Marlborough District Council - Liquefaction Assessment Guidelines

Version – FINAL
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Marlborough District Council
Liquefaction Assessment Guidelines

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Important Notice

This document has been prepared based on the guidance provided in the *Earthquake Engineering Practice: Modules 1 to 6* and the *Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes*, which were published by MBIE under Section 175 of the Building Act 2004, to assist parties to comply with their obligations under the Building Act 2004.

This document provides suitable foundation design solutions that:

(i) Take into account the likely future performance of the ground, under seismic loading,

(ii) Provides MDC with 'reasonable grounds' to be satisfied that the minimum performance standards of the Building Code (Clause B1) are satisfied, which will enable them to grant consent for foundation solutions, sited on potentially liquefiable ground.

While the Marlborough District Council and Fraser Thomas Ltd have taken care in preparing this document, it is only a guide, and, if used, does not relieve any person of the obligation to consider any matter to which that information relates, according to the circumstances of the case. All users should satisfy themselves as to the applicability of the content and should not act on the basis of any matter contained in this document without considering, and if necessary, taking appropriate professional advice.

This document may be updated occasionally, due to the ever-changing nature of geotechnical/earthquake engineering. The latest version can be found at www.marlborough.govt.nz.

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1.0 INTRODUCTION

1.1 GENERAL

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

The sequence of strong earthquakes in Canterbury in 2010 and 2011, resulted in 185 fatalities and extensive damage to buildings and infrastructure. Liquefaction occurred on several occasions in the city and nearby areas. The damaging effects of liquefaction included lateral ground spreading, ground settlement, building foundation failures, damage to infrastructure, subsidence of areas close to waterways and large volumes of sediment ejecta on the ground surface.

The majority of the houses affected by severe foundation damage, in the Canterbury earthquakes, were located in areas where moderate to severe liquefaction manifestation was observed. It is evident that appropriately assessing the liquefaction risk of site soils is critical for ensuring that foundation solutions are suitable for the site conditions.

1.2 THE LIQUEFACTION PHENOMENON

Earthquakes are sudden ruptures of the earth's crust caused by accumulating stresses resulting from internal processes of the planet. Energy radiates from the fault rupture as seismic waves. One of the principle geotechnical hazards associated with earthquakes is the liquefaction of soils. The term *'liquefaction'* is widely used to describe ground damage caused by earthquake shaking.

Liquefaction is associated with significant loss of stiffness and strength in the liquefied soil, and consequent large ground deformation as a result of the development of large excess pore water pressures within the soil.

The three key elements are all required for liquefaction to occur:

(i) Loose non-plastic soil (typically sands, and sandy silts),

(ii) Saturated soil (i.e., below the groundwater table),

(iii) Sufficient ground shaking (a combination of the duration and intensity of shaking).

Soil types that are susceptible to liquefaction are typically those that are geologically young, i.e., typically soils of Holocene age (less than 11,000 years old), which have been deposited in low energy environments, forming loose and soft normally consolidated layers of soils with negligible *"micro-structure".*

Materials such as clays and plastic silts are not considered to be suspectable to liquefaction (i.e., will not reach a condition of zero effective stress) but can generate excess pore pressures during shaking, which can result in soil softening (i.e., cyclic softening).

The possible consequences of liquefaction of the soils beneath a site may include:

(i) Ground settlement

(ii) Ejection of sand at the surface

(iii) Differential building foundation settlement as a result of differential ground settlement

(iv) Foundation settlement as a result of bearing capacity failure of the soils (both "sand like" and "clay like")

(v) Lateral displacement of the ground as a result of "lateral spread"

The soil liquefaction phenomenon, and the possible consequences, are referred to in this document as the earthquake induced soil liquefaction hazard.

Aerial view of liquefaction manifestation at the Blenheim Rowing Club, looking east.

2.0 OBJECTIVE

The Marlborough District is located in a seismically active region which contains certain areas which are considered to be vulnerable to liquefaction related damage.

Marlborough District Council (MDC), as part of their role to identify natural hazards affecting their region, have recently released a document titled *Liquefaction Vulnerability Study: Lower Wairau Plains* (dated May 2021), which identifies potentially liquefiable soils in the Lower Wairau Plains region surrounding Blenheim (referred to hereafter as the MDC study report).

Although the 2021 MDC study document identifies potentially liquefiable areas, it does not provide any guidance for stakeholders on how to determine the theoretical liquefaction triggering potential of the site soils, to predict the likely associated ground deformation, or how to design foundation solutions for sites on potentially liquefiable ground, which will likely meet the performance standards of the Building Code.

The objective of this document is to:

(1) Promote consistency of approach to assessing liquefaction risk in the whole Marlborough region,

(2) Provide sound guidelines for the determination of the theoretical liquefaction triggering potential of soils, due to seismic loading, to support rational foundation design, which are informed by the latest research and the MBIE Guidelines Modules (1 to 6),

(3) Provide suitable foundation design solutions that:

   (i) Take into account the likely future performance of the ground, under seismic loading,

   (ii) Provides MDC with 'reasonable grounds' to be satisfied that the minimum performance standards of the Building Code (Clause B1) are satisfied, which will enable them to grant consent for foundation solutions, sited on potentially liquefiable ground.

The science and practice of geotechnical earthquake engineering is advancing at a rapid rate. The geoprofessionals who use this document should familiarise themselves with the general principles of the simplified method for soil liquefaction analyses (Idriss and Boulanger 2008) and with recent advances, and interpret and apply the recommendations herein as time passes.

This guideline is not intended to be an overly prescriptive document. It is intended to balance the need for a consistent framework for liquefaction assessment in the Marlborough District, whilst simultaneously allowing the geoprofessional to apply their experience and judgement when assessing sites.

3.0 SCOPE

3.1 AUDIENCE

This document is intended for the engineering design and construction sectors and Marlborough District Council (in their role as a Territorial Authority).

The assessment of theoretical liquefaction triggering, and the assessment of the potential consequences of liquefaction, are complex problems. Geoprofessionals who are undertaking liquefaction assessment work should have a sound understanding and background in soil mechanics, geotechnical earthquake engineering and soil liquefaction theory, and should be suitably qualified and experienced chartered professional (CPEng) geotechnical engineers or engineering geologists.

3.2 GENERAL

The material in this document relates specifically to earthquake induced soil liquefaction hazard and should not be assumed to have wider applicability. It is intended to provide general guidance for earthquake engineering practice for the assessment of soil liquefaction and lateral ground spreading.

This document does not provide guidance on addressing other geotechnical hazards which may affect sites, such as soil swell/shrink, highly compressible soils and slope stability. Geoprofessionals are expected to address all potential geotechnical hazards when assessing sites, which could include, among others, potentially liquefiable soils.

The recommendations in this document are intended to be applied to everyday engineering practice (generally one to three storey residential structures) by qualified and experienced professional geotechnical engineers or engineering geologists, who are expected to also apply sound engineering judgment in adapting the recommendations to each particular situation. That being said, the determination of theoretical liquefaction triggering potential, and the general principles of foundation design, presented in this document, are considered to also be applicable to other structures, (such as commercial and industrial buildings).

Complex and unusual situations are not covered. In these cases, special or site-specific studies are considered more appropriate. Other documents may provide more specific guidelines or rules for specialist structures, and these should, in general, take precedence over this document.

Examples include:

(i) New Zealand Society on Large Dams- *Dam Safety Guidelines*

(ii) Waka Kotahi New Zealand Transport Agency (NZTA)- *Bridge Manual*

(iii) Transpower- *New Zealand Transmission Structural Foundation Manual*

Where significant discrepancies are identified among different guidelines and design manuals, it is the responsibility of the engineer to resolve such discrepancies as far as practicable. The recommendations made in this document may seem excessive for very small projects, such as minor building extensions or detached (IL1) structures. It is therefore intended that liquefaction hazards should be properly investigated and assessed at the subdivision stage of development (if possible). Simpler investigations and assessments would then be adequate for individual sites. Professional judgment needs to be applied in all cases.

This guidance document has been prepared using information obtained from the following documents:

(1) *Liquefaction Vulnerability Study: Lower Wairau Plains,* dated May 2021, prepared by the University of Auckland for the Marlborough District Council

(2) *Planning and engineering guidance for potentially liquefaction-prone land*, dated September 2017, prepared by MBIE

(3) *Earthquake Geotechnical Engineering Practice: Modules 1 to 6*, dated 2015 - 2017, prepared by MBIE

(4) *Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes*, dated 2012, prepared by MBIE

(5) *NZTA Bridge Manual SP/M/022.*

4.0 BUILDING ACT

As discussed in Section 2.0 of this document, one of the key objectives of these guidelines is to provide MDC with 'reasonable grounds' to be satisfied that the minimum performance standards of the Building Code (Clause B1) are satisfied, which will enable them to grant consent for foundation solutions, sited on potentially liquefiable ground. For this reason, it would be beneficial to provide some commentary regarding the NZ Building Act and the NZ Building Code.

Building Activities must comply with the requirements of the Building Act 2004 (the Act) and the relevant regulations. The Building Code is a regulation made under the Building Act 2004 (schedule 1 of the Building Regulations 1992).

The Building Act requires that all buildings must comply with the Building Code.

The Building Code is performance-based, outlining the performance that needs to be achieved under each of the Building Code clauses (covering aspects such as stability, protection from fire, moisture, safety of users etc). The Building Code does not prescribe how work should be done but states how completed building work and its parts must perform.

Acceptable Solutions and Verification Methods published by the MBIE, if followed, will result in building work that is deemed to comply with the Building Code. However, Alternative Solutions can be proposed and consented if sufficient evidence to satisfy the 'reasonable grounds' test that Building Code performance requirements will be met is provided to the building consent authority.

It is noted that the Christchurch City Council accepts the foundation design solutions provided in the MBIE guidance document (2012) as Alternative Solutions, and therefore the foundation solutions provided in the MBIE guidance document are considered, by CCC, to meet the minimum performance standards of the Building Code. Given that the geological conditions in the majority of the Marlborough region (in particular the Lower Wairau Plains area) are similar to that of the Canterbury region and given that the liquefaction assessment methodologies and foundation design solutions provided in this document are generally consistent with those proposed by the MBIE *Earthquake Geotechnical Engineering Practice: Modules 1 to 6* and the *Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes* documents, it is considered that this guidance document, if followed, provides MDC with 'reasonable grounds' to grant consent for foundation solutions, sited on potentially liquefiable ground.

The foundation design solutions provided in this guidance document should therefore be considered to be Alternative Solutions, as defined by the Building Act.

Building Code requirements to prevent structural collapse (B1.3.3)

Clause B1 (structure) of the Building Code is often the primary driver of the geotechnical and structural design aspects of a building. Amongst other things, B1 states that:

> *"Buildings, building elements and sitework shall have a low probability of rupturing, becoming unstable, losing equilibrium or collapsing during construction or alteration and throughout their lives"*

Buildings that are designed using AS/NZS 1170 are required to satisfy the ultimate limit state primary design case.

<u>Ultimate Limit State (ULS)</u>

The ULS design case is an extreme action, or combination of actions, that the building needs to withstand. ULS seismic loads for residential properties are based on a 1 in 500 year earthquake (a 10% chance of exceedance in 50 years). A building is expected to suffer moderate to significant structural damage in a ULS event, but not to collapse, and the ULS design case is therefore essentially a "life safety" design criteria.

Building Code requirements to prevent loss of amenity (B1.3.2)

Clause B1 (structure) of the Building Code also states that buildings should have:

> *"a low probability of causing loss of amenity.."*

Amenity is defined as *'an attribute of a building which contributes to the health, physical independence and well-being of the building's user but which is not associated with disease or a specific illness'.*

The meaning of 'loss of amenity' is not well described by the Building Act. The Canterbury MBIE Guidance describes loss of amenity as:

> *"All parts of the structure shall remain functional so that the building can continue to perform its intended purpose. Minor damage to structure. Some damage to building contents, fabric and lining. Readily repairable. Building accessible and safe to occupy. No loss of life. No injuries."*

Serviceability Limit State (SLS)

The SLS design case is a load, or combination of loads, that a building or structure is likely to be subjected to more frequently during its design life. If properly designed and constructed, a building should suffer little or no damage when subjected to a and SLS design load. All parts of the building should remain accessible and safe to occupy.

Services should be readily repairable at the perimeter and remain intact within the building. There may be minor damage to building fabric that is readily repairable possibly including minor cracking, deflection and settlement that do not affect the structural, fire or weathertightness performance of the building. SLS seismic loads for residential properties are based on a 1 in 25 year earthquake (refer to AS/NZS 1170.0).

5.0 EARTHQUAKE GEOTECHNICAL ENGINEERING PRACTICE: MODULES 1 TO 6

The New Zealand Geotechnical Society, in conjunction with MBIE, released Guidelines, in 2015 to 2017, with the objective of summarising current best practice in earthquake geotechnical engineering with a focus on New Zealand conditions. The main purpose of the Guidelines is to promote consistency of approach to everyday engineering practice in New Zealand and, thus, improve geotechnical earthquake aspects of the performance of the built environment.

The Guidelines consist of six modules (identified as Modules 1 to 6 inclusive), which are listed below:

(1) **Module 1**: Overview of the guidelines

(2) **Module 2**: Geotechnical investigation for earthquake engineering

(3) **Module 3**: Identification assessment and mitigation of liquefaction hazards

(4) **Module 4**: Earthquake resistant foundation design

(5) **Module 5**: Ground improvement; **Module 5A**: Specification of ground improvement for residential properties in the Canterbury region

(6) **Module 6**: Retaining Walls

The Guideline Modules are referred to throughout this document. It is recommended that Geoprofessionals undertaking liquefaction assessment work be familiar with the foregoing Modules.

It is noted that the *Earthquake Geotechnical Engineering Practice: Modules 1 to 6*, were published by MBIE as guidance under Section 175 of the Building Act 2004, to assist parties to comply with their obligations under the Building Act 2004. It is not mandatory to follow the guidelines, and if used does not relieve any person of the obligation to consider any matter to which that information relates according to the circumstances of the particular case. All users should satisfy themselves as to the applicability of the content of the Modules and should not act on the basis of any matter contained in the Modules without considering, and if necessary, taking appropriate professional advice.

6.0 MARLBOROUGH DISTRICT COUNCIL – LIQUEFACTION VULNERABILITY STUDY: LOWER WAIRAU PLAINS

6.1 GENERAL

Marlborough District Council (MDC), as part of their role to identify natural hazards affecting their region, have recently released prepared a document titled *Liquefaction Vulnerability Study: Lower Wairau Plains* (dated May 2021), which identifies potentially liquefiable soils in the Lower Wairau Plains region surrounding Blenheim (referred to as the MDC study report).

The MDC study report, dated May 2021, summarises the development of liquefaction vulnerability maps for the Lower Wairau Plains in Marlborough, and has been prepared based on procedures described in *Planning and engineering guidance for potentially liquefaction-prone land*, dated September 2017, prepared by MBIE (referred to as the 2017 MBE Guidelines).

The approximate extent of the Lower Wairau Plains study area is shown in Figure 1.

The primary objective of the study was to define the spatial distribution of liquefaction vulnerability across the region. The following data was collated to inform the study:

(i) Geological and digital elevation model data

(ii) Geomorphological mapping based on surface expression

(iii) Regional groundwater lithology models

(iv) Geotechnical site investigation data (CPT and machine excavated test pits)

(v) Groundwater models from hydrologic and geotechnical sources

(vi) Case history evidence of liquefaction manifestation, with a focus on the 2016 Kaikōura earthquake.

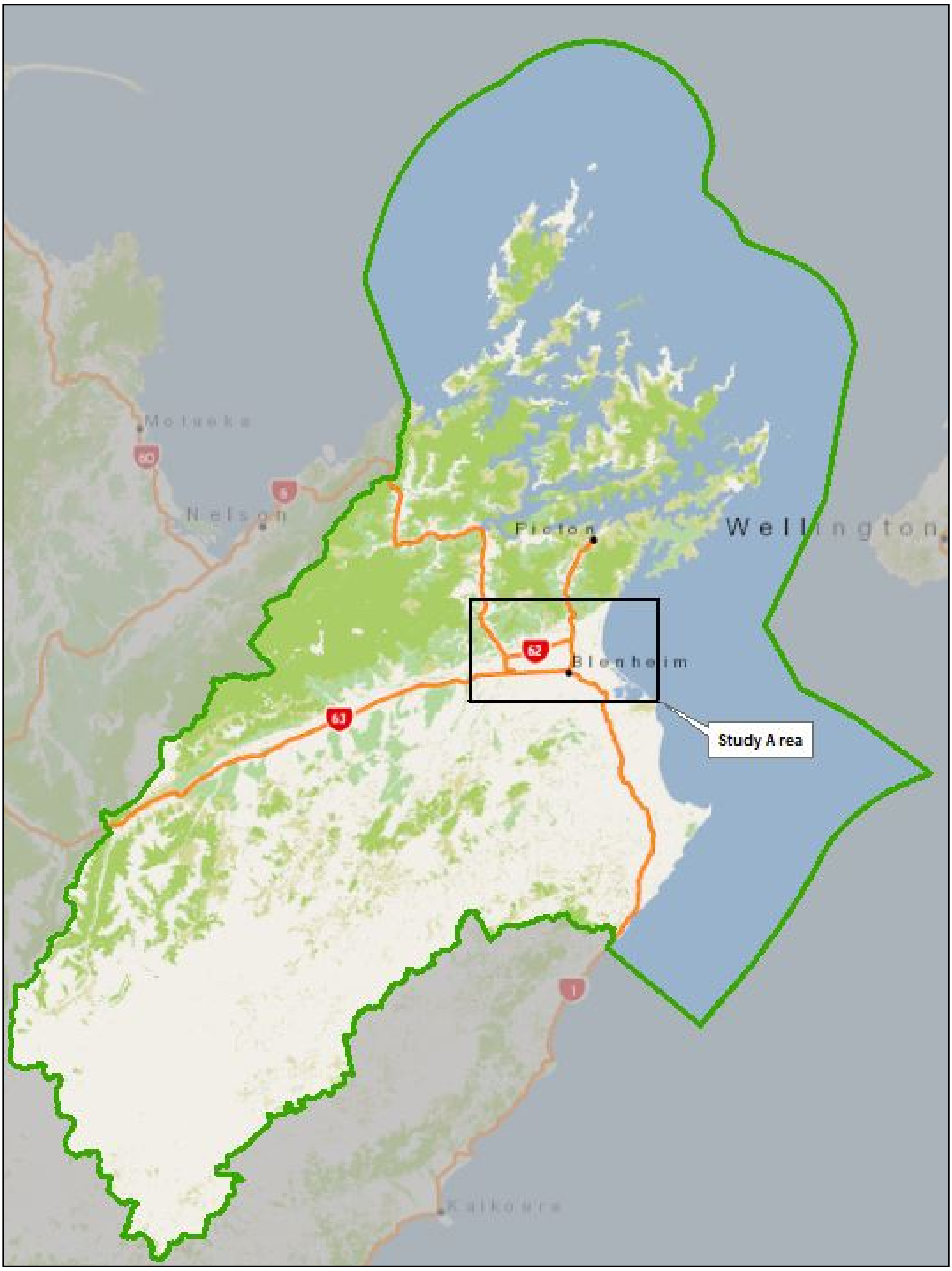

Figure 1 ***Geographic location of the Lower Wairau Plains and extent of the study area.***

6.2 LEVEL OF DETAIL IN STUDY

The 2017 MBIE guidelines provides guidance relating to the level of detail that is required for liquefaction assessments, which will be governed by the intended purpose of the study and how uncertainty in the assessment could affect the objectives.

The MBIE guidelines suggests four levels of detail, summarised below:

(1) Level A: Basic desktop assessment

(2) Level B: Calibrated desktop assessment

(3) Level C: Detailed area-wide assessment

(4) Level D: Site-specific assessment

6.3 SEISMIC HAZARD

The MDC study report summarises the seismicity of the Lower Wairau Plains.

