits adjacent to existing foundations or buried structures. This method is most suited to treating moderate permeability liquefiable gravels and sands. Some grouts are toxic in their liquid form and the spread of the grout is not easily controlled.

The risk of contamination groundwater and nearby waterways needs careful consideration.

Permeation grouting is typically expensive compared to other methods and therefore it is not discussed in detail here. Further information can be found in the reference texts.

9 Reinforcement methods

9.1 Outline

When saturated sand deposits are sheared during seismic loading, excess porewater pressure is generated reducing the stiffness and strength of the soil and increasing strains.

The aim of reinforcement is to reduce shear deformation in the ground during an earthquake to mitigate the development of excess porewater pressures. The increased composite strength of the reinforced ground also mitigates ground deformation and subsidence of the structure if liquefaction were to occur. These principles are illustrated in Figure 9.1 and 9.2.

Reinforcement of the ground involves either construction of a:

- lattice of intersecting walls to form containment cells, or
- a grid of closely spaced stiff vertical columns.

Typical layouts for lattice and isolated pile reinforcement arrangements are shown in Figure 9.2.

Open grid systems are relatively flexible compared to lattice systems and do not offer the same protection against the migration of excess pore water pressures or confinement of liquefied soils as a lattice with continuous perimeter walls. Because of the greater redundancy of lattice structures, they are a much more reliable method of reinforcement than grids of isolated piles.

Soil reinforcement is typically used to treat soils up to a depth of 20 m but greater depths are possible with some methods of construction and specialised equipment.

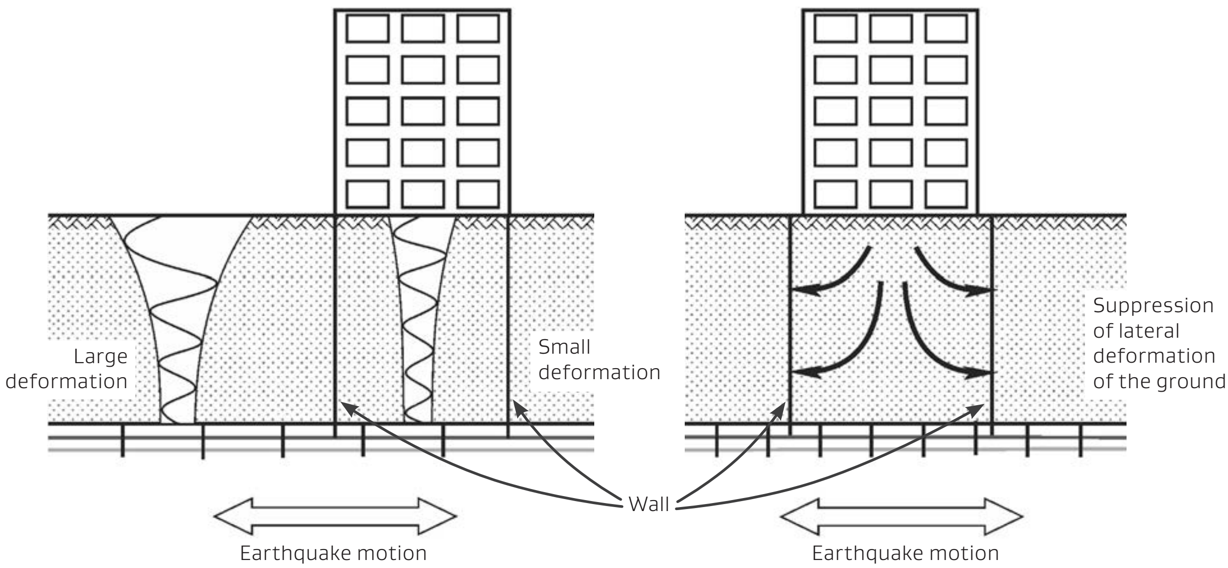
The advantages of reinforcement are that:

- it can be used for a range of ground conditions, including sites with silty soils and variable soils
- construction vibration is typically small, and
- with the use of jet grouting, ground beneath existing structures and on sites with limited space can be treated to improve their seismic performance.

Figure 9.1: Principle of reinforcement and containment:

a suppression of shear deformation in ground during earthquake

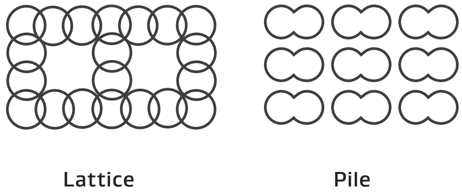
b suppression of lateral flow of ground after liquefaction (after JGS 1998)



With lattice reinforcement, the lattice perimeter wall can often be placed below the perimeter of the building unlike most densification and drainage methods. This is advantageous for ground improvement beneath buildings constructed close to the legal boundary.

One disadvantage of reinforcement techniques is that there are no simple methods to verify the effectiveness of the reinforcement to mitigate shear strain in the soil between the reinforcement elements. Compared with densification and drainage techniques, reinforcement methods are typically more costly.

Figure 9.2: General arrangements of structural elements for lattice and isolate pile reinforcement



9.2 Techniques for reinforcement

The lattice structures can be formed using overlapping DSM or jet grout columns, overlapping Continuous flight auger (CFA) piles or other contiguous pile and diaphragm wall techniques.

Piles within an isolated grid can be constructed using DSM or jet grout techniques, driven timber or precast concrete piles, or with conventional bored concrete piles, CFA or displacement auger concrete piles. With displacement auger piles and driven piles, there is added benefit of densification of soils between the columns. Jet grout columns can be reinforced by plunging a cage into the wet slurry similar to the reinforcement of CFA piles.

9.3 Site conditions suitable for reinforcement

Reinforcement can be used to treat most soil types including the treatment of sites with cohesive soils interbedded within liquefiable layers.

Methods that involve the mixing of additives with in situ soils (DSM and jet grouting) may not be suitable for sites with organic layers that are thicker than the diameter of the reinforcing elements, particularly layers of fibrous peat.

The treatment of gravels or soils containing dense layers, cobbles and other large inclusions can also be problematic. Further guidance on the suitability of ground conditions for DSM and jet grouting is provided in Section 11.

9.4 Design considerations

The spacing and sizing of reinforcement elements are typically based on methods developed from the results of numerical and centrifuge simulations of simplified soil profiles (Nguyen, 2013; Rayamajhi, 2014) or, for larger projects, numerical analysis with the site specific conditions.

> Early methods to calculate the layout of soil reinforcement elements, by Baez and Martin (1993) for example, assumed that the soil and reinforcement elements deform purely in shear and that the shear strain in the soil is equal to the shear strain in the reinforcement (commonly referred to as shear strain compatibility). Numerical and centrifuge studies (Goughnour and Pestana, 1998; Green, 2008; Oglan and Martin, 2008; Nguyen, 2013; Rayamajhi, 2014, 2015) have since found that the assumption of shear strain compatibility may greatly overestimate the magnitude of strain relief in the soil between reinforcement elements.

When designing the spacing of reinforcement elements, consideration needs to be given to the location of shallow foundations relative to the reinforcement elements. There may be some development of excess porewater pressure in soils between the reinforcement and having some footings directly supported on the reinforcement elements and others on the soil between the reinforcement should be avoided.

Unreinforced concrete, DSM and jet grout columns are susceptible to brittle failure. The tension stresses and shear stress in structural elements for both static and earthquake loads including loads transferred from the building above need to be evaluated during design, especially for individual columns that rely on some degree of fixity in soils above or below liquefied layers. Lattice structures have greater structural redundancy and limited cracking may be acceptable where the overall system remains ductile and structural integrity is not greatly reduced. Refer to discussion on damage modelled in grid walls by Namikawa et al (2007).

9.5 Design verification

Verification for deep soil mixing and jet grouting includes confirmation of the consistency of mixing and pile diameter, and the strength and stiffness of the columns.

For the construction of lattice structures using overlapping piles, the bond between adjacent piles needs to be verified. Joints where one day's work ends and another starts should be located in low stress areas.

9.6 Lattice reinforcement

Lattice reinforcement limits the horizontal squeeze of soils beneath shallow footings and prevents the migration of excess porewater pressures from adjacent liquefied soils. In many cases, the lattice need not extend far beyond the footprint of the building.

The effectiveness of lattice-type improvement to mitigate the development of excess porewater pressures in the soils contained within the cells of the lattice have been shown in numerical studies and centrifuge tests to be strongly dependent upon the grid spacing and the thickness and stiffness of the walls (Bradley et al, 2013; Kitazume and Takahashi, 2010; Funahara et al 2012).

Nguyen (2013) describes a simplified approach that can be used to calculate panel spacing and thickness for a given wall shear modulus. It is important to remember that this simplified approach is based on numerical simulations using a simplified model with lattice walls that fully penetrate the liquefiable layer. It does not consider the influence of external loads (eg from the building). Project specific numerical analysis should be considered for projects with important, heavy or irregular structures.

Stresses within the walls and the potential for cracking can be assessed using the simplified methods proposed by Nguyen (2013) or Orouke and Goh (1997).

Installing a drainage blanket over the improved area and a perimeter subsoil drain through the crust around the outside of the improved area to intercept and relieve water pressure and prevent soil seepage erosion is prudent where the improved zone does not fully penetrate liquefied layers. This layer will also give a more even transfer of loads from the footings to the improved ground.

A design example for lattice reinforcement is given in the Appendix (Example 6).

9.7 Stiff columnar reinforcement grids

The degree of strain relief and liquefaction mitigation from a grid of isolated reinforcement columns depends on:

› the spacing of the columns
› the stiffness and strength of the columns
› the degree of rotational fixity of the columns above and below potentially liquefiable soils
› the surface roughness of the reinforcing elements and magnitude of interface adhesion
› densification effects (only if soil is displaced during the installation of the columns)

The reinforcement effect increases with increasing area replacement, increasing flexural stiffness of the individual columns, rotational fixity at the top or bottom of the columns, especially if the columns work in double bending, and a rough interface between the reinforcement columns and the surrounding soil.

With the relatively high uncertainty in the ability of grids of stiff individual columns to supress liquefaction or the development of significant excess porewater pressures, to give some redundancy to the system, columns should extend down to a competent non-liquefiable layer and the area of treatment should extend beyond the perimeter of the building a suitable distance to protect against lateral deformation of the ground near the edge of the building. A granular load transfer platform and drainage blanket should be constructed across the top of the stiff pile reinforcement to relieve excess porewater pressures that develop in the soils between the piles during shaking and to distribute load between the building and the piles.

A grid of stiff columns is typically not suitable for mitigating lateral spreading unless the liquefiable layer is relatively thin, and the piles are designed to cantilever from the underlying non-liquefiable layer and are suitably reinforced for the bending and shear stress that will develop from the kinematic loading of the piles.

The simplified method based on dynamic numerical analysis of a pile through liquefied soil by Rayamajhi (2014) can be used to design the pile grid layout and estimate tension stress in the piles. A more detailed assessment of the bending and shear in the piles can be made by calculating the profile of horizontal ground displacements for the improved ground by integration of the soil shear strains over its depth and applying these to a pile in a beam on spring analysis together with a contribution of building inertia. The procedure is described in Module 4.

A design example for a grid of controlled modulus (weak cement) columns is given in the Appendix (Example 5).

10 Drainage methods

10.1 Outline

Drainage can mitigate liquefaction potential in two respects:

1. Drainage can desaturate potentially liquefiable soil, either by draw down of groundwater or gas entrainment.
2. Alternatively, vertical gravel or prefabricated drains typically at 1–2 m intervals can be installed to allow the rapid dissipation of excess pore pressures generated during shearing, preventing the condition of R_U=1 or liquefaction developing. Excess porewater pressure generated by cyclic loading is dissipated by installing permeable drains within the deposit. These methods rely on two mechanisms to reduce damage due to liquefaction:
   - Delaying the development of excess pore water pressure due to earthquake shaking
   - Preventing the migration of high excess pore water pressure from untreated liquefied zones into non-liquefied areas (say underneath the structure) to prevent secondary liquefaction caused by porewater pressure re-distribution.

Disadvantages of drainage methods are that there is no easy way to verify the effectiveness of the drains in the field and, should liquefaction be triggered in an earthquake, the damage may be just as severe as if no drains were installed. Furthermore, the spacing of the drains is sensitive to the permeability of the soil which is not readily measurable and often highly variable.

10.2 Permanent dewatering

Lowering the groundwater table increases the thickness of the non-liquefiable crust, and increases the effective stress for soils below the water table. If the water table is reduced to a level below the liquefiable soil layer, liquefaction is prevented because the absence of water makes the buildup of excess porewater pressure impossible (Cox and Griffiths, 2010).

Clearly, in order to retain effectiveness, it is necessary to maintain the low groundwater level in applying this method. Therefore, at sites with virtually unlimited recharge areas and those requiring large estimated improvement, the use of this technique is typically too costly due to the maintenance associated with continual pumping and the deterioration of pumping efficiency over time.

However, at some sites where it is possible to lower the water table to a designated depth over a long period of time, ie permanent dewatering, this technique can be a good method to prevent liquefaction-induced damage to structures. For example, Yasuda (2015) reported the application of this technique to several sites in Japan to improve the liquefiable soil of a large residential area. The studies indicated that the appropriate water table to prevent liquefaction damage to wooden houses is about 3 m below ground level. For this purpose, drain pipes and shallow wells were installed under roads and these were able to lower the water level under the houses.

Subsequent investigations indicated that porewater pressure decreased due to dewatering only at shallow depths. Based on these studies in 2014, the Japanese Ministry of Land, Infrastructure and Transport (MLIT) published a guideline on how to apply this remediation concept (Yasuda, 2015). Koseki et al (2015) also reported a case study in Japan where permanent dewatering was adopted as countermeasure against future liquefaction following the 2011 Tohoku earthquake. Using a network of drainage ditches constructed along the roads in the target area, plus installation of supplementary wells, the water table was lowered to 2.1 m below ground level.

Note

As a consequence of the increased effective stress due to dewatering, excessive settlement may occur due to consolidation of soft or loose layers at the site. Therefore, to supplement the application, in situ monitoring and numerical analyses are recommended to predict the associated long-term ground settlement.

10.3 Vertical drains

Vertical drains are typically installed either as column-like drains in a closely-spaced grid pattern or as backfill around underground structures to control the levels of maximum excess porewater pressure ratio during earthquake shaking. They can also be installed as wall-like or column-like perimeter drains around the perimeter of densified (treated) zones to isolate the migration of high excess porewater pressure from liquefied areas.

In the installation of gravel drains, a casing with an auger inside is drilled into the ground down to the specified depth. Crushed stone is then discharged into the casing and the gravel drain is formed by lifting the casing pipe.

Artificial drains can be made of geosynthetic composites or piles with drainage functions. Plastic drain consists of a plastic perforated pipe wrapped in geofabric to prevent clogging from soil particles. These can be easily installed; however, close spacing is usually required due to the limited capacity of each drain. The installation of prefabricated drains is illustrated in Figure 10.1.

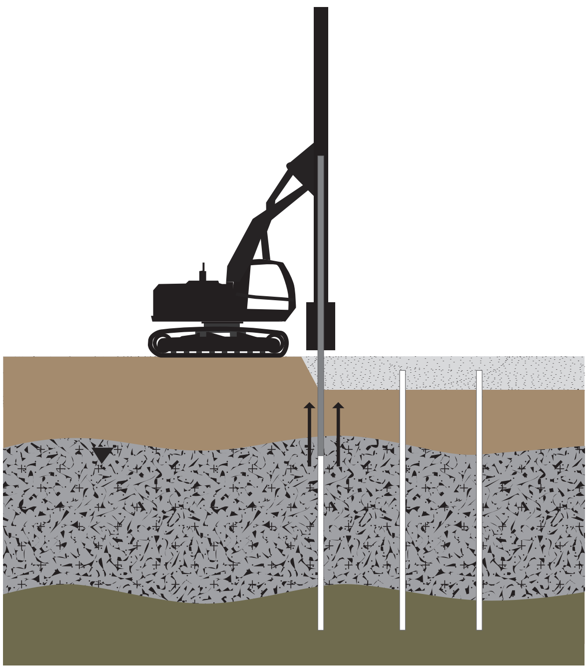
Design charts for drains were initially developed by Seed and Booker (1977) to control the maximum excess porewater pressure levels, but more recent design charts and analytical methods (eg Iai and Koizumi, 1986; Pestana et al, 1997) provide better methods of taking into account various factors affecting the drain performance, such as the hydraulic properties of the drain and permeability and volumetric compressibility of the native soil.

Drainage remediation methods are most suitable for use in sands with less than 5 percent fines. One of the greatest advantages of drains is that they induce relatively small horizontal earth pressures and can be installed with relatively low vibration during installation. Therefore, they are suitable for use adjacent to sensitive structures. In the design of drains, it is necessary to select a suitable drain material that has a coefficient of permeability substantially larger than the in situ soils.

Since the in situ soils improved by this method remain in a loose condition, the method has obvious disadvantages when compared to compacted deposits, such as negligible ductility and significant residual settlement of the treated soils should liquefaction be triggered. It is effective only if it successfully promotes sufficiently rapid dissipation of pore pressures to prevent the occurrence of liquefaction; if pore pressure dissipation is not sufficiently rapid during the relatively few critical seconds of the earthquake, this method does relatively little to improve post-liquefaction performance (Seed at al, 2003). Thus, the method is usually combined with densification methods, ie the surrounding ground is compacted to some extent during the drain installation.

Where drainage is the primary mechanism of treatment, both the aggregate and geosynthetics (geotextiles, geofabrics, and geocomposites) must have the appropriate permeability to dissipate the build-up of porewater pressures induced during shaking. Verification of the materials should be undertaken prior to construction. Drainage aggregate should have particle size distribution (PSD) tests undertaken to confirm the grading of the material is suitable in terms of filtration and permeability. Similarly, geosynthetics need to be tested to confirm they have an appropriate porosity, which will allow the egress of water and retention of soils, strength and filtration properties.

Figure 10.1: Prefabricated vertical drains to mitigate liquefaction



11 Ground improvement for residential construction

In response to the 2010–2011 Canterbury earthquake sequence, MBIE produced a series of guidelines to assist in the recovery and rebuild of houses affected by those events. The main document in that series is 'Repairing and rebuilding houses affected by the Canterbury earthquakes' (MBIE 2012–2015).

Parts of that document (specifically Section 15.3, and Appendix C) provide ground improvement design solutions for the rebuilding of houses on liquefiable 'TC3' ground. The design solutions presented in that document are based upon the results of the 2013 EQC ground improvement trials (Residential Ground Improvement: Findings from trials to manage liquefaction vulnerability), which were carried out to examine adapting ground improvement methods to residential house construction, on a scale that becomes affordable for that size of project. The NZGS/MBIE Module 5a was subsequently published to supplement the residential guidance document.

The MBIE residential guidance document should be referred to for in-depth information (both Section 15.3, as well as Appendix C4, and Module 5a). Some of the key points are presented below.

11.1 Applicability

The ground improvement methods in the residential guidance document are applicable to conventional one- to two-storey residential construction (see Section 1.4.3 of the residential guidance), for sites that fit the characteristics of Canterbury 'TC3' land. Outside the Canterbury region, this will need to be assessed based on local seismicity and expected performance during SLS and ULS design events.

Section 3.1 of the guidance document can be used to aid in this land assessment. In Canterbury however, 'Red Zone' land (ie land that is likely to be more vulnerable than TC3 land to the effects of liquefaction and particularly lateral spread) has been eliminated from the building stock, and this needs to be taken into consideration. It is suggested therefore that if the site is likely to be subject to severe area-wide lateral spread, or if land damage is likely to be severe (LSN>30) at 100-year return periods of shaking, then specific engineering design will need to be undertaken in lieu of simply selecting one of the guidance document solutions.

In all cases a CPEng geotechnical engineer with appropriate earthquake engineering knowledge is required to determine the applicability of each ground improvement method for the site in question, and to carry out any necessary design work. Some of the methods may have a relatively prescribed specification but they are only applicable where soil conditions are appropriate. Other methods will require a degree of design effort.

11.2 Design philosophy

Inherent in the design philosophy for the residential guidance document is the concept that the ground improvement works are part of an integrated foundation solution, comprising both the ground improvement works and either an overlying stiff foundation mat or raft slab, or a relevellable timber subfloor system (depending on the ground improvement option selected).

The design intent is not necessarily to eliminate liquefaction triggering in all the foundation soils — instead it is a performance based design philosophy, where the objective is to reduce damaging differential deformations (particularly flexural distortions) to tolerable levels in the overlying superstructure. This is achieved through control of deformations through both the stiffening or densification of the ground itself, as well as the stiffness provided by the overlying foundation raft slab (or in some cases the relevellability provided by the timber subfloor system).

The desired outcome at SLS levels of shaking is a low level of damage that is readily repairable. At ULS, a low probability of rupture of the structure is a requirement of the Building Code. An integrated foundation solution selected from the residential guidance should result in a foundation system that is unlikely to be the weak link in the total building system (an undesirable situation which caused considerable repair and rebuild cost in the Canterbury earthquake sequence). The performance at ULS will be such that recovery of the foundations will likely be feasible in most cases following such a design event.

11.3 Liquefaction mitigation strategies

A number of residential-scale ground improvement options are presented in the residential guidance document. The liquefaction mitigation strategy associated with the improvement methods comprises either:

- **shallow ground improvement (Figure 11.1 and 11.2)** — accepting that liquefaction will occur, and reducing the potential for damaging differential settlement and flexure of the house superstructure by constructing a non-liquefiable surface 'crust' in combination with a robust, stiffened foundation system; or
- **deep ground improvement (Figure 11.3)** — eliminating or greatly reducing the liquefaction potential (at design levels of shaking) throughout the depth of the soil profile expected to contribute to ground surface settlement (eg 8–10 m for lightweight residential structures). Again, this would be in combination with a suitable surface stiff foundation system.

The shallow options are further divided into those types which form a 'raft' of stabilised or densified materials, and those which rely on reinforcement with 'inclusions' (ie shallow stone columns, shallow columns of highly compacted aggregate (eg 'RAP') or driven timber piles). Design examples for a dense gravel raft and a reinforced cement-stabilised raft ground improvement are given in the Appendix (Examples 1 and 2).

11.4 Ground improvement mechanisms

The mechanisms of ground improvement for the methods presented in the residential guidance can be grouped as follows (noting that some methods can perform more than one of these functions, depending on soil conditions):

- **densification** of the in situ soils to eliminate or reduce triggering of liquefaction at design levels of ground shaking. Most effective in clean or low fines content sands.
- **replacement** of near surface weak soils with a stronger non-liquefiable soil to form a stiff crust. Effective in both sandy and silty soils.
- **stiffening** of the liquefiable soils to improve the integrated foundation system performance through a reduction of cyclic strains; sometimes in combination with increasing liquefaction resistance through densification. This can be effective in both sandy and silty soils — however in sandy soils densification is typically more effective than stiffening. In silty soils the stiffening effects may be primarily due to increases in lateral stresses (which can be lost if large lateral strains occur, eg during a lateral spread event).

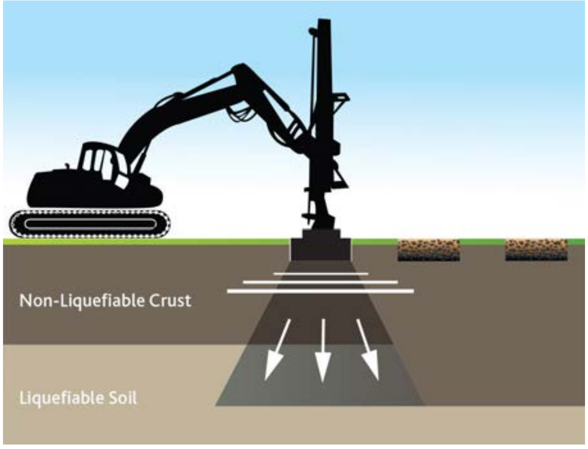
Although rarely used due to cost implications, deeper ground improvement options are included in the residential guidance document. This is for those cases where there is a need to reduce liquefaction-induced deformations at greater depths (for example, where the site also had a potential flooding issue with regard to finished floor levels if it were to settle excessively post-liquefaction).