The plate boundary between the Pacific and Australian plates passes through the Marlborough region, and consequently, this region is an area of high seismicity. The Marlborough region consists of a series of northwest-tilted blocks forming mountain ranges, hills and drowned valleys separated by major translucent faults such as Wairau, Awatere and Clarence Faults, each of which can give rise to frequent seismic events.

The Wairau Fault, which is a branch of Alpine Fault, divides Marlborough into two regions with divergent geological structures. The Wairau Plains are bounded by north-east trending mountain ranges (Richmond and Kaikoura Ranges) reflecting uplift along the Wairau and Awatere Faults which are part of the Marlborough Fault Zone (MFZ). This is a zone of north-east trending transgressional faulting associated with the offshore transition of the plate boundary (Rattenbury et al. 2006). The Wairau Fault is the closest active fault and is capable of rupturing in an earthquake event.

The MDC study report has determined peak ground acceleration (PGA) and earthquake moment magnitude (M_w) for the Lower Wairau Plains, based on a recent study by Cubrinovski et al. (2021). This study provided an update on the details in the NZTA Bridge Manual, based on the most up-to-date inputs that inform site-specific probabilistic seismic hazard analysis. The following seismic design loadings were used for the MDC study report:

(a) 100-year return period event - 0.26g (PGA), M_w = 6.8

(b) 500-year return period event - 0.52g (PGA), M_w = 7.3.

6.4 OBSERVATIONS FROM THE 2016 KAIKOURA EARTHQUAKE

The Kaikōura earthquake, occurred on 14 November 2016. It was a significant earthquake event (M_w 7.8), which resulted in the rupturing of several faults in the upper South Island.

The MDC study report provides an assessment of the land damage that was observed to have occurred in the Lower Wairau Plains, in response to seismic loading associated with the 2016 Kaikoura earthquake.

The MDC study report indicates that strong to severe shaking was felt across the Marlborough region during the Kaikōura earthquake. All Marlborough communities were subjected to earthquake damage. The main impact was to buildings, farm assets, horizontal infrastructure, river control works, the transportation networks and water supply networks.

There are two strong motion stations (SMS) in the Lower Wairau Plains, identified as BWRS and MCGS. These motion stations recorded earthquake shaking during the 2016 Kaikōura earthquake event. BWRS is a rock site on the edge of the Plains and MCGS is a deep soil site in Blenheim. The geometric mean horizontal peak ground accelerations recorded at these SMS were 0.15g and 0.26g respectively.

Across the Wairau Plains, peak ground accelerations would be expected to be slightly greater than 0.26g, moving towards the south-east of Blenheim, and would have likely been less than 0.26g, moving to the west and north. The 2016 Kaikōura earthquake is considered to have imposed seismic loadings in the Marlborough region, which would approximate the seismic design loadings associated with a 100-year return period event.

Post-earthquake reconnaissance surveys, aerial photography, and discussions with local engineers and the Marlborough District Council officers provided a comprehensive summary of the liquefaction-related impacts and manifestations in the Wairau Plains following the 2016 Kaikōura earthquake. These are summarised in detail by Stringer et al. (2017) and in GEER (2017). Within the Wairau Plains, liquefaction and lateral spreading was the major feature of ground damage and was largely observed along the Lower Wairau and Opaoa Rivers.

The approximate locations of observed liquefaction manifestations, following the 2016 Kaikōura earthquake, are shown on Figure 2.

Severe manifestations were recorded in the area of the Equestrian Park and the Blenheim Rowing Club but, as very few buildings were present in these areas, the engineering impacts were generally low. Some moderate liquefaction manifestations were observed in a few locations within Blenheim, but these again had limited impact. Localised liquefaction and associated lateral spreading occurred proximal to the Opaoa River within Blenheim. Liquefaction and lateral spreading related damage was confined to the inner-banks of meander bends of the rivers or associated paleo-channel, with damage observed on the outer-banks of the meander bends. Localised manifestations were also observed adjacent to the Taylor River within central Blenheim. Sand boils were observed at Lansdowne Park which is located adjacent to the southern bank of the Opaoa River, on the northern edge of Blenheim (Stringer et al. 2017, GEER 2017).

The observed distribution of liquefaction manifestations in this event further reinforces that fluvial geomorphology and the depositional processes of the meandering rivers are important factors for the interpretation of the distribution and sediment types in areas which are susceptible to liquefaction.

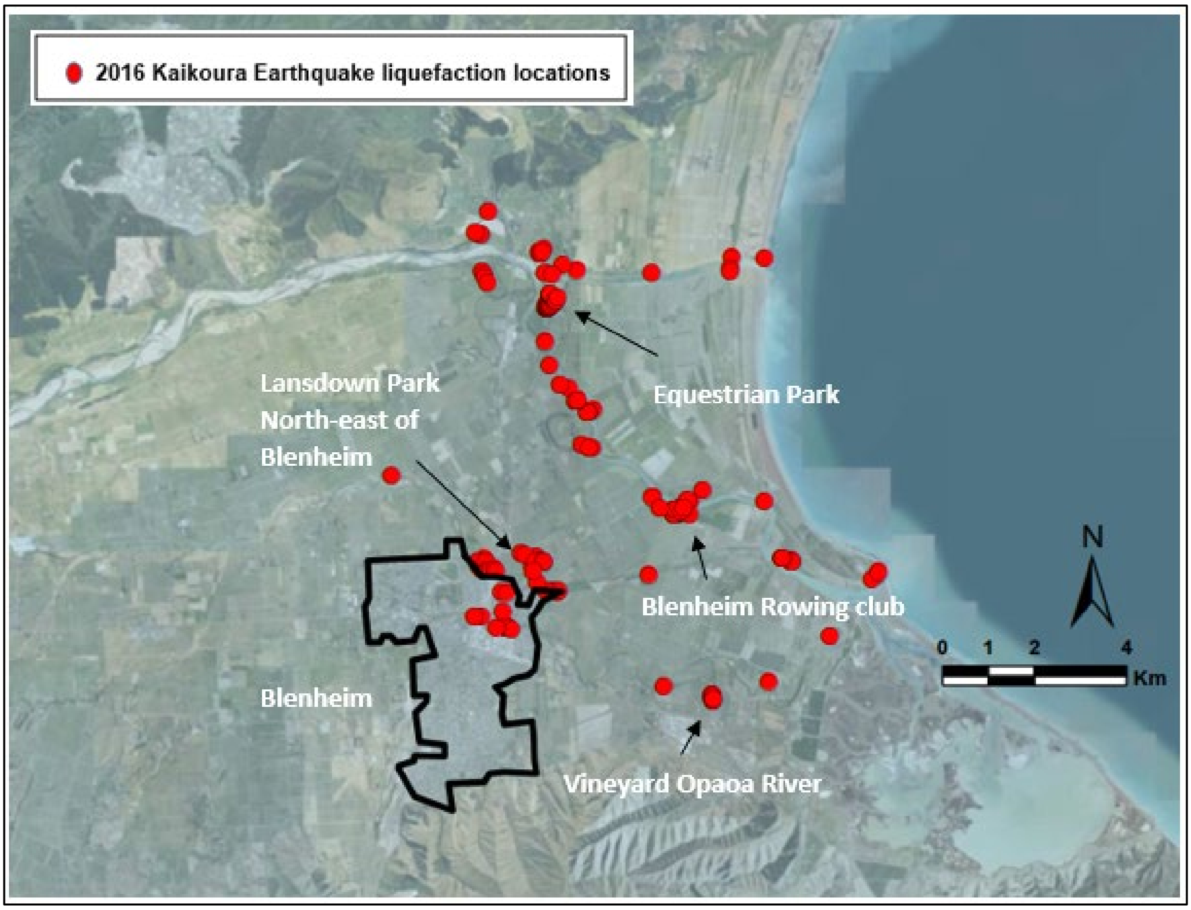

Figure 2 ***Locations of liquefaction manifestation from the 2016 Kaikōura Earthquake[1].***

6.5 GEOLOGY

The MDC study report provides a good summary of the geology of the study area.

The Lower Wairau Plains are located in the north-east of the South Island of New Zealand in the region of Marlborough. The region is intersected by many active crustal faults such as the Wairau, Awatere, and Clarence Faults (Rattenbury et al. 2006).

The Lower Wairau Plains are predominantly flat to gently undulating alluvial plains, underlain by Holocene age marine and estuarine silts and sands of the Dillons Point Formation, and alluvial gravels and sands of the Rapaura Formation. The soils of the Dillons Point Formation are observed to vary significantly in their composition and degree of consolidation, varying between loose sands and soft silts to very dense sands and very dense clayey silts (MDC 2012). The alluvial sediments,

[1] Figure adopted from Figure 10 of the *Liquefaction Vulnerability Study: Lower Wairau Plains*, dated May 2021

to the eastern margin of the Wairau Plains, are inter-fingered with lagoonal muds and coastal sands, silts, and gravels which reflect coastline progradation and marine regression following the mid-Holocene high stand 6,000 years ago (Basher 1995).

Near-surface sediments present in the Lower Wairau Plains, towards the coast, are postglacial swamp, lagoonal estuarine and beach deposits that overlie fluvial and glacial outwash deposits.

Figure 3 summarises the surface geological deposits present in the Lower Wairua Plains.

The MDC study report also provides a detailed assessment of the various geomorphic units in the Lower Wairau Plains. The geomorphic map for the Lower Wairau Plains, determined for the MDC study report, is shown on Figure 4.

The MDC study report indicates that detailed geomorphic characteristics can be used to refine the evaluation of the liquefaction potential of soil deposits.

The geomorphology of the Lower Wairau Plains has been assessed regarding the potential for liquefaction manifestation occurring in the various geomorphic units, using literature related to the performance of typical geomorphological formations in previous earthquake events.

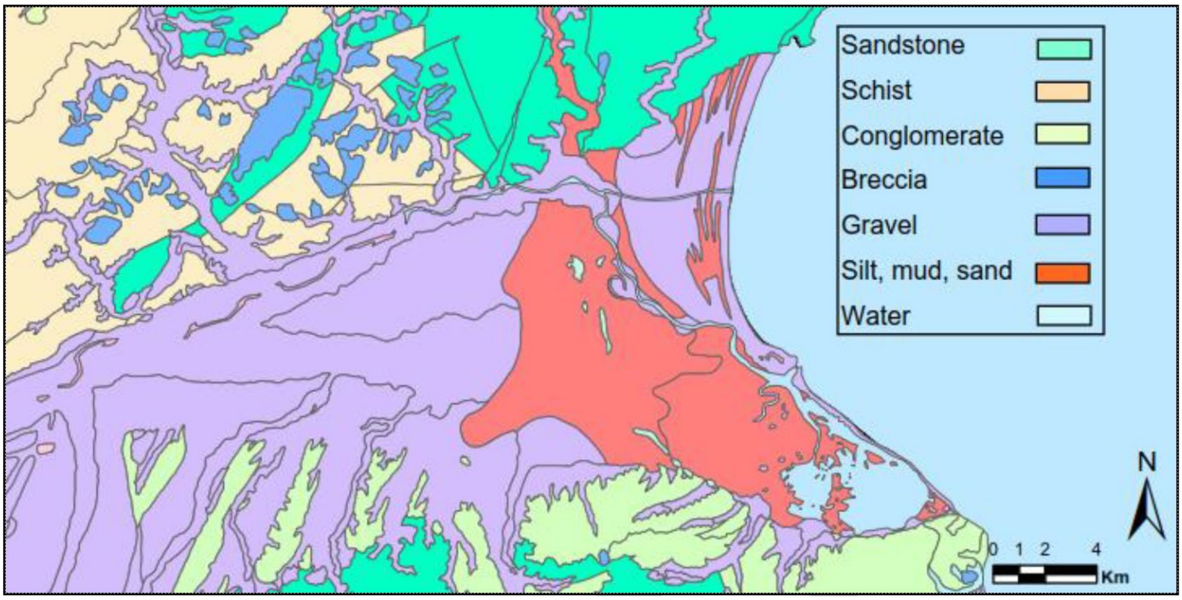

Figure 3 Surface geologic map of the Lower Wairau Plains[2].

[2] Figure adopted from Figure 6 of the *Liquefaction Vulnerability Study: Lower Wairau Plains*, dated May 2021

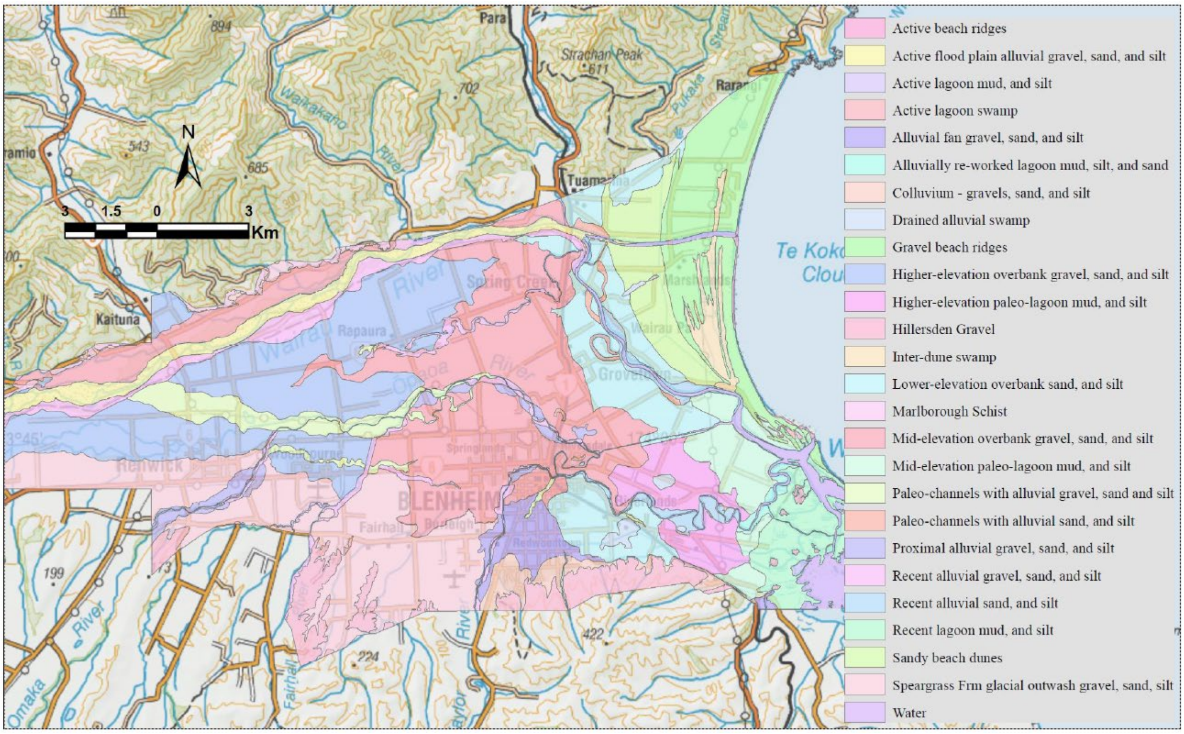

Figure 4 ***Geomorphic map of the Lower Wairau Plains[3].***

Based on the foregoing, the MDC study report provides suggested *"liquefaction vulnerability sub-categories"* based on the nature of the geomorphological formations. The results of this assessment are summarised in Table 1.

[3] Figure adopted from Figure 7 of the *Liquefaction Vulnerability Study: Lower Wairau Plains*, dated May 2021

Table 1 ***Summary of liquefaction vulnerability sub-categories for the Lower Wairau Plains based on geology and geomorphology[4].***

Geomorphological Unit	Surface Geology	Formation Type	Liquefaction Vulnerability Category	Sub-category Based on Geomorphology
Active flood plain alluvial gravel, sand and silt	Holocene River deposits dominated by gravel	Rapaura Formation	Liquefaction Damage Possible	Less susceptible
Alluvial fan gravel, sand and silt	Holocene River deposits dominated by gravel	Rapaura Formation		Less susceptible
Inter-dune swamps	Holocene silty deposits with sand, gravel and peat	Dillons Point Formation		More susceptible
Drained alluvial swamps	Fine sand grading to silts	Dillons Point Formation		More susceptible
Active lagoon swamp	Holocene aged estuary deposits mainly consist of silts with peat and sand	Dillons Point Formation		More susceptible
Active lagoon mud and silt	Holocene silty deposits with sand, mud and peat	Dillons Point Formation		More susceptible
Paleo-channels with alluvial gravel, sand and silt	Holocene alluvial deposits with sand, gravel, and silt	Rapaura Formation		More susceptible
Proximal alluvial gravel, sand and silt	Holocene alluvial deposits with sand and silt	Rapaura Formation		More susceptible
Lower-elevation overbank gravel mud and silt	Holocene alluvial deposits with gravel, sand and silt	Rapaura Formation		More susceptible
Mid-elevation overbank gravel, sand and silt	Holocene alluvial deposits with gravel, sand and silt	Rapaura Formation		Less susceptible
Mid-elevation paleo-lagoon mud and silt	Holocene river deposits consist of silts, mud and peat.	Dillons Point Formation		More susceptible

[4] Table adopted from Table 3 of the *Liquefaction Vulnerability Study: Lower Wairau Plains*, dated May 2021

Geomorphological Unit	Surface Geology	Formation Type	Liquefaction Vulnerability Category	Sub-category Based on Geomorphology
Higher-elevation overbank gravel, sand and silt	Dominated by gravels towards the east coast and silty towards west.	Dillons Point Formation	Liquefaction Damage Possible	Less susceptible
Higher elevation paleo-lagoon mud and silt	Holocene River deposits dominated by Silts	Dillons Point Formation		Less susceptible
Active beach ridges	Holocene shoreline deposits dominated by gravel	Dillons Point Formation		Less susceptible
Sandy beach ridges	Holocene shoreline deposits dominated by gravel	Dillons Point Formation		Less susceptible
Gravel beach ridges	Holocene shoreline deposits dominated by gravel	Dillons Point Formation		Less susceptible
Alluvially re-worked lagoon mud, silt and sand	A mixture of river deposits with swamp deposits. Mostly silty with the inclusion of sand and gravel	Dillons Point Formation		Less susceptible
Recent alluvial gravel, sand and silt	Holocene alluvial deposits with gravel, sand and silt	Rapaura Formation		More susceptible
Recent alluvial sand and silt	Holocene alluvial deposits with sand and silt	Rapaura Formation		More susceptible
Drained alluvial swamp	Holocene alluvial deposits consist of silts, mud and peat	Rapaura Formation		More susceptible
Speargrass Formation glacial outwash gravel, sand and silt	Late Pleistocene river deposits with gravel, sand and silt	Speargrass Formation		Less susceptible

6.6 BASIN GEOLOGICAL MODEL

The MDC study report also presented interpretations of a geological basin model for the Lower Wairau Plains, so as to better determine what deposits are present both across the plains and the variation of these deposits with depth.

White et al. (2016) developed a detailed geologic model of the basin beneath the Wairau Plains to better understand groundwater-surface interactions. Observations of lithology from 1,165 wells were used to develop a continuous 3D distribution of de-facto probabilities for the occurrence of three sediment classes: gravel, sands and clays. This model was used in the MDC study to provide a more detailed representation of the stratigraphy across the Wairau Plains, as related to the potential for liquefaction manifestation.

As the model identifies the presence of different sediment classes, it is used to differentiate between locations where surface gravels would dominate the potential surface manifestation severity and those where sands would dominate. A lack of surface manifestation of liquefaction, due to the presence of an upper non-liquefiable crust, is well documented by Ishihara (1985), Youd and Garris (1995), and Bouckovalas and Dakoulas (2007).

The research indicates that a non-liquefiable *"crust"* thickness of 5m would act to prevent surface manifestation. For the Wairau Plains case, there is the potential for young, looser surface gravels to liquefy, so here the depth to the base of the surface gravels is used to differentiate between locations where the underlying sands and silts could liquefy and control performance and those where the gravel could liquefy and control performance.

The depth to the base of the surface gravel, above sand and silt deposits, at each location is controlled by either the base of the gravel layer or the water table depth.

Figure 28, presented in the MDC study report, indicates the parts of the Lower Wairau Plains where the effective non-liquefiable crust is inferred to be greater than 5 m.

6.7 SUMMARY OF LIQUEFACTION VULNERABILITY

6.7.1 General

Liquefaction vulnerability category maps (for Level A and Level B assessments) are presented in the MDC study report.

The age of deposits across the Wairau Plains and the relatively shallow depth to groundwater means much of the area is classified as "*Liquefaction damage is possible*".

The surrounding hills are classified as *"Very low liquefaction vulnerability"* and areas with deeper groundwater, along the edge of the plains, are classified as *"Liquefaction damage is unlikely"*.

Some alluvial deposits in the plains, dominated by stiff gravel, are classified as *"Low liquefaction vulnerability"*, with investigations in this area suggesting an absence of pockets of loose sandy deposits.

Figure 30, presented in the MDC study report, provides a suggested liquefaction vulnerability category map for the Lower Wairau Plain study area, and has been determined based on the assessment of all of the available data (which is discussed in the MDC study report). This map is based on a Level B assessment, as defined by the 2017 MBIE guidelines.

Figure 30 of the MDC study report, is shown below, and is identified as Figure 5, for the purposes of these guidelines.

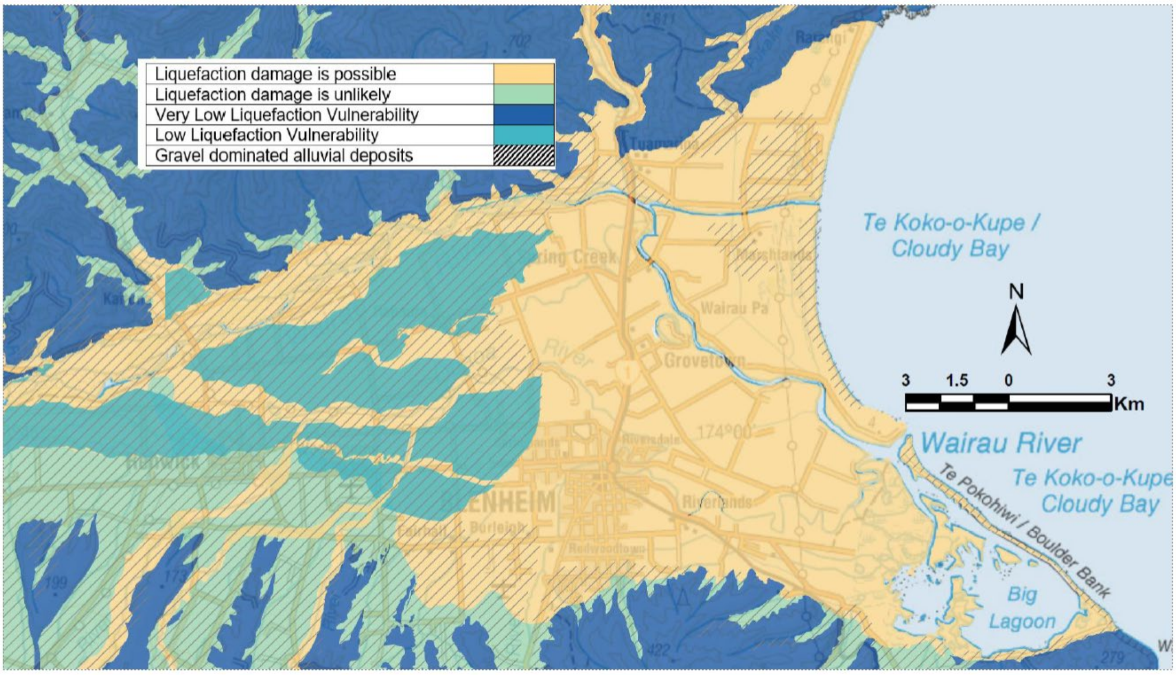

Figure 5 ***Summary of liquefaction vulnerability category map for the Lower Wairau Plains and base of gravel greater than 5 m[5].***

6.7.2 Lateral Ground Spread

The MDC study report indicates that liquefaction induced lateral ground spread can cause disproportional damage to urban infrastructure, over and above, that from the vertical settlement effects of liquefaction alone. However, lateral ground spreading, in particular the nature and quantum of lateral ground strains, is very difficult to reliably predict, as the theoretical analyses is a highly complex process, dependent upon multiple variables, including:

- The elevation difference between the base of the free-face (i.e., a road cutting, old terrace or a riverbank) and the elevation of the land at the point of interest;
- The distance (L) from the base of the free face to the point of interest;
- The earthquake ground motions including Peak Ground Accelerations (PGA) and earthquake magnitude (M_w);

[5] Figure adopted from Figure 30 of the *Liquefaction Vulnerability Study: Lower Wairau Plains*, dated May 2021

- The thickness, relative density and location of liquefying layers within the soil profile; and
- Additional topographic and geological boundary conditions.

When considering the potential for lateral spreading adjacent to a free-face, the *Planning and engineering guidance for potentially liquefaction-prone land*, dated September 2017, prepared by MBIE, notes that *"It is less likely (but not impossible) for lateral spreading to occur if there is no liquefied soil within a depth of 2H of the ground surface (where H is the height of the free-face"*.

Severe lateral spreading was observed as a result of the Kaikōura earthquake along the Opaoa River which greatly impacted the adjacent land and the cross-sectional characteristics of the river. The latter was identified by MDC and locals through observing flooding in the sections of the Opaoa River, close to the Blenheim Township, during smaller rainfall events than those prior to the Kaikōura earthquake.

Ogden (2018) identified, by thorough investigation of lateral spreading manifestations in the region and predictions, that for lateral-spreading no one measurement or prediction tool can be used to comprehensively model or estimate the potential effects. The study by Ogden (2018) also highlighted that the simplified liquefaction procedures provided a reasonable estimation of liquefaction vulnerability, evaluated against observations in relatively uniform profiles comprising fine-grained non-plastic deposits. However, there was a substantial proportion of sites at which there was computed over-prediction from the simplified methods. Potential inaccuracies in the ground motion and groundwater surfaces that were developed for the region could account for a small proportion of the false positive predictions. However, the largest source of over-prediction was found at sites with significant degrees of interlayering present in the subsurface profile.

In the absence of evidence to provide region specific guidance, the MDC study report suggests that there should be particular attention given to the potential for liquefaction-induced lateral ground spread occurring for land located within a horizontal distance of 100 m of a free-face (with a height less than 2 m), and for land located within a horizontal distance of 200 m of a free-face (with a height greater than 2 m).

7.0 LIQUEFACTION POTENTIAL ASSESSMENT

7.1 GENERAL

As discussed in Section 2.0 of this document, some of the key objectives of these guidelines are to:

(i) Promote consistency of approach to assessing liquefaction risk in the whole Marlborough region,

(ii) Provide sound guidelines for the determination of the theoretical liquefaction triggering potential of soils, due to seismic loading, to support rational foundation design, which are informed by the latest research and the MBIE Guidelines Modules (1 to 6)

Sections 7.0 to 14.0 inclusive of these guidelines, provides guidance, so as to achieve the foregoing objectives.

Module 3: Identification assessment and mitigation of liquefaction hazards, suggests a three-step process for the liquefaction assessment of sites, generally being:

(i) Step 1: Assessment of liquefaction susceptibility,

(ii) Step 2: Triggering of liquefaction,

(iii) Step 3: Consequences of liquefaction.

It is recommended that liquefaction potential assessments of soils be generally undertaken using the methods suggested by the Module 3 guideline.

7.2 ASSESSMENT OF LIQUEFACTION SUSCEPTIBILITY

The following soils are generally considered to be susceptible to liquefaction:

(a) Young (typically Holocene age) alluvial sediments (typically fluvial deposits laid down in a low energy environment) or man-made fills,

(b) Poorly consolidated/compacted sands and silty sands,

(c) Areas with a high groundwater level.

As discussed in Sections 6.5, 6.6 and 6.7 of this document, the MDC study report provides information relating to the relative liquefaction susceptibility of the various soils for different parts of the Lower Wairau Plains area. The information provided in the MDC study report has been assimilated, in order to determine recommended Liquefaction Investigation Zones (LIZ) for the Lower Wairau Plains. The purpose of the Liquefaction Investigation Zones, is to provide guidance as to the level/nature of geotechnical investigation and appraisal works that would be expected to be undertaken, for different parts of the Lower Wairau Plains, in order to assess the liquefaction potential of the soils.

The Liquefaction Investigation Zones for the Lower Wairau Plains, and their definitions, are provided in Table 2 below:

Table 2 ***Recommended Liquefaction Investigation Zones***

Liquefaction Investigation Zones (LIZ)	General Development	Subdivisions-creating 3 or more lots
LIZ A	Requires detailed liquefaction triggering analyses using deep investigation data (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils	Requires detailed liquefaction triggering analyses using deep investigation data (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils
LIZ B	Requires detailed liquefaction triggering analyses using deep investigation data (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils, and a lateral ground spread assessment	Requires detailed liquefaction triggering analyses using deep investigation data (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils, and a lateral ground spread assessment
LIZ C	Requires a desktop study and shallow investigation (as a minimum comprising hand augered boreholes and/or machine excavated test pits). May require detailed liquefaction triggering analyses, if potentially liquefiable soils are encountered, such as saturated silty sands, sandy silts and sands (depends on results of desktop study and shallow investigation works).	Requires detailed liquefaction triggering analyses using deep investigation data* (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils
LIZ D	Requires desktop study and shallow investigation (as a minimum comprising hand augered boreholes and/or machine excavated test pits)	Requires detailed liquefaction triggering analyses using deep investigation data* (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils
LIZ E	Requires desktop study	Requires desktop study
LIZ F	Area located outside the scope of the MDC *Liquefaction Vulnerability Study: Lower Wairau Plains* (dated May 2021). Requires, as a minimum, a desktop study and shallow investigation (as a minimum comprising hand augered boreholes and/or machine excavated test pits). May require detailed liquefaction triggering analyses, (depends on results of desktop study and shallow investigation works).**	May require detailed liquefaction triggering analyses using deep investigation data* (such as CPT sounding data), in order to determine the theoretical liquefaction potential of the soils. It is likely, however, that proposed large subdivisions may not require deep ground investigation (due to the geological conditions).**

* See Section 8.4 of these Guidelines, regarding suitable deep ground investigation methods within gravelly soils.

** It is likely that the low lying coastal parts of Havelock and Picton/Waikawa could potentially be underlain by liquefiable soils, and that detailed liquefaction triggering analyses could therefore be required for sites in these areas.

Further guidance relating to "shallow investigations" and "deep investigations" is provided in Section 8.0 of these guidelines.

It should be noted that the foregoing recommended investigation works are only considered relevant for the determination of one potential geotechnical hazard, that being liquefaction potential. It is likely that other types of investigation and appraisal works will be required for sites within these zones, in order to assess the risk of other potential geotechnical hazards, which are not addressed in this document.

The approximate location and extent of the various Liquefaction Investigation Zones for the Lower Wairau Plains are shown on Figure 6 and Figure 7.

The Liquefaction Investigation Zones, have been determined using the following justification/rationale:

LIZ A - As indicated on Figure 5, area determined by the MDC study report as being "*Liquefaction damage is possible*", and, as indicated in Table 1, having a liquefaction vulnerability sub-category of "*more susceptible*".

LIZ B - As indicated on Figure 5, area determined by the MDC study report as being "*Liquefaction damage is possible*", and within a horizontal distance of approximately 200m of the Lower Wairau River or the Opaoa River, or within a horizontal distance of 100m of Taylor River, Fairhall River, Omaka River, the Upper Wairau River, Doctor Creek, Roses Overflow, Murphy's Creek, Fulton's Creek, Old Fairhall Stream and Casey's Drain.

LIZ C - As indicated on Figure 5, area determined by the MDC study report as being "*Liquefaction damage is possible*", and, as indicated in Table 1, having a liquefaction vulnerability sub-category of "*less susceptible*".

LIZ D - As indicated on Figure 5, area determined by the MDC study report as being "*Liquefaction damage is unlikely*", and "*Low Liquefaction Vulnerability*".

LIZ E - As indicated on Figure 5, area determined by the MDC study report as being "*Very Low Liquefaction Vulnerability*".

LIZ F - Area located outside the scope of the MDC *Liquefaction Vulnerability Study: Lower Wairau Plains*, dated May 2021).

Figure 8 summarises the process for determination of suitable geotechnical investigations for liquefaction potential assessment purposes. It should be noted, for simplification of presentation purposes, that the LIZ B zone is not shown on Figure 8.

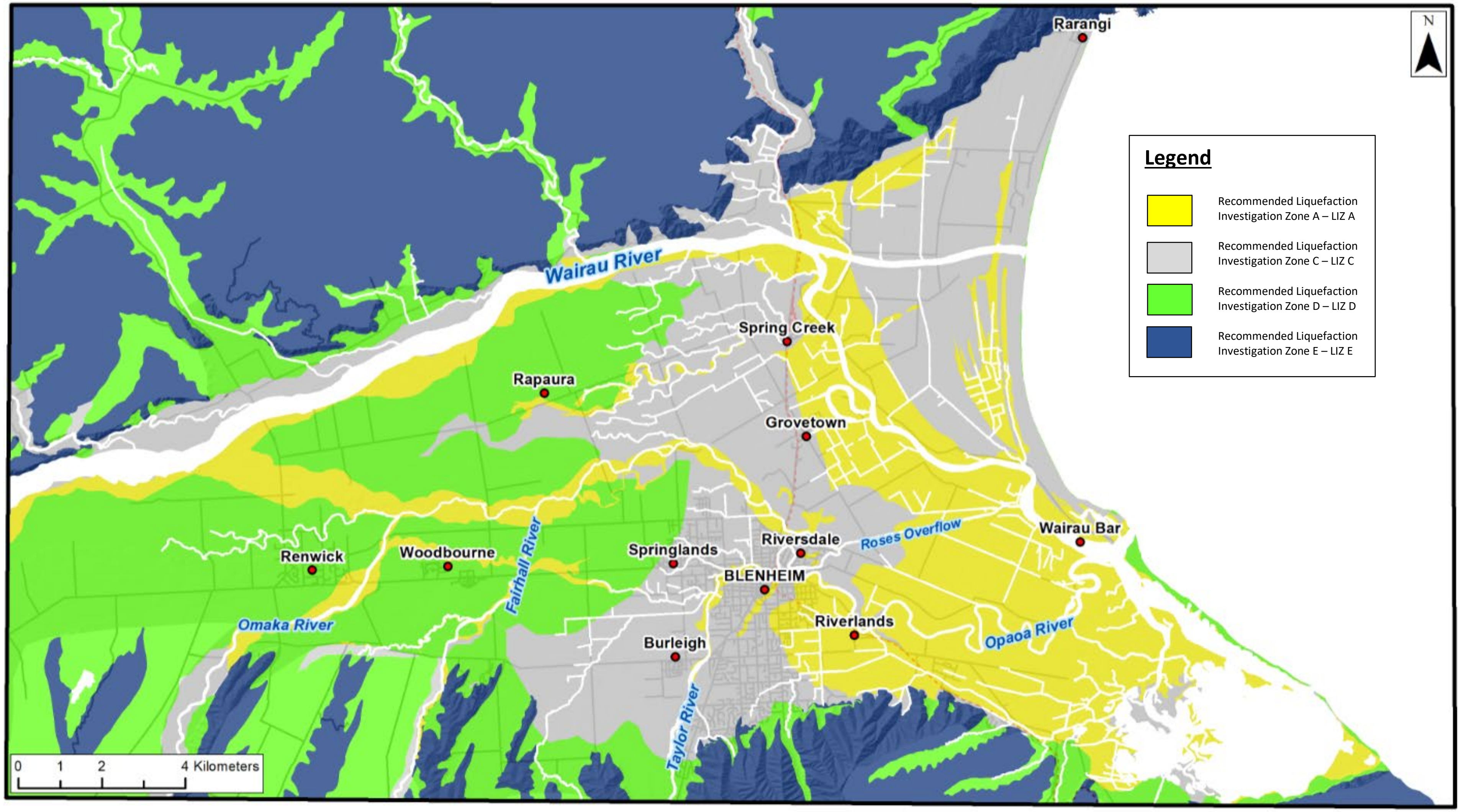

Figure 6 Liquefaction Investigation Zones (LIZ) for the Lower Wairau Plains – Overall Plan.

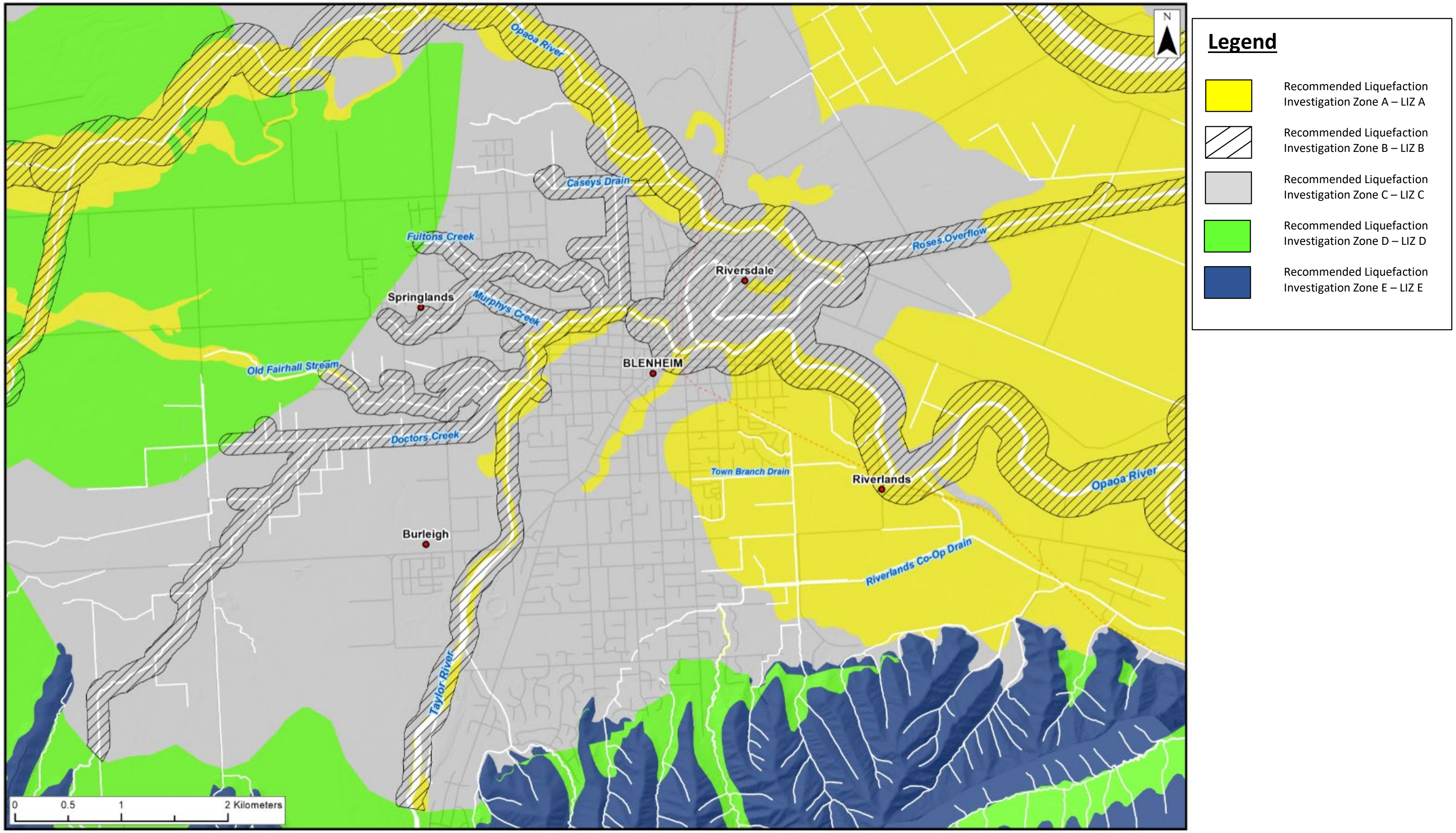

Figure 7 Liquefaction Investigation Zones (LIZ) for the Lower Wairau Plains – Blenheim Area.