Figure 11.1: Shallow raft-type residential ground improvement options

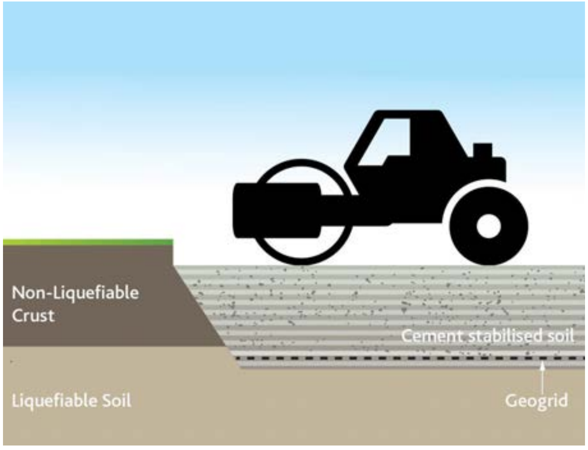
The shallow 'raft type' options comprise:

Excavate and recompact (2 m)

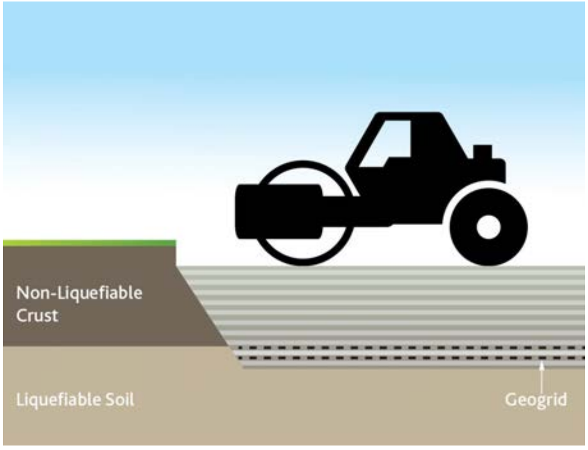
Rapid impact compaction/dynamic compaction



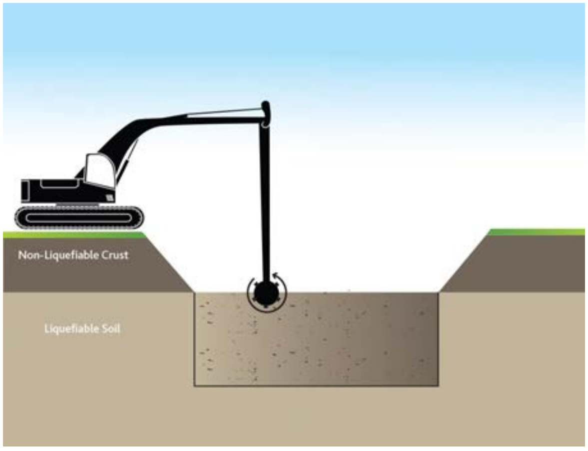
Cement stabilised raft (1.2 m)



Reinforced gravel raft (1.2 m)



Cement stabilised raft (in situ mixing) (2 m)



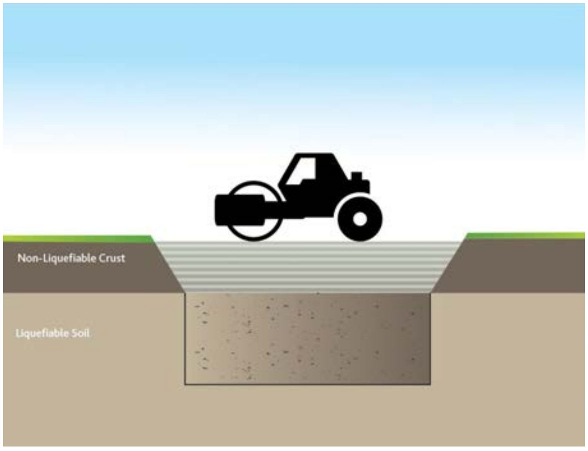
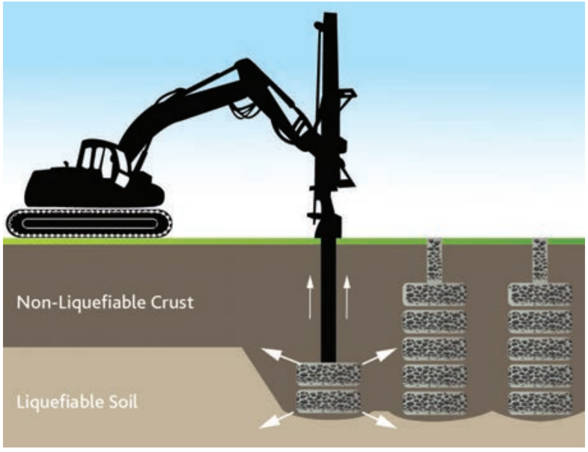


Figure 11.2: 'Crust reinforced with inclusions' residential ground improvement options

The shallow 'inclusion-reinforced' options comprise:

Shallow stone columns, RAP (4 m)



Driven timber piles (4 m)

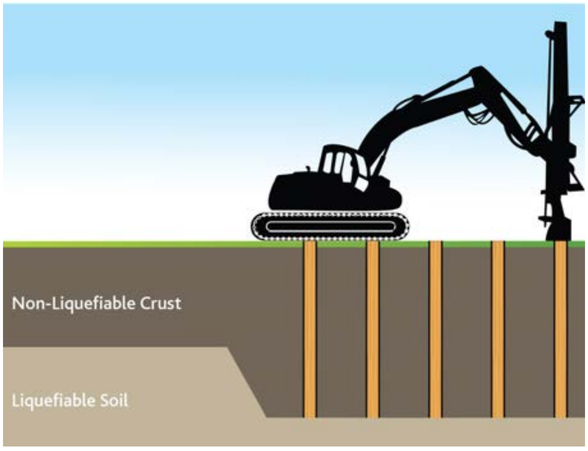
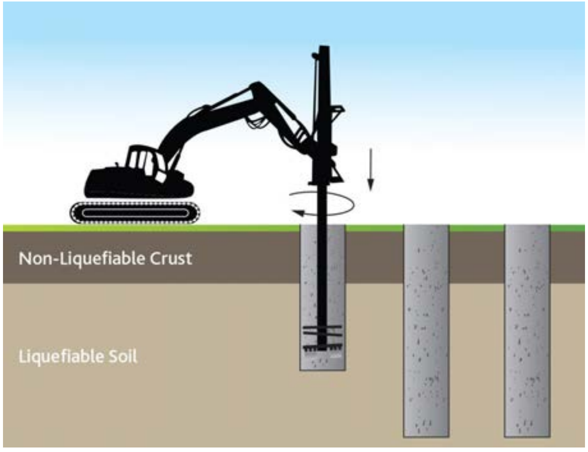


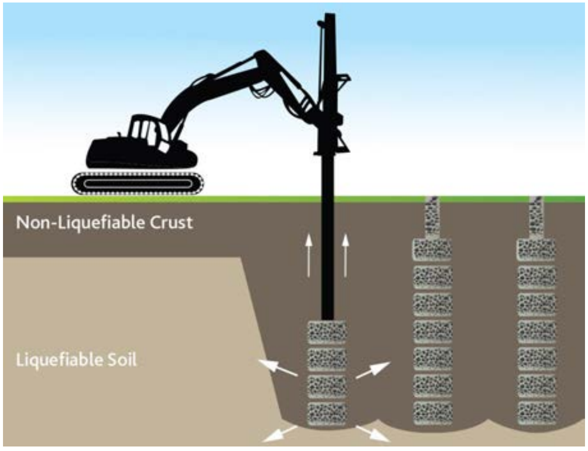
Figure 11.3: 'Deep' residential ground improvement options

The 'deep' options comprise:

Deep soil mixing, jet grouting (8 m)



Deep stone columns (8 m)



11.5 Selection criteria

Each method in the residential guidance is limited to some extent in the scope of its applicability, and the surface foundation components that are suitable for use in conjunction with that method. This is outlined in Table 15.4 in the guidance document. The selection of an appropriate solution depends on several site considerations and constraints, including:

- Soil type (eg fines content)
- Lateral spread potential for the site
- Likely post-treatment ground settlements at SLS and ULS
- Location within the soil column of the liquefiable layers
- Depth to groundwater (ie if dewatering might be required or not)
- Site access (for the necessary plant and equipment)
- Stockpile areas available
- Proximity to structures that might be affected by vibrations or batter instability
- Contractor availability.

As an example, with regard to soil types, in sandier materials the 2013 EQC trials found that columns of highly compacted aggregate (eg RAP) performed better than most other methods tested in eliminating or reducing the onset of liquefaction at design levels of ground shaking. However, as the fines content of the soil increased, the effectiveness of this method to densify the soil decreased. Nonetheless, it was noted that the installation of the columns still acted to stiffen the overall soil mass which resulted in a reduction in triggering of liquefaction up to moderate levels of ground shaking.

On a site containing silty soils discretely layered with clean sands, columns of highly compacted aggregate or conventional stone columns may be effective in both densifying the sandy layers and stiffening the siltier soils, and thereby adequately reduce the liquefaction hazard. However, during construction in some cases, the lower permeability layers may impede pore pressure dissipation and therefore reduce the effectiveness of the improvement of the sands. For a predominantly silty sand site, a replacement method such as a cement stabilised raft or reinforced crushed gravel raft would be a preferred option if total settlement is not a concern.

11.6 Specification, construction and quality control

Appendix C4 of the Canterbury rebuilding houses guidance (MBIE, 2012) provides a simplified method statement for the construction of a number of ground improvement options. It also provides some useful information on construction quality control. Module 5a provides a detailed standard construction specification for the ground improvement options — this specification can be directly incorporated into construction contracts.

11.7 Findings from EQC ground improvement trials

The performance of various ground improvement methods for residential structures was assessed as part of post Canterbury Earthquake Sequence inspections and full scale EQC ground improvement trials (EQC, 2015).

The soil profiles (geology) from properties that have performed well, ie did not experience liquefaction-related damage, through the Canterbury Earthquake Sequence provide the best example of the CPT and crosshole geophysical parameters for soils that are likely to perform well during earthquakes. Generally, properties with thicker, denser or stiffer near-surface soils (non-liquefying crusts) performed better during the Canterbury Earthquake Sequence compared with properties with thinner, looser and less stiff near-surface soils. These profiles were found to be similar to the post-ground improvement CPT and crosshole geophysical profiles for soils that performed well during the T-Rex shake testing and blast-induced liquefaction testing at EQC trials sites (EQC, 2015).

The shallow ground improvement methods for residential construction all aim to thicken and/or stiffen the near-surface soil layers to reduce liquefaction vulnerability, ie replicate the characteristics of natural soil sites that performed well during the Canterbury Earthquake Sequence. The results from the EQC Trials showed that shallow ground improvements do not significantly reduce ground surface subsidence as a result of the liquefaction of the underlying soil layers but they improve the crust rigidity and reduce the differential ground surface subsidence that damages buildings on top of the improved ground. The rapid impact compaction and rammed aggregate piers ground improvement methods work well in building thicker non-liquefying crusts, reducing liquefaction vulnerability. Stiff soils (stiff surface crust) behave more rigidly compared to less stiff crusts, reducing the likelihood of differential ground surface subsidence (undulations, tilt and differential settlement). The 1.2m thick shallow reinforced soil-cement and reinforced gravel rafts work well in improving crust rigidity. Driven timber poles do not prevent liquefaction triggering in the near-surface soils but they help to redistribute the weight of the house and make the liquefaction-induced ground surface subsidence more uniform, which means a reduction in differential ground surface subsidence (EQC, 2015).

12 Procurement for design and construction

A holistic approach that considers the ground and the structure together when building on liquefaction vulnerable land will provide more options and better outcomes. Selection of an appropriate procurement strategy for design and construction is key to the success of any project involving ground improvement. Consideration needs to be given to the scale and complexity of the project, the proportioning of risk between owner and contractor and the overall procurement strategy for the building.

Generally, specifications for ground improvement should include provisions on:

› ground improvement method (materials, equipment, and construction procedure)
› performance and acceptance criteria
› verification testing
› vibration and noise control
› monitoring
› field Inspections, and
› certification of the improved ground.

Contracting approaches used for developing ground improvement specifications include method specifications, performance specifications and a hybrid approach.

With the performance type approach, the engineer specifies the minimum performance requirements of ground improvement, and the contractor develops the design and installation method for ground improvement to meet the performance requirements. The engineer prepares documents that states loading requirements of the structure and performance requirements of the foundations (ultimate bearing strength and strength reduction factors) and settlement tolerances. The contractor (or his subconsultant) determines the amount, arrangement, and properties of the improved ground necessary to satisfy the performance requirements. The performance type approach puts more risk on the ground improvement contractor.

With the method approach, the engineer carries out the design and specifies the scope of work, installation, and quality control as well as quality assurance requirements of ground improvement. The engineer develops a detailed set of drawings and specifications, which are incorporated into the tender documents. The equipment, materials, and installation techniques for ground improvement are prescribed by the engineer. In this approach, the contractor is not responsible for performance of the improved ground. For example, with the method approach, the ground improvement contractor is not responsible for the strength of the improved ground. This approach puts more risk on the engineer.

FHWA-HRT-13-046 (FHWA, 2013) recommends a hybrid approach as the most appropriate contracting approach for ground improvement projects that equitably distributes the responsibilities and risks between the engineer and the contractor. With the hybrid approach, the engineer carries out the overall design but relies on the ground improvement contractor to define the means for achieving the required parameters of ground improvement, eg strength of improved ground.

Close interaction between the geotechnical and the structural designers and the ground improvement contractors is required through the design and construction process. On some past projects, the design of ground improvement was carried out in separation from the structural design. In such cases the geotechnical designers were required to provide an improved building platform with bearing capacity and maximum settlement/ differential settlement and lateral displacements specified by the structural designers. This approach may not result in cost-effective design and should be avoided where possible. Where this approach is used, ground improvement can be carried out under a stand-alone contract and not be part of the main construction contract.

Where the structural and the geotechnical designers work together, the integration of structural and geotechnical design solutions to meet the performance requirements for the building in mitigating the effects of liquefaction and lateral spreading normally results in the most cost-effective design outcomes. The interaction between the structural and the geotechnical designers should also continue through the construction phase, as some adjustments to the structural design may be required depending on the achieved level of ground improvement.

Ground improvement can be procured separately from (as part of the early works or site preparation) or together with other parts of the building projects under:

- 'design-bid-build'
- design-build (where a specialist ground improvement contractor undertakes most of the detailed design) or
- engineering, procurement, and construction management contracting arrangements.

For techniques such as stone or sand columns, soil mixing, grouting, bio-improvement and compaction methods, the specific equipment used for construction and the skill and experience of the contractor can have a profound impact on the effectiveness of the ground improvement. The capability of the contractor is therefore a key aspect to getting quality end product with these methods.

The cost of ground improvement is substantial and for many projects can be comparable to the cost of the structure. Early ground improvement contractor's involvement should be considered to confirm at an early stage of the project the availability of equipment, efficiency of a particular ground improvement method, local experience, project programme and cost of ground improvement. Where there is substantial uncertainty about the applicability of a particular ground improvement technique (with respect to level of improvement that can be achieved or effect on environment such as vibration level, effect on ground water, ground heave, etc), ground improvement trials can be required. Ground improvement trials should be specified under a stand-alone contract or as part of the main construction contract.

Undertaking ground improvement as part of a design and build project requires clear communication and coordination among parties, including the client, whose performance requirements need to be clearly specified, neighbouring property owners, consenting officials, contractors and designers.

Pre-construction building condition surveys on close neighbouring properties are encouraged for ground construction works to verify or mitigate concerns about potential vibration damage and provide evidence should there be complaints about damage.

Building condition surveys before and after ground improvement should be used to determine whether any cosmetic or structural damage has occurred to neighbouring buildings as a result of vibration. Surveys before ground improvement are important as some buildings and structures may already have cracked walls, which could be incorrectly attributed to vibration associated with ground improvement. Therefore, surveys should be made for all properties where construction vibration is predicted to exceed the acceptable limits.

Prior to commencement of ground improvement, properties should be identified that are at risk of damage based on predicted vibration levels. These should be inspected to determine the pre-works existing condition. Depending on the severity of the predicted vibration, measurements of the actual vibration from key sources may be undertaken at the start of ground improvement works. This information can then be used to establish the requirement for on-going condition surveys as the works progress.

Following the completion of ground improvement works, a final condition survey should be undertaken. Specialist surveyors should be employed to conduct building condition surveys and their reports both before and after construction should contain as a minimum:

- Buildings addresses and location.
- A description of the buildings condition and any cosmetic and/or structural damage.
- Sketches and photographs showing the location and extent of any damage.
- Verification of the report by the surveyor and building owner.

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Appendix. Worked examples

Introduction

Six examples of ground improvement design are presented to demonstrate application of the principals outlined in Module 5 to the practical deign of ground improvement to mitigate liquefaction effects.

The six examples cover common scenarios and ground improvement techniques used in New Zealand for light weight residential structures, industrial and heavy buildings. One of the key criteria in selecting a ground improvement technique is the suitability of the ground conditions. Each of the six examples pertain to site conditions at one of two fictional sites, Site A with silty liquefiable soils or Site B with relatively clean sandy soils.

Both sites are situated on level ground with no free faces or water bodies nearby. Hence, lateral spreading is not considered in any of the examples. Furthermore, the ground improvement techniques demonstrated in some examples may not be appropriate for the same scenarios if there was potential for lateral spreading.

For the purposes of simplifying these examples, each site is characterised with a single CPT and some laboratory testing. For many projects, more rigorous site investigations are needed to suitably characterise the site and assess the liquefaction hazard. Scoping of a suitable site investigation are discussed in Module 2. The liquefaction hazard has been assessed using the simplified methods described in Module 3 with the Module 1 seismic hazard estimates.

Importantly, ground improvement is only part of the overall building and foundation system and needs to be designed to work with the other components to achieve the performance requirements, be these the minimum set out in the building code or higher performance defined by the owner. To a degree, these issues are discussed in each example and design procedures and methods for analysis of foundation performance are suggested but not always demonstrated. These issues are discussed further in Module 4.

Calculations using simplified methods to assess the mitigation of liquefaction triggering in improved ground, which are the starting point for seismic design of ground improvement on liquefiable sites, are demonstrated in the examples. These methods have limitations and more sophisticated analysis methods, for example finite element or finite difference analysis will be necessary for some situations to have enough confidence that the design will meet the performance requirements.

Good construction quality control and quality assurance are crucial to confirming the design objectives are realised and the success of any ground improvement project. Construction issues and environmental effects that that need to be considered in the design are broadly discussed with each example. Example specifications for some of the techniques presented are in Module 5a.

The sites

SITE A

The first site, Site A, is in Napier and has a profile of silty sands and silt that is most suited to replacement, solidification (soil mixing for example) or reinforcement ground improvement techniques.

SITE B

Site B, the second site, is in Christchurch and has a sandier profile with less silt that is generally suited to densification techniques like stone columns and dynamic compaction.

Examples outline

EXAMPLE 1 — GRAVEL RAFT REPLACEMENT

This example demonstrates the design of undercut of weak near surface natural soils and replacement with geogrid reinforced densely compacted gravel fill to support a single storey light weight residential structure on Site A.

The example design approach and philosophies have been adapted from the Canterbury Residential Technical Guidance, MBIE 2012.

Note
The more detailed approach outlined in Example 3 could be taken to optimise the ground improvement and foundation design for this scenario.

EXAMPLE 2 — CEMENT-SOIL MIXED RAFT

This example demonstrates the design of a cement soil mixed raft as an alternative to the gravel raft in Example 1 for the same light weight structure on Site A. As with Example 1, the design approach is based on the Canterbury Residential Technical Guidance, MBIE 2012 and the more detailed approach outlined in Example 3 could be used to optimise the design.

EXAMPLE 3 — STONE COLUMNS

This example describes the design of stone column ground improvement and shallow foundations for a new steel portal frame warehouse at Site B.

EXAMPLE 4 — DYNAMIC COMPACTION

This example demonstrates the design of dynamic compaction for the same building and site as Example 3, a new steel portal frame warehouse at Site B.

EXAMPLE 5 — CONTROLLED MODULUS COLUMNS

This example considers the design of controlled modulus (weak concrete) columns to mitigate the effects of liquefaction for a medium rise concrete frame building with a grillage of intersecting foundation beams located on Site A.

EXAMPLE 6 — DEEP SPOIL MIXED LATTICE

This example considers the design of lattice reinforcement using walls constructed from contiguous deep soil mix (DSM) columns to mitigate the effects of liquefaction for a medium rise concrete frame building with a mat foundation located on Site A.

Site A

Site A is located in a low-lying alluvial plain in the Napier region. The site and general area is on level ground with no waterways or free faces in the vicinity.

Ground stratigraphy is reasonably uniform across the site and comprises interbedded silty SAND, non plastic sandy SILT and SAND to a depth of 8.5 m. Between depths of 8.5 m and 12 m is a 3.5 m thick layer of moderately plastic firm clayey SILT. Below 12 m the ground comprises thickly bedded layers of medium dense to dense SAND and firm to stiff SILT.

The corrected cone resistance, friction resistance, dynamic porewater pressures and soil characterisation from a typical CPT for the site are presented in Figure A.A.1. The generalised ground profile and engineering soil properties are described in Table A.A.1. Atterberg testing and particle size analysis on samples taken from a borehole are summarised in Table A.A.2 and Figure A.A.2.

Four hand augers and scala penetrometers have also been carried out at the site to a depth of 2 m and show the natural crust is typically soft and variable having a scala penetration resistance between 50 mm and 150 mm per blow.

Groundwater level varies seasonally. Monitoring shows that the winter water level is 1.5 m below ground level. There are no artesian or sub-artesian conditions at this site and groundwater in the vicinity is not used for water supply.

Table A.A.1: Ground profile for Site A

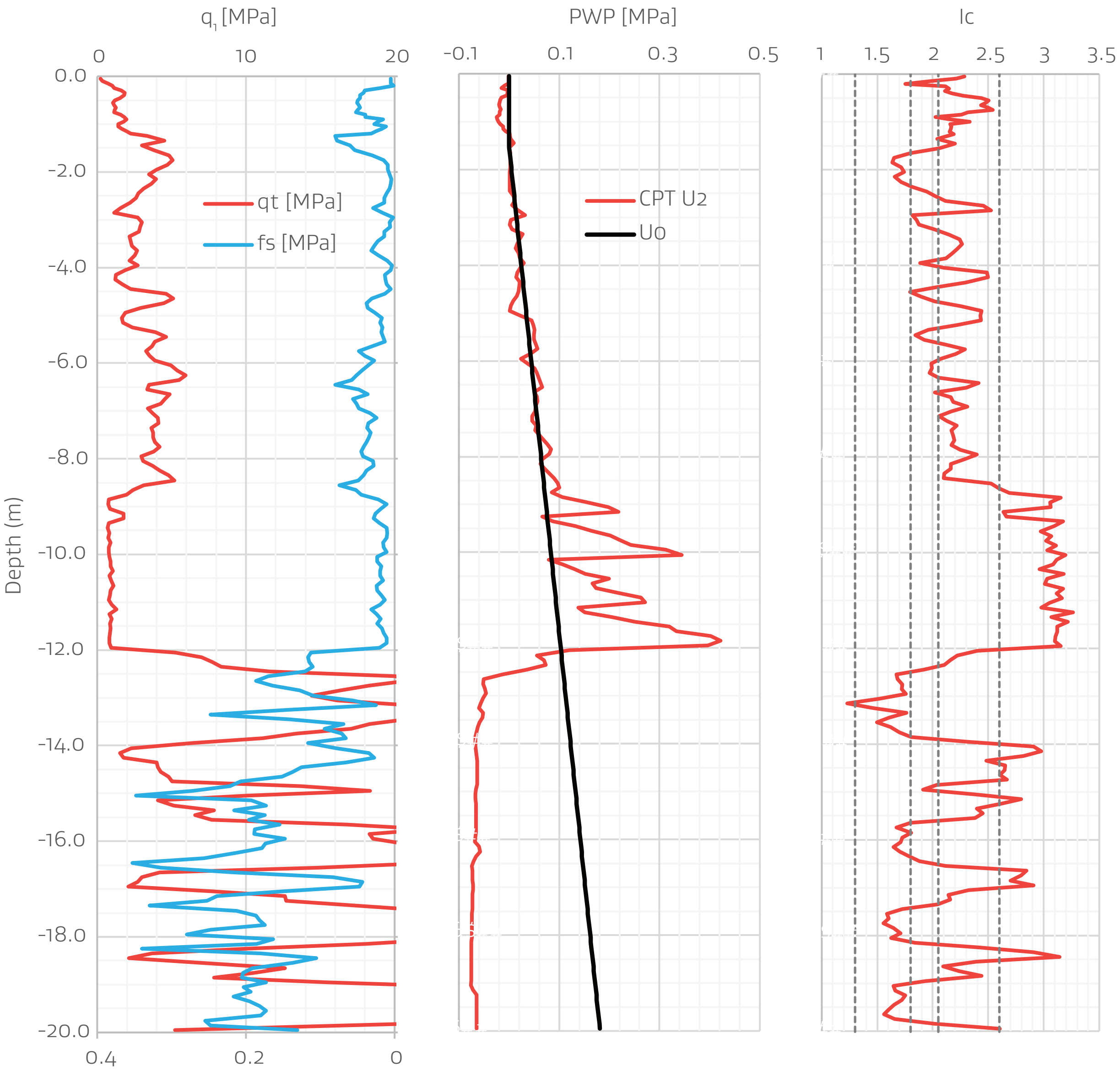
DEPTH		UNIT DESCRIPTION	BULK UNIT WEIGHT, γ (kN/M3)	ANGLE OF SHEARING RESIST, Φ' (º)	COHESION, c' (kPa)	UNDRAINED SHEAR STRENGTH, su (kPa)
0	0.2	TOPSOIL. SILT with some sand. Soft, moist, slightly plastic.	N/A	N/A	N/A	N/A
0.2	8.5	Interbedded silty fine to medium SAND, sandy SILT and fine to coarse SAND, loose	18	30(1)	0	N/A
8.5	12.0	Clayey SILT, firm, mod plastic.	17	22	0	40
12.0	12+	SAND, fine to coarse, medium dense to dense and firm to stiff SILT	20(2) 8(3)	38(2) 30(3)	0(2)	N/A(2) 50 – 150(3)

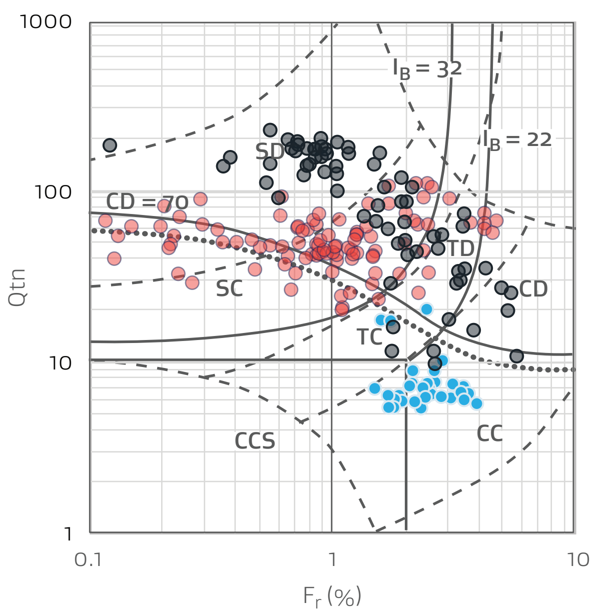
1 Soil in the top 1.5 m have an angle of shearing resistance of 25 degrees

2 Sands

3 Silt.

Figure A.A.1: CPT measurements and soil behaviour characterisation, Site A





Soil behaviour type

1 **CCS** Clay-like, contractive, sensitive
2 **CC** Clay-like, contractive
3 **CD** Clay-like, dilative
4 **TC** Transitional, contractive
5 **TD** Transitional, dilative
6 **SC** Sand-like, contractive
7 **SD** Sand-like, dilative

$CD = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$

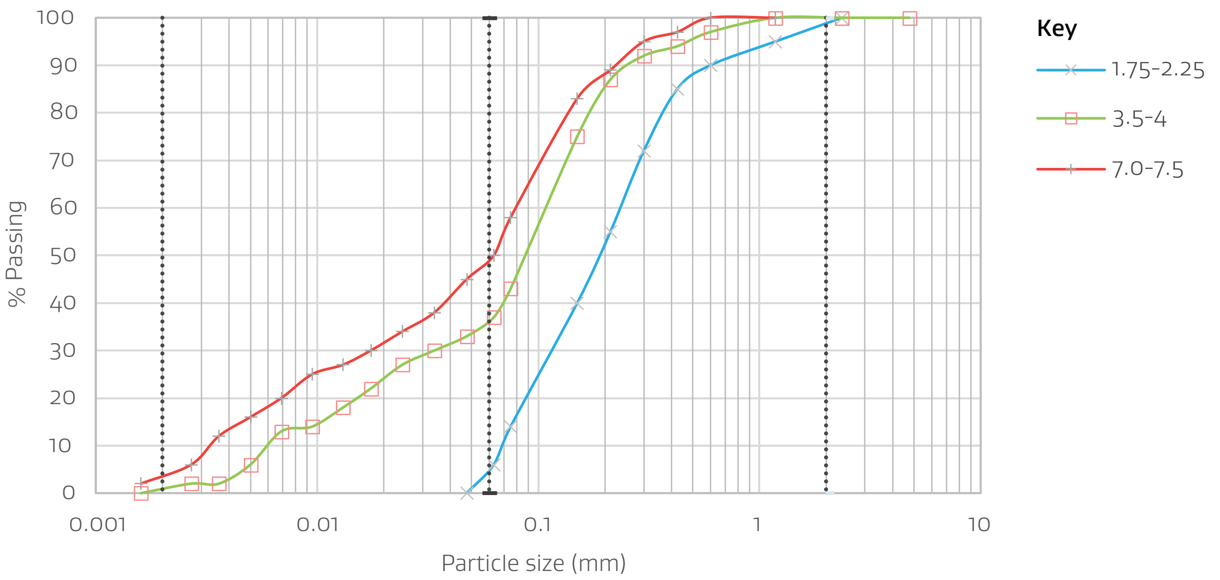
$I_B = 100\,(Q_{tn} + 10)/(70 + Q_{tn}F_r)$

0 m – 8.5 m
8.5 m – 12.0 m
12.0 – 20 m

Table A.A.2: Atterberg limits for soil samples from Site A

SAMPLE DEPTH	WATER CONTENT	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	LIQUIDITY INDEX
1.5	30	35	28	7	0.29
3.5	32	–	–	N.P.	–
7.5	28	–	––	N.P.	–
10.0	47	62	27	35	0.57

Figure A.A.2: Particle size analysis, Site A



Seismicity

A site-specific hazard assessment has been carried out to provide ground motion parameters for the site. Table A.1 summarises the peak ground accelerations and mean magnitudes at PGA for a range of return periods calculated from the PGA.

Table A.A.3: Magnitude and peak ground acceleration (PGA) values for Site A

ID	PRINCIPAL LOCATION	25 YRP		50 YRP		100 YRP		250 YRP		500 YRP		1000 YRP		2500 YRP	
		a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W
PL2	Napier	0.12	6.4	0.18	6.5	0.26	6.7	0.42	7.0	0.58	7.2	0.79	7.2	1.12	7.2

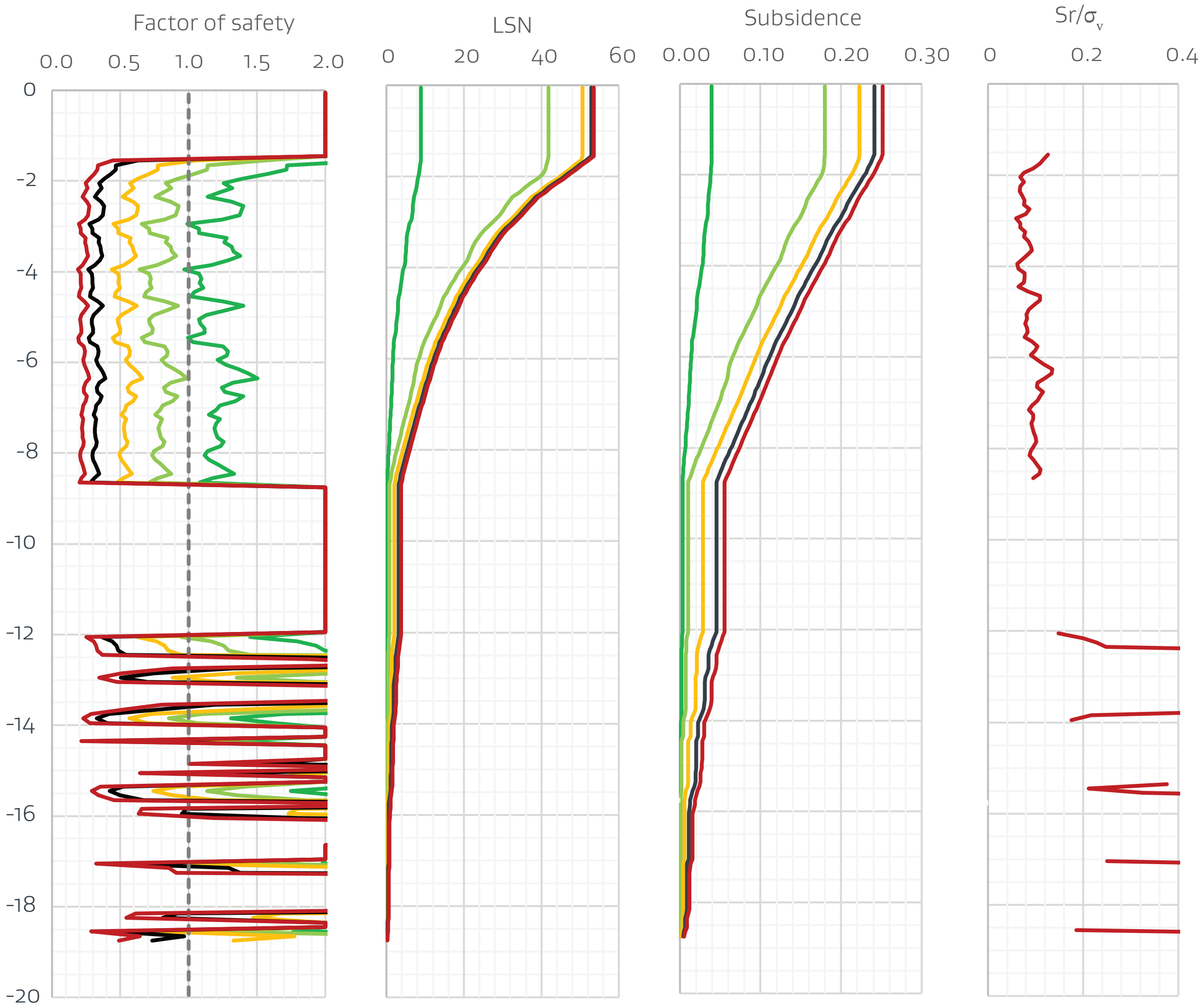
Liquefaction

The site liquefaction hazard is evaluated in accordance with Module 3. Inspection of I_c, the soil descriptions and the plasticity testing carried out shows that apart from soils between 8.5 m and 12 m deep and the interbedded silts below 12 m, the remaining sand and silt layers below the water table are susceptible to liquefaction.

Liquefaction triggering evaluation has been carried out using the simplified empirical method by Boulanger and Idriss (2014) with 15th percentile cyclic resistance (P_L=0.15) and fines contents calculated using the relationship with I_c. The calculated fines content for all layers is reasonably consistent with the measured fines contents in the laboratory particle size analyses and the borehole descriptions, therefore a calibration factor, C_{FC} of 0.0 has been applied to all layers. For triggering assessment, the groundwater level is set at 1.5 m below ground level. Figure A.A.3 shows the results of the triggering analysis for return periods of 25, 50, 100, 250 and 500 years.

Liquefaction is not triggered in a 25 y return period earthquake although some excess porewater pressure may develop resulting in minor ground subsidence and ground damage. The layers of loose silty sands and sandy silts between 1.5 m depth and 9 m depth are the most prone to liquefaction at this site and may liquefy in a 50 y return period earthquake or at greater levels of ground shaking. Below a depth of 12 m, liquefaction may develop in the layers of medium dense sand in earthquakes with return periods of 100 years to 200 years.

Figure A.A.3: Liquefaction triggering analysis for return periods of 25 y, 50 y, 100 y, 250 y, 500 y, Site A



While the clayey silt between 8.5 m and 12 m deep is not susceptible to liquefaction (PI>18, wc/LL <0.8), it may be susceptible degradation of stiffness and strength in large earthquakes. Using the procedure by Boulanger and Idriss (2007), large shear strains (3 percent or more) may develop in the clayey silt in earthquakes with a return period of more than about 200 years. This silt is not sensitive (LI<1.5) and its residual strength after earthquake loading in a 500 year earthquake is anticipated to be between 0.6 and 0.8 times its monotonic strength.

As the area is approximately level and there are no watercourses or free faces in the vicinity, there is no risk of lateral spreading or global lateral movement. Minor lateral stretch and cracking of the ground surface may result from differential cyclic ground movements in a strong earthquake

LSN is greater than 40 for earthquakes with a return period of 50 year or higher. Significant ground damage including ejection of soils at the surface and up to a few tens of centimetres of ground subsidence is possible at the 50 y return period or higher levels of earthquake shaking. Therefore, in the next 50 years there is approximately a 65 percent probability of significant ground damage occurring at this site from earthquake induced liquefaction. The residual undrained strength of the liquefied soil between 1.5 m and 8.5 m is calculated to be 0.08 times the overburden pressure using the method by Idriss and Boulanger (2008).

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Site B

Site B is located on soils known to be liquefiable. The site is on level ground with no waterways or free faces in the vicinity.

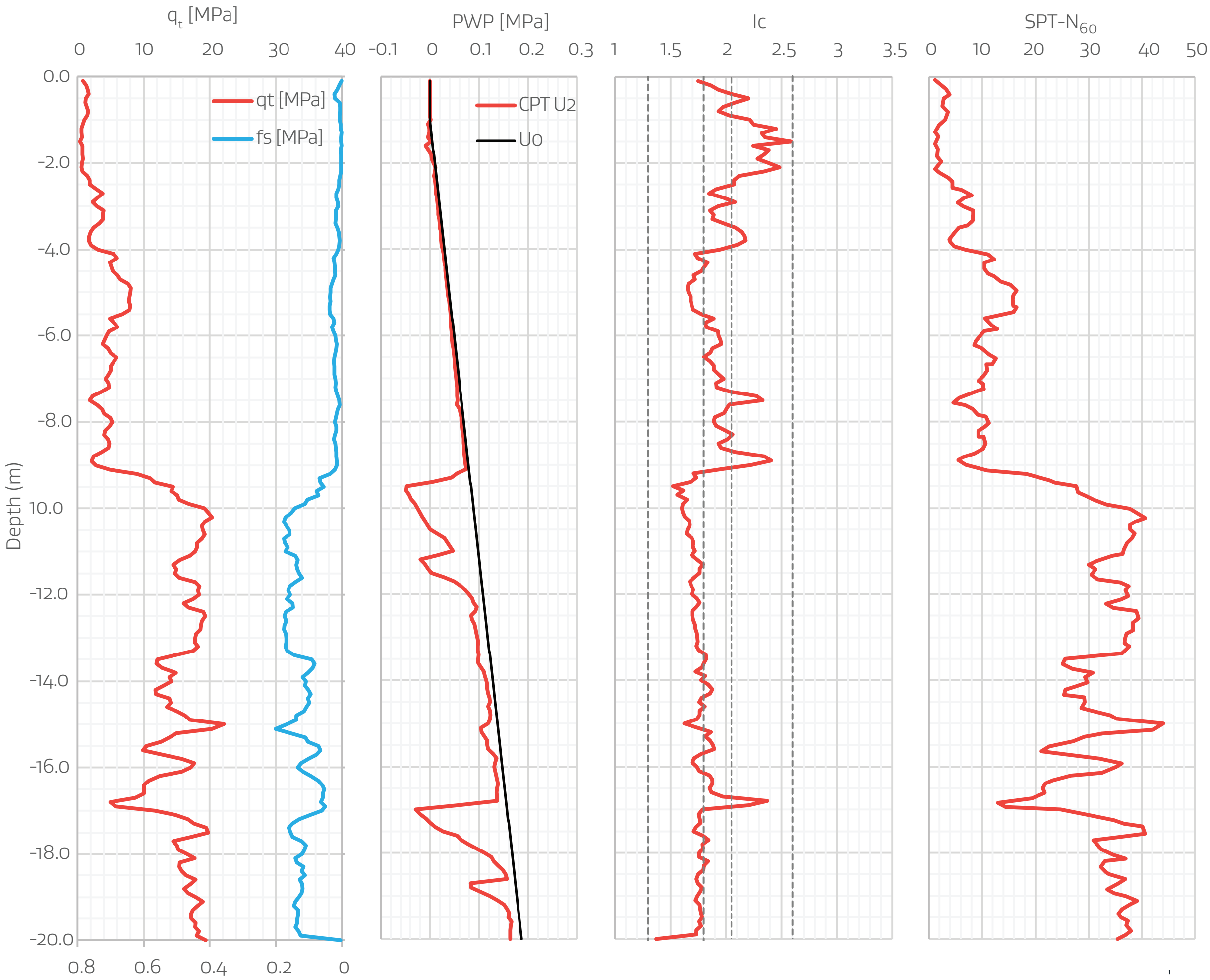
This site is predominantly underlain by loose to medium dense SAND but also has layers of firm to stiff non-plastic to moderately plastic SILT. The generalised ground profile at Site B and soil engineering properties for each layer are described in Table A.B.1. The corrected cone resistance, friction resistance, dynamic porewater pressure (u_2) and soil behaviour index, I_c for a typical CPT are presented Figure A.B.1. Particle size analyses on samples taken from a borehole are shown in Figure A.B.2.

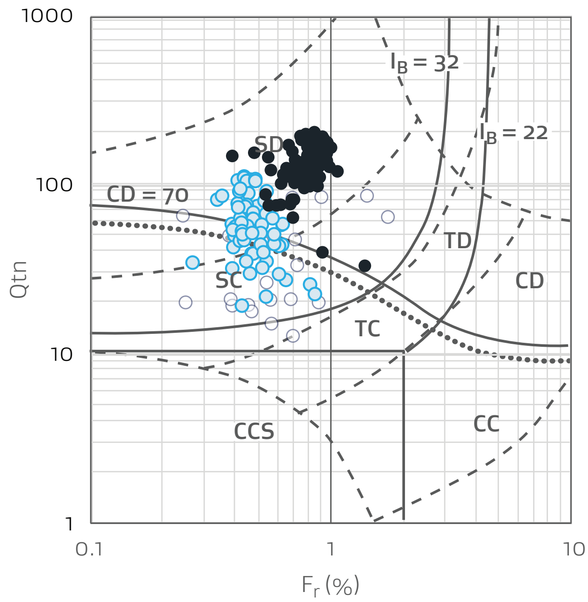
Water table is 1.5 m below ground level on average but varies seasonally by 0.5 m. A winter groundwater table of 1.0m below ground level has been adopted in analysis. There are no artesian or sub-artesian conditions at this site and groundwater in the vicinity is not used for water supply.

Table A.B.1: Ground profile for Site B

DEPTH		UNIT DESCRIPTION	Avg qt (MPa)	UNIT WEIGHT (kN/m³)	ANGLE OF SHEARING RESIST, ϕ ' (º)	COHESION, c' (kPa)
0	0.2	TOPSOIL. SILT with some sand. Soft, moist, slightly plastic.	N/A	18	N/A	N/A
0.2	2.1	Silty SAND, firm, moist, low plasticity (PI from lab testing = 5).	1.0	18	30	3
2.1	9.2	SAND, with a trace to minor silt. Loose, wet, non-plastic. Sand is fine grained. Occasional thin silt lenses.	5.0	18	32	0
9.5	20	SAND, with some fine gravel. Medium dense well-graded. Occasional thin silt lenses.	16.0	19	37	0

Figure A.B.1: CPT measurements and soil behaviour characterisation, Site B





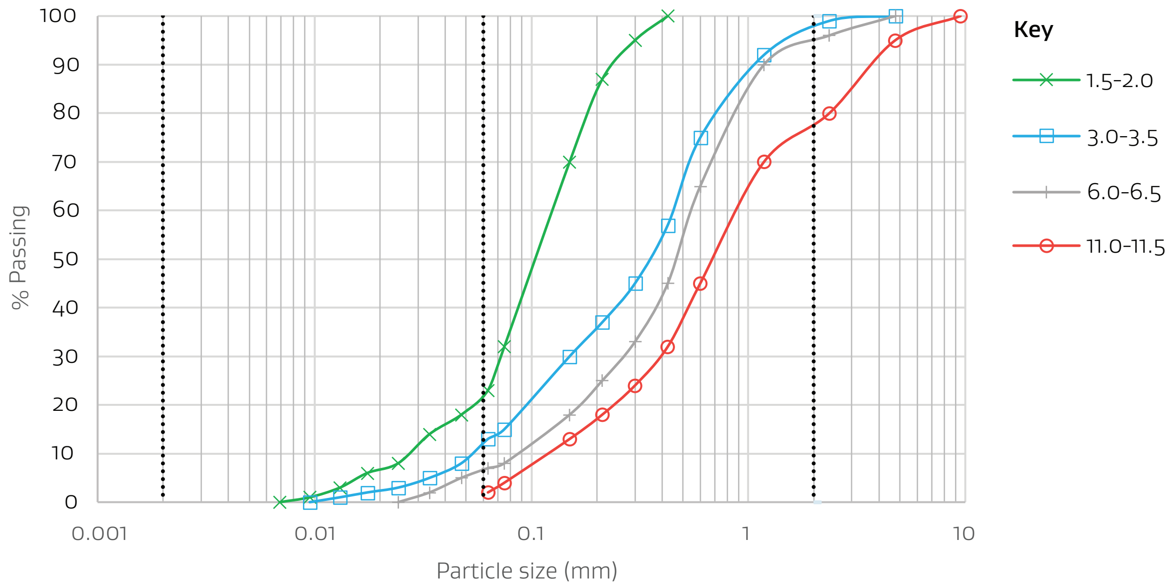
Soil behaviour type

1 **CCS** Clay-like, contractive, sensitive
2 **CC** Clay-like, contractive
3 **CD** Clay-like, dilative
4 **TC** Transitional, contractive
5 **TD** Transitional, dilative
6 **SC** Sand-like, contractive
7 **SD** Sand-like, dilative

$CD = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$

$I_B = 100\,(Q_{tn} + 10)/(70 + Q_{tn}F_r)$

Figure A.B.2: Particle size distributions, Site B



Seismicity

Ground motion parameters for the site have been assessed using the parameters for liquefaction analysis in the Canterbury region as specified in Module 1 and adjusted using the R values from NZS 1170.5. (2004)

Table A.B.2: Magnitude and peak ground acceleration (PGA) values for Site B

ID	PRINCIPAL LOCATION	25 YRP		50 YRP		100 YRP		250 YRP		500 YRP	
		a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W	a_{max} (g)	M_W
PL2	Christchurch	0.09	7.5	0.13	7.5	0.18	7.5	0.26	7.5	0.35	7.5

Liquefaction

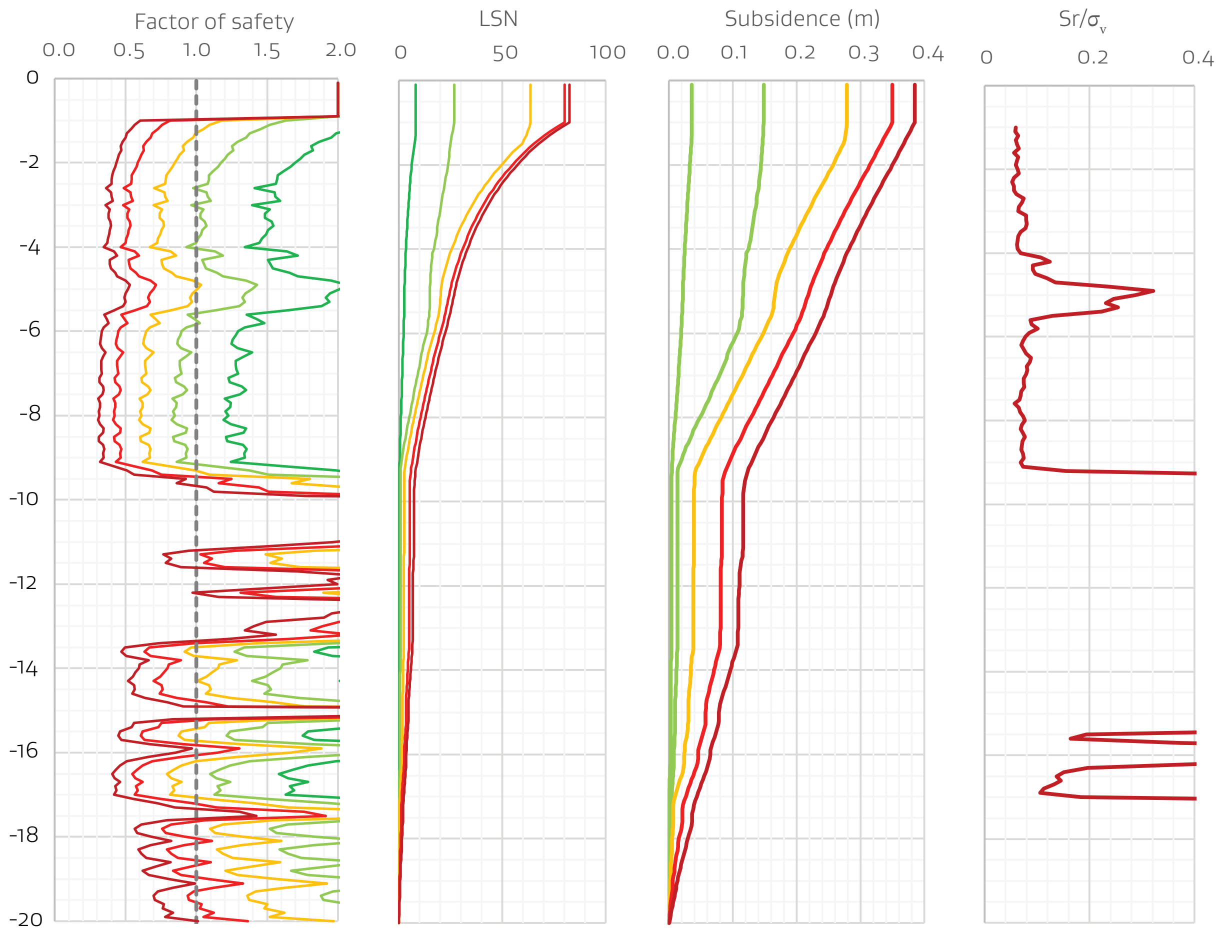
The site liquefaction hazard is evaluated in accordance with Module 3.

Inspection of Ic, the soil descriptions and the plasticity testing carried out on samples of silty sands in the upper 2.1 m of the soil profile shows practically all soils below the water table at this site are susceptible to liquefaction.
Liquefaction triggering evaluation has been carried out using the simplified empirical method by Boulanger and Idriss (2014) with 50th percentile cyclic resistance (P_L=0.5) and fines contents measured in the PSD's. Conventionally, liquefaction triggering is assessed using 15th percentile cyclic resistance. 50th percentile resistances are used in this example as observed land performance in previous earthquakes indicates that this is reasonable, and the proposed ground improvement reduces the sensitivity of foundation seismic performance to the liquefaction resistance.
For triggering assessment, the groundwater level is set at 1.0 m below ground level. Figure B.3 shows the results of the triggering analysis for return periods of 25, 50, 100, 250 and 500 years.

Liquefaction is not triggered in a 25 y return period earthquakes and ground damage is expected to be minor at this level of shaking. The layers of loose sands between 1 m depth and 9.5 m depth are the most prone to liquefaction at this site and may liquefy in a 50 y return period earthquake. Between 9.5 m and 1.3.5 m depth the sands are typically too dense to liquefy. Below a depth of 13.5 m, liquefaction may develop in the looser layers of the medium dense sand in earthquakes with return periods of 100 years to 200 years or more.

LSN is greater than 50 for earthquakes with a return period of 100 year or higher. Significant ground damage including ejection of soils at the surface and a few tens of centimetres of ground subsidence is possible at 50 y return period or higher levels of earthquake shaking. Therefore, in the next 50 years there is a 50 percent to 60 percent probability of significant ground damage occurring at this site from earthquake induced liquefaction.

Figure A.B.3: Liquefaction triggering analysis for return periods of 25 y, 50 y, 100 y, 250 y, 500 y, Site B



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NZS 1170.5:2004. *Structural Design Actions* — Part 5 Earthquake Actions — New Zealand. Standards New Zealand.

Example 1: Shallow undercut and replacement with a dense gravel raft

Scenario

This example considers construction of a single storey light weight residential structure on Site A with undercut of the near surface natural soils and replacement with reinforced densely compacted gravel fill.

The example design approach and philosophies have been adapted from the Canterbury Residential Technical Guidance, MBIE (2012), v3 and are consistent with the guidance in Module 4 and Module 5. The first steps of this process, evaluating liquefaction, lateral spreading and site stability are presented in the Site A evaluation.

LAND TECHNICAL CATEGORY

Land in Canterbury was zoned and categorised based on its potential for damage from liquefaction in earthquakes (Canterbury residential technical guidance, MBIE 2012, v3). The land categorisation gives an indication of future seismic performance and foundation types that are suitable for new residential buildings. While this system was developed specifically for the rebuild of Canterbury following the Canterbury earthquake sequence, the system can be applied outside of Canterbury for initial assessment of foundation systems that may be suitable on liquefiable sites.

Using the Canterbury residential technical guidance, Site A is categorised as TC3 as ULS land settlement from reconsolidation of soils above a depth of 10 m exceeds 100 mm (refer to Table 3.1 in the residential technical guidance) and therefore foundations have a higher risk of damage from liquefaction than TC1 or TC2 category sites. Within the TC3 category, the site is sub-classified as having minor to moderate vulnerability to liquefaction because subsidence in a SLS earthquake is less than 100 mm, lateral spread is not expected and lateral stretch across the footprint is likely to be less than 200 mm (refer to Section 12 in the residential technical guidance).

BUILDING FORM

The building is 12 m wide x 20 m long timber framed building clad with weatherboards and a profiled steel roof. The superstructure is therefore light weight and tolerant to moderate levels of differential settlement.

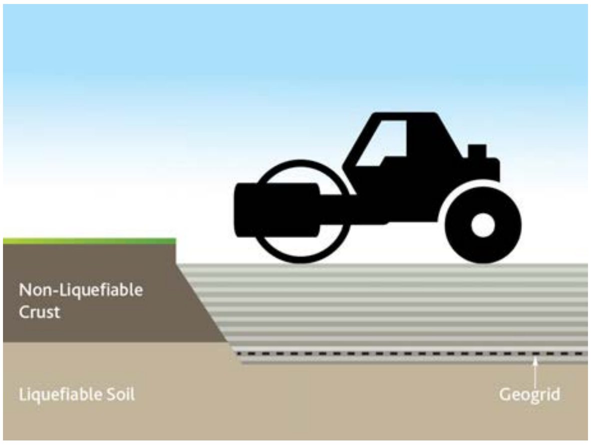
GRAVEL RAFT GROUND IMPROVEMENT

Construction of the gravel raft involves:

- Excavation and removal of soils to the base of the undercut with dewatering and shoring as required.
- Placement of a filter fabric at the base and side of the excavation.
- Import, placement and compaction of the well graded strong angular gravel fill in uniform lifts of a thickness that prevents segregation but ensures adequate compaction.
- Installation of a stiff of layers of stiff geogrid near the base of the excavation to improve confinement in the lower part and edges of the gravel raft and aid compaction.
- Quality assurance testing on the materials, dimensions and compaction.

A schematic typical section of a gravel raft under construction is shown in Figure 4.

Figure A.1.1: dense gravel raft construction



Design

STEP 1: ESTABLISH PERFORMANCE REQUIREMENTS AND DESIGN CRITERIA

The building is an Importance Level 2 structure in terms of NZS 1170.5 (2004) and has a design life of 50 years. The minimum seismic performance requirements are described in the New Zealand building code and summarised as:

› **For the serviceability limit state**, the building must maintain its amenity following an earthquake with a return period of 25 years. At this level of shaking all parts of the structure shall remain functional so that the building can continue to perform its intended purpose. Minor, readily repairable damage to the structure and some damage to building contents, fabric and lining are acceptable.