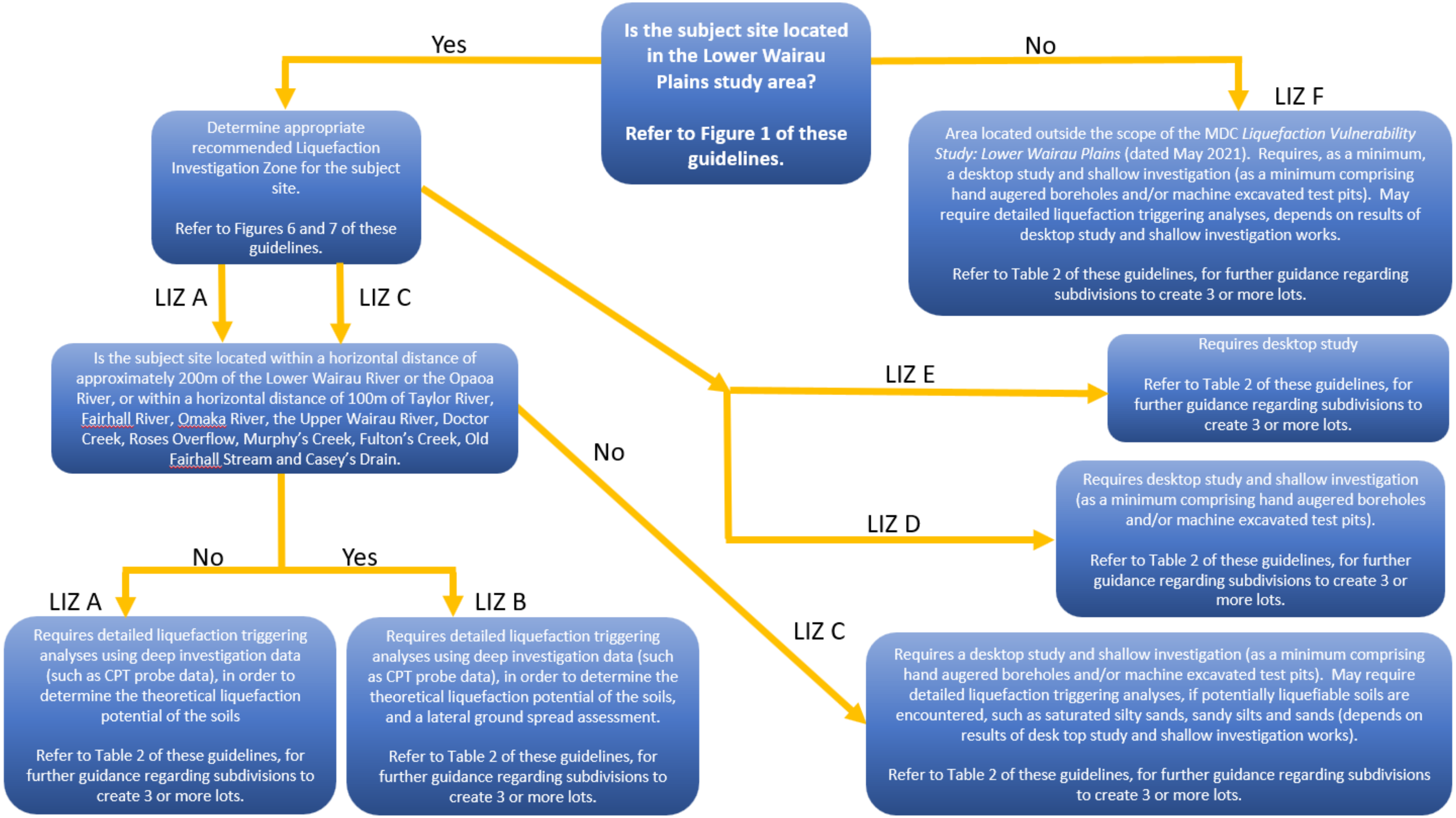

Figure 8 ***Flowchart: Determination of suitable geotechnical investigations for liquefaction potential assessment purposes (general).***

8.0 SITE ASSESSMENT

8.1 INTRODUCTION

The purpose of this section, is to provide some guidance/clarity as to the definitions and expectations for the various levels of assessment works, ranging from desktop study to deep ground investigations.

8.2 DESKTOP STUDY

As indicated in Table 2, sites located within LIZ C, LIZ D, LIZ E and LIZ F, require a desk top study, as part of the liquefaction susceptibility assessment for these areas. That being said, it is recommended that an initial desk top study be undertaken for any site, in order to identify likely geotechnical hazards affecting the site (this should also include LIZ A and LIZ B sites).

Before any site investigation works are undertaken, it is vital that the Geoprofessional undertakes a thorough desk top study of the subject site, in order to gain a broad understanding of the site and its vulnerability to any liquefaction related damage. This study will guide the Geoprofessional when they assess the level of geotechnical investigation works required to gain the required understanding of the risk posed to the subject site, from liquefaction related damage.

A poorly performed initial desk top study may result in the Geoprofessional not correctly assessing the level of geotechnical investigation works required, to gain sufficient understanding of the vulnerability of the subject site to liquefaction related damage, which can result in further investigation works being required at later stages. On the contrary, a poorly performed desktop study can also result in excessive geotechnical investigation works being performed, which are not appropriate for the level or importance of development taking place on the subject site. Thus, it is important that the Geoprofessional allows for due time to undertake the desktop study, prior to any field investigation works being performed.

Note that in the following list, emphasis is placed on the word “relevant”. It is important when performing a desktop study that information that is not valid or irrelevant to the subject site is not relied upon when the Geoprofessional makes their assessment of the subject site. An example of such a situation would be if Cone Penetration Test (CPT) sounding data was relied upon, when the test was performed at a significant distance from the subject site and thus in a different geomorphic zone. The Geoprofessional could be misled by the results of this CPT sounding when undertaking their assessment of the subject site and the resultant site investigation works required. It is expected that the Geoprofessional will use their professional judgement when determining whether existing information is “relevant”.

It is expected that this study would include but is not limited to the following:

- A study of relevant geological maps to develop an initial understanding of the origin and type of soil likely to be encountered at the subject site,

- A study of the MDC *"Liquefaction Vulnerability Study: Lower Wairau Plains"* (dated January 2021),

- Developing an understanding of the proximity of the site in relation to faults,

- A study of the New Zealand Geotechnical Database in order to find relevant existing geotechnical information,

- A study of historical and current aerial photos for the subject site,

- A study of any relevant previous water bore or well records.

As part of the desktop study, it is recommended that the Geoprofessional consider undertaking a site walkover. The site walkover will help the Geoprofessional to gain a better understanding of the on-site conditions; and obtain a level of understanding that cannot be gained from aerial imagery of the site.

8.3 SHALLOW GROUND INVESTIGATIONS

It is expected, as part of a liquefaction assessment for a given site, that the subsurface conditions are confirmed by a shallow ground investigation. The amount of shallow ground investigation tests undertaken should be sufficient for the Geoprofessional to obtain a good understanding of the nature and consistency of the surficial subsoil condition across the subject site and sufficient for the Geoprofessional to obtain a tactile appreciation of the subsoil conditions.

The shallow ground investigation results are expected to complement the deep ground investigation results, if undertaken.

It is expected that any shallow ground investigation would consist of machine excavated test pits or hand augered boreholes (or a combination of both). The soil profiles obtained from these test positions should be logged in accordance with the methods described in the NZGS Field Description of Soil and Rock- Guideline for the field classification and description of soils and rock for engineering purposes (2005).

It is expected, where practicable, that the in-situ undrained shear strength of the soils, using hand held shear vane testing equipment, should be measured down the soil profile, for cohesive soils.

It is expected that Dynamic Cone Penetrometer (DCP) scala testing may also be undertaken, in certain soils (typically cohesionless soils), in order to provide additional subsoil information. It should be noted, however, that the DCP test is a crude test. The use of the DCP test alone will not reliably determine the nature and consistency of soils, and in particular, will not identify important geotechnical issues, such as buried topsoil, peat layers, fill, expansive soils etc. Therefore, it is recommended that DCP testing is always accompanied by boreholes or test pits.

8.4 DEEP GROUND INVESTIGATION

A variety of different testing procedures currently exist within the engineering community, which are able to provide “deep ground” subsoil information for the purpose of assessing the liquefaction triggering risk of soils.

These different methods are described in detail in *Module 2: Earthquake geotechnical engineering practice*. It is recommended that Geoprofessionals familiarise themselves with the information provided in Module 2.

The deep ground investigation methods which are commonly available in New Zealand are the Cone Penetration Test (CPT) and the Standard Penetration Test (SPT).

The SPT is commonly available in New Zealand, however, the usage of the SPT for determining the theoretical liquefaction triggering potential of soils is steadily declining in the engineering community, due to the prevalence and wealth of research that has taken place in recent years in relation to the CPT.

The SPT has the following disadvantages when compared to the CPT:

- It is necessary to undertake a machine borehole investigation, in order to enable the SPT testing to be undertaken, which is typically more expensive than CPT testing
- Test depth intervals are widely spaced, resulting in a non-continuous subsoil profile
- The SPT generally has poor repeatability, when compared to the CPT sounding
- The SPT energy needs to be measured for every test in order to validate results.

Due to these disadvantages, usage of the SPT as a tool to determine the theoretical liquefaction triggering potential of soils is discouraged, and where possible, it is recommended that data obtained from a Cone Penetration Test (CPT) sounding be used.

If the SPT is to be relied upon for determining theoretical liquefaction triggering potential, then the results should be carefully interpreted and corrected according to the recommendations of Seed et al. (1985), as summarised in Youd et al. (2001) and Idriss and Boulanger (2008).

The Cone Penetration Test (CPT), using an electronic cone (preferably CPTU where pore water pressure is measured), is the preferred in situ test procedure for determining the theoretical liquefaction triggering potential of soils, because of its sensitivity, repeatability, and ability to provide continuous profiling and to detect thin strata. Some CPT rigs are also able to recover soil samples, using push-in devices.

The disadvantage of a CPT sounding, is that, at some sites, the CPT sounding may not be able to be progressed through dense gravel soils, and therefore the CPT sounding may not be able to be progressed to the desired depth. It may be necessary for some sites, and for some projects (as the level of subsoil information can depend on the nature of the development), that machine borehole and/or machine excavated test pit investigations may be required to be undertaken, in order to determine the depth and nature of gravel soils and the groundwater levels.

The CPT sounding is a sophisticated investigation tool. If the investigation is not undertaken by an appropriately qualified and experienced CPT operator, in accordance with the required testing methodologies, then there is a risk that the data provided could be unreliable. It is therefore recommended that only CPT data provided by an experienced CPT operator, and testing undertaken in accordance with the latest version of ASTM D 5778 testing standards, be relied upon.

Alternative deep ground investigation methods are also available, such as geophysical testing. Simplified procedures for assessing liquefaction triggering, based on shear wave velocity (V_s) measurements have been developed, however a study in Christchurch (EQC study) indicates that the results of the V_s-based simplified procedures did not fit well with the field observations of liquefaction manifestation. The use of the V_s-based simplified procedure, as the only means of determining the liquefaction triggering is therefore not recommended.

8.5 INDICATIVE SPATIAL DENSITY OF DEEP GROUND INVESTIGATIONS

There are various guidelines and papers available, which provide opinions as to the minimum spatial density of deep ground investigation required, for various purposes.

This document is careful not to provide 'black and white' rules, as to the level of deep ground investigation required for sites in the Marlborough region, as the quantum of testing is dependent on factors such as:

(i) The nature of the geology of the site

(ii) The quantum and reliability of existing deep ground investigation data in close proximity to the site

(iii) The nature and complexity of the proposed development

(iv) The purpose of the geotechnical investigation and reporting i.e., is the investigation in support of an application for a plan change or a building consent.

This guidance document is intended for use by appropriately qualified and experienced Chartered Geoprofessionals, who are expected to have the ability to determine the nature and quantum of field investigation works required to satisfy themselves, as to the nature, consistency and liquefaction potential of the site soils. However, in order to provide some "broad-brush" indication as to expected spatial density of deep ground investigations, and to encourage consistency amongst Geoprofessionals, the following indicative spatial density of deep ground investigation is provided in Table 3.

Table 3 ***Indicative Spatial Density of Deep Ground Investigation***

Purpose of Investigation/Reporting	Average Investigation Density	Minimum Total Number of Test Positions
Plan Change	0.1 to 1 per Ha	1 if area 0.25 to 1 Ha 3 if area > 1 Ha
Subdivision Consent	1 to 4 per Ha	1 if area < 0.25 Ha 2 if area 0.25 to 1 Ha 5 if area 1 Ha to 5 Ha
Building Consent	2 to 30 per Ha	1 within or close to the proposed building footprint

Notes:

(1) It should be noted that it is unlikely that any deep ground investigation, for the purposes of determining the liquefaction potential of sites, would be required for sites within the LIZ E zone

(2) It is possible that existing deep ground investigation is available for the site (on the NZGD). This existing data, if available, could also be used as part of the assessment of the site soils

8.6 GROUNDWATER

The groundwater level that is used for liquefaction analysis should be based upon the water table measured at the locations of shallow investigation test positions, within the subject site. Care should be taken, when measuring groundwater levels in cohesionless (impermeable) soils, as the groundwater may require time to equilibrate following drilling, i.e., the groundwater level measured immediately following drilling may be artificially low, as the groundwater will rise to the phreatic surface (once the groundwater has had time to equilibrate).

In most cases, it would be expected that the groundwater level measured at the locations of shallow investigation positions will be more accurate than that measured by a pore pressure sensor of a CPT cone.

In the absence of any available shallow investigation data, the Geoprofessional should refer to any relevant groundwater data, obtained in their desk top study. Drilling log data can be useful in determining likely groundwater conditions within an area.

8.7 LABORATORY TESTING

Common soil laboratory testing methods and descriptions are provided in detail in *Module 2: Earthquake geotechnical engineering practice.*

The susceptibility of soils to liquefaction triggering is a function of the fines content of the soils. The simplified procedure B&I (2014) method for determining liquefaction triggering contains a fitting parameter called C_{FC}. The C_{FC} parameter adjusts the empirical relationship between the predicted fines content (FC) of a soil (predicted from the CPT sounding data) and the soil behaviour type index (I_c).

Laboratory testing can be undertaken on disturbed soil samples to more reliably determine the fines content of soils, and to more reliably assess the liquefaction triggering potential of soil layers.

For high risk/high consequence projects, it is recommended that CPT testing should be complemented by drilling and soil sampling of potentially problematic soils to verify the I_c correlations with FC (or make FC correlations manually in the analyses).

The recommended values of the C_{FC} and I_c parameters, that should be assumed for the theoretical liquefaction triggering analyses in the Marlborough region, are discussed in Section 11.2 of these guidelines.

9.0 COMMENTS REGARDING SUBDIVISIONS

For proposed subdivisions (residential and industrial), which will create 3 or more lots, it is recommended that site specific deep ground investigation be undertaken, in order to determine the nature and consistency of the soils, for liquefaction assessment purposes. Unless suitable justification is provided to MDC, this recommendation applies to Recommended Liquefaction Investigation Zones LIZ A, LIZ B, LIZ C, LIZ D and LIZ F. It is likely, in particular, for sites within LIZ F (i.e outside of the MDC study report area), that proposed large subdivisions may not require deep ground investigation (due to the geological conditions).

10.0 TRIGGERING OF LIQUEFACTION

If the results of the assessment of liquefaction susceptibility indicate that the soils underlying the site are likely to be susceptible to liquefaction, then it is recommended that detailed investigation and analyses are undertaken to determine the theoretical liquefaction triggering potential of the site soils.

Module 3: Identification assessment and mitigation of liquefaction hazards, provides guidance on the identification of liquefaction hazards, and also provides details regarding different methodologies for determining theoretical liquefaction triggering.

The Module 3 guideline refers to the methods suggested by *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils,* dated October 2001. The Module 3 guidelines recommends that theoretical liquefaction triggering, when using CPT and SPT data, be determined using the *"simplified procedure"* originally proposed by Seed and Idriss (1971), and as amended by Boulanger and Idriss (2014).

The simplified procedure states that:

FL = CRR/CSR

- where FL = Liquefaction Triggering Factor of Safety

CRR = Cyclic Resistance Ratio (ability of soils to resist liquefaction)

CSR = Cyclic Stress Ratio (seismic demand on soil caused by earthquake)

When FL < 1.0 - Liquefaction is assumed to occur within the soil layer.

Generally, the calculation of the CRR value for a certain soil is determined taking into account the soil type, density and the depth (confinement) of the soil layer.

Generally, the CSR value for a certain soil is determined taking into account the theoretical PGA resulting from an earthquake and the depth (confinement) of the soil layer.

Computer programs are available which can compute the CRR and CSR values for soils using the data obtained from CPT soundings and SPT data.

11.0 THEORETICAL ANALYSIS OF LIQUEFACTION TRIGGERING POTENTIAL AND EXPECTED GROUND SETTLEMENTS

11.1 PEAK GROUND ACCELERATION (PGA) VALUES ASSUMED FOR ANALYSIS

The following design earthquake events should be assessed for the purposes of any theoretical liquefaction triggering analyses:

(a) Serviceability Limit State (SLS) – 25 year return period,

(b) Intermediate Limit State (ILS) – 100 year return period,

(c) Ultimate Limit State (ULS) – 500 year return period.

Module 1: Overview of the Guidelines, indicates that generally, in New Zealand, the unweighted seismic hazard factors and corresponding effective earthquake magnitude presented in the NZTA Bridge Manual (2014) should be used in liquefaction triggering analyses. However, the design seismic loadings obtained using the Bridge Manual are considered to be conservatively low for the Marlborough region, and Geoprofessionals should use the values provided in Table 4 of these guidelines, which have been obtained from a recent study by Cubrinovski et al. (2021).

The theoretical PGA values and corresponding earthquake Moment Magnitudes (M_w) for liquefaction potential assessments for the SLS, ILS and ULS design conditions for the Lower Wairau Plains, are presented in Table 4 of this document.

Module 4: Earthquake resistant foundation design, provides some discussion regarding the use of the Intermediate Limit State (ILS) design earthquake event in liquefaction triggering assessments. Module 4 states the following:

> *"Under verification method B1/NVM1 and NZS 1170.0-2002, there is no requirement to consider earthquake events intermediate between the SLS and ULS levels of shaking, the assumption being that there would be a continuum of performance of the structure between the SLS and ULS limit states (except SLS for IL4 buildings). With liquefaction triggering at a site, however, there may be a pronounced degradation in foundation performance and this is likely to happen at a shaking level which is intermediate between the SLS and ULS earthquakes. Where liquefaction triggering is likely at modest, intermediate return period (eg less than 100 year return period for a building of normal importance) the resulting level of damage may be excessive and inappropriate for such a high likelihood of occurrence."*

Module 4, goes on to state:

> *"Tolerable impact limits for these intermediate cases will depend on the return period. The return period for earthquake shaking required to trigger consequential liquefaction at a site should be calculated, and design measures taken to limit building damage to an appropriate level for that return period."*

Based on the foregoing, it is recommended, when undertaking liquefaction triggering analyses using the B&I (2014) method of analyses, that, in addition to the SLS and ULS design earthquake events, that an "intermediate" design strength earthquake (ILS- 100 year return period) also be analysed.

In order to provide for a robust foundation solution, it is recommended that the 'index' theoretical ground settlement and LSN values, calculated using the earthquake loading parameters for the ILS design earthquake event, also be considered when assessing the theoretical liquefaction potential for sites in the Marlborough region.