› **For the ultimate limit state**, the building is expected to suffer moderate to significant structural damage in an earthquake with a return period of 500 years, but must not to collapse. It may be uneconomic and/or not feasible to repair a building or structure that has been subjected to an ULS load.

A more detailed description of the design requirements for residential structures is in the Canterbury repair and rebuild technical guidance, Section 8.2 (MBIE 2012, v3).

Design criteria for residential buildings can be developed from the indicator criteria for repair and rebuild in the Canterbury repair and rebuild technical guidance, Section 2.3 (MBIE 2012, v3). For this example, the following criteria are adopted to meet the seismic performance requirements:

1 **For the ultimate limit state (500 year return period)**: The foundation bearing strength calculated using a strength reduction factor of 1.0 shall be greater than the post earthquake design bearing pressure

2 **For the serviceability limit state (25 year return period)**: Total subsidence shall not exceed 50 mm and the slope of the floor shall not exceed 1 in 200 between any two points 2 m apart.

For light weight structures, Bray and Macedo (2017) recommend a minimum post earthquake factor of safety of 1.0 to suitably limit subsidence. Assuming unfactored post earthquake loads on the foundation, this suggests that in a limit state approach, a strength reduction factor of 1.0 is acceptable in this situation.

A strength reduction factor of 1.0 is adopted for this example considering the structure is relatively tolerant to differential subsidence and the and the capability of the waffle slab to redistribute load should there be loss of support in some areas.

> **Note**
> A flood risk has not been identified at this site with global settlement of up to 300 mm.

While not mandatory to meet the requirements of the Building Code, it is good practice to consider performance across the range of return periods and identify where step changes in performance occur. At Site A, damage from liquefaction may be significant in earthquakes with a return period of 50 years, the return period where liquefaction is triggered.

An intermediate limit state is not assessed as part of this example. However, it would be reasonable to assess performance at a return period of 100 years and to limit damage at this intermediate limit state such that releveling the floor may be necessary but reconstruction or repair of services under the building would not be required. Suitable design criteria for the intermediate limit state could be limiting differential subsidence across the floor to 100 mm.

STEP 2: ASSESS WHETHER THE NATURAL CRUST CAN SUPPORT THE STRUCTURE

Whether the natural crust is thick enough and competent enough to support the foundation loads and meet the performance requirements can be assessed using qualitative and quantitative methods (refer to Section 4.5 of Module 4).

Based on the TC3 classification of the site (see site A liquefaction evaluation) and the variability and low stiffness of the near surface in situ soils, the natural crust is not considered sufficiently competent for the proposed dwelling.

Note

To simplify these examples, Site A has been characterised using only one CPT and the ground conditions are assumed to be relatively uniform across the building footprint. Generally, at least two CPTs would be used to assess the uniformity of the deeper soil profile across the site and the potential for tilt or low angular distortion of the shallow foundations from variations in the ground conditions and liquefaction potential beneath.

Further qualitative assessment of the crust could be carried out by comparison of LSN with observations of foundation performance related to LSN and the liquefaction induced ground damage chart by Ishihara, 1985 (refer to example 3, step 9) or quantitative assessment using the methods by Bray and Macedo (2017) for example to estimate bearing capacity and settlement and compare these to the design criteria.

STEP 3: SELECTION OF GROUND IMPROVEMENT AND FOUNDATION CONCEPT

Ground improvement and the structure itself work as an integrated system and need to be designed together to meet the performance requirements. Section 15 of the Canterbury residential technical guidance is referred to in the selection and design of the ground improvement and foundation system for the new dwelling.

Site A is a minor to moderate TC3 site and a waffle slab with gravel raft replacement ground improvement is one of the acceptable foundation solutions for this site category in the Canterbury residential technical guidance, Table 15.2.

Gravel raft replacement

Replacement ground improvement is suitable for the treatment of the silty sands, sandy silts and silts at this site. Environmental effects must be considered together with other technical aspects, access and space for construction when considering the feasibility of undercut and replacement ground improvement. In this example, we assume there are no constraints on the size of excavation or vibration and noise during construction, that major dewatering will not be necessary and that there is sufficient access and space for construction plant and materials.

Where significant dewatering and or temporary retaining is required for construction, the system needs to be designed to prevent damage to neighbouring property.

The principals of replacement ground improvement are described in Module 5, Section 6 and Section 15 of the Canterbury technical guidance, MBIE (2012). The dense non-liquefiable gravel fill is designed to limit differential subsidence and mitigate the potential for ejecta within the building footprint in the event of an earthquake that causes liquefaction. The gravel raft spreads the building load to the underlying liquefiable soils and evens out subsidence at the surface. A dense gravel raft will not eliminate subsidence entirely, but it does limit the shear deformation beneath the building foundations which is generally more damaging than global subsidence from reconsolidation of liquefied soils.

The gravel fill may also provide a path for groundwater that will tend to flow upward from underlying liquefied layers. The fabric filter around the gravel fill prevents intrusion of foundation soils into the gravel crust zone from seepage erosion as porewater pressures in liquefied areas dissipates. This drainage will help to supress liquefaction effects in the underlying soils. A suitable drainage outlet at the perimeter and use of a well-sealed damp-proof membrane will stop direct ingress of water to the building.

Foundations

The foundation comprises a stiff reinforced concrete waffle slab to bridge areas localised areas of differential settlement. The foundations system will be similar to option 4 described in the MBIE guidance for repairing and rebuilding houses as shown in Figure A.1.2. Specific design of the reinforced concrete mat foundation is beyond the scope of this example.

Following the recommendation in MBIE Guidance for Repairing and Rebuilding houses affected by the Canterbury earthquake (Dec. 2012), and to meet serviceability limit state floor slope requirements, the building reinforced concrete waffle slab is to be designed to withstand the following loss of support:

› 2 m width from the edge, ie the raft is designed to cantilever 2 m at the edge.
› Any 4 m wide area in the middle of slab, away from the edge.

STEP 4: DETERMINE GRAVEL RAFT THICKNESS AND EXTENTS

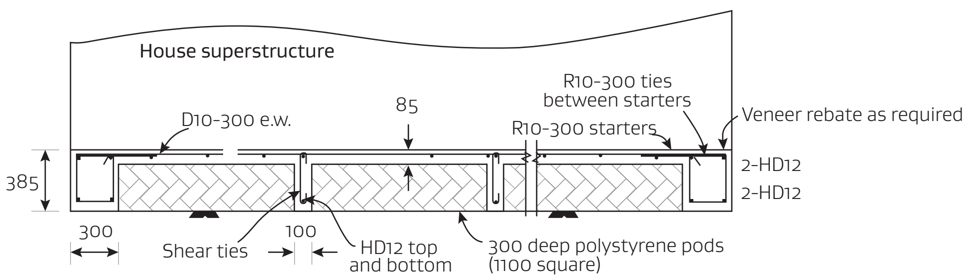
Generally, in areas with liquefying soils, residential homes founded on thicker and stiffer non-liquefying crusts were observed to perform better than residential homes with thinner or less stiff non-liquefying crusts (EQC, undated, Residential ground improvement, Findings from trials to manage liquefaction vulnerability).

Observations of the performance of light weight residential dwellings on liquefiable ground in the Canterbury Earthquake Sequence and in the EQC ground improvement trials suggest that the seismic performance of well-engineered residential foundations is satisfactory when there is a minimum of 1 m to 1.5 m thickness of uniformly stiff crust overlying liquefiable soils at sites with a uniform ground profile and a low risk of lateral spreading.

A minimum gravel raft thickness of 1.2 m is recommended in the Canterbury technical guidance (MBIE, 2012) and is used as a starting point to assess bearing capacity and settlement against the design criteria.

The gravel raft needs to extend sufficiently beyond the waffle slab to spread the load at the edge of the waffle slab and to allow any excess porewater pressure in the gravel raft to dissipate beyond the perimeter of the waffle slab rather than into the building. For this example, the base of the gravel raft extends 1 m beyond slab footprint as recommended in the Canterbury technical guidance.

Figure A.1.2: Waffle slab foundation (copied from MBIE 2012)



ULS requirements

Post earthquake bearing capacity following a ULS earthquake is calculated using the method by Meyerhof and Hanna (1978) for 2-layered soil profiles as recommended by Bray and Macedo (2017) using post earthquake soil strengths. The ultimate bearing capacity is calculated as:

$$q_u = 5.14C_2 + 2\frac{C_a D_1}{B} + \gamma_1 D_f \leq 5.14C_1 + \gamma_1 D_f$$

$$Ca = C_1 \left(-0.58 \times \left(\frac{C_2}{C_1}\right)^2 + 0.96 \times \left(\frac{C_2}{C_1}\right) + 0.612\right)$$

Where C_1 and C_2 are the average post earthquake strength of the gravel fill and liquefied soils respectively, γ_1 is the bulk weight of the granular fill and D_1 is the thickness of the gravel raft (1.2 m) and D_f is the embedment depth of the waffle slab below ground level (0.2 m)

A constant volume angle of shearing resistance of 38° and a bulk density of 22 kN/m3 is assumed for the compacted well graded granular raft fill. The average shear strength of the gravel fill assuming the groundwater table is at the surface (allowing for groundwater migration into the raft from upward seepage of water from the underlying liquefied layers) is:

$$C_1 = 0.5 \times 1.2\text{m} \times \left(22\ \frac{\text{kN}}{\text{m3}} - 9.8\ \frac{\text{kN}}{\text{m3}}\right) \times \tan 38°$$
$$C_1 = 5.7\ \text{kPa}$$

Effective stress at middle of the liquefied layer (4 m depth) is:

$$\sigma_{vo} = 1.2\text{m} \times 22\ \frac{\text{kN}}{\text{m3}} + 2.8\ \text{m} \times 18\ \frac{\text{kN}}{\text{m3}} - 2.5\text{m} \times 9.8\ \frac{\text{kN}}{\text{m3}} = 52.3\ \text{kPa}$$

And the average undrained strength of the liquefied layer beneath the raft, C_2 is therefore:

$$C_2 = 0.08 \times 52.3\ \text{kPa} = 4.2\ \text{kPa}$$

$$C_\alpha = 5.7\ \text{kPa} \times \left(-0.58 \times \left(\frac{4.2\ \text{kPa}}{5.7\ \text{kPa}}\right)^2 + 0.96 \times \left(\frac{4.2\ \text{kPa}}{5.7\ \text{kPa}}\right) + 0.612\right)$$

$$C_\alpha = 5.7\ \text{kPa}$$

The ultimate bearing capacity for the 12 m wide waffle foundation is:

$$q_u = \min \rightarrow 5.14 \times 3.3\ \text{kPa} + 2 \times \frac{5.5\ \text{kPa} \times 1.2\text{m}}{12\text{m}} + 22\frac{\text{kN}}{\text{m3}} \times 0.15\text{m},\ 5.14 \times 5.7\ \text{kPa} + 22\frac{\text{kN}}{\text{m3}} \times 0.15\text{m} = 34.9\ \text{kPa}$$

With a strength reduction factor of 1.0, the design bearing strength is 34.9 kPa

The waffle slab foundations apply an approximately uniform vertical pressure of 15 kPa to the gravel raft. This load is the unfactored deadload and the post earthquake live-load, for the load combination G, E_u, γ_{cQ}.

Note
$E_u = 0$ for this post earthquake case.

The post earth-quake bearing strength (34.9 kPa) is greater than the design bearing pressure (15 kPa) therefore the design meets the ULS design requirements.

SLS requirements

While some excess porewater pressure may be generated, liquefaction is not expected to trigger in a SLS earthquake, the ground profile and liquefaction hazard across the building footprint is reasonably uniform and land subsidence calculated in the liquefaction evaluation is 35 mm. Total settlement is therefore likely to be less than 50 mm and reasonably uniform in an SLS earthquake.

As the waffle slab is designed to bridge areas of local differential subsidence, and considering the low level of ground settlement predicted in an SLS earthquake and the ability of the stiff gravel raft to reduce differential settlement at the base of the waffle slab over short distances, floor slopes are unlikely to exceed 1 in 200 over a distance of 2 m.

The foundation system therefore meets the SLS design requirements.

Note
If liquefaction was predicted to trigger at the SLS level of shaking, the methods presented in Example 3 could be used to assess subsidence and design the foundation system.

Intermediate limit states

Performance in earthquakes between SLS and ULS is not included in the scope of this example. However, the methods in Example 3 could be used to assess the foundation seismic performance at intermediate limit states where liquefaction is triggered.

STEP 5: DETAILS

Selection of the filter fabric should consider its filtration and strength characteristics. A fabric with strength class B and filtration class 2 NZTA F7 is specified for this example. The fabric extends up the side of the excavation to prevent lateral intrusion of the natural soils into the gravel raft.

Two basal layers of geogrid are included in the granular raft, the first is installed above the geotextile at the base of the excavation and the second is placed 1 lift of compacted fill (approximately 150 mm) above. Stiff tri-axial that is not susceptible to large creep elongation is used in this example gravel raft. The geogrid should be placed taught before placement and compaction of gravel. An additional three layers of 2.4 m wide grid are installed at each edge above the 2 basal layers of basal grid at depth intervals of 300 mm for additional confinement of the edge.

A welded HDPE damp proof membrane is installed on top of the gravel raft and below the waffle slab foundation to both protect the building from dampness and prevent egress of excess porewater pressure from the gravel fill into the building.

STEP 6: STABILITY OF THE EXCAVATION AND DEWATERING

Dewatering will may be required to properly compact the lower layers of the gravel fill. The detailed design of dewatering is beyond the scope of this example. Construction dewatering and groundwater control (Powers et al, 2007, 3rd ed) gives guidance for the design of construction dewatering. The stability of an open excavation and the need for shoring depends on the space available and the method of dewatering. This is also outside the scope of this example.

Dewatering will cause temporary decrease of ground water table in the surrounding ground. The associated potential ground settlement can be assessed using empirical methods in Powers et al or using other suitable methods.

Construction

Further details of the construction of dense gravel rafts are contained in MBIE Guidance for Repairing and Rebuilding houses affected by the Canterbury earthquake (Dec. 2012) and specifications for construction of stiff granular rafts are in NZGS Guidance Module 5a.

References

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Meyerhof, G. G., & Hanna, A. M. (1978). *Ultimate bearing capacity of foundations on layered soils under inclined load*. Canadian Geotechnical Journal, 15(4), 565-572.

Ministry of Building Innovation and Employment, (2012), *Repairing and rebuilding houses affected by the Canterbury earthquakes*, version 3.

NZS 1170.5:2004. *Structural Design Actions — Part 5 Earthquake Actions* — New Zealand. Standards New Zealand.

Powers, J. P., Corwin, A. B., Schmall, P. C., & Kaeck, W. E. (2007). *Construction dewatering and groundwater control: new methods and applications*. John Wiley & Sons.

Example 2: Shallow undercut and replacement with a reinforced cement-soil mixed raft

Scenario

This example demonstrates the design of a cement soil mixed raft as an alternative to the gravel raft in example 1 for the same light weight structure with a stiff waffle slab foundation on the same site, Site A. A reinforced soil-cement raft is a good alternative to a gravel raft when crushed gravel is scarce or expensive.

The example design approach and philosophies have been adapted from the Canterbury Residential Technical Guidance, MBIE (2012) and are consistent with the guidance in Module 4 and Module 5. Example 1 should be read in conjunction with this example as much of the design of shallow reinforced cement-soil mixed raft is similar to the design of the gravel raft in Example 1. The first steps of this process, evaluating liquefaction, lateral spreading and site stability are presented in the Site A evaluation.

REINFORCED CEMENT-SOIL MIXED RAFT

Construction of a reinforced soil-cement mixed raft involves:

- Excavation of soils to the base of the undercut with dewatering and shoring as required. Excavated soils are stockpiled on site.
- Mixing of uniform batches of the excavated soils with cement and water in a pug mill or rotovator.
- Placement and compaction of the cement mixed soil in uniform lifts of a thickness that ensures adequate compaction.
- Installation of a stiff of layers of stiff geogrid near the base of the excavation to protect against cracking and aid compaction and near the top of the soil mixed raft to protect against wide tensile cracking that could cause high angular distortion of the foundations if the raft hogs.
- Quality assurance testing on the materials, dimensions and compaction

Design

STEPS 1 AND 2, PERFORMANCE REQUIREMENTS AND ASSESSMENT OF THE NATURAL CRUST

Step 1, setting performance requirements and design criteria and step 2, assessing the suitability of the natural crust are the same as for example 1.

STEP 3: SELECTION OF GROUND IMPROVEMENT AND FOUNDATION CONCEPT

Ground improvement and the structure itself work as an integrated system and need to be designed together to meet the performance requirements. Section 15 of the Canterbury residential technical guidance is referred to in the selection and design of the ground improvement and foundation system for the new dwelling.

Site A is a minor to moderate TC3 site and a waffle slab with a reinforced cement-soil mixed raft is one of the acceptable foundation solutions for this site category in the Canterbury residential technical guidance, Table 15.2.

A reinforced cement soil mixed raft is suitable for ground improvement of the silty sands, sandy silts and silts at this site. There is sufficient access and space for construction and the environmental effects are manageable. Reinforced cement-soil mixed rafts are generally stiffer than gravel rafts, and as with gravel rafts they form a thick non-liquefiable layer to mitigate damage in earthquakes that cause liquefaction but will not prevent subsidence entirely.

Ex-situ mixing is selected to stabilise the natural soils in this example as significant dewatering is not likely to be required, it generally enables more effective mixing and lower cement contents compared to in situ mixing and geogrid can be installed within the raft to mitigate wide cracking and potentially high angular distortions of the foundations.

STEP 4: DETERMINE THE RAFT THICKNESS AND EXTENTS

A minimum raft thickness of 1.2 m is recommended in the Canterbury technical guidance (MBIE, 2012) with the raft extending 1 m beyond the perimeter of the waffle slab and these dimensions are adopted for this design example.

A design minimum 28-day unconfined compressive strength of 1 MPa is selected for the cement-soil mixture. Cement contents to achieve this strength may range between 3 percent and 8 percent of the dry weight of the soil. Dosage rates to achieve minimum strengths can be assessed from laboratory testing and confirmed during construction.

The raft has the same dimensions as the gravel raft in Example 1 but is stiffer and stronger, It will therefore meet the design requirements for the ultimate limit state and serviceability limit state as demonstrated for the gravel raft in example 1.

STEP 5: DETAILS

Two layers of stiff triaxial geogrid are included in the soil-cement mixed raft, the first is installed 150 mm above the base and the second is installed 150 mm below the top. The grid is installed to protect against wide cracking and high angular distortion developing in the raft.

STEP 6: STABILITY OF EXCAVATION AND DEWATERING

Refer to Example 1 for comment on this design aspect.

Construction

Further details of the construction of dense gravel rafts are contained in MBIE Guidance for Repairing and Rebuilding houses affected by the Canterbury earthquake (Dec. 2012) and specifications for construction of stiff granular rafts are in NZGS Guidance Module 5a.

References

Ministry of Building Innovation and Employment, (2012), *Repairing and rebuilding houses affected by the Canterbury earthquakes*, version 3.

Example 3: Stone columns

Scenario

This example describes the design of stone column ground improvement and shallow foundations for a new steel portal frame warehouse at Site B, in a new industrial subdivision. The nearest neighbouring building is 50 m from the site.

The design process follows the basic steps outlined in Module 4, Figure 4.1 and design of the ground improvement follows the principals outlined in Module 5. The first steps of this process, evaluating liquefaction, lateral spreading and site stability are presented in the Site B evaluation.

STRUCTURE AND GROUND IMPROVEMENT CONCEPT

Ground improvement

Stone column ground improvement comprises columns of densely compacted stone installed in either a triangular or square grid across the site The columns are typically spaced 1.5 m – 4 m apart and have a diameter of 0.6 m–1.2 m. The soil between the columns are compacted to mitigate the potential for liquefaction and large shear strains developing in earthquakes. Refer to Module 5, Section 7.7 for a more detailed description of stone column ground improvement.

The depth and lateral extent of improvement are designed to mitigate differential settlement of the foundations and superstructure to meet their seismic performance requirements

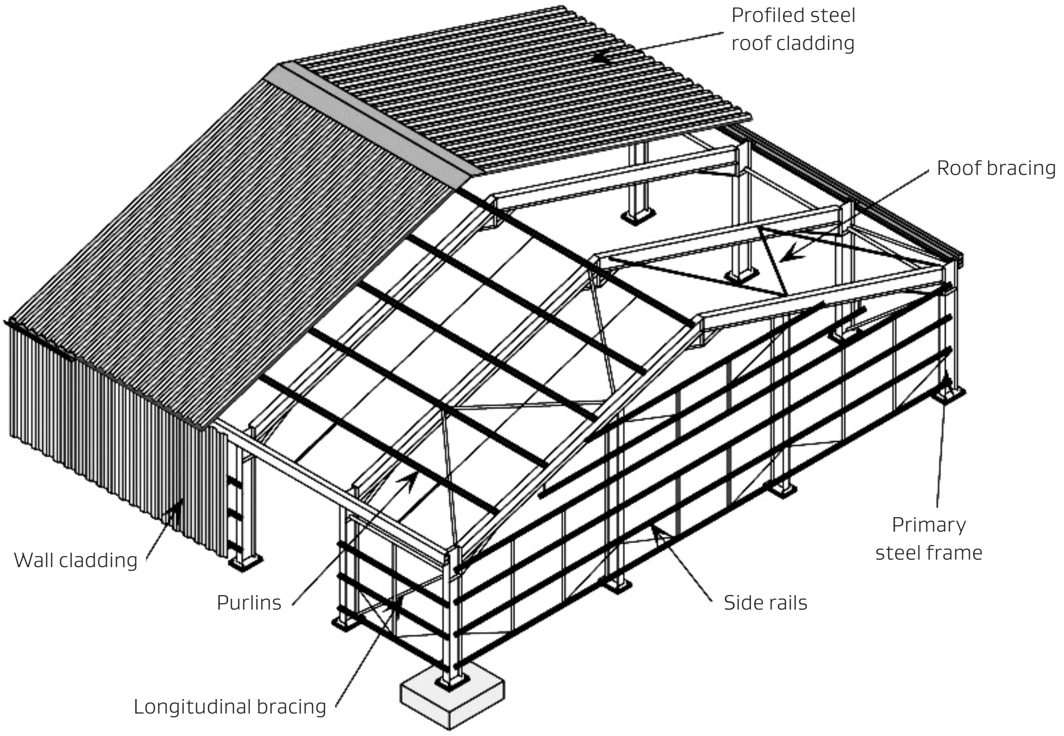
Superstructure

The basic structural form of the new 30 m wide x 72 m long warehouse is shown in Figure 4 and comprises:

1. Steel portal frames in the at 8 m centres. The portals span the full width of the structure with no central supports and are braced in the longitudinal direction.
2. Tilt up precast reinforced concrete wall cladding
3. Shallow strip foundations around the perimeter
4. A 125 mm thick mesh reinforced concrete floor supported on the ground. The floor is integral with the strip footings and relied upon to limit stretch of the building from differential lateral ground movement during an earthquake and to support the precast panels as cantilever elements in a fire.

The building stiffness and mass is reasonably uniformly distributed and will not include a mezzanine floor. The structure is considered type Eb in terms of the MBIE guidelines 'Assessment, repair and rebuild of earthquake-affected industrial buildings in Canterbury' (2014).

Figure A.3.1: Warehouse structural form (https://www.steelconstruction.info/images/0/07/Portal-1.jpg)

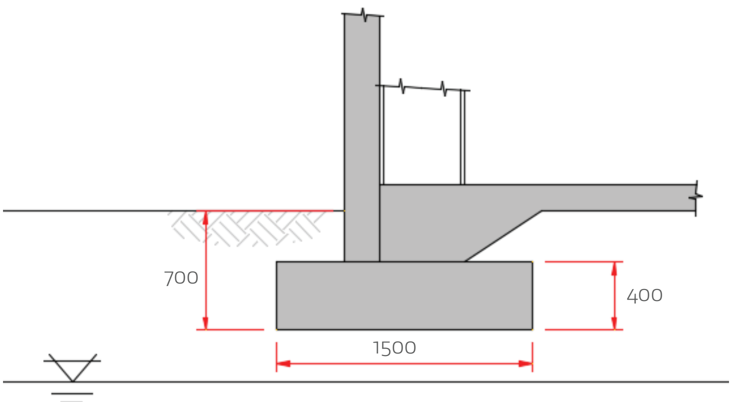


Foundations

The footing compressive load for assessment of post-earthquake bearing capacity and subsidence is 120 kN/m (including the self-weight of a 1.5 m wide x 0.4 m thick reinforced concrete footing). The load is applied approximately centrally to the footing. The design bearing pressure, qd at the base of the 1.5 m wide footing is therefore 80 kPa.

The floor dead and live load for post-earthquake bearing and settlement assessment is 12 kPa. The total building weight (dead and live) for post-earthquake settlement assessment is 50,400 kN and the foundation pressure averaged across the building footprint is 23.3 kPa.

Figure A.3.2: Warehouse reinforced concrete footing detail



Design

STEP 1: ESTABLISH PERFORMANCE REQUIREMENTS AND DESIGN CRITERIA

The building is an Importance Level 2 structure in terms of NZS 1170.5 (2004) and has a design life of 50 years. The building will be used for medium height storage (up to 8 m high) requiring fork-lift access to racks. Additional to the minimum requirements for serviceability and life safety, the client has stipulated that the warehouse must be capable of normal operations within 15 working days of an earthquake having a probability of occurrence of 40 percent over the design life of the building to reduce their commercial risk and insurance premiums.

To meet the seismic performance requirements stipulated by the building code and by the owner, the design criteria in Table A.3.1 have been developed with the structural engineer and agreed with the owner for the earthquake serviceability and ultimate limit states and an intermediate limit state having a return period of 100 years (40 percent probability of exceedance in the next 50 years).

The MBIE guide 'Assessment, repair and rebuild of earthquake-affected industrial buildings in Canterbury' (2014) gives useful commentary on the performance of similar structures on sites that liquefied in the Canterbury earthquakes and damage criteria that can been used to develop the design criteria for new industrial structures.

In this case, the intermediate limit state operational requirements, the choice of precast concrete cladding and the floor having a structural function will likely have a considerable impact on the extent and cost of ground improvement. An alternative of designing to the minimum requirements in the building code, the use of a more flexible cladding system and making the floor non-structural would reduce the extents of ground treatment required. On the other hand, if there is a desire to further reduce the vulnerability of the business to liquefaction, future-proof the site to allow for heavier structures or consider multiple earthquakes that trigger liquefaction (eg in aftershocks), deeper ground improvement may be necessary. These factors should be discussed with the client and structural engineer when developing the performance requirements and design criteria.

Table A.3.1: Performance requirements and design criteria

	SLS	ILS	ULS
Return period	25 y	100 y	500 y
Performance requirements	Minor cosmetic damage that is repairable without major disruption.	Limited damage.[1] Operations restored within 15 days. Permanent repairs to NBS possible within 6 months.	Life safety protection. Safe egress. Complete reconstruction may be required
Max footing gradient[2]	1 in 500	1 in 300	1 in 80
Max floor gradient[2]	1 in 400	1 in 200	1 in 80
Ejecta	Nil in building	Nil in building	Moderate ejecta acceptable

1 Reconstruction or major relevelling of the floor should not be required in an ILS event

2 Over a distance of 2 m.

STEP 2: ASSESS WHETHER THE NATURAL CRUST CAN SUPPORT THE STRUCTURE

For example, it is reasonably evident that the 1 m thick natural raft is insufficient to support shallow foundations that are likely to be embedded at least 0.5 m deep with liquefaction triggering over several meters depth below the natural raft at a return period of 50 years. Therefore, ground improvement is required with shallow foundations.

Whether the natural crust is thick enough and competent enough to support the foundation loads and meet the performance requirements can be assessed using qualitative and quantitative methods (refer to Section 4.5 of Module 4). Methods used to evaluate the depth of ground improvement in the following steps can be used for qualitative and quantitative assessment of natural rafts.

STEP 3: ASSESS SUITABILITY OF GROUND CONDITIONS FOR STONE COLUMN GROUND IMPROVEMENT

Stone columns can be constructed using a variety of methods as described in Module 5. The method of construction, the construction plant and the capability of the plant operator have a significant influence on the effectiveness of stone column ground improvement. Stone columns are suitable for a wide range of soil types as shown in the Figure 7.3 but are best suited to soils with a fines content less than 15 percent and $I_c < 1.8$ but may be suitable for soils with up to 25 percent fines and I_c up to 2.1.

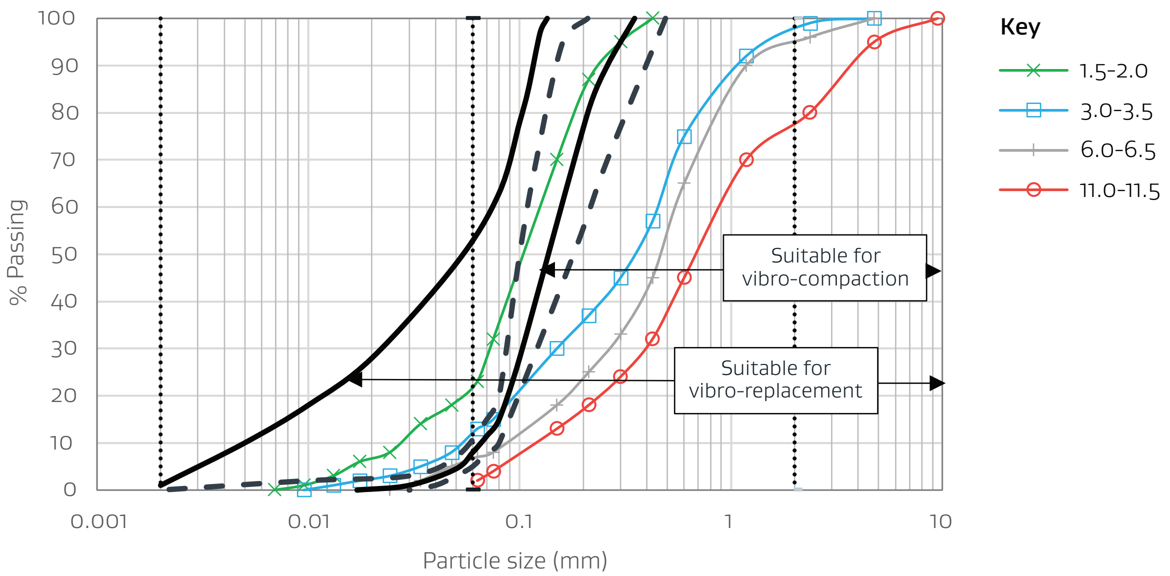
Comparison of the gradings to grading limits suitable for vibro-compaction and vibro-replacement and inspection of the soil behaviour index from the CPT suggests that the liquefiable sands at this site, while potentially not ideal, are suitable for stone column improvement. The sandy silt to a depth of 2.1 m may be difficult to improve with stone columns.

> **Note**
> Approximately 1 m of this layer will be removed at the perimeter for construction of the footings and drainage blanket.

Environmental factors including noise and vibration from column installation, the potential for damage of surrounding buildings and infrastructure, effects on the natural environment and contamination of water supply should also be considered when assessing the feasibility of stone column ground improvement. Guidance on acceptable noise and vibration is given in Module 5 and 5A.

As this structure is in an industrial subdivision, there are no artesian pressures and the nearest building is 50 m away, stone columns are considered feasible in this regard.

Figure A.3.3: Suitability of particle size for vibro-replacement



STEP 4: CALCULATE THE TARGET DENSITY FOR GROUND IMPROVEMENT

Stone columns primarily improve liquefaction resistance of the ground via densification of the soil through the vibration and displacement of the ground with the addition of stone aggregate. The columns also reinforce and may increase the permeability of the improved ground, that can further improve seismic performance. However, as improvement from reinforcement and increased permeability are small or difficult to quantify, the design only considers improvement from densification.

The target level of densification can be defined by setting a minimum SPT N60 value or CPT tip resistance to achieve between the columns and within the columns themselves. The improved ground can be difficult to penetrate with a CPT once improved and here we elect to use SPT's for quality assurance and thus define a target SPT N60 profile.

Target minimum SPT N60 values are back calculated for a pre-determined factor of safety (FoS) against liquefaction using the simplified method for evaluating liquefaction triggering. The target FoS for each limit state is selected considering the design criteria and the potential for softening within the improved ground at lower factors of safety.

Note

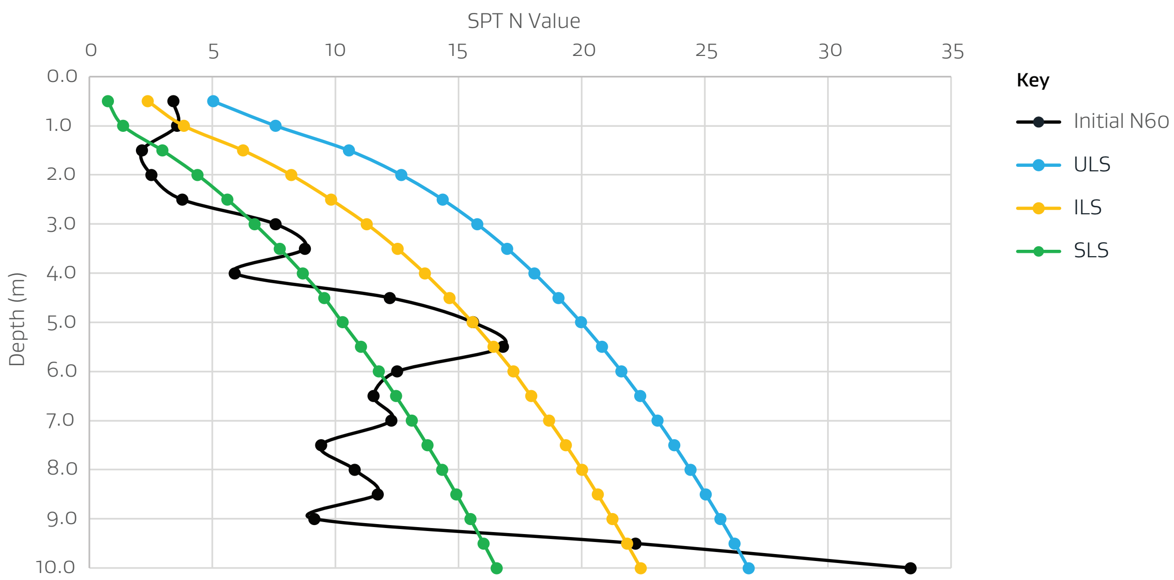
For factors of safety less than about 2, some development of excess porewater pressure is likely within the improved ground, refer to Module 5.

However, the increased density of the improved zone will limit accumulation of strains despite the increase in porewater pressure. The factors of safety selected for this example are:

- SLS 25 yr – 2.0
- ILS 100 yr – 1.5
- ULS 500 yr – 1.2

The procedure by Boulanger and Idriss (2014) is used to calculate the minimum SPT N_{60} required at each depth and for each limit state. The calculations are iterative. Targets at all depths and for all limit states shown in Figure 6. Higher N_{60} values are required to achieve a target FoS of 1.2 in the ULS earthquake, therefore this is the critical limit state for the design of column spacing. Factors to convert CPT to SPT are wide ranging, here N_{60} of the natural ground is calculated in a similar method to the densification targets using CRR calculated from the CPT triggering analysis.

Figure A.3.4: Target N_{60} values versus depth



STEP 5: ESTIMATE STONE COLUMN SIZE AND SPACING

An initial stone column spacing can be calculated using empirical charts that relate either the penetration resistance or the increase in penetration resistance to area replacement ratio, A_r. The area replacement ratio is the area of a single column divided by the area of ground that is treated by the column.

$$A_r = \frac{A_{sc}}{A}$$

Where A_{sc} is the area of the stone column and A is the area of the unit cell.

The actual column spacing should be confirmed with a full scale field trial. For this example, charts from the Japanese Geotechnical Society reclamation manual in Figure A.3.5 and the chart by Barksdale and Bachus (1983), Figure A.3.6 are used to estimate A_r

From Figure A.B.1 see:

- at 1.5 m depth, the in situ SPT N_{60} is 2 and needs to be increased to 10.5
- at 3.5 m depth, the in situ SPT N_{60} is 8.5 and needs to be increased to 17.
- at 8.0 m depth, the in situ SPT N_{60} is 11 and needs to be increased to 25.

Based on Figure A.3.6, the estimated A_r required to achieve the minimum increase in penetration resistance (and therefore density) at depths of 1.5 m, 3.5 m and 8.0 m is 15 percent, 8 percent and 15 percent respectively. A normalised clean sand SPT N160-cs of 23. Therefore, a minimum A_r of 15 percent is required.

Figure A.3.5: Area replacement ratio, Japanese Geotechnical Society, 1998. Fines content < 20%

a Relationship between original N-value N_o and N-value between piles N1

b Relationship between original N-value N_o and N-value after center of pile N_p

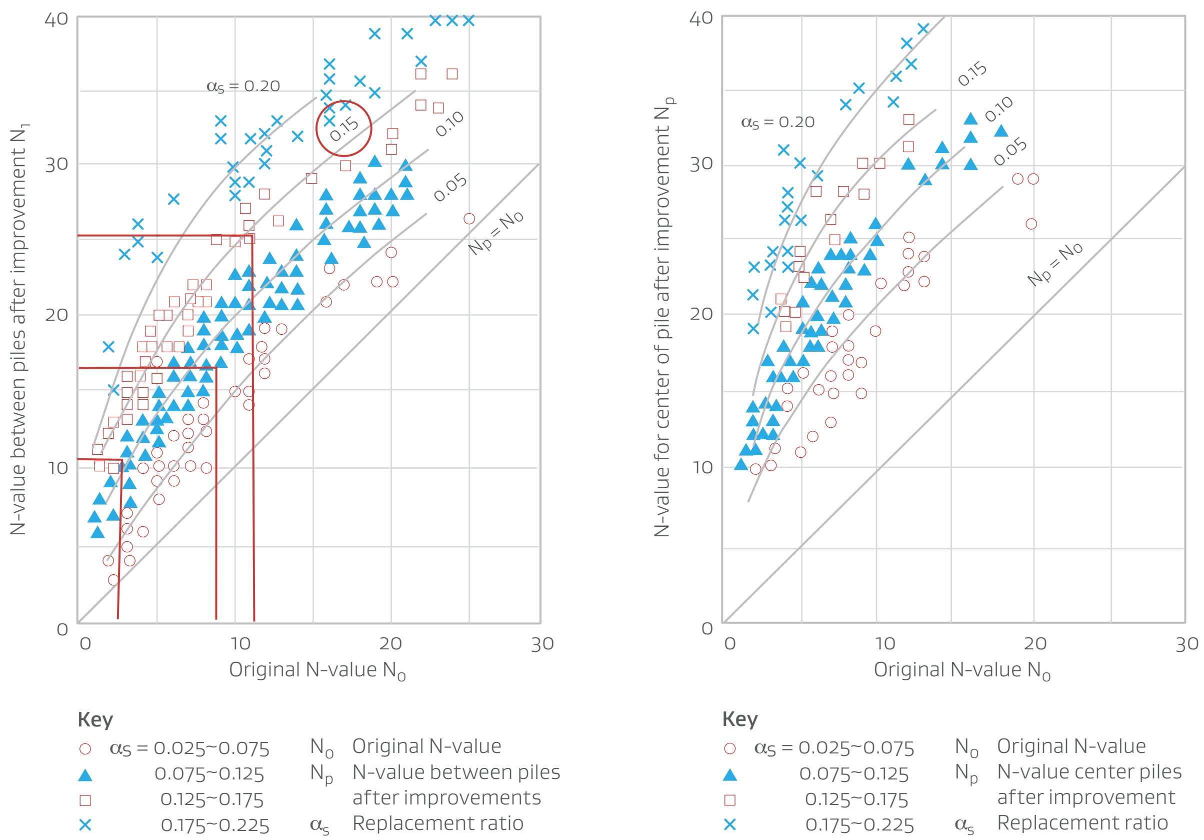
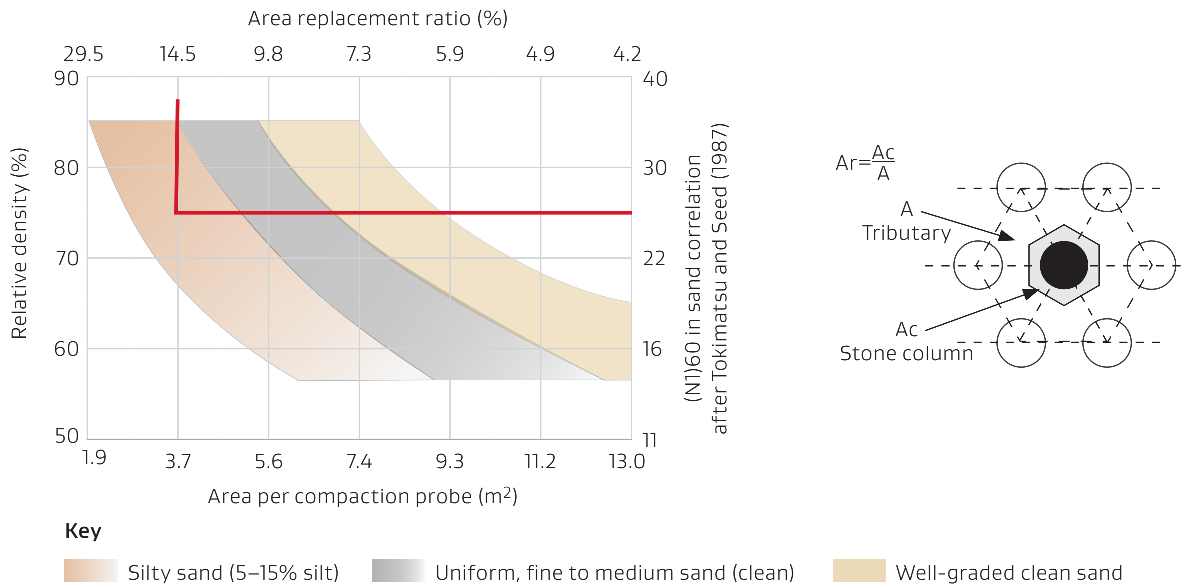


Figure A.3.6: Area replacement ratio, Barksdale and Bachus 1983



A symmetrical pattern is selected for the stone column arrangement so that stiffness is uniform. Columns are typically installed in a square or triangular grid as shown in Table A.3.2.

For a triangular grid and assuming a column diameter of 0.65 m, the initial stone column spacing is 1.6 m, calculated as:

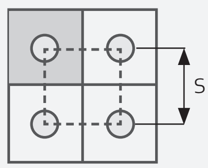
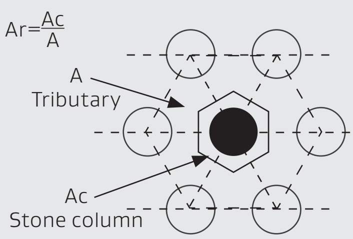
$$A_{sc} = \pi 0.325^2 = 0.33m^2$$

$$A = \frac{0.33m^2}{0.15} = 2.21m^2 = \frac{\sqrt{3}}{2}\text{ spacing}^2$$

$$\text{Spacing} = 1.6m$$

A trial will need to be carried out to confirm column spacing.

Table A.3.2: Possible arrangement for columnar ground improvement

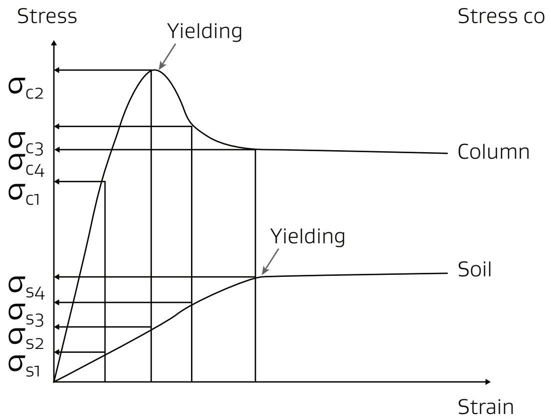
SHAPE OF GRID	TRIBUTARY AREA (AS FUNCTION OF GRID SPACING)	LAYOUT EXAMPLE
Squared	$A_r = s^2$	
Triangular/Hexagonal	$A = A \frac{\sqrt{3}}{2} s^2$	

STEP 6: CALCULATE COMPOSITE STRENGTH OF IMPROVED GROUND

To determine the improved strength of the ground, equivalent parameters (c_{eq}, ϕ_{eq}) are assumed for the composite ground. The equivalent parameters are estimated based on the area average of each parameter for the stone columns and surrounding soil (Han, 2015).

Peak angles of shearing resistance of stone columns typically vary between 35° and 45°. The stone column design angle of shearing resistance is selected considering the materials and installation methods to be employed for the project and the relative shear stiffness of the columns and surrounding soil. In this case, a bottom feed casing based method will be used to install clean well graded angular gravel that filters the surrounding ground. Therefore, the columns will have a relatively high peak friction angle. As the strength in the surrounding soils will mobilise at relatively high strains compared to the stone columns (see Figure A.3.7), an assumed constant volume angle of shearing resistance of 35° is selected for the stone columns.

Figure A.3.7: Typical stress strain curves for stone columns and surrounding soil (Han, 2015)



In this example, the in situ peak angle of shearing resistance is considered reasonable to assume for the surrounding soil. However, we note that there may be some improvement in strength from densification with the installation of the columns. The composite strength of the upper two layers is therefore calculated as:

$$c_{eq} = c_s\,(1 - \alpha_s) + c_c\,\alpha_s$$

$$\varphi_{eq} = \arctan\,(\alpha_s \tan\varphi_c + (1 - \alpha_s)\tan\varphi_s)$$

c_s = soil cohesion

c_c = column cohesion (typically 0)

ϕ_s = soil friction angle

ϕ_c = column friction angle

2 – 2.1 m depth

$$c_{eq} = 0 \text{ kPa}$$

$$\varphi_{eq} = \arctan\,(0.15\tan35+(1 - 0.15)\tan30)$$

$$\varphi_{eq} = 31^{o}$$

2.1 m to 9.2 m depth

$$c_{eq} = 0 \text{ kPa}$$

$$\varphi_{eq} = \arctan(0.15\tan35+(1\text{-}0.15)\tan32)$$

$$\varphi_{eq} = 33^{o}$$

Note: the composite unit weight can be calculated in a similar way

STEP 7: CALCULATE FOOTING DIMENSIONS FOR NON-SEISMIC LOAD COMBINATIONS

Details of these calculations are not included in this example. Bearing capacity and settlement should be assessed for the critical gravity ULS and SLS load combinations and footing sizes designed accordingly.

> **Note**
> The ULS loads are typically higher than the loads used to assess post earthquake bearing capacity. Strength reduction factors between 0.4 and 0.65 should be applied to ultimate capacities to calculate design bearing strengths (see Module 4, Section 5.3).

An infinite extent of ground improvement can be considered initially, and the design bearing calculated using the composite strength from Step 5. Additional checks may be required once the extent of ground improvement is determined to check punching for example. Where stone columns are installed through a weak crust and only a few columns support heavy loads, shear failure of the columns themselves may limit bearing capacity. Han (2015) Section 5.3.3 present a method for assessing the capacity of individual columns.

Average settlement can be evaluated using the stress reduction, improvement factor or elastic-plastic methods presented in Han (2015) Section 5.3.4 or using a suitable composite Young's Modulus for the improved ground.

> **Note**
> Some methods, originally developed for stone column improvement of soft clay soils only consider volume deformation.

Settlement from shear deformation should be added to estimate of settlement from volume deformation. Settlement may be calculated assuming ground improvement over the full zone of influence of the foundations initially. But this assumption should be confirmed once the extent of ground improvement and the size of the foundations is determined in subsequent steps and updated if required.

Differential settlement will be affected by the stiffness of the structure (walls and foundations), the distribution of loads on the foundation and variability of the ground stiffness. Differential settlement can be evaluated considering these factors using a beam on winkler spring analysis. In this analysis the coefficient of vertical subgrade reaction can be calculated as the average applied foundation pressure divided by the calculated settlement. In reality spring stiffness will not be uniform along footing and sensitivity to spring stiffness should be assessed.

STEP 8: DESIGN FOOTINGS FOR LATERAL STABILITY FOR THE CYCLIC PHASE OF THE BUILDING RESPONSE

The footings should be designed for lateral stability and bearing capacity for the G, Eu, Q load combination. The procedure and methods for the design and analysis of shallow foundations for lateral seismic loading are described in Module 4, 5.3.2. The details of these calculations are beyond the scope of this example.

STEP 9: ASSESS THE MINIMUM DEPTH OF GROUND IMPROVEMENT USING QUALITATIVE METHODS

The minimum depth of ground improvement beneath the building is estimated using qualitative metrics initially. These methods have a low level of reliability and the design is confirmed using quantitative methods in subsequent steps. For this example, we will consider LSN and the liquefaction induced ground damage chart by Ishihara, 1985 to prevent ejecta.

Relationships between LSN and ground and foundation damage for residential structures have been developed using a large dataset following the Canterbury earthquake sequence (van Ballegooy et al, 2012), see Figure A.3.8 . The relationship between LSN and foundation performance is weak and may not be applicable to commercial or heavily loaded structures. The design criteria used to determine the initial depth of ground improvement for this example case are:

› LSN < 5 for SLS
› LSN < 20 for ILS
› LSN < 40 for ULS

A minimum depth of improvement of 5 m below existing ground level is required to meet the above criteria and is dictated by the intermediate limit state, see Figure A.3.9. With 5 m of ground improvement, the LSN at each limit state is:

› LSN = 3 for SLS
› LSN = 20 for ILS
› LSN = 27 for ULS

The liquefaction induced ground damage chart by Ishihara (1985) confirms that surface ejecta is unlikely in an SLS or ILS event with a 5 m thick improved ground crust but is possible in a ULS event that has a PGA of 0.35 g.

Figure A.3.8: Relationship between LSN and ground and foundation damage

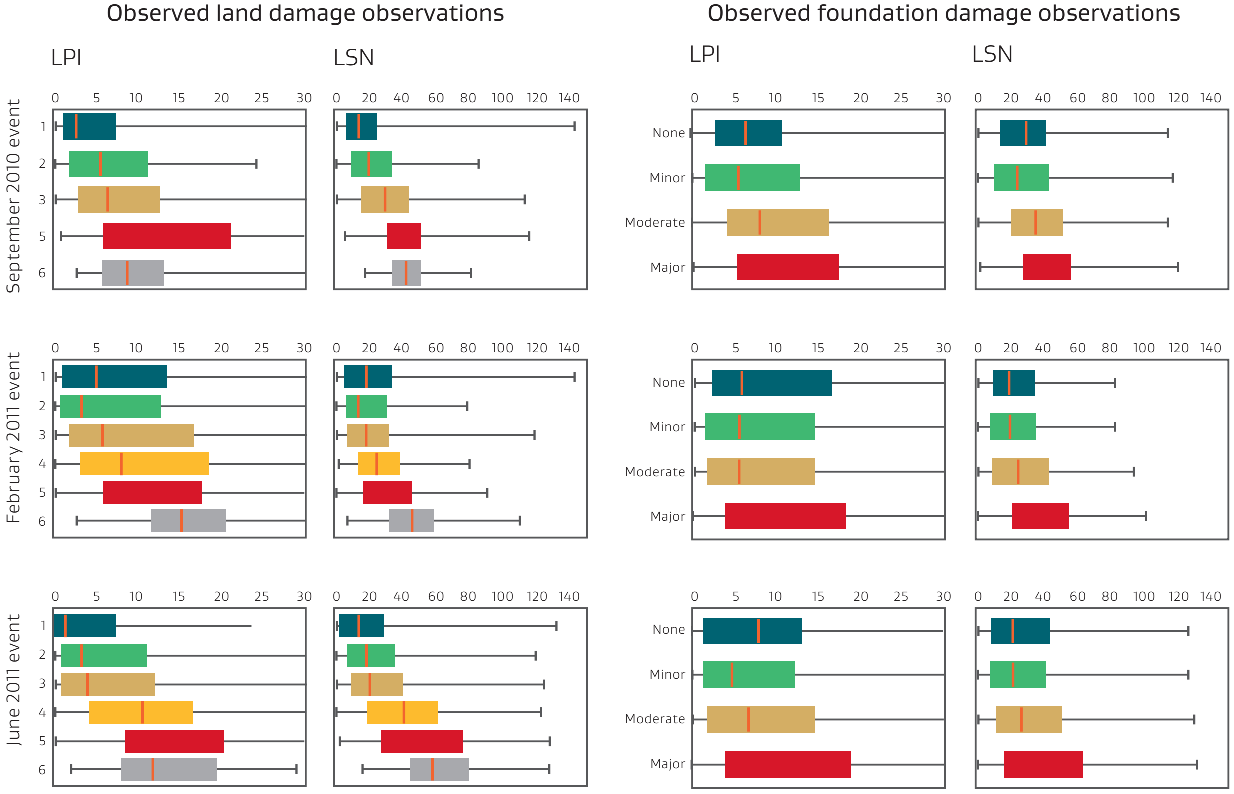
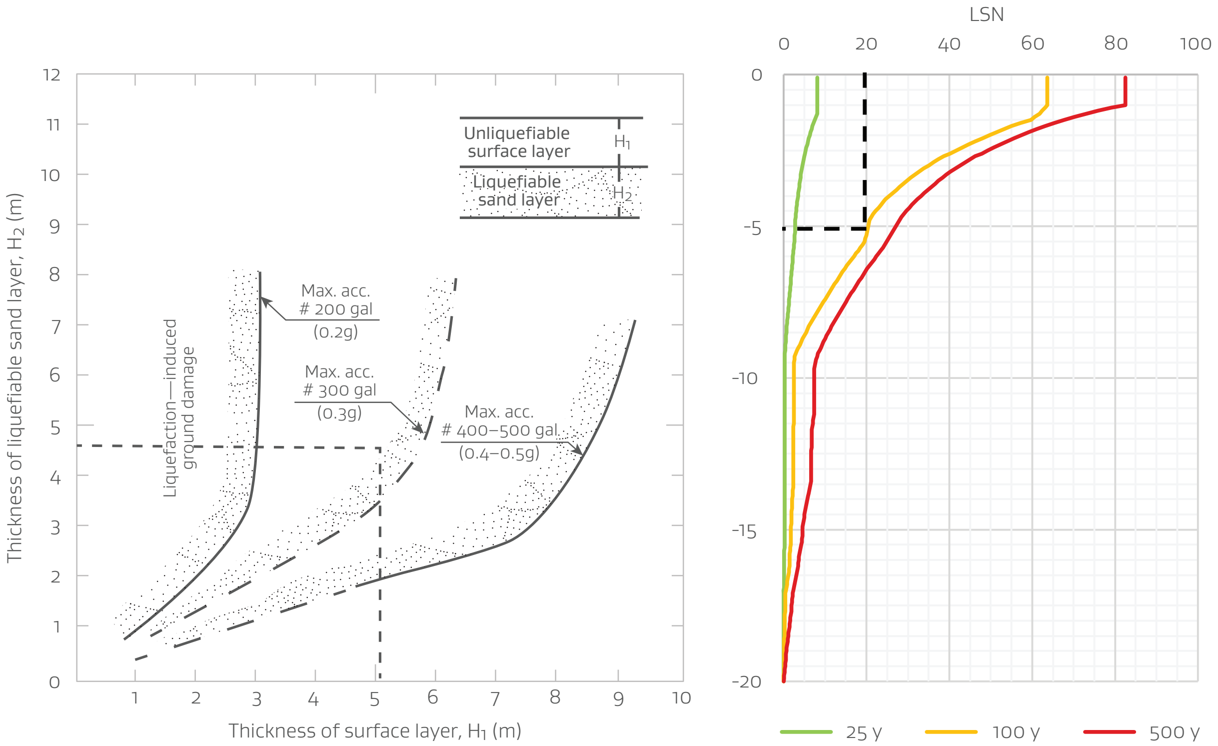


Figure A.3.9: Liquefaction induced ground damage chart based on thickness of liquefied and non-liquefied layers (Ishihara, 1985) and LSN for the intermediate limit state



STEP 10: CALCULATE POST-EARTHQUAKE BEARING CAPACITY OF THE FOOTING FOLLOWING A ULS EARTHQUAKE

Failure modes

Three potential failure mechanisms as shown in Figure A.3.10 are considered for this example:

1 General shear failure within the improved zone (Figure A.3.10a)

2 Punching of the footing through the improved zone into the underlying liquefied soil (Figure A.3.10b), and

3 Shear failure within the liquefied ground from the distributed footing load through the improved ground (Figure A.3.10c)

When the breadth of improvement is narrow, the potential for punching failure of the improved ground into any underlying liquefied soil should also be checked. But this is not the case for this example where the improvement will be at least the width required to distribute the load to the liquefied soil beneath the improved ground.