Table 4 ***Recommended Design Peak Ground Acceleration (PGA) Values for Assumed Design Conditions.***

Design Condition	Earthquake Return Period (years)	Design Peak Ground Acceleration (PGA) (proportion of gravity acceleration (m/s^2))	Earthquake Moment Magnitude (M_w)
SLS	25	0.12g	6.4
ILS	100	0.26g	6.8
ULS	500	0.52g	7.3

11.2 METHOD OF ANALYSES

11.2.1 General

In order to quantify the liquefaction potential of a site and to subsequently determine suitable foundation solutions, it is recommended that the *'free-field'* theoretical ground settlement *"index number"* is calculated for the site soils, using the following methodology:

(i) assess liquefaction induced settlement only for the upper 10 m of subsoils under SLS, ILS and ULS seismic load conditions,

(ii) the liquefaction triggering analyses should be undertaken using the simplified procedure suggested by Boulanger & Idriss (2014).

It should also be noted that the MBIE Module 3 guidelines also recommends using the Boulanger & Idriss (2014) methodology, for determining theoretical liquefaction triggering.

11.2.2 Fines Content Correlations

The B&I (2014) method uses a fitting parameter called C_{FC} to fit the relationship between the predicted FC (fines content) and the I_c (soil behaviour type index).

B&I (2014) states the following:

> *"The revised CPT-based liquefaction triggering procedure [i.e. the B&I- 2014 methodology] included a recommend relationship and approach for estimating FC and soil classification from the Ic index when site specific sampling and lab testing data are not available. For analyses in the absence of site-specific soil sampling and lab testing data, it would be prudent to perform parametric analyses to determine if reasonable variations in the FC and soil classification parameters have a significant effect on the final engineering recommendation."*

B&I (2014) recommends that in the absence of reliable site-specific fines content data, that the user undertakes a sensitivity analysis in relation to the C_{FC}, by varying the C_{FC} between -0.29 and +0.29.

Further information and discussion relating to the C_{FC} parameter is provided in the MBIE Module 1 guideline.

Lees, et al (2015) used the results of an extensive geotechnical investigation dataset collected following the 2010/2011 Canterbury earthquake sequence to examine the correlations of the liquefaction susceptibility and FC with I_c for the Christchurch soils.

Borehole and CPT data were used to assess the appropriateness of the FC-I_c correlations, presented in B&I (2014), as well as the I_c cut-off threshold. The results of the study indicate, for Christchurch soils, that the default C_{FC} value of 0.0 will generally over-predict liquefaction triggering, and that a C_{FC} parameter of 0.2 is appropriate for Christchurch soils. However, in the absence of any detailed research relating to the correlation of liquefaction susceptibility and FC, it is recommended, for liquefaction assessments in the Marlborough region, that a C_{FC} parameter of 0.0 be assumed for most analyses.

11.2.3 Probability of Liquefaction (P_L)

The P_L parameter presented in the B&I (2014) procedure is defined as the probability of liquefaction triggering at a FL equal to 1.0. For the normal deterministic procedure outlined in these guidelines, B&I (2014) suggests that a P_L parameter of 16% should be used.

It is noted that the MBIE Module 3, for site assessments being carried out for purposes of compliance with the Building Code, also recommends that the normal probability of 16% be used in the liquefaction triggering analysis.

11.2.4 Soil Behaviour Type Index (I_c) Cut-Off

Idriss and Boulanger (2008) classified soils as either sand-like or claylike in their behaviour, where sand-like soils are susceptible to cyclic liquefaction and clay-like soil are not.

Robertson et al (1986) have developed charts, using data obtained from CPT soundings, to identify soil types by predicting the soil behaviour type (SBT). Robertson (1990) recommended that the Q_t-F_r chart was the most reliable in predicting soil type behaviour., where Q_t is the normalised cone resistance and F_r is the normalised cone skin friction.

Jefferies and Davies (1993) and Roberson and Wride (1998) identified that a soil behaviour index I_c, could represent the SBTn zones in the Q_t-F_r chart. The contours of I_c can be used to approximate the SBTn boundaries. Robertson and Wride (1998) had suggested that I_c = 2.6 was an approximate boundary between soils that were either more sand-like or more claylike.

It is recommended that an Ic value of 2.6 be used when undertaking liquefaction triggering analyses in the Marlborough region, using the simplified procedure.

11.2.5 Thin Sand Layer "Transition Zones"

Robertson, Idriss and Boulanger et al recognise that the reliability of CPT based theoretical liquefaction triggering analyses, can be affected by an effect known as the *"thin sand layer"* transition zone. This occurs because the CPT sounding provides readings from a soil influence zone, which is located some distance in front of the cone tip (the influence zone varies with soil types), which can underestimate the cone resistance of sand layers (particularly when sandwiched between soft cohesive soil layers), which can consequently incorrectly estimate liquefaction triggering for some layered sandy soils. Liquefaction triggering analysis that uses uncorrected data may overestimate the theoretical liquefaction induced ground settlement.

Numerically intensive methods such as Boulanger and De Jong are available which attempt to remove the *"noise"* associated with the thin layers and modify the CPT data so as to produce a more realistic CPT profile.

Robertson (2009) provides a simpler method for adjusting for this effect in the CLiq program. The adjustment is based on the rate of change of the Soil Behaviour Type Index (I_c). The method will automatically ignore zones in which the I_c is between two values and also changing at a rate which exceeds a given rate of change.

It should be noted, however, that recent research has shown that there is currently no reliable method for dealing with the problem of thin sand layer 'transition zones', and that Geoprofessionals should therefore be prudent when attempting to allow for the *"thin sand layer"* transition zone, in their analyses. It is recommended that Geoprofessionals use their engineering judgement, particularly when assessing highly layered soils, and that a sensitivity analysis may be warranted for these types of soil profiles.

11.2.6 Summary

In the absence of any further research in the Marlborough region or site-specific laboratory testing, the input parameters, provided in Table 5, are recommended for theoretical liquefaction triggering analyses, using the simplified procedure B&I (2014):

Table 5 ***Recommended Input Parameters for Liquefaction Analyses***

Input parameter	Value adopted	Comments
Design Seismic Loading	See Table 4	See Section 11.1
I_c cut-off	2.6	Recommended value for simplified procedure, for soils in the Marlborough region
Probability of Liquefaction (P_L)	16%	Deterministic value - in accordance with B&I (2014)
FC Fitting Parameter C_{FC}	Assume 0.0	Recommended value for simplified procedure, for soils in the Marlborough region

12.0 QUANTIFING THEORETICAL LIQUEFACTION RISK

12.1 GENERAL

In order to quantify the theoretical liquefaction risk, in order to determine a suitable shallow foundation solution, it is recommended that analyses be undertaken, in order to determine the following for the site soils:

(a) the *'free-field'* theoretical ground settlement *"index number"*

(b) the Liquefaction Severity Number (LSN).

12.2 FREE-FIELD THEORETICAL GROUND SETTLEMENT “INDEX NUMBER”

In order to quantify the liquefaction potential of a site and to subsequently determine suitable foundation solutions, it is recommended that the *‘free-field’* theoretical ground settlement *“index number”* is calculated for the site soils, using the following methodology:

(i) assess liquefaction induced settlement only for the upper 10 m of subsoils under SLS, ILS and ULS seismic load conditions,

(ii) the liquefaction triggering analyses should be undertaken using the simplified procedure suggested by Boulanger & Idriss (2014).

12.3 LIQUEFACTION SEVERITY NUMBER (LSN)

Following the Canterbury Earthquake Sequence (CES), S. van Ballegooy, et al (2013) developed an unweighted assessment methodology, to assess the vulnerability of land to liquefaction-induced damage. The methodology suggests the use of a dimensionless number termed the Liquefaction Severity Number (LSN).

The LSN, is defined as:

$$LSN = 1000 \int \frac{\varepsilon_v}{z} dz$$

- where εv is the calculated post-liquefaction volumetric reconsolidation strain, and z is the depth below the ground surface in metres.

The theoretical value of LSN varies from 0 (representing no liquefaction vulnerability) to more than 100 (representing very high liquefaction vulnerability).

S. van Ballegooy, et al (2013) suggest a range of LSN values, which relate to three categories of expected degree of liquefaction-induced ground damage, namely:

(i) None to minor,

(ii) Minor to moderate,

(iii) Moderate to severe.

The original LSN ‘boundary’ values, suggested by Ballegooy (2013), have been amended by more up-to-date studies, and these have been adopted by the MDC study report, for the Marlborough region.

The suggested range of LSN values for each ground damage category, for the Marlborough region, are presented in Table 6.

The typical consequences at the ground surface, for the various categories presented in Table 6 are described in Table 2.2 of the MBIE guidance document, titled *“Planning and Engineering Guidance for Potentially Liquefaction Prone Land”*, dated September 2017.

Table 6 ***LSN Range – Corresponding to Expected Liquefaction-induced Ground Damage***

LSN	Expected liquefaction-induced ground damage category
< 13	None to minor
13 –18	Minor to moderate
18+	Moderate to severe

The LSN should be calculated over the full depth of the CPT sounding (minimum depth of 10 m soil profile, if possible). Note that, as the depth of the discrete soil layer increases, the influence of that soil layer upon the final LSN value will decrease.

13.0 FOUNDATION TECHNICAL CATEGORY

The MBIE *Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes* document, dated 2012, provides guidance relating to the construction of new foundations on ground susceptible to liquefaction. The principal objective of the 2012 MBIE guidance document is to provide building repair and reconstruction solutions and options that:

(i) are appropriate to the level of land and building damage experienced;

(ii) take account of the likely future performance of the ground;

(iii) meet Building Act and Building Code requirements.

The 2012 MBIE guidance document provides expected future land performance for various *“Foundation Technical Categories”.* The Foundation Technical Categories used for the 2012 MBIE guidelines, and their corresponding future land performance expectation in response to liquefaction are summarised in Table 7.

Table 7 ***Suggested Foundation Technical Categories (Sourced from the MBIE Guidelines – Canterbury, 2012)***

Foundation Technical Category	Future Land Performance Expectation in Response to Liquefaction
TC1 (where confirmed)	Liquefaction damage is unlikely in a future large earthquake
TC2 (where confirmed)	Liquefaction damage is possible in a future large earthquake
TC3 (where confirmed)	Liquefaction damage is possible in a future large earthquake

For the purposes of these guidelines a similar approach has been adopted. Foundation Technical Categories will need to be determined, in order to identify the potential liquefaction characteristics of sites, so as to enable suitable foundation solutions to be provided for sites in the Marlborough region.

The 2012 MBIE guidelines for Canterbury generally used the *'free-field'* theoretical ground settlement *'index'* values, determined using the simplified procedure, to define the various Foundation Technical Categories.

For the purposes of these guidelines, more emphasis will be placed on the LSN value, when determining the Foundation Technical Categories, as the LSN value takes into account the depth of potential liquefiable layers, and the adverse effects of shallow liquefiable layers, and is therefore considered to be more critical than the *'index'* ground settlement number (which applies no 'weighting' to the depth of the layer).

Table 8 provides expected future land performance for the various Foundation Technical Categories, for the Marlborough region.

Table 8 ***Expected Future Land Performance for Various Foundation Technical Categories.***

Foundation Technical Category	Future Land Performance Expectation in Response to Liquefaction	LSN value- in Response to an SLS Strength Earthquake	LSN value- in Response to an ULS Strength Earthquake
TC1	Liquefaction damage is unlikely in a future large earthquake	< 13*	< 13
TC2	Liquefaction damage is possible in a future large earthquake	13– 18**	13 – 18
TC3	Liquefaction damage is possible in a future large earthquake	13- 18	18+

* If the *'index'* theoretical ground settlement value is greater than 25 mm, under SLS design earthquake loading, the site should be considered to be a TC2 site, for foundation design purposes.

** If the *'index'* theoretical ground settlement value is greater than 60 mm, under SLS design earthquake loading, the site should be considered to be a TC3 site, for foundation design purposes.

As discussed in Section 11.1 of these guidelines, in order to provide for a robust foundation solution, it is recommended that the 'index' theoretical ground settlement and LSN values, calculated using the earthquake loading parameters for the ILS design earthquake event, also be considered when assessing the theoretical liquefaction potential for sites in the Marlborough region, and therefore a suitable Foundation Technical Category, for the purposes of determining a suitable 'robust' foundation system for the site conditions.

In the absence of any site-specific geotechnical testing and analyses to determine the theoretical liquefaction triggering potential of site soils, indicative Foundation Technical Categories, for the various Liquefaction Investigation Zones (LIZ) are provided in Table 9. These 'indicative' categories are considered to be useful when considering simple light-weight building extensions, light weight 'simple' residential structures and small residential subdivisions (less than 3 lots).

Table 9 ***Indicative Foundation Technical Categories for Various Liquefaction Investigation Zones (LIZ).***

Liquefaction Investigation Zone (LIZ)	Indicative Foundation Technical Category
LIZ A	Not applicable
LIZ B	Not applicable
LIZ C	TC2
LIZ D	TC2
LIZ E	TC1
LIZ F	Not applicable

The process for the determination of the theoretical liquefaction potential characteristics of soils is summarised in the flowchart in Figure 9.

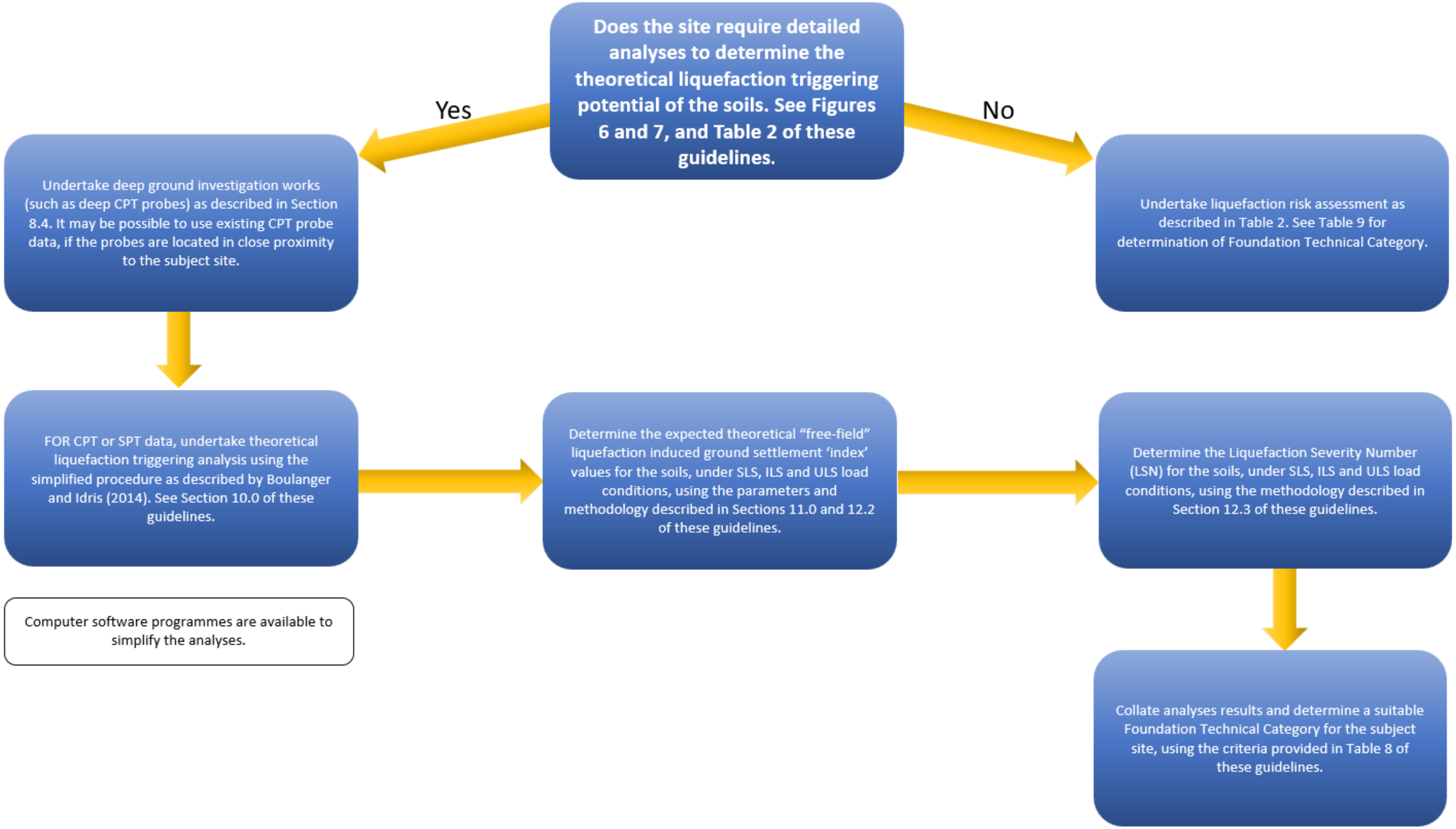

Figure 9 ***Flowchart, Determination of theoretical liquefaction potential characteristics.***

14.0 LATERAL GROUND SPREAD RISK

As discussed in Section 6.7.2 of this document, in the absence of evidence to provide region-specific guidance, the MDC study report suggests that particular attention should be given to the potential for liquefaction-induced lateral ground spread occurring, for land located within a horizontal distance of 100 m of a free-face (with a height less than 2 m), and for land located within a horizontal distance of 200 m of a free-face (with a height greater than 2 m).

When soil liquefies under seismic loading, it loses a significant amount of strength and stiffness. In soil that is located in sloping ground, the effect of the loss of strength and stiffness of the soil when it liquefies can manifest as lateral displacement of the slope. This phenomenon is known as "lateral ground spread" and typically occurs in gently sloping ground that is located in proximity to watercourses, due to the depositional nature of the soils that tend to be located near watercourses.

It is recommended, for sites located in LIZ B, that liquefaction triggering analyses, using deep investigation data (such as CPT sounding data), be undertaken, in order to determine the theoretical liquefaction potential of the soils, and that a lateral ground spread assessment be undertaken. However, Geoprofessionals are expected to address the risk of liquefaction induced lateral ground spread risk for all sites located in close proximity to a free-face, including some sites located outside of the LIZ B area.

It is the experience of the author that there are currently no theoretical analysis methods available that can reliably predict the liquefaction-induced lateral ground strains, expected to occur in soils located in close proximity to a free-face.

Where possible, it is recommended that Geoprofessionals use observational methods to determine the actual performance of sites, in response to recent seismic loading, when assessing the risk of liquefaction induced ground spreading occurring.

Earthquake geotechnical engineering practice, *Module 3: Identification, assessment and mitigation of liquefaction hazards*, provides some comments relating to suitable methods for the prediction of liquefaction-induced lateral ground spread displacements.

There are several empirical methods available for evaluation of lateral ground spread displacements (Youd et al (2002), Tokimatsu and Asaka (1998), Zhang et al (2004)).

The assessment of the risk of lateral ground spread occurring can also be undertaken by means of limit-equilibrium analyses methods, using appropriate soil and excess pore water pressure parameters.

Estimates of lateral ground spread should consider several of the available methods and also consider the range and variability of the predictions and possible extent of the hazard. Cubrinovski and Robinson (2015) provide guidance for a more systematic evaluation of lateral ground spread based on a comprehensive study of liquefaction-induced lateral ground spreads observed to have occurred in response to the 2010/2011 Canterbury earthquake sequence.

15.0 FOUNDATION DESIGN

15.1 GENERAL

One of the objectives of these guidelines is to provide foundation design solutions, for foundations sited on potentially liquefiable ground, which will likely meet the performance standards of the Building Code.

Earthquake geotechnical engineering practice: Module 4; Earthquake resistant foundation design, dated November 2016, provides good guidance for earthquake resistant foundation design of foundations in New Zealand.

Module 4 covers all aspects of earthquake resistant foundation design, including:

(a) Site assessment and foundation selection

(b) Soil/structure interaction

(c) Soil liquefaction

(d) Deep foundations

(e) Ground improvement.

The assessment of the likely effects of liquefaction induced ground deformation, soil/structure interaction under dynamic loading and the estimation of expected differential foundation settlement is a complex problem.