General shear failure

For vertical loads applied centrally to the footing, the ultimate bearing capacity of the improved ground is:

$$q_u = q'\lambda_{qd} N_q + \frac{1}{2}\gamma B'\lambda_{\gamma d} N_\gamma$$

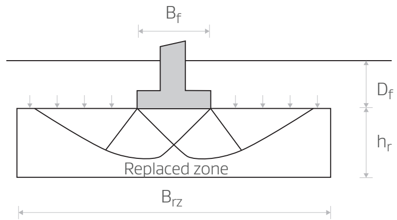
The bearing capacity factors N_q and N_γ and the depth factors λ_{qd} and $\lambda_{\gamma d}$ are calculated using the method in B1/VM4 for the improved ground composite friction angle of 31°, $B_f = 1.5$ m and $D_f = 0.7$ m as $N_q = 20.6$, $N_\gamma = 23.6$, $\lambda_{qd} = 1.1$ and $\lambda_{\gamma d} = 1.0$.

Ground improvement to factors of safety against liquefaction less than 1.5 to 2.0 will not entirely prevent the development of excess porewater pressure within the improved zone. Furthermore, as the full depth of liquefiable soils is not improved, there will be migration of excess porewater pressure from the lower liquefied layers into the improved zone. These effects will temporarily reduce the strength of the improved ground beneath the building and should be considered in the foundation design (see Module 4, Section 4.10). The soils will regain strength as porewater pressures dissipate in the hours to weeks after the earthquake.

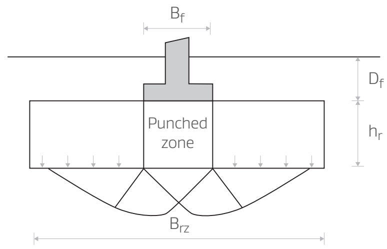
For this example, the overburden stress at the base of the footing and the effective unit weight of the composite improved block are calculated considering the potential for the development of some excess porewater pressure within the improved ground and the overlying crust.

Figure A.3.10: Possible failure mechanisms (Han, 2015)

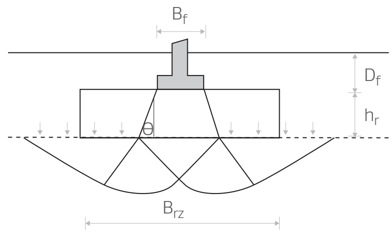
a General failure within replaced zone



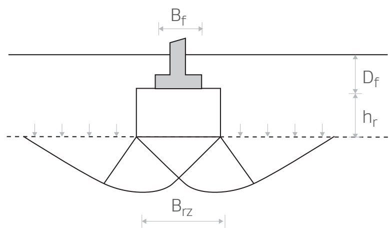
b Punching failure through replaced zone



c Failure of distributed foundation



d Punching failure of replaced zone



For the ULS, the factor of safety against liquefaction in the improved ground is 1.2. From Figure A.3.11, the excess pore pressure ratio in the improved zone is estimated to be 0.3. The same excess pore pressure ratio is assumed for the soil above the water table.

The effective overburden pressure at the base of the footing is:

$$q' = D_f\gamma(1 - r_u)$$

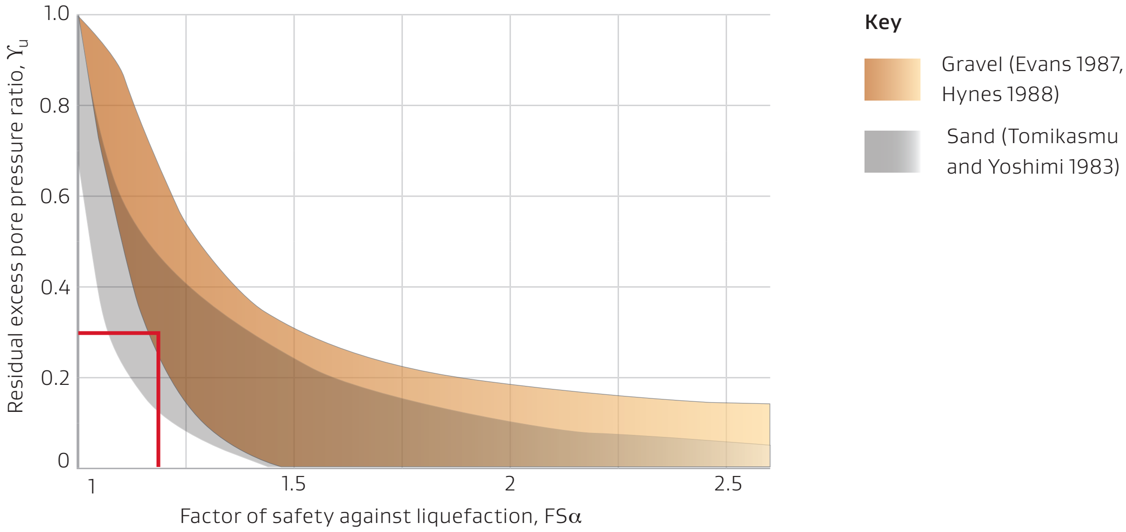
$$q' = 0.7\text{m} \times 18\ \frac{\text{kN}}{\text{m3}}\ (1 - 0.3) = 8.8\ \text{kPa}$$

The effective unit weight of the improved ground is calculated as:

$$\gamma' = (\gamma - \gamma_w)(1 - r_u)$$

$$\gamma' = \left(18\ \frac{\text{kN}}{\text{m3}} - 9.8\ \frac{\text{kN}}{\text{m3}}\right)(1\text{-}0.3) = 5.7\ \frac{\text{kN}}{\text{m3}}$$

Figure A.3.11: Excess pore pressure ratio vs factor of safety against liquefaction triggering (Module 4)



The ultimate bearing capacity is:

$q_u = 8.8 \text{ kPa} \times 1.1 \times 20.6 + 0.5 \times 5.7 \frac{\text{kN}}{\text{m3}} \times 1.5\text{m} \times 1 \times 23.6 = 300 \text{ kPa}$

To limit subsidence from shear deformation within the improved soil and ensure a reasonable level of foundation performance reliability a strength reduction factor of 0.5 is adopted for the general shear case. Using a strength reduction factor of 0.5, the design bearing strength, qdbs is:

0.5 x 300 kPa = 150 kPa

The design bearing strength is greater than the design bearing pressure (q_d = 80 kPa) so OK.

In this case, the footings are supported on a group of columns and the natural soil surrounding the columns is not overly soft so the assumption that columns and improved soil work as a composite is reasonable. In instances where there are a few stiff columns within a soft crust supporting the footing, the columns will take much of the vertical load and the ultimate capacity of the improved ground may be governed by the capacity of the columns themselves.

Punching of the footing through the improved zone

The method by Meyerhof and Hanna (1978) is used to calculate the ultimate bearing capacity of the improved zone overlying liquefied soil.

$$q_{ult} = q_b + \frac{U_p P_h \tan\varphi + U_p\, h_r\, c_1 - W_{pz}}{A_f}$$

Where:

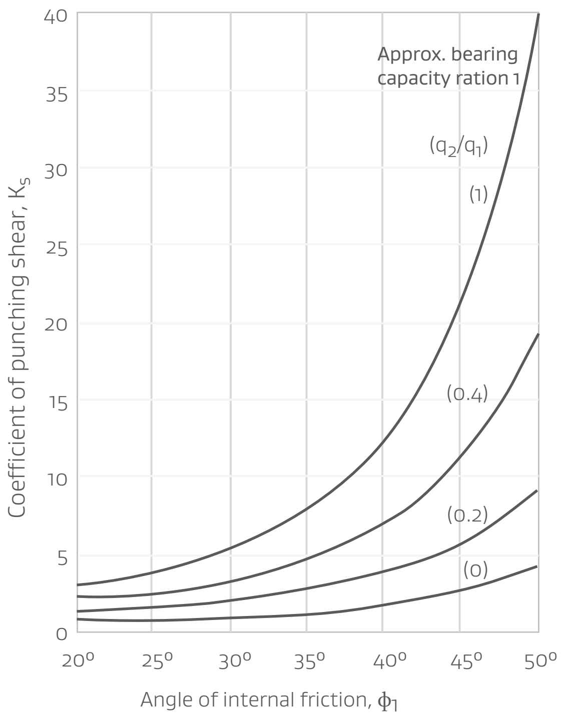
- q_b = ultimate bearing capacity of the liquefied soil beneath the improved zone calculated using the dimensions of the footing and the residual undrained strength of the liquefied soil (see Site B liquefaction assessment)
- U_p and h_r = perimeter length and height of the punched zone
- W_{pz} = weight of the punched zone
- A_f = area of footing
- c_1 and $\varnothing_1$ = composite cohesion and friction of the improved ground
- P_h = lateral earth pressure acting on the perimeter surface of the punched zone

$$P_h = K_s\,(\gamma'_1\, D_f\, h_r + 0.5_{\gamma'1}\, h_r^2)$$

Where:

- γ'_1 = unit weight of the composite improved ground
- K_s = the coefficient of punching shear and can be estimated from Figure A.3.12 assuming it is constant within the improved zone.

Figure A.3.12: Coefficient of punching shear, K_s



q_1, q_2 = ultimate bearing capacity of a strip footing of width B_f on the surface of the improved zone and on the surface of the liquefied soil respectively.

$$q_1 = c_1 N_{c1} + 0.5\gamma'_1 B_f N_{\gamma 1}$$

$$q_2 = c_2 N_{c2} + 0.5\gamma'_2 B_f N_{\gamma 2}$$

For the footings in this example:

$$q_{ult} = q_b + \frac{U_p P_h \tan\varphi + U_p h_r c_1 - W_{pz}}{A_f}$$

Calculate q_b

$$q_b = c\lambda_{cd} N_c + q'\lambda_{qd} N_q$$

c = 3.5kPa, N_c = 5.1, q' = 50.8 kPa, N_q and λ_{qd} = 1.0 ($\varnothing'$ = 0, liquefied soil), and

$$\lambda_{cd} = 1 + 0.4\text{atan}\frac{D_f + h_r}{B}$$

= 1 + 0.4 x atan[(0.7m + 4.3m) / 1.5m] = 3.4

q_b = 3.5kPa x 3.4 x 5.1 + 53.3 kPa x 1 x 1 = 114 kPa

$$P_h = K_s (\gamma'_1 D_f h_r + 0.5\gamma'_1 h_r^2)$$

Calculate K_s

Determine q_1 and q_2

$$q_1 = c_1 N_{c1} + 0.5\gamma'_1 B_f N_{\gamma 1}$$

$$q_2 = c_2 N_{c2} + 0.5\gamma'_2 B_f N_{\gamma 2}$$

$N_{\gamma 1}$ = 23.6, γ'_1 = 18.5 kN/m³ (assuming the weight of the columns is 22 kN/m³), c_1 = 0 kPa

$N\gamma 2$ = 0 ($\varnothing$=0), N_{c2} = 5.1, c_2 = 3.5 kPa for the liquefied ground (calculated as S_r/σ'_v x σ'_v at 5 m depth, with s_r/σ'_v = 0.07 and σ'_v = 5m x 18.5kN/m³ – 4m x 9.8kN/m³ =53.3 kPa)

q_1 = 0 + 0.5(18.5kN/m³ – 9.8kN/m³).1.5m.23.6 = 145kPa

q_2 = 3.5kPa.5.1 + 0 = 18kPa

$$\frac{q_2}{q_1} = \frac{18\text{kPa}}{145\text{kPa}} = 0.12$$

$\varnothing$= 32°, Ks = 1.5 from Figure A.3.12.

P_h = 1.5[(18.5 – 9.8)kN/m³.0.7m.4.3m + 0.5.(18.5 – 9.8)kN/m³.(4.3m)²]=160kN/m

W_{pz} = 1.5m.4.3m.1m.18.5kN/m³ – 1.5m.4m.1m.9.8kN/m³ = 60.5kN

$$q_{ult} = q_b + \frac{U_p P_h \tan\varphi 1 + U_p h_r c_1 - W_{pz}}{A_f}$$

$$q_{ult} = 114\text{kPa} + \frac{2\text{m}.160\text{kN/m}.\tan 32° + 0 - 60.5\text{kN}}{1.5\text{m}.\ 1\text{m}}$$

= 207kPa

Bray and Macedo (2017) suggest seismic performance will be poor if foundations have a post earthquake factor of safety less than 1.5 for heavy structures or 1.0 for light weight structures when considering shallow foundations on a non-liquefied crust overlying liquefied soil. Assuming post-earthquake loads on footings typically have a factor of 1.0, the strength reduction factor to get an equivalent factor of safety is 0.65 for heavy structures and 1.0 for light structures.

The footings in this example are moderately loaded and a strength reduction factor of 0.75 is adopted, the design bearing strength, q_{dbs} is:

0.75 * 207 kPa = 155 kPa

The design bearing strength is greater than the design bearing pressure (q_d = 80 kPa) so OK.

Shear failure within the liquefied ground from the distributed footing load

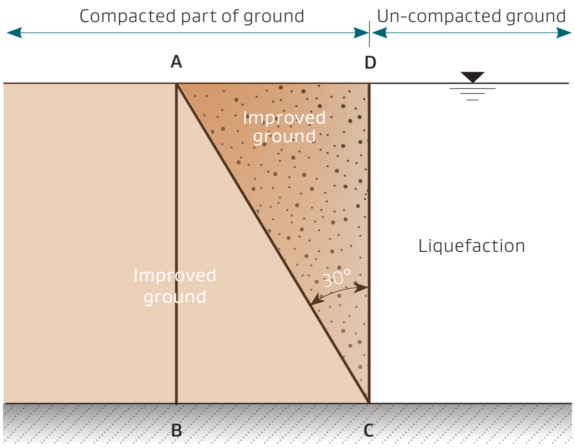
Foundation performance is affected by the confinement and strength of the soil adjacent to the foundations. With densification techniques like stone columns to improve liquefaction resistance, it is common practice to improve the ground to a distance of half the depth of the improved zone from the edge of the structure. This also protects against the softening effects of lateral porewater pressure migration into the improved zone during an earthquake event. Module 5 recommends ground improvement extend a horizontal distance beyond the foundations of at least AD, see Figure A.3.13. Many buildings are sited near the property boundary, hence there is limited space for ground improvement beyond the building footprint (Module 4, Section 4.6). In these cases, consideration can be given to

Cantilevering support for a foundation edge beam over the zone of ineffective ground improvement,

Concrete columns in zone ABCD that are tied together at surface with a reinforced concrete slab or stiffly reinforced granular raft,

A cantilever sheet, secant diaphragm wall at the boundary to laterally constrain the stone column improved ground

Figure A.3.13: Lateral extents of compaction ground improvement



For this example, the improvement needs to extend beyond the edge of the footings will be

$AD = 4.3m.\ \tan(30°) = 2.5m$

3.5 m wide vehicle access is required around the entire perimeter of the building so there is sufficient space to extend the stone columns beyond the building footprint.

The width over which the footing load is distributed at the base of the ground improvement is therefore:

$B_d = 1.5m + 2 \times 2.5m = 6.5m$

The ultimate bearing capacity at the base of the improved ground from the distributed footing load is:

$q_b = c_2\, \lambda_{cd}\, N_c + q'_2\, \lambda_{qd}\, N_q$

c_2 is the residual undrained strength of the liquefied soil = 3.5 kPa,

The effective overburden pressure at the base of the ground improvement is

$q'_2 = 5m.\ 18.5kN/m^3 - 4m.9.8kN/m^3 = 53.3kPa$

The bearing capacity factors N_c and N_q equal 5.1 and 1 respectively and $\lambda_{qd} = 1$ as $\varnothing = 0$.

Depth factor λ_{cd} is calculated using the B1/VM4 with the depth and width of the footing as $D_f = 5$ m and Bd = 6.5 m as:

$$1 + 0.4.\text{atan}\left(\frac{5}{6.5}\right) = 1.3$$

The ultimate bearing capacity of the liquefied ground at the base of the stone column improvement is therefore:

$q_b = 3.5kPa.1.3.5.1 + 53.3kPa = 76.5\ kPa$

The ultimate bearing capacity at the base of the footing is

$$q_{ult} = \frac{B_D}{B_f}(qb - \gamma'_1\, h_r) = \frac{6.5m}{1.5m}(76.5kPa - (18.5 - 9.8)kN/m^3.4.3m) = 169kPa$$

Adopting a strength reduction factor of 0.75, the design bearing strength, q_{dbs} is:

$0.75 \times 169\ kPa = 127\ kPa$

The design bearing strength is greater than the design bearing pressure (q_d = 80 kPa) so OK.

STEP 11: CALCULATE SEISMIC SUBSIDENCE

Mechanisms for seismic subsidence of buildings are described in Module 4, Section 4.9. Subsidence can result from shear and volume deformation in the improved ground and the unimproved ground and the internal erosion of foundation soils. Example calculations are provided for the intermediate limit state only here. A similar approach can be adopted to asses subsidence for other limit states.

The simplified method by Bray and Macedo (2017) is used to assess the subsidence form shear deformation and subsidence from volume deformation is estimated using the method by Zhang et al (2002) that are calculated in the site liquefaction evaluation, see Figure A.B.3. Surface ejecta is not expected at ILS and the columns will be designed to filter the surrounding natural ground, hence no significant subsidence from internal erosion is anticipated.

In this example, the site is characterised using one CPT for convenience. For a real project the full range of site investigations should be used to evaluate differential subsidence and foundation performance.

Subsidence from shear deformation

In the method by Bray and Macedo (2017), shear induced liquefaction building subsidence is estimated using the equation:

$$L_n(D_s) = c1 + 4.59 \times LN(Q) - 0.42 \times LN(Q)^2 + c2 \times LBS + .58 \times Ln\left(Tanh\left(\frac{HL}{6}\right)\right) - 0.02 \times B + 0.84 \times LN(CAV\delta p) + 0.41 \times Ln(Sa1) + \varepsilon$$

Where:

- Ds = the shear induced foundation subsidence (mm)
- c1 = -8.35 for LBS ≤ 16, or -7.48 otherwise
- c2 = 0.072 for LBS ≤ 16, or 0.014 otherwise
- Q = the foundation contact pressure (kPa)
- LBS = liquefaction induced building settlement index
- HL = liquefiable layer thickness (m)
- B = the foundation width (m)
- CAVδp = standardised cumulative absolute velocity (g-s)
- Sa1 = the spectral acceleration at a period of 1 s.
- ε = is a normal random variable with zero mean and 0.5 standard deviation in Ln units

CAVδp is estimated using Campbell and Bozorgniaina (2011) with estimated earthquake parameter values selected considering the type and activity of faults that are likely to be the greatest contributors to the sites seismic hazard. Excel files with the implementation of this GMPE can be downloaded from: http://peer.berkeley.edu/ngawest/nga_models.html CAV is calculated as:

Mean CAVδp = 0.72 g-s for ILS and 1.0 g-s for ULS

The spectral acceleration at 1s has been calculated from NZS 1170.5 for Christchurch as:

$$Sa(1s) = C_h\ (1s).Z.R.N(T,D)$$

Where Z = 0.22 (Christchurch), Ch(1s) = 1.93 (Tbl 3.1, NZS 1170.5, Class D), R = 0.5 (100 y return period for the ILS), N(T,D) = 1.0 (no faults within 20 km of the site).

$$Sa(1s) = 1.93 \times 0.22 \times 0.5 \times 1 = 0.21g$$

$$LBS = \int W \times \frac{\varepsilon_{shear}}{z}\, dz$$

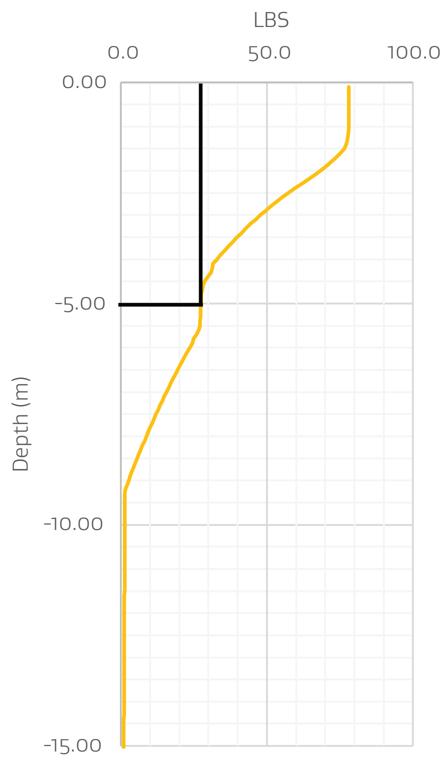
Where:

- z = depth measured from the ground surface (m)
- W = foundation weighting factor = 0.0 for z less than Df (the embedment depth of the footing)

ε_{shear} = liquefaction induced shear strain estimated using Zhang et al (2002) CPT-based procedure (%)

LBS for the intermediate limit state is plotted against depth in Figure A.3.14. Ground improvement will prevent liquefaction, therefore the LBS with ground improvement to a depth of 5 m is 28.

Figure A.3.14: LBS for the intermediate limit state



In the Canterbury earthquakes, similar buildings on liquefiable sites generally experienced:

› Near uniform subsidence around the perimeter of the building with differential movement across the floor. The perimeter was lower relative to the centre of the floor. This could have been caused by a combination of greater soil shear deformation at the perimeter and high upward seepage pressures under the floor.
› Tilt across the structure with either a uniform or non-uniform slope.

Further details are in the MBIE guide 'Assessment, repair and rebuild of earthquake-affected industrial buildings in Canterbury' (2014).

As the majority of load is concentrated at the strip footings along the sides of the building and considering the observed performance of similar structures on liquefied sites in the Canterbury earthquakes, shear induced subsidence is calculated for both the individual strip footings and the total building footprint. These two estimates are then used to assess differential subsidence.

Average overall building footprint subsidence

Subsidence from shear deformation for the overall structure with a width of 30 m and an average bearing pressure of 23.3 kPa for the intermediate limit state is:

$$\mathrm{Ln}(D_s) = -7.48 + 4.59 \times \mathrm{LN}(23.3) - 0.42 \times \mathrm{LN}(23.3)^2 + 0.014 \times 28 + .58 \times \mathrm{Ln}\left(\mathrm{Tanh}\left(\frac{4.5}{6}\right)\right) - 0.02 \times 30 + 0.84 \times \mathrm{LN}(0.71) + 0.41 \times \mathrm{Ln}(Sa10.21) + \varepsilon$$

$$D_s = 4 \text{ mm} + \varepsilon$$

Subsidence from 1D reconsolidation of the liquefiable soil below the ground improvement (below 5 m depth) is 165 mm for the intermediate limit state.

The average overall subsidence of the structure is therefore 170 mm.

Subsidence of the perimeter footings

Subsidence from shear deformation for the 1.5 m wide perimeter footings with a bearing pressure of 80 kPa is:

$$\mathrm{Ln}(D_s) = -7.48 + 4.59 \times \mathrm{LN}(80) - 0.42 \times \mathrm{LN}(80)^2 + 0.014 \times 28 + .58 \times \mathrm{Ln}\left(\mathrm{Tanh}\left(\frac{4.5}{6}\right)\right) - 0.02 \times 1.5 + 0.84 \times \mathrm{LN}(0.71) + 0.41 \times \mathrm{Ln}(Sa10.21) + \varepsilon$$

$$D_s = 43 \text{ mm} + \varepsilon$$

ε = -17 mm and +27 mm for the 16th and 84th percentiles, hence the 16th and 84th percentile subsidence is 26 mm and 70 mm.

Subsidence from 1D reconsolidation of the liquefiable soil is 165 mm for the ILS and total subsidence is 210 mm.

Differential subsidence

Differential subsidence after an earthquake is the sum of settlement that occurred prior to the earthquake (Step 7) and the seismic settlement. As with the gravity load settlement, seismic subsidence will be affected by the structure stiffness and mass distribution. Variations in the stiffness of the improved ground and variation in the extent and liquefaction potential of the layers below the improved ground will also affect seismic subsidence and differential subsidence.

Assuming the ground conditions and liquefaction potential of the unimproved ground is reasonably consistent across the site and the gravity load differential settlement between the centre of the floor and the footings is 10 mm, using the average calculated footing subsidence, the average slope of the floor after an ILS earthquake is in the order of:

$$1 \text{ in } \frac{15000 \text{ mm}}{40 \text{ mm} + 10 \text{ mm}} = 1 \text{ in } 300$$

This is less than the ILS floor gradient design criteria (1 in 200) so OK. Considering ground and liquefaction potential variability and uncertainty in settlement estimates, differential subsidence across the floor could vary between 1 in 150 and 1 in 1000.

Differential subsidence along the footings after an earthquake can be calculated using the same winkler spring analysis described in step 7 by applying vertical displacements to the free-field ends of the vertical soil springs. Even on relatively uniform sites, there will be some variability of subsidence and the sensitivity of foundation performance should be assessed using a range of ground subsidence profiles.

For the ILS in this example, the improved ground has a factor of safety against liquefaction triggering in excess of 2 so there is unlikely to be any significant reduction in stiffness of the improved ground and no change to the spring stiffness. The soil springs should have no tension capacity.

STEP 12: DESIGN THE GRADING OF THE AGGREGATE FOR THE COLUMN

To avoid the migration of fines into the columns with the dissipation of excess porewater pressure during or after an earthquake, the grading of the aggregate should be designed to filter the surrounding soil. The stone should be well graded, angular, sound stone. Although drainage is not depended on, the fines content of the aggregate should be less than 8 percent (Federal Highway Administration, 2017). Module 5A gives further guidance on suitable materials for stone column construction.

$$20D_{S15} < D_{G15} < 9D_{S85}$$

D_{S15} = the diameter of soil passing 15%

D_{G15} = the diameter of gravel passing 15%

D_{S85} = the diameter of soil passing 85%

Based on the grading results for Site B

$D_{S15} = 0.15\text{mm}$

$D_{S85} = 1.5\text{mm}$

Therefore

$20 * 0.15 < D_{G15} < 9 * 1.5$

$D_{G15} = 3 - 13.5\text{mm}$

A second method for determining a suitable grading of backfill material for stone columns was developed by Brown based on settling rate of backfill in water (Han, 2015; Brown, 1977).

$$S_N = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}}$$

Table A.3.3. Suitability of backfill (source: Brown 1977)

Suitability number	0–10	10–20	20–30	30–40	>50
Rating	Excellent	Good	Fair	Poor	Unsuitable

STEP 13: DESIGN THE DRAINAGE BLANKET

The stone columns will have a higher permeability than the surrounding soil and may act as vertical drains that drain water from liquefied layers below the improved ground or any excess porewater pressure that develops in the soils between the columns. A granular drainage blanket is therefore required beneath, and extending beyond, the building footprint to direct water away from the underside of the floor and mitigate the potential for heaving and cracking of the floor.

The drainage layer should comprise well graded gravel with less than 10 percent sand and no fines and with a permeability not less than 10^{-4} m/s. Drainage layers are typically 300 – 500mm thick with a geotextile separation layer between the top of the stone columns and the base of the drainage layer to prevent fines migration (Han, 2015). The damp proof membrane can be laid directly above the drainage layer

As well as performing a drainage function, this aggregate layer helps to distribute the building loads between the stone columns. Where stone columns are being designed as stiff elements to carry building loads (as opposed to solely for liquefaction remediation), the drainage blanket should be designed as a load transfer platform following the method set out in BS 8006 (2011).

Construction

FIELD TRIAL

- A field trial should be carried out to verify that the design level of improvement can be achieved with the calculated spacing and column size. Typically three different arrangements (spacings) are tested to refine the column spacing need to gain the minimum density (penetration) requirements.
- More than one trial may be required if ground conditions vary across the site or if different construction techniques are to be considered, eg casing based method vs dry bottom feed vibro-replacement.
- It may take several days to two weeks for excess porewater pressures caused from column construction to dissipate and the full benefit of the stone columns to be realised. This needs to be considered in the construction programme.

CONSTRUCTION QUALITY ASSURANCE AND QUALITY CONTROL

Advice on quality assurance and quality control for stone column construction is included in Module 5a. Many stone column rigs have automated onboard monitoring equipment to measure the rate of construction, compactive effort, verticality and the volume of stone installed per meter. Information gained from the trial should be used to set construction quality control criteria for the production columns and select areas for quality assurance testing.

Quality assurance should include as a minimum:

- Laboratory testing of the aggregates used for the stone columns and the drainage blanket to confirm source and production properties are within specified limits.
- Penetration testing between columns and through columns to ensure compaction meets minimum requirements through the full treatment depth.
- Checks on the column spacings, levels of finished columns.
- Placement of the filter fabric.
- Measurement of finished dimension, levels and the degree of compaction of the drainage blanket.

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Example 4: Dynamic compaction

Scenario

This example considers the design of dynamic compaction for the same building and site as Example 3, on Site B. In Example 3 we establish that the natural crust is not suitable to support the proposed structure.

Parts of the design of dynamic compaction and the building foundations are similar to the design of stone columns in Example 3. These are not repeated in this example and Example 3 should be read in conjunction with this example. The first steps of this process — evaluating liquefaction, lateral spreading and site stability are presented in the Site A evaluation.

STEP 1: SET PERFORMANCE REQUIREMENTS AND DESIGN CRITERIA

The performance requirements and design criteria in Example 3, Step 1 are also adopted for this example.

STEP 2: DETERMINE IF THE GROUND CONDITIONS AT THE SITE ARE SUITABLE FOR DYNAMIC COMPACTION

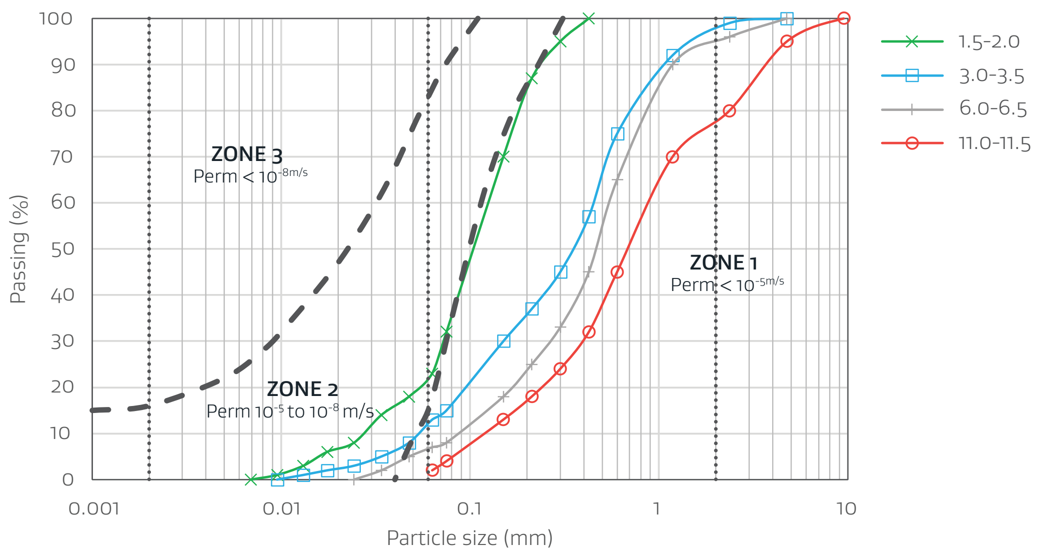
Dynamic compaction works best on, free-draining soils. In saturated or nearly saturated sandy soils, excess pore water pressures develop on impact, which then dissipate and allow the soil grains to rearrange into a denser state of packing. The permeability of the soil therefore affects the effectiveness of dynamic compaction.

Deep dynamic compaction is not typically suitable for soils with an $I_c > 2.0$, or a fines content greater than about 10 percent. PSDs from the site soils are compared to grading envelopes in the Federal Highway Administration's document, Ground Modification Reference Manual Volume 1 (FWHA, 2017) in Figure A.4.1.

- Zone 1 includes coarse grained materials with high permeability that are best suited for dynamic compaction.
- Zone 2 represents intermediate deposits where improvement can be achieved provided the energy is applied in multiple passes to allow pore-pressure dissipation between passes.

Based on the PSDs of soils at our site and CPT I_c, the soils at Site B can likely be improved with deep dynamic compaction but multiple passes may be required. The silty sands in the upper 2.1 m may be problematic as they will hamper dissipation of excess porewater pressure and reduce the effectiveness of each drop.

Figure A.4.1: Figure showing the range of suitable particle size distributions for dynamic compaction (Federal Highway Administration, 2017)



Dynamic compaction creates high levels of noise and vibration and can be damaging to neighbouring structures. The environmental effects limit the feasibility of dynamic compaction to green field sites away from existing structures. Guidance on acceptable noise and vibration is given in Module 5 and 5A. Step 6 gives an example of vibration assessment that can be used to assess feasibility of dynamic compaction.

STEP 3: DETERMINE DEPTH AND EXTENT OF GROUND IMPROVEMENT

For this example, we will assume that there will be a slight increase in the strength of compacted soils and adopt a friction angle of 33 degrees for the improved sands. The procedure used to calculate the extents of improvement is then the same as Example 3, Steps 6 to 10.

The depth of improvement required below the existing ground level is 5 m.

STEP 4: SELECT TAMPER AND CALCULATE MINIMUM DROP HEIGHT

Tamper weights typically range from 5 to 35 tonnes and are dropped from heights of 10 to 40 m. The depth of treatment is related to the potential energy of the falling weight. For a given drop height and tamper weight, the depth of improvement can be estimated as:

$$D = n(WH)^{1/2}$$

Where:

› D = depth of improvement (m)
› n = empirical coefficient, selected from Table A.4.1
› W = mass of tamper in metric tonnes (t)
› H = drop height (m)

There are silty layers within the improvement depth and an of 0.4 is adopted. The tamper selected for primary compaction has a diameter of 1.8 m and a mass of 15 t, the required drop height for a treatment depth of 5 m is therefore calculated as:

$$H = \frac{\left(\frac{5\ \text{m}}{0.4}\right)^2}{15\ \text{t}} \qquad H = 10.4\ \text{m}$$

STEP 5: DESIGN THE GRID LAYOUT, NUMBER OF PASSES AND NUMBER OF DROPS

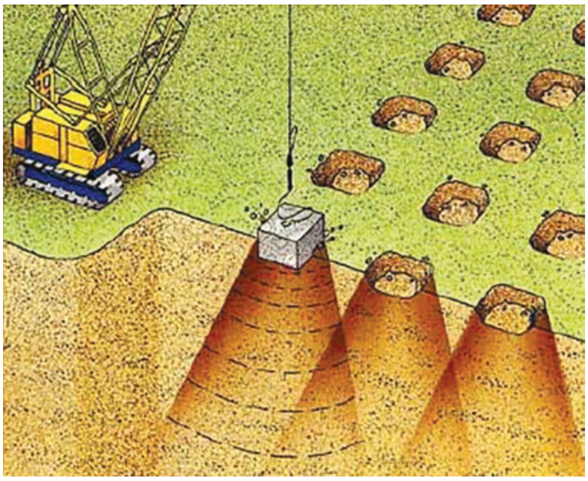
Multiple phases and passes are typically used for dynamic compaction, each targeting soils of a different depth or to mitigate excess pore-pressure effects. The first phases target deeper soils, while the last phase is classed as an 'ironing' phase or 'blanket' phase. This phase involves less energy (smaller weight or lower drop) and is employed to even out the ground surface and improve the surface layers of soil. The grid spacing of each phase and pass will typically be offset from the previous pass to allow for pore-pressure dissipation between passes and the craters from the prior pass may be infilled with gravel if necessary. The ironing phase may be carried out at a smaller grid spacing than previous passes.

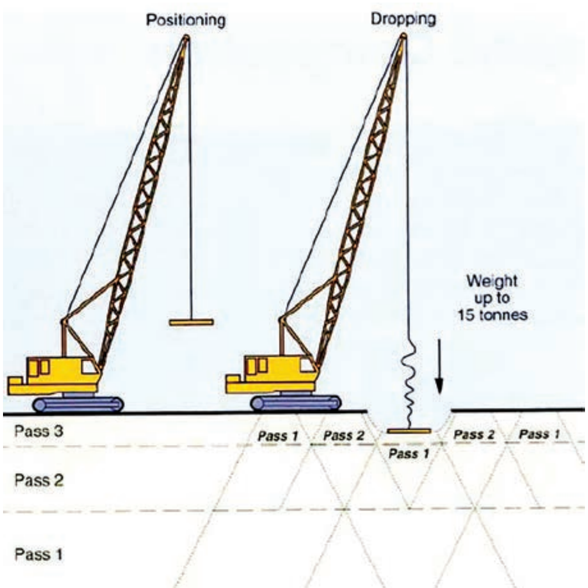
Table A.4.1: recommended n values for different soil types (FHWA, 1995)

SOIL TYPE	DEGREE OF SATURATION	RECOMMENDED *n* VALUE*
Previious soil deposits — granular soils	High	0.5
	Low	0.5 – 0.6
Semipervious soil deposits — primarily silts with plasticity index of <8	High	0.35 – 0.4
	Low	0.4 – 0.5
Impervious deposits — primarily clayey soils with plasticity index of >8	High	Not recommended
	Low	0.35 – 0.40 Soils should be at water content less than the plastic limit

* For an applied energy of 1 to 3 MJ/m^2 and for a tamper drop using a single cable with a free spool drum.

Figure A.4.2: Images showing the typical arrangement of each pass (Department of Trade and Industry, 2003)



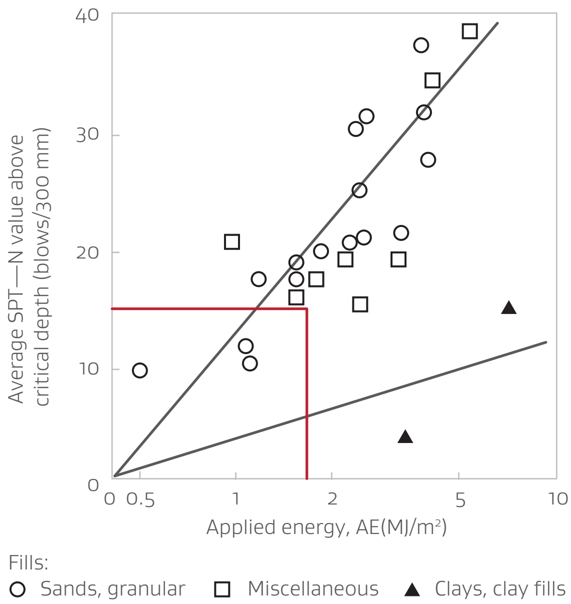


The number of passes, drops and drop spacing is calculated initially based on the estimated total energy needed to treat the soils. Grid spacings, drop heights and number of drops are then confirmed with a field trial prior to commencing production ground improvement. Initial estimates of the required energy for treatment can be taken from Figure A.4.3.

In example 3 we saw that an average SPT N_{60} of 15 (and a maximum of 20) is required to supress liquefaction over the 5 m treatment depth. From Figure A.4.3, the required energy per square metre to compact the sandy soils at this site is assumed to be 1800 kJ/m².

Note: this could range from 800 kJ/m² to 3000 kJ/m².

Figure A.4.3: Guidelines on applied energy requirements for dynamic compaction (Federal Highway Administration, 2017)



The grid spacing is typically selected as 1.5 to 2.5 times the diameter of the tamper. A square grid spacing of 2.6 m is selected for this example. The effective area per drop location is therefore:

Area per drop location = 2.6 m x 2.6 m = 6.76 m²

For a 15 t mass and a 11 m drop height, the applied energy per drop is:

15 t x 9.8 m/s² x 11 m = 1617 kJ

The number of drops required at each grid location is therefore:

$$\frac{1800 \text{ kJ/m}^2 \times 6.76\text{m}^2}{1617 \text{ kJ/drop}} = 8 \text{ drops}$$

For safety, crater depths should be limited to the height of the tamper plus 0.3 m. Crater depths can be estimated as:

$$d_{cd} = 0.028 N_d^{\,0.55} \sqrt{W_t H_d} \qquad (3.18)$$

For a more accurate estimate:

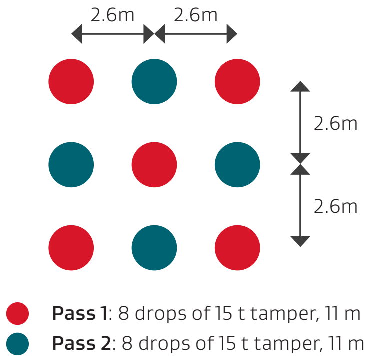
$$\log d_{cd} = -1.42 + 0.553 \log N_d + 0.213 \log H_d + 0.873 \log W_t - 0.435 \log\left(\frac{S_d}{d_t}\right) - 0.118 \log p \qquad (3.19)$$

Where:

H_d = drop height (m)
W_t = tamper weight (tons)
N_d = number of drops
S_d = drop spacing (m)
d_t = tamper width or diameter (m)
p = contact pressure in (t/m²)

(Extract from Han 2015, after Rollins and Kim, 2010.)

Figure A.4.4: Grid layout and specifications of each pass of compaction



For eight drops of a 15 t hammer from a height of 11 m, the estimated crater depth is therefore:

$$0.028 \times 8^{0.55}\sqrt{15t \times 11\ m} = 1.13\ m$$

As the tamper is 1.5 m high, crater depths at eight drops are unlikely to be a major safety concern.

As discussed previously, dynamic compaction is often sequenced with multiple phases and/or passes. To mitigate the effects of excess porewater pressure generation, primary compaction could be carried out in two passes. The first pass involving eight drops of the 15 t tamper from 11 m at every second grid point and the second pass involving eight drops from 11 m at the intermediate grid locations as shown in Figure A.4.4. An ironing phase would be carried out after primary compaction. The grid layout, sequence and drop height can be adjusted in the trial to achieve the required level of compaction and optimise efficiency.

STEP 6: VIBRATION ASSESSMENT

For sites where there are neighbouring structures or occupancy, assessment of vibration effects needs to be carried out to ensure vibration levels are within acceptable tolerances.

Vibration effects from dynamic compaction can be estimated using Figure A.4.5 from FHWA, 2017. As the nearest neighbouring building is at 50 m distance from the site, the scaled energy factor is calculated as:

$$\text{Scaled energy factor} = \frac{\sqrt{1617\ kJ / 9.8m / s^2}}{50\ m}$$

$$\text{Scaled energy factor} = 0.26$$

The site soils correspond to category 6 and the calculated particle velocity at a distance of 50 m is therefore 5 mm/s but could be as high as 10 mm/s.

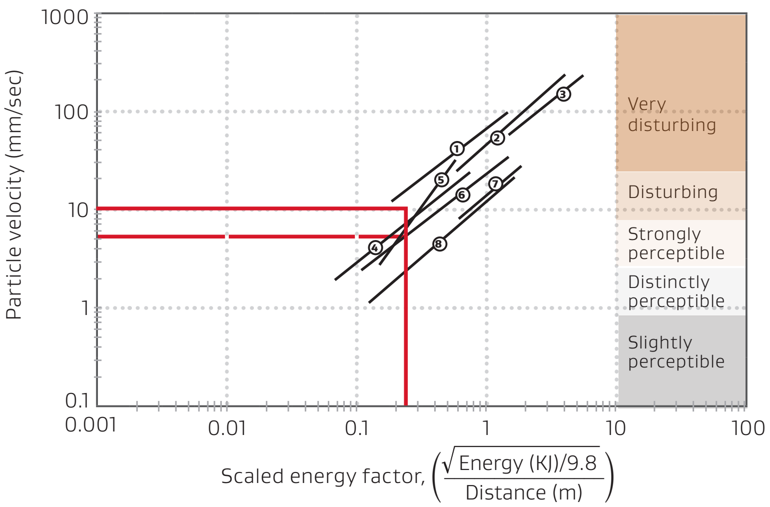
Threshold vibration levels for annoyance and cosmetic damage to buildings are generally about 1 mm/s and 10 mm/s respectively but depends on the nature of adjacent land use, building types and condition.

With an estimated particle velocity of 5 mm/s to 10 mm/s, the neighbouring building is unlikely to suffer significant damage from dynamic compaction, but the vibration may not be tolerable for building occupants. The level of nuisance will also be affected by the construction duration with longer periods of vibration being generally less tolerable.

STEP 7: CALCULATE THE TARGET PENETRATION RESISTANCE FOR GROUND IMPROVEMENT

This involves the same procedure as the one used in Example 3.

Figure A.4.5: Estimate of particle velocities as a result of dynamic compaction for different soil types (Federal Highway Administration, 2017)



Key

4	Very stiff silty clay
5	Medium dense micaceous sand
6	Clayey mine soil
7	Building rubble
8	Collapsible silt
9	Loose sand
10	Loose sand fill
11	Loose decomposed garbage

Construction

PRE-CONSTRUCTION

- Vibration monitors should be installed in the surrounding area or on any nearby buildings to determine baseline vibration levels to compare against construction vibrations.
- Pre-condition assessment of neighbouring buildings is normally required, so that any damage that does occur can be quantified. Refer to Module 5A and the ASCE Guideline for structural condition assessment of existing buildings (2000).
- A field trial should be carried out to verify the number passes, time between passes, drops, drop height, and grid spacing required to meet the design objectives, backfilling procedures and that vibration levels are acceptable. Based on the depth of crater, height of groundwater table, and softening effects witnessed during the field trial, a granular surface layer may be required as a working platform for the plant. Installation of piezometers below the water table should be considered to measure the rate of dissipation of porewater pressure. This will help determine the waiting time between passes.

CONSTRUCTION

- Dynamic compaction should start around the perimeter of the site and move progressively towards the centre in a grid formation for each pass. This will reduce vibration and heave effects on neighbouring properties.
- Crater size and heave as compaction continues should be monitored. Where craters are larger and deeper, indicating looser deposits, additional energy may be required.
- Quality assurance testing typically involves in-situ penetration tests (CPT or SPT) to ensure adequate compaction and depth of compaction is achieved. These can be carried out as compaction proceeds; however, dynamic compaction is known to have delayed strength gains. These occur mostly in silty soils but have also been recorded in clean granular soils. Where possible, there should be a delay of at least two weeks between finishing the ground improvement and carrying out final quality assurance testing to allow all pore pressures to dissipate and the full improvement to be measured.

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Example 5: Controlled modulus columns

Scenario

This example considers the design of controlled modulus (weak concrete) columns to mitigate the effects of liquefaction for a medium rise concrete frame building with a grillage of intersecting foundation beams located on Site A.

The columns are installed using a displacement auger. The auger displaces the ground as it is inserted to the base of each column. Weak concrete is then placed through the hollow auger stem and compacted as the auger is withdrawn. Construction of CMC's using a displacement auger therefore mitigates liquefaction through a combination of reinforcement and densification.

This example presents calculations to assess the reinforcement contribution of the columns to the mitigation of liquefaction using simplified methods and discusses other aspects of the seismic design. For projects of this type, preliminary and detailed design would typically require field trials and numerical modelling (finite element or finite difference) to give enough confidence in the design.

The field trials and numerical analysis will inform the increase in stiffness and liquefaction resistance of the soils between the columns, how loads are transferred through the building foundations and load transfer platform to the columns and soil under normal gravity loads and in earthquakes and the structural demands in the columns.

While not presented in this example, the first stages of the design process involve setting the performance requirements and confirming the need for ground improvement. The next stage is to confirm the site is suitable for CMC's, then design the ground improvement to meet the gravity load serviceability and ultimate limit state performance requirements.

Design

CALCULATION OF THE CSR REDUCTION FROM REINFORCEMENT EFFECTS

In the past, the reduction in cyclic stresses in soils between stiff columns has been calculated assuming the shear strains that develop in the soil and the stiff columnar inclusions are the same, ie shear strain compatibility. Researches have since found that this is incorrect and methods based on the assumption of shear strain compatibility significantly overestimates the reduction in cyclic shear stress in the soil between stiff columns.

To account for shear strain incompatibility between rigid inclusions, such as the controlled modulus columns (CMC), and the surrounding liquefied soil and the effects of column flexure, Rayamajhi (2014) developed a modified cyclic shear stress-ratio reduction factor (R_{CSR}), as follows:

$$R_{CSR} = \frac{CSR_{improved}}{CSR_{unimproved}} = R_{rd} \times R_{a,max}$$

Where $R_{a,max}$ is the ratio of peak surface accelerations for improved and unimproved cases, and R_{rd} is the ratio of shear stress-reduction coefficients for the improved and unimproved cases.

A value of $R_{a,max}$ of 1.0 is commonly adopted in practice for preliminary design, ie the dynamic response on unimproved and improved ground are considered to be the same. Although this is not usually the case, it is also difficult to estimate the peak surface accelerations for improved ground without undertaking dynamic analysis. In this working example, $R_{a,max}$ is assumed to be 1.0, R_{rd} is calculated using the following equation:

$$R_{rd} = \frac{1}{G_r \times [A_r \times \gamma_r \times C_G + \frac{1}{G_r} \times (1 - A_r)]} \leq 1$$

Where G_r is the ratio of the column shear modulus and the soil shear modulus, A_r is the area replacement ratio and γ_r is the shear strain ratio, the ratio between shear strain in the columns and shear strain in the surrounding soil and C_G is the equivalent shear factor and depends on the shape of the columns.

CALCULATE AREA REPLACEMENT RATIO, A_r

The design is an iterative process with the column spacing, diameter and modulus are changed until the design requirements are met. We start by assuming a triangular grid of 0.45 m diameter columns with 0.9 m spacing centre-to-centre to calculate A_r as:

$$Ar = \frac{A_c}{A} = \frac{\pi \times \frac{\text{diameter}^2}{4}}{\frac{\sqrt{3}}{2}\text{spacing}^2} = \frac{\pi \times \frac{0.45^2}{4}}{\frac{\sqrt{3}}{2}0.9^2} \times 100\% = 22.7\%$$

CALCULATE SHEAR MODULUS RATIO, G_r

For this example, we assume the columns are constructed of weak concrete with a 28-day unconfined compressive strength, f_c of 8 MPa.