The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes", dated 2012, prepared by MBIE, provides shallow foundation design solutions, for foundations sited on potentially liquefiable ground. These foundation solutions have generally been adopted, for the various Foundation Technical Categories described in Section 13.0 of this document. Details of the recommended shallow foundation design solutions, for the various Foundation Technical Categories, are provided in Table 10.

Table 10 ***Suitable Shallow Foundation Solutions for TC1, TC2 and TC3 Sites.***

Flooring System Type	Foundation Technical Category	Suitable Foundation Design Solution Type
Suspended timber flooring system	**TC1**	Shallow foundation system designed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.
Concrete slab-on-ground flooring system	**TC1**	Shallow foundation system designed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings. (as modified by B1/AS1)
Suspended timber flooring system	**TC2**	Shallow foundation system designed in accordance with the requirements of NZS 3604: 2011, New Zealand Standard, Timber Framed Buildings.
Concrete slab-on-ground flooring system	**TC2**	Shallow 'enhanced' concrete foundation system, such as a concrete waffle slab type foundation system, designed in accordance with the requirements *The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes*, dated 2012, for new buildings constructed in the TC2 zone.
Suspended timber flooring system	**TC3**	Shallow "Surface Structure' as defined by *The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes*, dated 2012 (see Table 11 for more information)
Concrete slab-on-ground flooring system	**TC3**	Shallow "Surface Structure' as defined by *The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes*, dated 2012, with ground improvement (see Table 11 for more information).

It is considered that the shallow foundation design solutions provided in Table 10, if designed and constructed in accordance with the relevant New Zealand Standard Codes of Practice, should appropriately mitigate the risk of any significant liquefication induced differential foundation movement, and are expected to meet the minimum performance standards of the Building Code.

It should be noted that the foregoing recommended foundation solutions are considered suitable for the mitigation of the potential liquefaction hazard. It is likely that other potential geotechnical hazards could affect sites, which may also dictate the configuration and nature of foundation solutions. It is important that the foundation solution selected for any sites is suitable to mitigate the effects of all potential geotechnical hazards.

Further foundation design parameters and recommendations, provided in the *Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes* document, for shallow foundations sited in the TC1 and TC2 zones, are presented in Appendix A of these guidelines.

Further design details for recommended shallow foundation design solutions, for TC3 sites, with varying SLS 'Index' theoretical ground settlement values, are provided in Table 11.

Table 11 ***Suitable Shallow Foundation Options for TC3 Sites.***

Flooring System Type	SLS 'Index' Theoretical Ground Settlement	Suitable Foundation Design Solution Type
Suspended timber flooring system	< 100 mm	Timber floor on enhanced NZS 3604 subfloor **(Type 1 Surface Structure)** or Timber floor over concrete underslab on gravel raft **(Type 2A Surface Structure)**
Suspended timber flooring system	>100 mm	Timber floor over concrete underslab on gravel raft **(Type 2B surface structure)**
Suspended timber flooring system	>200 mm	Specifically designed subfloor grid **(Type 3 surface structure)**
Concrete slab-on-ground flooring system	< 100 mm (pre-treatment)	Shallow *"Surface Structure"* as defined by "*The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes"*, dated 2012, supported on a 'G1d' reinforced gravel raft (see Appendix C).
Concrete slab-on-ground flooring system	> 100 mm (pre-treatment)	Refer to Section 15.3.8 of "*The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes"*, dated 2012.

Further foundation design parameters and recommendations, provided in the *Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes* document, for shallow foundations sited in the TC3 zone, are presented in Appendix B of these guidelines.

It should be noted that the foundation design solutions presented in Tables 10 and Table 11 are considered to be relevant for the following situations:

(1) One to two storey residential structures

(2) Foundations sited on generally level ground, generally at the ground surface (i.e. no deep basements)

(3) Significant liquefication induced lateral ground spread

(4) Building footprints not likely to be affected by:

(i) Soil swell/shrink

(ii) Ground movement associated with slope instability

(iii) Highly compressible soils.

Additional foundation design recommendations may be required, in order to mitigate the potential effects of the foregoing geotechnical hazards.

The design of any shallow foundations for heavy or unusual structures (which are not covered by the foundation solutions presented in Tables 10 and 11), and any deep foundations, i.e., piles, should be undertaken in accordance with the requirements of *"Earthquake geotechnical engineering practice: Module 4; Earthquake resistant foundation design",* dated November 2016, and the relevant New Zealand Standard Codes of Practice. For deep foundations, on sites that could be subject to liquefaction, particular care should be taken to ensure that piles are not founded within, or directly above, potentially liquefiable soil layers.

15.2 GROUND IMPROVEMENT

Ground improvement for mitigating the harmful effects of liquefaction induced ground deformation is a complex problem. It is recommended that Geoprofessionals familiarise themselves with *"Earthquake geotechnical engineering practice: Module 5; Ground improvement od soils prone to liquefaction",* and the information provided in Appendix C of *"The Guidance; Repairing and rebuilding houses affected by the Canterbury earthquakes".*

15.3 DETACHED GARAGES

Detached garages are considered to be an Importance Level 1 (IL1) structures.

IL1 structures have no performance requirements under SLS seismic load conditions, and therefore have no amenity requirements relating to liquefaction induced ground deformation in response to an SLS design earthquake event.

IL1 structures do, however, have performance requirements under ULS seismic load conditions.

It is likely that a *'life safety'* design requirement at Ultimate Limit State (ULS) for a 1/100 year event, should be able to be provided, in most cases on a TC2 site, by a suitably detailed structure on a TC1 type foundation system.

Based on the foregoing, for foundation design purposes relating to a proposed detached garage structure located in a TC2 zone, it is likely that a shallow foundation system suitable for a Foundation Technical Category 1 (TC1) site should be suitable for the site conditions.

Likewise, for foundation design purposes relating to a proposed detached garage structure located in a TC3 zone, it is likely that a shallow foundation system suitable for a Foundation Technical Category 2 (TC2) site should be suitable for the site conditions.

16.0 DATA COLLECTION AND REPORTING

All soils and rock should be logged in accordance with the methods described in the *NZGS Field Description of Soil and Rock- Guideline for the field classification and description of soils and rock for engineering purposes* (2005).

Some guidance on suggested geotechnical reporting is provided in detail in *Module 2: Earthquake geotechnical engineering practice.*

It is recommended, when reporting the results of the liquefaction triggering analyses, that the assumptions used for the analyses be made clear, and that the following information be provided in the report (as a minimum):

(i) Design seismic loading

(ii) I_c cut=off

(iii) Probability of Liquefaction (P_L)

(iv) FC Fitting Parameter (C_{FC})

The New Zealand Geotechnical Database (NZGD) provides a large amount of geotechnical investigation data, and is available online. The database was set up, following the 2010/2011 Canterbury earthquake sequence, and provides a useful resource for undertaking geotechnical assessments for sites. The volume of geotechnical investigation data available on the NZGD (in Canterbury) sometimes enables Geoprofessionals to undertake detailed liquefaction triggering analyses, using existing CPT data obtained from the NZGD (negating the need, sometimes, to undertake site specific CPT testing).

The volume of geotechnical investigation data in Marlborough region, and other parts of the country, is significantly less than that in Canterbury, however, it is hoped, if Geoprofessionals around the country upload their field test results to the NZGD (in particular CPT data and machine borehole logs) that the quantum of data available on the NZGD will increase, which will enable more analyses to be undertaken, using existing geotechnical field information.

Procedures for uploading the data to the NZGD are described in Module 2. It is recommended, as a minimum, that Geoprofessionals upload any CPT data and machine borehole logs to the NZGD, at the completion of their investigation works, so that the Geoprofessionals in the region can benefit from the NZGD (as has been the experience in Canterbury).

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Appendix A

Extracts from the *MBIE Guidance: Repairing and rebuilding house affected by the Canterbury earthquakes*, for design of shallow foundations in the TC1 and TC2 zones

Extract from: Ministry of Business, Innovation and Employment (December 2012, updated May 2018), Repairing and rebuilding houses affected by the Canterbury earthquakes, Version 3, Section 5: New Foundations in TC1 and TC2, pg. 5.1 to 5.22.

5. New foundations in TC1 and TC2

5.1 General

UPDATE: December 2012

This section covers both foundations for new houses and situations where foundations are completely rebuilt for existing houses in the Green Zone on the flat. These foundation solutions are primarily for properties classified TC1 or TC2. Some of these foundation solutions may also be applicable on some sites currently classifed TC3, following site-specific investigation and assessment (refer to section 13.6). Refer to Table 2.3 for guidance on whether a foundation can be relevelled or should be rebuilt.

New foundation options are outlined in sections 5.2 and 5.3, and guidance for specific engineering design is provided in section 5.4. Additional considerations for replacement foundations beneath existing houses are provided in section 5.5. Detailing considerations for services are outlined in section 5.6.

New foundations for the above situations will require a foundation system suitable for the foundation technical category confirmed for the site. The choice of foundation option for TC1 and TC2 will depend on the results of a shallow subsurface investigation (refer to section 3.4.1).

An overview of the process for new foundations on TC1 and TC2 sites is provided in Figures 5.1 and 5.2 respectively.

In TC1, foundation Types A and B can be built as per NZS 3604. Type C foundations will require reinforced concrete slabs as provided in NZS 3604 Timber Framed Buildings, as modified by B1/AS1, which requires ductile reinforcing in slabs: refer to the Ministry's information sheet at www.dbh.govt.nz/seismicity-info

UPDATE: December 2012

For all three foundation types in TC1, the geotechnical ultimate bearing capacity must be greater than 300 kPa in order to use standard foundation details unmodified, without specific consideration of actual building weights, imposed bearing stresses and actual soil strengths. Alternatively, a stiffened raft in accordance with section 5.3 may be used if the geotechnical ultimate bearing capacity is greater than 200 kPa, otherwise a specific engineering design is required. (This will primarily consist of a simple calculation of specifically imposed bearing stresses and actual soil strengths (section section 3.4.1)).

In TC2, new foundations will need to be capable of resisting tension effects from nominal lateral spreading. They must also be capable of accommodating settlement of the ground beneath the house. Options 1 to 5 in this section are considered to be suitable for TC2. Specific information regarding deep pile options is provided in Part C. The deep pile options will require deep geotechnical investigation and specific design.

> Refer to Part C for TC3 foundation options. Specific design will be required for any deep piled raft option or any alternative designs and will need to be undertaken in consultation with a geotechnical engineer.

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Dwellings that require reconstruction because they are a total loss can normally be designed to provide more resilience than existing structures. It is noted that light dwellings are likely to perform better than heavy dwellings. They can be more easily re-levelled or repaired if damaged in a future large earthquake and are likely to undergo lower amounts of settlement. Therefore the **use of light timber or steel framing, light-weight cladding systems and light-weight roofing materials is recommended** wherever possible for rebuilding houses and building new houses, particularly where liquefaction is possible.

UPDATE:
December 2012

Figure 5.1: Overview of process for new foundations on TC1 sites

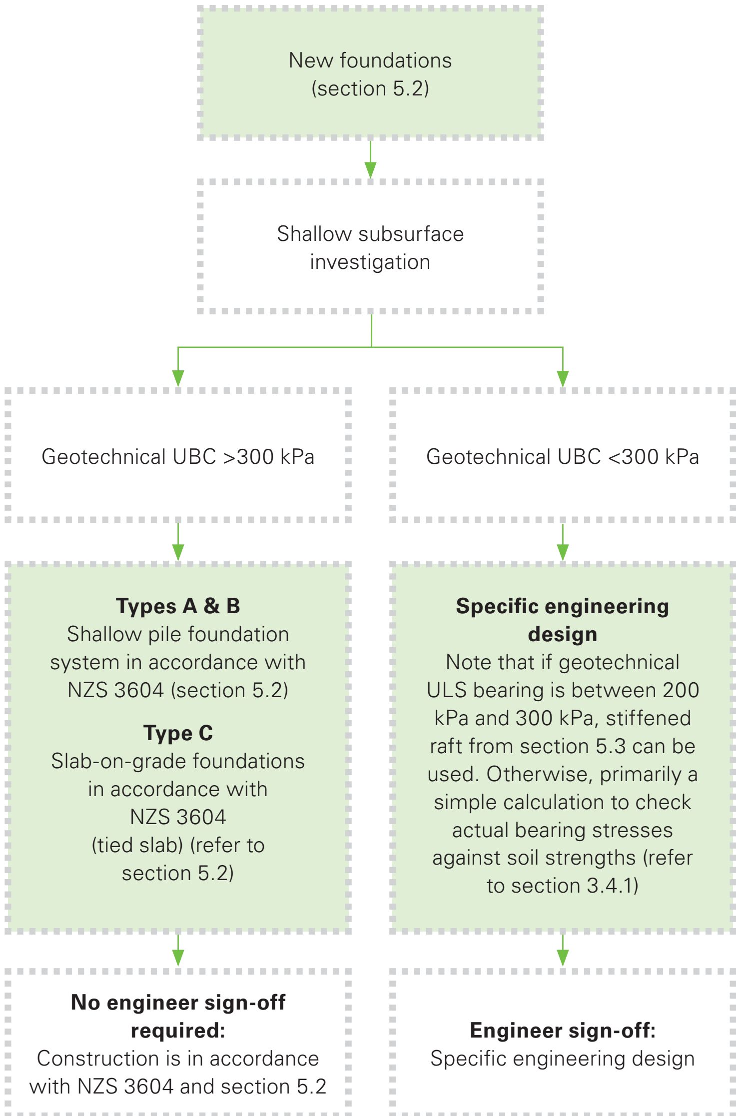
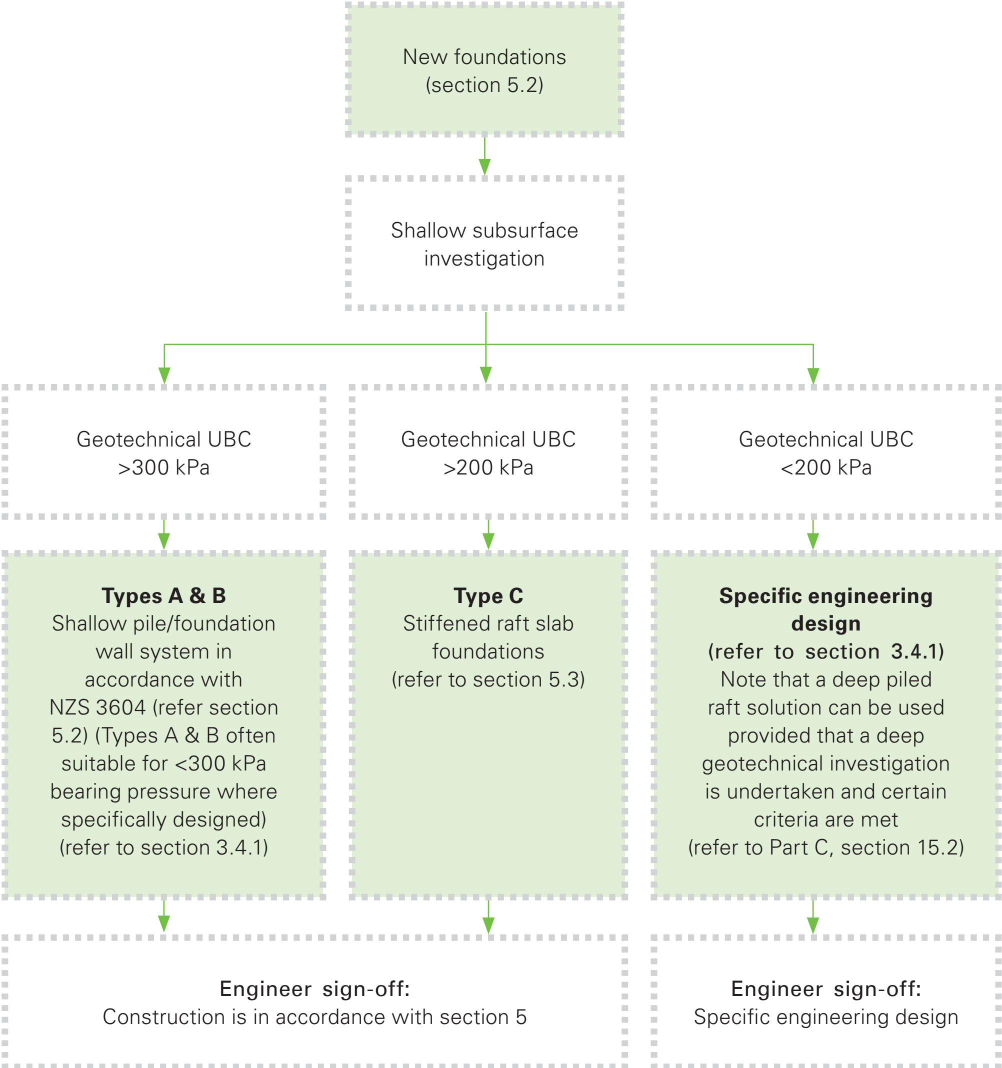


Figure 5.2: Overview of process for new foundations on TC2 sites



UPDATE:
December 2012

The use of NZS 3604 for the design of the superstructure (ie, everything from the ground floor plate up) is acceptable for the construction of any house within the scope of NZS 3604 (ie, the dimensional limitations are adhered to, and the use is limited to Importance Level 2 (AS/NZS 1170.0)).

5.2 Overview of new foundation options

TC1

Type A dwellings within the scope of NZS 3604 could generally be founded on shallow piles, if ground conditions permit, in TC1. While Type B dwellings are now rarely constructed on the flat in Christchurch, they are still suitable for TC1. NZS 3604 Type C foundation options, (with B1/AS1 modifications) are considered suitable.[1]

TC2

A light clad house structure supported fully on short timber or concrete piles (Type A) is considered to be a valid option in TC2. It is the most easily repaired form of dwelling construction. Type B construction is also considered suitable for TC2 areas. Provisions are given in this section.

The principal objectives in designing new concrete slab foundation systems for rebuilding in TC2 ground damaged land should be that any settlements that occur in future earthquakes will be constrained to cope with settlements outlined in Table 3.1. In many areas of greater Christchurch, the 'good ground' provisions of NZS 3604 may not apply, and therefore the concrete foundation and flooring provisions of that Standard should not be used in these areas without specific engineering design input (see section 3.4.1).

Providing stiffened and better-tied-together floor slabs for Type C houses in TC2 areas will reduce hogging or other undue deformation of the slab as a result of future earthquake induced land damage and will enable them to be more readily relevelled.

UPDATE: December 2012

Foundation Options 1 to 4 in this section are considered to provide sufficient stiffness to accommodate the expected future ground movements for TC2 for all but two-storey houses with heavy-weight cladding extending over both storeys. Thickening of the slab in Option 2 will allow its use with heavy-weight (brick venner) cladding and a heavyweight roof. Structure cladding weight limits are also specified for Options 3 and 4, above for which specific engineering design would be required to stiffen the options to satisfy the performance criteria in section 5.4. A summary of the wall and roof cladding weight limits for Options 1 to 4 is provided in Table 7.2

Options 1 to 4 are expected to be able to bridge a length of up to 4 m of settled soil (or sudden lack of support) beneath the foundation and cantilever a distance of up to 2 m over settled soil at the building footprint extremities, within acceptable deformation limits.

While it is not envisaged that these foundation and floor options will require specific engineering design, their documentation will require oversight by structural engineers.

UPDATE: December 2012

Flood risk mitigation requirements may require the building platform to be constructed to a height greater than the land surrounding the dwelling (see section 8). However, the potential for future liquefaction-induced settlement in properties in TC2 leads to the geotechnical requirement to limit the increase in mass added to the land. The maximum recommended increase in height of building platforms[2] above the surrounding land is 400 mm (refer to Figure 5.3). Greater increases may be allowable on a site-by-site basis following geotechnical investigation.

(1) Refer to www.dbh.govt.nz/UserFiles/File/Publications/Building/
(2) See the glossary for definition of 'building platform'.