The shear modulus of the columns is calculated as:

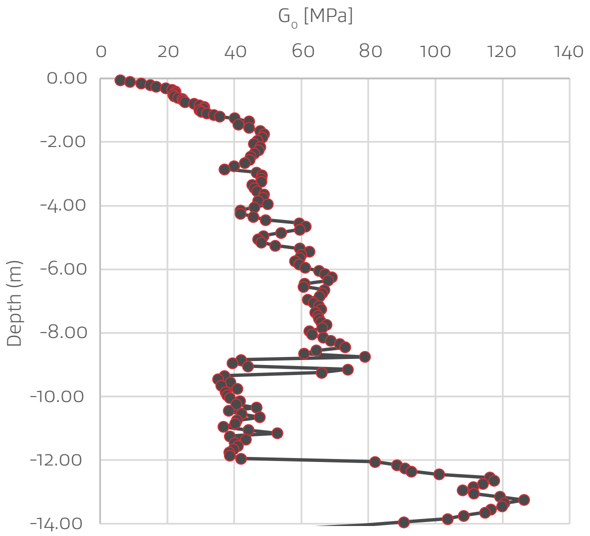
$$G_C = \frac{E_c}{2 \times (1 + \upsilon)}$$

Youngs modulus of the concrete columns, E_c is estimated as 400.f_c for this example but would be confirmed with laboratory testing. Poisons ratio of the concrete columns is assumed to be 0.2 and

$$G_C = \frac{(400 \times 8 \text{ MPa})}{2 \times (1 + 0.2)} = 1330 \text{ MPa}$$

The shear modulus of the soil, G_s, can be estimated as the low strain modulus factored by 0.4 to 0.6 to calculate the shear modulus at working strain levels. The low strain shear modulus of the soils is ideally determined from cross-hole or down-hole shearwave velocity measurement or seismic CPT's but can also be estimated using correlations with CPT penetration resistances.

Figure A.5.1: The calculated G_0 profile



For this example, G_0 is calculated using the correlations by Rix and Stokoe (1991) for soils with $I_c < 2.6$:

$$G_0 = 1634(q_t)^{0.250} (\sigma'_v)^{0.375}$$

And the method by Mayne and Rix (1993) for soils with $I_c \geq 2.6$

$$G_0 = 406(q_t)^{0.695}\, e^{-1.130}$$

To a depth of 4.5 m,

$$G_s = 0.5 \times 46 \text{ MPa} = 23 \text{ MPa}$$

And

$$G_r = \frac{G_C}{G_s} = \frac{1330 \text{ MPa}}{23 \text{ MPa}} \approx 58$$

Between 4.5 m and 8.5 m

$$G_s = 0.5 \times 62 \text{ MPa} = 31 \text{ MPa}$$

And

$$G_r = \frac{G_C}{G_s} = \frac{1330 \text{ MPa}}{31 \text{ MPa}} \approx 43$$

CALCULATE THE SHEAR STRAIN RATIO

To a depth of 4.5 m,

$$\gamma_r = 1.04 \times G_r^{-0.65} - 0.04 = 1.04 \times 58^{-0.65} - 0.04 = 0.034$$

Between 4.5 m and 8.5 m,

$$\gamma_r = 1.04 \times G_r^{-0.65} - 0.04 = 1.04 \times 43^{-0.65} - 0.04 = 0.05$$

Calculate the ratio of shear stress-reduction coefficients for the improved and unimproved cases, R_{rd}

$$C_G = 1.0 \text{ (for circular columns)}$$

To a depth of 4.5 m,

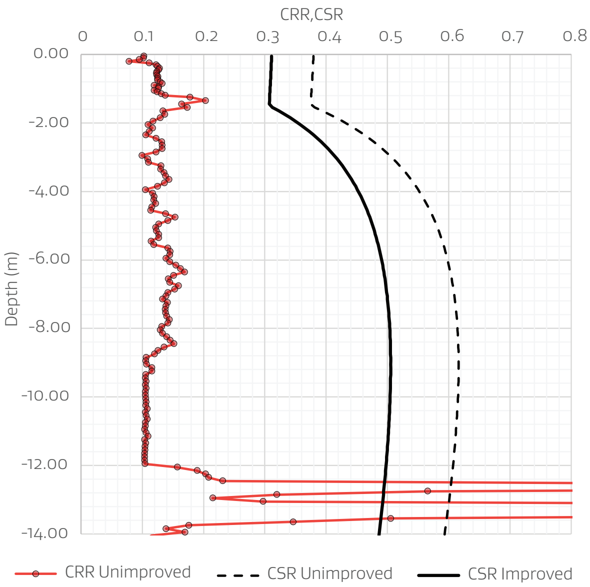
$$R_{rd} = \frac{1}{58 \times [0.227 \times 0.034 \times 1.0 + \frac{1}{58} \times (1 - 0.227)]} = 0.82$$

Between 4.5 m and 8.5 m,

$$R_{rd} = \frac{1}{43 \times [0.227 \times 0.05 \times 1.0 + \frac{1}{43} \times (1 - 0.227)]} = 0.79$$

Assuming $R_{a,max}$ is 1.0, the cyclic shear stress-ratio reduction factor (R_{CSR}) is therefore 0.82 and the CSR for the soil between the columns is calculated as 0.82 x $CSR_{Unimproved}$ as shown below for a ULS earthquake with PGA=0.51 g and Mw=7.2. $CRR_{Unimproved}$, the cyclic stress resistance of the unimproved soil between the columns is also plotted.

Figure A.5.2: Cyclic stress ratio for improved and unimproved ground



As CRR unimproved is below CSR improved, the design cannot rely solely on reinforcement affects to prevent liquefaction of the soil between the columns in a ULS earthquake. CRR of the improved ground needs to be quantified with a field trial to determine whether the design has the required factor of safety against liquefaction for soils between the columns.

DETERMINE CRR IMPROVED

Construction of CMC's densifies the soil between the columns thereby increasing their cyclic resistance ratio, CRR. Charts produced to estimate the increase in CRR from densification for different area replacement ratios are generally for soils with less than 25 percent fines that are densified using vibratory installation methods (eg vibroflots). These charts may significantly overestimate the degree of densification improvement in this case where the liquefiable silts have a fines content in the order of 40 percent and the construction uses a displacement auger.

A field trial is required to confirm the column spacing that gives sufficient factor of safety against liquefaction from both reinforcement and densification mechanisms.

Note
Prevention of liquefaction altogether in the soils between the columns may not be necessary to meet the performance requirements.

EXTENT OF IMPROVEMENT

The length of the CMC's and areal extents are designed to maintain stability and limit displacements for the ULS, SLS and any intermediate limit states. Based on the ground conditions, it seems likely that the CMC's would extend to a depth of about 12.5 m below existing ground level to meet design requirements.

Procedures by Poulos (2001), Horikoshi and Randolph (1999) for the design of piled raft foundations and procedures by Han (2001) for the design of DSM column ground improvement with suitably degraded strength and stiffness for can be used to calculate post liquefaction bearing capacity and for preliminary estimates seismic subsidence.

Stiff inclusions such as CMC's do not rely on the confinement of the surrounding soil for internal strength. With suitable structural design of the CMC's and adequate distribution of loads from the structure to the columns, the CMC's should not need to extend beyond the perimeter foundation beams.

STRUCTURAL DESIGN OF THE CMC'S

Bending and shear demands on the CMC's during cyclic loading in earthquakes can be calculated using the method in Module 4 where horizontal ground displacements are applied to the free end of the horizontal pile — soil springs along the length of the pile and inertia and compression loads from the superstructure are applied at the top. Axial stress along the piles can be calculated in a similar way with the inclusion of soil friction and end bearing springs.

Peak cyclic horizontal ground displacements applied to the free field end of soil springs can be estimated by integration of the soil shear strains over the depth of the column. The peak cyclic shear strain in non-liquefied soil between the columns can be calculated as:

$$\gamma(z) = \frac{CSR_{imp}\sigma'_v}{G_{imp}}$$

Where CSR_{imp} is the CSR of the improved ground, σ'_v is the is the initial effective overburden pressure and G_{imp} is the secant shear modulus of the soil between the columns. If liquefaction develops, γ can be estimated using the method by Tokimatsu and Asaka (1998).

It is possible to reinforce CMC's by plunging reinforcing steel into the columns before the concrete sets. Plunging of cage reinforcement is also possible with suitably designed concrete with an unconfined compressive strength in excess of 20 MPa to develop bond with the reinforcement. The axial compressive, bending and shear capacities of the piles can be calculated using the New Zealand steel and concrete standards, NZS 3404 or NZS 3101 respectively.

DESIGN OF LOAD TRANSFER PLATFORM (LTP)

The load transfer platform (LTP) distributes loads from the building foundation beams to the CMCs. In this example, the LTP is a geogrid reinforced dense well graded crushed high-quality gravel layer. The gravel LTP will also act as a drain to relieve any excess porewater pressure that develops in the soil between the columns.

The Collin method (2004) that assumes a pyramid-shaped region of gravel forms between the columns as shown in Figure A.5.3 to design the LTP is presented below. There are other methods for the design of LTP's which are based on different soil arching assumptions that may be suitable or even preferable to the Collin method. Most methods have been developed for column supported embankments that apply a uniform load to the LTP. For buildings, the distribution of loads can vary considerably across the footprint. Complex foundation loading can be considered in the design of the LTP using numerical analysis.

A minimum of three geogrid layers is recommended with a minimum spacing of 200 mm between each layer. The minimum thickness of the LTP (h) is taken as a minimum of 0.6 m or the height of the pyramid ha calculated as:

$$ha = \frac{(s-d)}{2} \text{ x } \tan\alpha = \frac{(0.9\text{m} - 0.45\text{m})}{2} \text{ x } \tan 45^\circ$$
$$= 0.225\text{m}$$

Note

We have not included pile caps and d equals the diameter of the columns. In this worked example, the LTP thickness is 600 mm with 3 layers of geogrid and a basal filter fabric.

The tensile load in the geogrid per unit length, T_n, is calculated for each geogrid layer based on 2D tension membrane theory to support the pyramid of soil between the columns as:

$$T_n = \gamma_{fill} \text{ x } h_n \text{ x } \frac{L_n^2 + L_{n+1}^2}{L_n^2} \text{ x } \frac{D_n}{2} \text{ x } \Omega$$

Where,

- h_n is the thickness of the soil layer between geogrids n and n + 1
- L_n is the length of the side of the equilateral triangle formed by the intersection of the pyramid and geogrid layer n
- Ω is a dimensionless factor determined from strain level [Ω = 0.97 for 5% strain]
- D_n is design span, which for a triangular grid is given by:

$$D_n = \frac{\sqrt{3}}{2} \text{ x } L_n$$

The length of the first layer of the geogrid (n=1) is given by:

$$L_1 = s - d = 0.9 - 0.45 = 0.45 \text{ m}$$

For a triangular column grid with pyramid-shaped region of loose soil and taking α as 45 degrees, L_{n+1} can be calculated as:

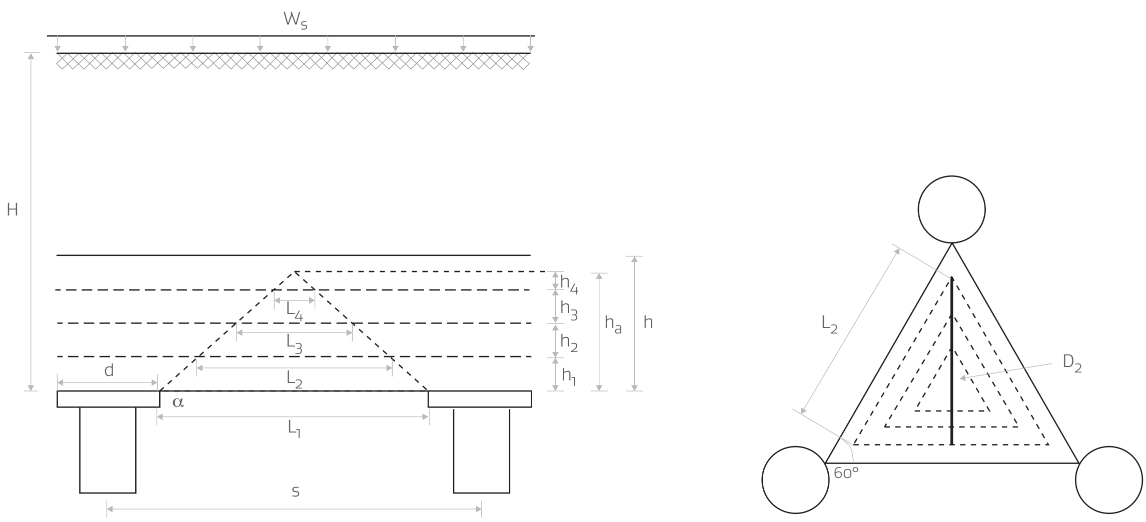
$$L_n + 1 = Ln - 2 \text{ x } h_n$$

$$L_2 = L_1 - 2 \text{ x } h_1 = 0.45 - 2{\cdot}0 \text{ x } 20 = 0.05 \text{ m}$$

$$D_1 = \frac{\sqrt{3}}{2} \text{ x } L_1 = \frac{\sqrt{3}}{2} \text{ x } 0.45 = 0.39 \text{ m}$$

$$T_1 = \gamma_{fill} \text{ x } h_1 \text{ x } \frac{L_1^2 + L_2^2}{L_1^2} \text{ x } \frac{D_1}{2} \text{ x } \Omega = 20 \text{ kN/m}^3 \text{ x } 0.20 \text{ m}$$
$$\text{x } \frac{0.45^2 + 0.05^2}{0.45^2} \text{ x } \frac{0.39}{2} \text{ x } 0.97 = 0.8 \text{ kN/m}$$

Figure A.5.3: Pyramid-shaped region of loose soil of Collin's method



Construction

QUALITY ASSURANCE AND QUALITY CONTROL

Quality control should be fully automated a give real-time information to the plant operator of installation parameters, including:

- Speed of rotation
- Rate of advancement and withdrawal of auger
- Torque, pull-down, down-pressure, drilling energy
- Depth of column
- Time of installation
- Pressure and volume of grout

Quality assurance should include:

- Testing of materials (concrete slump, unconfined compressive strength, LTP gravel source and production properties)
- Geogrid placement and LTP compaction
- CPT testing of soils between the columns to confirm that the soils between the columns have been compacted to the minimum required levels.

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Example 6: Lattice reinforcement

Scenario

This example considers the design of lattice reinforcement using deep soil mix (DSM) contiguous pile walls to mitigate the effects of liquefaction for the same medium rise concrete frame building in example 5 but with a mat foundation located on Site A.

This example presents calculations to determine the geometry of the lattice to mitigate liquefaction of the soils within the lattice using simplified methods and discusses other aspects of the seismic design. For projects of this type, preliminary design would typically require numerical modelling (finite element or finite difference) to assess how loads from the structure are transferred to the lattice stresses within the lattice and its overall settlement performance. This is beyond the scope of this example.

While not presented in this example, the first stages of the design process involve setting the performance requirements and confirming the need for ground improvement. The next stage is to confirm the site is suitable for deep soil mixing and then design the ground improvement to meet the gravity load serviceability and ultimate limit state performance requirements.

Design

The method by Nguyen (2013) is used to determine the cyclic shear stress reduction factor for the soils within the lattice (R_{CSR}), as follows:

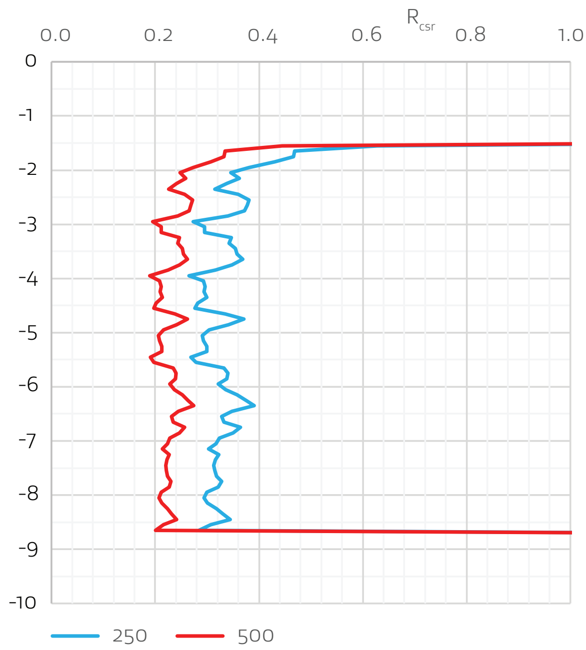
$$R_{CSR} = \frac{CSR_{improved}}{CSR_{unimproved}} = R_{rd} \times R_{a,max}$$

Where $R_{a,max}$ is the ratio of peak surface accelerations for improved and unimproved cases, and Rrd is the ratio of shear stress-reduction coefficient for improved and unimproved cases. The factor of safety against liquefaction, FS is calculated as:

$$FS(z) = \frac{CRR_{M,\sigma'v}}{R_{CSR} \times CSR}$$

The value of R_{CSR} required for a factor of safety of 1.0 for the 250 year and 500 year return period earthquakes calculated using the equation above for the CPT at site A are shown in Figure A.6.1.

Figure A.6.1: R_{csr} required for factor of safety against liquefaction



A value of $R_{a,max}$ of 1.0 is commonly adopted in practice for preliminary design, ie the dynamic response on unimproved and improved ground are considered to be the same. Although this is not usually the case, estimating the peak surface accelerations for improved ground typically requires undertaking dynamic analysis. In this working example, $R_{a,max}$ is assumed to be 1.0,

R_{rd} is calculated using the following equation:

$$R_{rd} = \frac{1}{G_r \times [A_r \times \gamma_r \times C_G + \frac{1}{G_r} \times (1 - A_r)]}$$

Where G_r is the ratio of the lattice wall shear modulus and the soil shear modulus, A_r is the area replacement ratio and γ_r is the shear strain ratio, the ratio between shear strain in the lattice walls and shear strain in the soil within the lattice and C_G is the equivalent shear factor.

DETERMINE PROPERTIES OF CEMENT MIXED SOIL

The in-situ strength of the cement mixed soil depends on the properties of the soil to be treated, the method of mixing (wet or dry), the binder type and dosage, the efficiency of the mixing plant and the skill of the operator. The strength and stiffness of DSM materials may vary considerably because of variations in the ground conditions and variations in construction.

Laboratory testing on laboratory mixed samples should be carried out to get an initial estimate of strength and Young's modulus. Field properties can vary between a third to double laboratory measured strengths and the dosage rates, construction methods and properties of the DSM need to be verified with field trials before construction of the production columns comences.

For this worked example, the design unconfined compressive strength of the soil mix, f_{sm} is assumed to be 4 MPa and Young's modulus, E_{sm}, is assumed to be $400f_{sm}$.

> **Note**
> Young's modulus as a function of compressive strength can varies widely.

Design properties for the cement mixed soil in this example are:

The poisson's ratio of the cement mixed soil,

$$\upsilon_{sm} = 0.25$$

$$G_{sm} = \frac{E_{sm}}{2 \times (1 + \upsilon_{sm})} = \frac{(400 \times f_{sm})}{2 \times (1 + \upsilon_{sm})} = \frac{(400 \times 4 \text{ MPa})}{2 \times (1 + 0.25)}$$
$$= 640 \text{ MPa}$$

CALCULATE SHEAR MODULUS RATIO, G_r

Calculation of soil shear modulus for site A is presented in example 5, and not repeated here. The shear modulus ratio Gr is:

To a depth of 4.5 m,

$$G_r = \frac{G_{sm}}{G_s} = \frac{640 \text{ MPa}}{23 \text{ MPa}} \approx 28$$

Between 4.5 m and 8.5 m

$$G_r = \frac{G_{sm}}{G_s} = \frac{640 \text{ MPa}}{31 \text{ MPa}} \approx 31$$

Where G_s is the secant shear modulus of the unimproved soil.

CALCULATE THE WALL SPACING

Calculating the wall spacing is an iterative process. Published case studies, centrifuge tests and numerical studies can be used to make an initial assessment of grid spacing and wall thickness. Calculations for a 6 m grid spacing with G_r=30 are presented here. The lattice walls will be constructed of overlapping 1200 mm diameter columns spaced 900 mm apart giving an effective wall width of 800 mm.

The area replacement ratio (A_r) is defined by the area of improved ground, in this case the area of lattice walls, divided by the total plan area of the grid cell. For a square grid with equal wall spacing in both direction the A_r is given by:

$$A_r = \frac{A_{walls}}{A} = 1 - \left(1 - \frac{t}{s}\right)^2$$

$$A_r = 1 - \left(1 - \frac{0.8}{6.0}\right)^2 = 0.249$$

Where t is the wall thickness, s is the grid spacing of the walls. The shear strain ratio, γ_r is calculated as:

$$\gamma_r = \left[1 - (1 - A_r)^{1.3} \times \left(\frac{G_r - 1}{185}\right)^{0.4}\right] \times \min\left(\frac{H}{s}, 1\right) =$$
$$\left[1 - (1 - 0.249)^{1.3} \times \left(\frac{30 - 1}{185}\right)^{0.4}\right] \times 1 = 0.544$$

Where H is the height of the lattice

$$C_G = 1 - 0.5 \times \sqrt{1 - A_r} = 1 - 0.5 \times \sqrt{1 - 0.249} = 0.567$$

$$R_{rd} = \frac{1}{30 \times \left[0.249 \times 0.544 \times 0.567 + \frac{1}{3} \times (1 - 0.249)\right]}$$
$$= 0.35$$

Assuming $R_{a,max}$ = 1.0, R_{csr} = 0.35.

Therefore, with this lattice arrangement and properties, liquefaction will start to trigger (FS=1) in a 250 year return period earthquake. Because the lattice confines the soil and the matt foundation can be designed to span the lattice walls, preventing liquefaction for the ULS earthquake is not essential if stability is maintained and evacuation is not impeded. This spacing may therefore be adopted or reduced as necessary to meet the design criteria.

EXTENTS

As the lattice laterally confines the soil beneath the foundation and prevents migration of excess porewater pressures laterally into the lattice, the perimeter walls of the lattice can align with the edge of the mat foundation.

The depth of the lattice is designed to maintain stability during and after an earthquake and to limit differential movement to acceptable levels. Considering the ground conditions, the lattice base would need to be at least 9 m below existing ground level but potentially extend to a depth of 12.5 m to meet performance requirements.

The firm clayey silt between 8.5 m and 12 m below ground level will likely have a marked effect on the ground shaking experienced in the liquefiable soil above and at the surface. The inclusion of stiff ground improvement to 12.5 m depth may reduce seismic subsidence and tilt, but could significantly increase the inertial demands on the building and the lattice. Dynamic analysis is necessary to understand the relative merits of improvement to 9 m and improvement to 12.5 m depth.

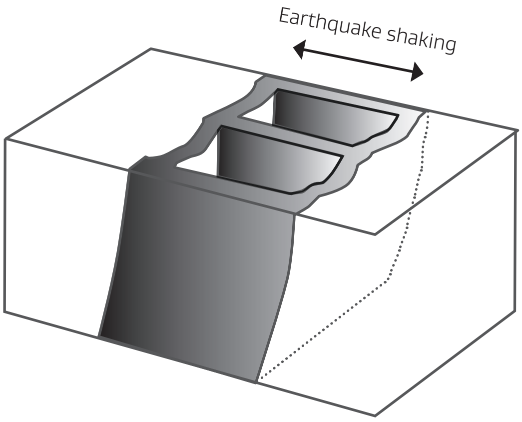
The overall stability of the lattice can be evaluated using a pseudo static approach considering the lattice to be a rigid block. Stability can be checked for different phases of the earthquake, for example during strong shaking before liquefaction has developed, once liquefaction develops outside the lattice, and at the end of shaking.

Subsidence could be estimated initially, using an equivalent pier method. For the case with the lattice terminating at the top of the clayey silt (9 m depth), cyclic shear deformation of the silt could be a significant contributor to differential subsidence. If the post-earthquake bearing capacity factor of safety for the lattice with reduced soil strengths for liquefaction and cyclic softening is less than 1.5, dynamic numerical analysis may be necessary to adequately evaluate seismic performance.

STRUCTURAL DESIGN OF THE LATTICE

The lattice walls will experience both shear and flexure during an earthquake from ground movement and inertia and loads from the building. Figure A.6.2 shows the typical deformation of a lattice founded on competent ground for one direction of shaking.

Figure A.6.2: Lattice shear and flexure in earthquakes (O'Rouke and Goh 1997)



For structural design of the lattice, stresses within the lattice walls for the seismic case can be evaluated using an approach like the method described by O'Rouke and Goh 1997 with base shear and compressive loads from the building applied to the top of the lattice. This method involves 2D dynamic analysis of a transformed 3D cross section through the lattice and plate analysis to assess the bending in the out of plane walls. To simplify the calculations, the dynamic analysis could be substituted with an equivalent pseudo static analysis of both the pre-liquefaction and post liquefaction cyclic phases.

Dynamic numerical analysis, either 2D or 3D should be considered if the simplified analysis indicates low structural redundancy in the lattice or for design optimisation through better understanding of the system response.

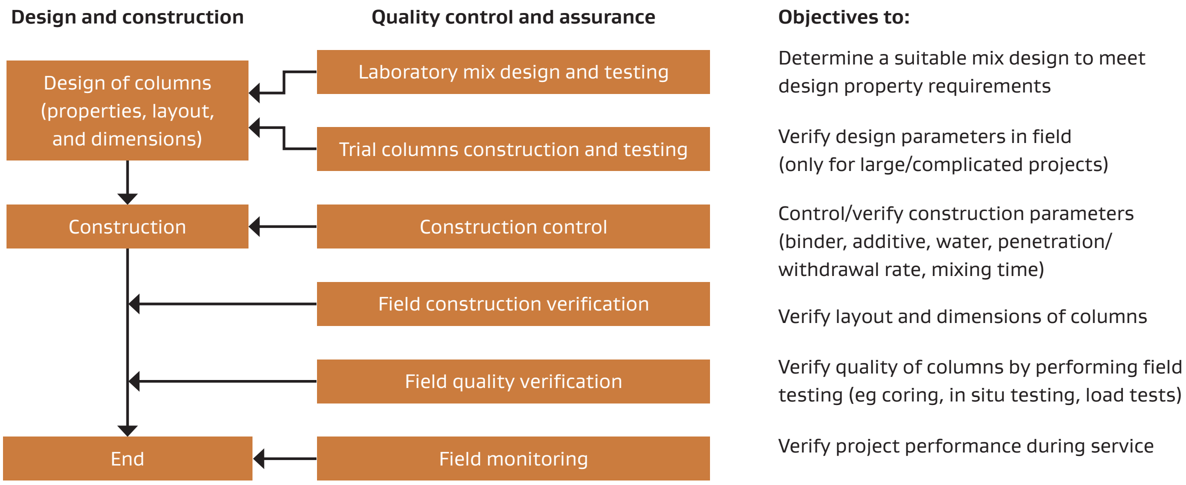
Construction

A typical approach to quality control and quality assurance for deepsoil mixing is shown in Figure A.6.3. A trial is usually undertaken during the design phase to confirm the construction method, properties of the columns and production rates.

Modern soil mixing rigs have onboard equipment to measure penetration, rotation and dosage rates and mixing times for quality control.

Field verification will frequently involve inspection of the top of the lattice with a section partially excavated to check column consistency and overlap. Cores are taken to confirm mixing consistency with depth and to retrieve samples for UCS testing to check compressive strength and modulus.

Figure A.6.3: Typical Quality Control and Quality Assurance for Deepsoil Mixing (Han 2015)



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