Figure 5.3: Maximum building platform heights above surrounding ground (TC1 and TC2)

In uncategorised areas on the flat, a geotechnical engineer should be engaged to undertake a site-specific investigation to determine which of the above foundation technical categories best fits the site and recommend appropriate investigations and foundations accordingly.

A summary of proposed foundation solutions for the three technical categories is given in Table 5.1, and the corresponding geotechnical requirements are given in Table 5.2.

UPDATE:
December 2012

Table 5.1: Summary of proposed foundation solutions for rebuilt foundations or new foundations on the flat

TC1 Future liquefaction unlikely	TC2 Minor liquefaction likely and SLS spreading <50 mm	TC3 Future liquefaction expected and SLS spreading >50 mm
NZS 3604 timber piles and floor or tied concrete slabs (as modified by B1/AS1) where ULS bearing capacity > 300 kPa (shallow subsurface investigation required[1]) otherwise Raft foundations (Options 1-4) or Specific engineering design[3] (including deep piles)	Light construction with timber floors and shallow piles as per NZS 3604 where ULS bearing capacity > 300 kPa (shallow geotechnical investigation required[1]) or Enhanced perimeter foundation wall (see section 4.2) and shallow piles as per NZS 3604 (shallow geotechnical investigation required[1]) or Raft foundations (Options 1–4) or Specific engineering design[3] (including deep piles)	Deep piles (section 15.2)[2] or Site ground improvement (section 15.3)[2] or Surface structures with shallow foundations (section 15.4)[2], whichever is the most appropriate for the site, or Specific engineering design[3]

(1) Shallow subsurface investigation – refer to section 3.4.1
(2) See Part C
(3) See section 3.4.1

In uncategorised areas on the flat, a geotechnical engineer should be engaged to undertake a site-specific investigation to determine which of the above foundation technical categories best fits the site and recommend appropriate investigations and foundations accordingly.

UPDATE:
December 2012

Table 5.2: Geotechnical requirements for rebuilt or new foundations on the flat

Foundation technical category	Geotechnical requirements
TC1	Foundations for new dwellings should include a shallow[1] subsurface investigation to determine the bearing capacity of the soil. 1. If the investigation determines the site is 'good ground' (geotechnical ULS bearing capacity is greater than 300 kPa), NZS 3604 timber piles or tied NZS 3604 slabs are acceptable. 2. If the investigation determines the site's geotechnical ULS bearing capacity is greater than 200 kPa but less than 300 kPa, use TC2 enhanced slab solutions (Options 1-4) or other specific engineering design (including deep piles). 3. If the investigation determines the site's geotechnical ULS bearing capacity is less than 200 kPa or affected by other hazards (eg, peat), foundations should be specifically designed.
TC2	Foundations for new dwellings should include a shallow[1] subsurface investigation to determine the bearing capacity of the soil (or for deep piles, a deep investigation[2]). 1. If the investigation determines the site's geotechnical ULS bearing capacity is greater than 300 kPa, NZS 3604 timber piled foundations (Type A) or an enhanced perimeter foundation wall as per Figure 4.2 (Type B) may be used, or specific engineering design carried out. 2. If the investigation determines the site's geotechnical ULS bearing capacity is greater than 200 kPa, use enhanced slab TC2 solutions (Options 1 - 4) or other specific engineering design[1]. 3. If the investigation determines the site's geotechnical ULS bearing capacity is less than 200 kPa, foundations should be specifically designed[1]. TC2 sites generally require only a shallow investigation to provide the information necessary for foundation assessment. However, in some circumstances deep investigations may have been carried out in TC2 areas for other reasons. If a TC2 site has been 'well-tested' by the Canterbury earthquakes (refer to section 13.5.1) and damage to the land or foundations is not greater than implied by the TC2 categorisation, then the site observations implicit in the TC2 categorisation, as well as the actual site observations, provide strong evidence that the TC2 foundation assessment process is appropriate, at the discretion of a CPEng. geotechnical engineer. (In applying engineering judgement to reach a balance between predicted settlement and observed damage, consideration could be given to factors such as the severity of liquefaction and strength-loss predicted, the depth below the surface where liquefaction is predicted, and the thickness and quality of the surface crust).
TC3	A site-specific deep investigation[2] including CPTs or deep boreholes (or data from an appropriate area-wide investigation), and geotechnical analysis of the site is required to determine the land performance in future SLS and ULS events. 1. If data confirms TC3 performance then a range of technical solutions are given in Part C. 2. If the data shows the site has performance equal to a TC2 site then TC2 solutions from this document can be implemented. 3. In some cases, the data will show that the site is a 'hybrid' between TC2 and TC3 (ie, part of the site has TC2 characteristics and part has TC3 characteristics; solutions for this are contained in Part C.

(1) Shallow subsurface investigation – refer to section 3.4.1.
(2) Deep geotechnical investigation – refer to section 3.4.2.

5.3 Description of indicative new foundation and floor options

Site investigation requirements are as outlined in Table 5.2.

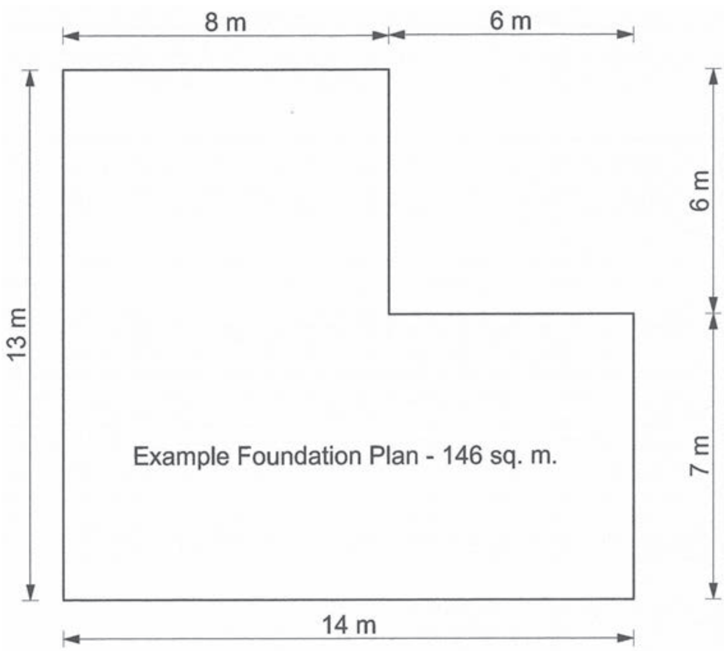
Site preparation should ensure that all grass and topsoil is removed before the placement of foundations or gravel fill. A well-graded sandy gravel aggregate that can be adequately compacted with a plate compactor (eg, pit run, river run, AP 40 or AP65) should be used as subgrade fill beneath any new concrete slabs. The aggregate should be placed in maximum 200 mm layers compacted with (as a minimum) a plate compactor. The top 75 mm of fill should be AP40 to ensure a finer grading in contact with the damp proof course where pit run or river run has been used for bulk filling.

Poorly graded river gravels (tailings or 20/40 rounded river stone) that have commonly been used in Christchurch as subgrade material should not be used. This type of material is prone to forming unstable stone arrangements (bridges) that may collapse with future vibrations, leading to a localised loss of support to the overlying slab. There is also a tendency for finer subgrade materials to migrate into the tailings, particularly when wet and subjected to vibration. Compacted, well-graded sandy gravels will provide additional stiffness and therefore better performance in seismic conditions.

Insulation has not been shown beneath the floors in the proposed options. Insulation requirements will need to be established in conjunction with the insulating characteristics of the walls and roof of the dwelling.

The representative floor plan on which the development and modelling of these details has been based on is shown in Figure 5.4. The details in this section should only be applied to simple house plan shapes such as rectangular, L, T or boomerang shapes.

Figure 5.4: Representative floor plan



5.3.1 Reinforced concrete floor construction in TC2

Several options may be used, but each has limitations that must be recognised. In all options the NZS 3604 ground clearances adjacent to the house foundation must be complied with. Note that for clarity the damp proof membrane (DPM) has not been shown in these representative details.

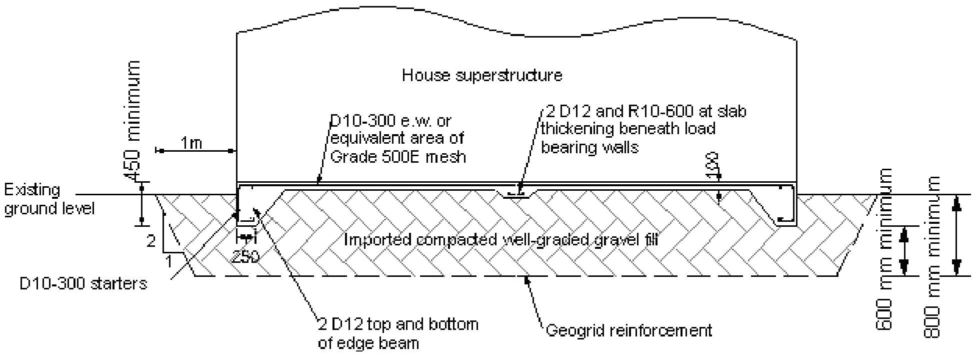
New flood freeboard requirements will also need to be considered if there has been uniform settlement over several properties (see section 8).

Option 1 – Excavation and replacement of the upper layers of soil with compacted, well-graded gravels and construction of a reinforced NZS 3604 slab foundation.

The ground immediately beneath the compacted gravel fill must have a minimum geotechnical ultimate bearing capacity of 200 kPa, or the slab should be subject to specific engineering design (see section 3.4.1).

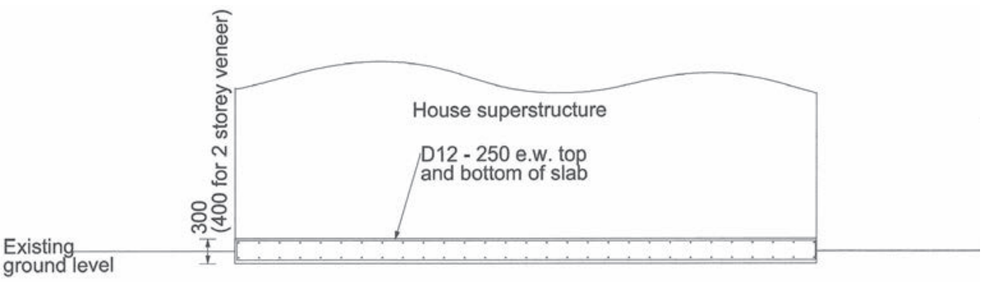
External service lines will need to be beyond the outer extent of the gravel raft and/or have flexible connections (refer to section 5.6).

Figure 5.5: Enhanced foundation slab – Option 1



Option 2 – Construct a thick slab foundation over the existing soil.

Figure 5.6: Enhanced foundation slab – Option 2



Note: NZS ground clearances adjacent to house foundation must be complied with. DPC omitted for clarity.

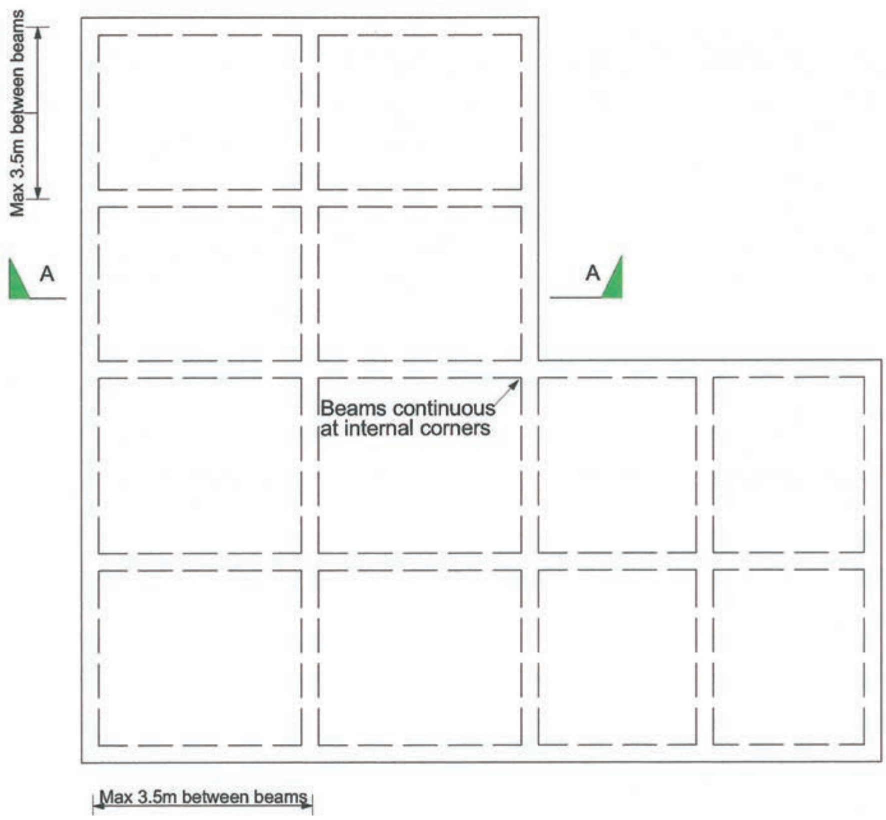
The ground immediately beneath the slab must have a minimum geotechnical ultimate bearing capacity of 200 kPa, or the slab should be subject to specific engineering design (see section 3.4.1). **Note: The thickness needs to increase to 400 mm for two-storey heavy-weight (brick veneer) construction with either a heavy or light roof cladding.**

UPDATE: December 2012

The treatment of service lines as they enter and travel within the slab requires careful consideration (refer to section 5.6).

Option 3 – Construct a generic beam grid and slab foundation.

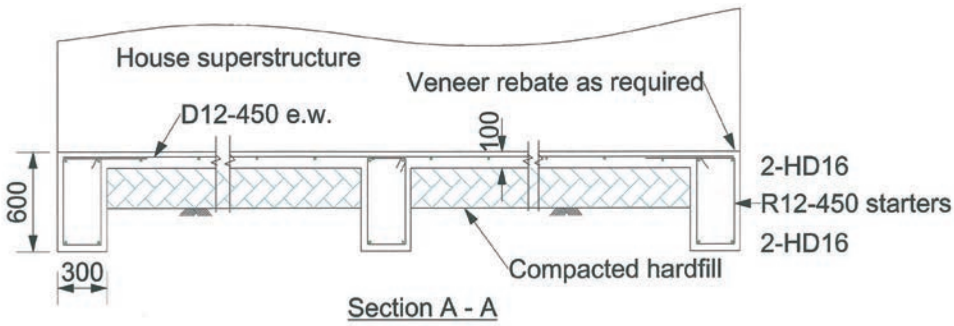
Figure 5.7: Enhanced foundation slab – Option 3 plan



Note: Reinforcing details are not sufficient for two-storey heavy-weight cladding (brick veneer) with a heavy roof but can be used for a two-storey heavy-weight cladding with a light-weight roof.

UPDATE: December 2012

Figure 5.8: Enhanced foundation slab – Option 3 cross-section



The ground immediately beneath the slab must have a minimum geotechnical ultimate bearing strength of 200 kPa, or the slab should be subject to specific engineering design (see section 3.4.1).

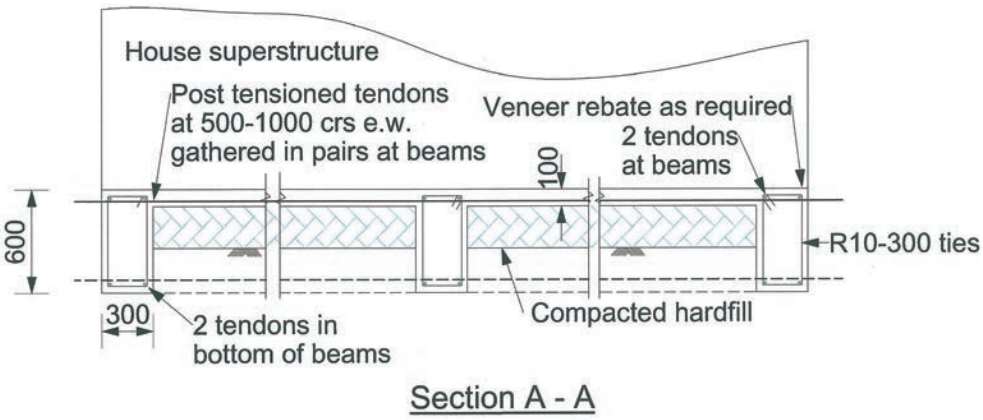
A variation to this option involves post-tensioning the slab using single 12.9 mm or 15.2 mm strand tendons in an unbonded format. The factory-applied greased and sheathed strands are supported in the slab on bar chairs and tensioned through mono-strand anchorages fixed at both ends through the perimeter formwork. Tensioning is carried out using calibrated centre-hole hydraulic jacks.

Post-tensioned slabs are tensioned to between 0.5 and 1 MPa (in time) to overcome drying shrinkage and give some bridging capacity. Spacing of the tendons is nominally 1 m centres each way.

This option requires specific engineering design.[3]

UPDATE: December 2012

Figure 5.9: Enhanced foundation slab – Option 3 variation with post tensioning



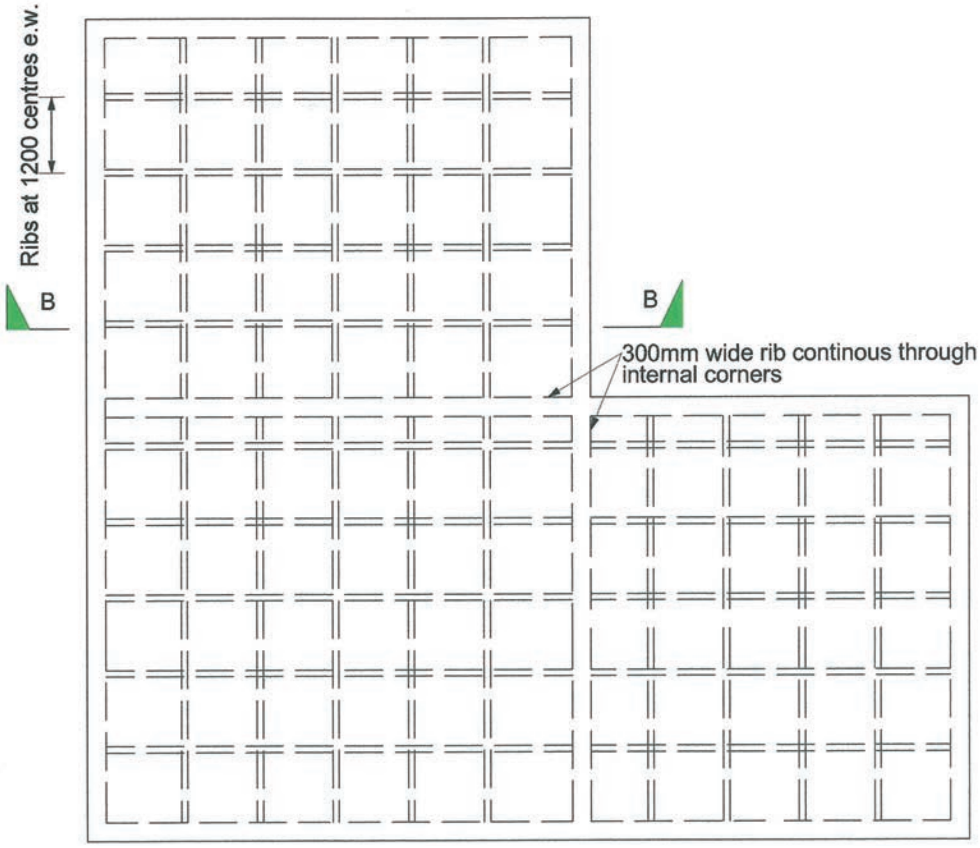
Note: Post tensioning strands are either 12.9 mm or 15.2 mm diameter and factory coated with grease inside an HDPE sheath, giving an overall outside diameter of 17 to 20 mm respectively. Strands are tensioned to provide 0.5-1.0 MPa compressive stress in the concrete.

For both Option 3 variations, it may be easier and more economical to construct the concrete foundation by replacing the compacted hardfill and soil beneath the slab down to the underside of the beams with polystyrene pods.

(3) Refer also to U.S. Post Tensioning Institute publications: Design and Construction of Post-Tensioned Slabs-On-Ground and Construction and Maintenance Procedures Manual for Post-Tensioned Slabs-On-Ground

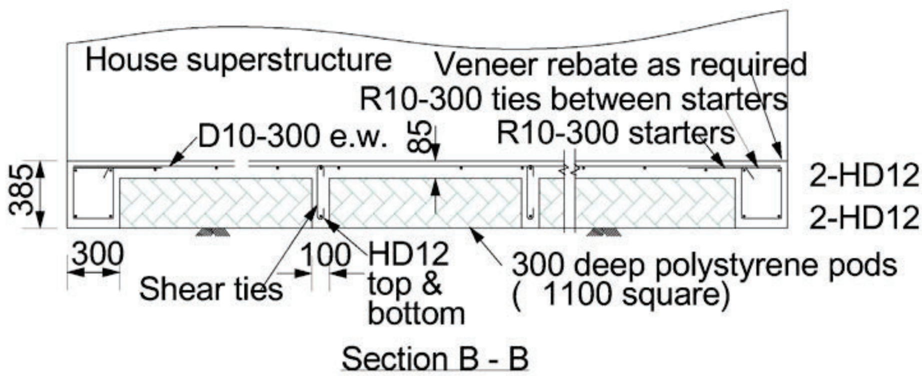
Option 4 – Construct a waffle slab over the existing soil

Figure 5.10: Enhanced foundation slab – Option 4 plan



Note: Reinforcing details are not sufficient for two-storey heavy-weight cladding (brick veneer) with either a heavy or light roof.

Figure 5.11: Enhanced foundation slab – Option 4 cross-section



The ground immediately beneath the polystyrene and ribs must have a minimum geotechnical ultimate bearing strength of 200 kPa, or the system should be subject to specific engineering design (refer to section 3.4.1). Shear ties in accordance with NZS 3101 are required in the ribs.

UPDATE:
December 2012

DELETION:
December 2012
Guidance on the use of deep piles is contained in Part C. Figure 5.11 has been deleted and is superseded by new guidance in Part C.

Option 5 – Deep piles

Install piles to a dense non-liquefiable bearing layer and construct a floor slab (refer to section 15.2)

5.3.2 Timber floor construction in TC2

Timber floors in combination with light-weight claddings and roofing provide several advantages with regard to ease of repair and relevelling.

A rebuilt timber ground floor should generally be constructed in accordance with NZS 3604. The advantage of this type of floor is that it is easy to relevel or repair because of the easy access, and its elemental nature allows straightforward replacement of damaged elements. Bracing demand will be low and standard details can be used.

The soil conditions at each site should be confirmed as suitable in accordance with the modified NZS 3604 procedure, as detailed in Table 5.2 and section 3.4.1.

Driven timber piles to NZS 3604 are suitable under suspended floors.

The level of timber floors should be set to provide a minimum crawl space under the joists of at least 450 mm (NZS 3604 requirement).

Type A dwellings

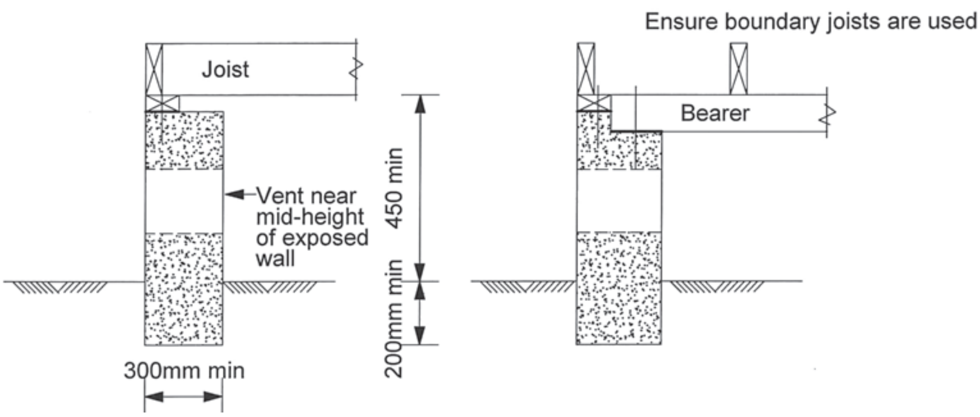
A one or two storey house with a light roof and light- or medium-weight wall cladding supported fully on an NZS 3604 shallow timber or concrete pile foundation is considered to be a valid option in TC2.

Type B dwellings

New foundation walls for one or two storey dwellings with light- or medium-weight cladding and roofing in TC2 should follow the details in Figure 5.12 below. Reinforcing details should be as shown in Figure 4.2a.

Deep piles installed under foundation walls are not within the scope of NZS 3604.
A suitable driving set and founding depth will be required to achieve the required bearing capacity, and the foundation wall will also need to be designed to span between the piles.

Figure 5.12: Timber floor with perimeter walls



Note: Reinforcement details as per Figure 4.2a

The vents in the foundation wall must be positioned near the middle of the wall below the top reinforcing bar, and not notched out of the top of the wall as is common in older houses in Christchurch.

Floor construction details in NZS 3604 are generally adequate, but in practice the jointing between members often falls short of what is required. This is particularly important where resistance to lateral spreading is required. The following should be noted:

- Pile to bearer connection: Ordinary pile connections in Figure 6.3 of NZS 3604. Braced pile connections in Figures 6.6 to 6.8. Anchor pile connection in Figure 6.9.
- Bearer to foundation wall connection: See Figure 6.17 of NZS 3604.
- Bearer butt end joints: See Figure 6.19 of NZS 3604.
- Joist butt end joints: See Figure 7.1 of NZS 3604.

5.4 Guidance for specific engineering design

UPDATE:
December 2012

In many cases the '300 kPa' requirement for 'good ground' or the '200 kPa' requirement for Options 1 – 4 may not be met. Often, simple calculations of actual bearing stresses will allow redimensioning of foundations (refer section 3.4.1 for details). In other cases, specifically designed solutions other than those provided above may be devised. In these cases, the following criteria should be satisfied:

- Geotechnical investigations of the site in accordance with Table 5.2 are to be carried out before designing the foundation system.
- Design for the potential for lateral ground spreading to the extent indicated from the geotechnical investigation.

For Type C house foundations in TC2

- Design Type C house foundations for the potential for differential settlement of the supporting ground that will allow a maximum unsupported length for the ground floor of 4 m beneath sections of the floor and 2 m at the extremes of the floor (ie, ends and outer corners).
- Design to ensure that the floor does not hog or sag more than:
  - 1 in 400 (ie, 5 mm hog or sag at the centre of a 4 m length) for the case of no support over 4 m (see Figure 5.13), and
  - no more than 1 in 200 for the case of no support of a 2 m cantilever at the extremes of the floor (see Figure 5.13).
- Appropriate provision should be made for 'flexible' services entry to the dwelling to accommodate the potential differential settlement of the foundation as indicated in the geotechnical report.
- Designs should accommodate settlements as indicated in Table 5.3.

Table 5.3: Liquefaction design settlements of new building foundations in TC2

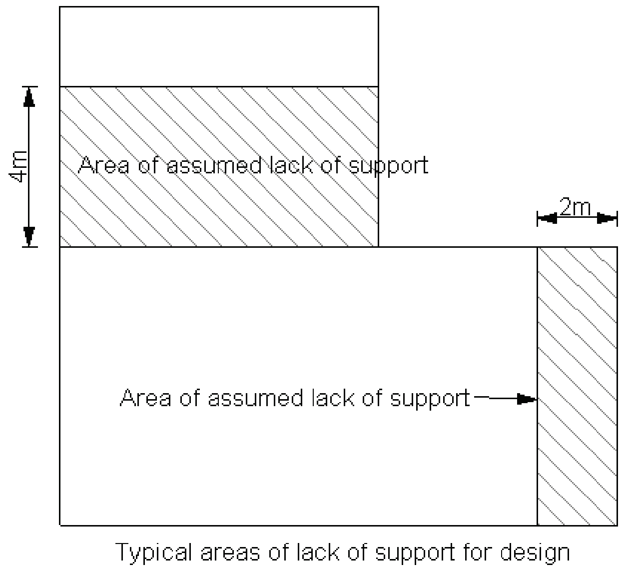
Type	SLS[1]	ULS[2]
Total settlement (mm)	Up to 50 mm	Up to 100 mm

(1) SLS – serviceability limit state
(2) ULS – ultimate limit state
(3) Part C covers liquefaction settlement limits for foundations in TC3.

UPDATE: December 2012

Where possible, reinforced concrete foundation systems should have sufficient stiffness to permit relevelling by jacking at perimeter points, accompanied by pressure grouting or resin injection beneath the house interior. With regard to lateral spreading, the foundation system should also have sufficient tensile strength to permit sliding of the house in relation to the ground without breaking or distorting. The strength should be sufficient to withstand forces equal to frictional resistance to sliding over half the house footprint. Timber floors are expected to be readily relevellable by the packing of piles.

Figure 5.13: Foundation plan showing design criteria for specific design



5.5 Replacing foundations (retaining the superstructure)

A house superstructure that is still reasonably intact may be able to be temporarily lifted off existing foundations so that new foundations can be built. The new foundation will be required to fully comply with the Building Code.

Figure 5.1 shows the process for TC1 and Figure 5.2 shows the process for TC2. A summary of the steps for each foundation type in TC1 and TC2 is provided in Table 5.4 and in more detail on subsequent pages.

Replacement approaches for TC3

Appropriate replacement solutions for TC3 will involve undertaking a geotechnical investigation and making decisions based on the results of this investigation.

Guidance for house foundation replacement options in TC3 is given in Part C. Specifically engineered solutions (eg, stiffened surface structures, deep piles, ground improvement) are required to meet the performance requirements of the Building Code.

For foundations on hillsides that rely on retaining walls for support of either the structure or the ground immediately above or below the structure, see section 6.

UPDATE: December 2012

Table 5.4: Summary of foundation rebuilding approaches for TC1 and TC2

Foundation type	Foundation rebuild		
	Rebuild strategy	Occupancy during rebuild operations	
Type A Foundation: Timber-framed suspended timber floor structures supported only on piles Cladding: Light- and medium-weight	Remove base skirt, disconnect services if adjacent to works, repile affected area, reconnect services and reskirt perimeter Re-establish minimum ground clearances in accordance with section 2.6	No	Usually only minor disruption to occupants. Need to consider distress to framing, trusses and bracing at this level of foundation damage
Type B1 Foundation: Timber-framed suspended timber floor structures with perimeter concrete foundation Cladding: Light- and medium-weight	Disconnect services, temporarily raise house as necessary, remove perimeter concrete foundation wall and replace, repile, reconnect services and reinstate ground to wall Re-establish minimum ground clearances in accordance with section 2.6	No	As for Type A regarding pile relevelling. Replacing the wall will require vacancy as the perimeter of the house will be disrupted
Type B2 Foundation: Timber-framed suspended timber floor structures with perimeter concrete foundation Cladding: Heavy-weight (veneer)	Disconnect services, remove exterior cladding, temporarily raise house as necessary, remove perimeter concrete foundation wall, and replace, repile, reconnect services and reinstate ground to wall, replace cladding Re-establish minimum ground clearances in accordance with section 2.6	No	Perimeter will be disrupted to give access to wall and will disrupt services
Type C1 Foundation: Timber-framed dwelling on concrete floor (slab-on-grade) Cladding: Light- and medium-weight	Disconnect services, temporarily raise house superstructure, remove and replace slab, reinstate house and reconnect services.	No	Severely cracked slab with differential settlement over 150 mm may have caused severe damage to the timber framing, trusses and bracing

Foundation type	Foundation rebuild		
	Rebuild strategy	Occupancy during rebuild operations	
Type C2 Foundation: Timber-framed dwelling on concrete floor (slab-on-grade) Cladding: Heavy-weight (veneer)	Disconnect services, remove exterior cladding, temporarily raise house superstructure, remove and replace slab, reinstate house and reconnect services, replace cladding	No	Severely cracked slab with differential settlement over 150 mm may have caused severe damage to the timber framing, trusses and bracing

Note: It may be necessary to remove decking and paths in order to expose the foundation wall (Types A and B) or the perimeter foundation (Type C) for relevelling and rebuilding works.

Type A foundation – concrete or timber piles throughout

In these instances, it may be possible to lift the superstructure, including the floor, rebuild the pile system beneath the house and remediate any damage caused to the claddings and linings of the structure.

Provided the geotechnical ULS bearing capacity is greater than 300 kPa, the process will be very similar to that employed by a house removal company engaged to relocate or repile a house. A summary of the process is given in Table 5.4 with a more detailed process description included in Appendix A2. If the geotechnical ULS bearing capacity is less than 300 kPa, then specific engineering design is required (see section 3.4.1).

Type B1 perimeter concrete foundation wall (light or medium-weight claddings)

There will be cases where only sections of the foundation wall will need to be replaced. The building work, which is the repair of a building element (the section of perimeter wall), needs to comply with the Building Code and therefore should be designed as if the perimeter foundation wall was new. For guidance, refer to section 4.2.

In TC1, provided the geotechnical ULS bearing capacity is greater than 300 kPa, this would amount to simple replacement of the existing foundation wall with an NZS 3604 foundation wall, as liquefaction and future settlement is not anticipated. Otherwise, specific engineering design is required (see section 3.4.1).

In TC2, provided the geotechnical ULS bearing capacity is greater than 300 kPa, an enhanced reinforced foundation wall would be required to withstand the differential settlement anticipated with future minor liquefaction. Refer to Figure 4.2a and section 5.3.2 for indicative cross-sections. Otherwise, specific engineering design is required (see section 3.4.1).

A summary of the process is given in Table 5.4 with a more detailed process description included in Appendix A2.

Type B2 perimeter concrete foundation wall (heavy veneer cladding)

In these instances, it may be very difficult to lift the superstructure, including veneer cladding, without causing irreparable damage to the cladding. It will probably be necessary to demolish the veneer and rebuild it once the new foundation has been constructed and the house superstructure has been re-installed on the new foundation.

If the veneer is removed, the owner may choose to have insulation installed in the exterior walls if this was not already in place, but this will be at the owner's expense.

In TC1, provided the geotechnical ULS bearing capacity is greater than 300 kPa, this would amount to simple replacement of the existing foundation wall with an NZS 3604 foundation wall, as damaging liquefaction and future settlement is not anticipated. Otherwise specific engineering design is required (see section 3.4.1).

In TC2, provided the geotechnical ULS bearing capacity is greater than 300 kPa, an enhanced reinforced foundation wall (refer to Figure 4.2a and section 5.3.2) would be required to withstand the differential settlement anticipated with future minor liquefaction. Otherwise specific engineering design is required (see section 3.4.1).

The veneer may be rebuilt on the new foundation. Alternatively, the owner may choose an alternative lighter cladding system if acceptable to the insurance company.

For cases where partial replacement of the perimeter foundation wall may be all that is required to reinstate the foundation, see Type B1 above and section 4.2 for guidance.

A summary of the process is given in Table 5.4 and a more detailed process description included in Appendix A2.

Type C1 slab-on-grade floors (light- or medium-weight claddings)

The degree of settlement that has occurred in this instance will be such that the floor slab and edge thickening are expected to be heavily damaged and not easily repairable. The slab is likely to be deformed and cracked. The repair process will involve lifting the superstructure (including the bottom plates), demolishing the existing slab, building a new foundation, and re-installing the superstructure on the new foundation. If foundation Option 1 is used then the house will need to be temporarily moved off the site to allow construction of a compacted gravel raft.

In TC1, provided the geotechnical ULS bearing capacity is greater than 300 kPa, the foundation replacement may be in accordance with NZS 3604 (as modified by B1/AS1). If the geotechnical ULS bearing capacity is between 200 kPa and 300 kPa, stiffened raft foundations (Options 1 to 4 in section 5.3) could be used or specific engineering design. If the geotechnical ULS bearing capacity is less than 200 kPa, specific engineering design is required (see section 3.4.1).

In TC2, provided the geotechnical ULS bearing capacity is greater than 200 kPa and there are no other geotechnical constraints (eg, peat deposits), the new foundation will need to be a stiffened raft foundation (Options 1 to 4 in section 5.3). If the geotechnical ULS bearing capacity is less than 200 kPa, specific engineering design is required (refer to section 3.4.1).

Alternatively, replace the foundation with a shallow pile and timber floor option in accordance with NZS 3604. The superstructure is then reconnected to the new foundation system.

A summary of the process is given in Table 5.4 with a more detailed process description included in Appendix A2.

UPDATE:
December 2012

Type C2 slab-on-grade floors (heavy veneer cladding)

The process for Type C2 is the same as for Type C1, with the following additional guidance.

The veneer must be demolished to allow the superstructure to be lifted off the existing concrete slab. The repair process will involve lifting the superstructure (including the bottom plates), demolishing the existing slab, building a new foundation, and re-installing the superstructure on the new foundation. If foundation Option 1 is used then the house will need to be temporarily moved off the site to allow construction of a compacted gravel raft.

If the veneer is removed, the owner may choose to have insulation installed in the exterior walls, if this was not already in place, at the owner's expense.

The veneer may be rebuilt on the new foundation. Alternatively, the owner may choose an alternative lighter cladding system if acceptable to the insurance company.

A summary of the process is given in Table 5.4 with a more detailed process description included in Appendix A2.

5.6 Garage structures and outbuildings

Uninhabited detached garages (ie, that are not constructed as an integral part of a house) and outbuildings are considered to be Importance Level 1 (IL1) structures. If these structures are currently habitable or of significant value, Importance Level 2 (IL2) applies. Refer to DBH Codewords No 35 – March 2009 'Guidance on garage classification' www.dbh.govt.nz/codewords-35-1.

UPDATE:
December 2012

IL1 structures have no seismic load requirements (under AS/NZS 1170.0) at Serviceability Limit State (SLS), and therefore have no amenity requirements relating to liquefaction deformations at SLS levels of shaking. This leaves a 'life safety' design requirement at Ultimate Limit State (ULS) for a 1/100 year event, which should be able to be provided in most cases on a TC2 site by a suitably detailed structure on a TC1 type foundation system. Alternatively, a specific design can be determined by applying the 1/100 year design event loadings at ULS.

Conversely, attached or integral garages need to be designed to the same level of performance as the main structure.

Refer to Section 11.3 for garage structures and outbuildings on properties designated as TC3.

5.7 Services

If lateral spread or differential settlement of the ground occurs, there is potential for damage to services, and provision must be made for the design and installation of services to minimise the effects of ground movement. This is particularly important when services penetrate or are attached to concrete floor systems. Flexibility in service lines is the key to good performance.

Drinking water

Modern drinking water supply to a property is delivered via flexible 'plastic' pipes. When installed in a trench, they may be laid down in a snake pattern, which provides extra length should ground extensions occur. Where the pipe penetrates the foundation and the floor slab, a duct/sleeve 125 mm greater in diameter than the pipe should be provided to allow the pipe to move independently. The sleeve may be filled with a compressible filler, which allows differential movement but which also provides limited access beneath the slab should a leakage issue arise.

Sewer pipes

Sewer pipes from the house to the sewer in the street are generally formed in uPVC plastic, which possesses some flexibility in itself. Waste pipes may pass through the floor of the dwelling to serve plumbing fixtures such as baths, showers, basins, and soil pipes from toilets. These pipes will pass below the floor in Options 1 and 2 (see section 5.3.1), although there is scope (while maintaining the required falls) for passing the waste pipes through the beams and ribs of the foundation in Options 3 and 4. If there is vertical or horizontal movement between the foundations and the ground in Options 3 and 4, the expected failure plane is across the bottom of the beams or ribs. Consideration will need to be given to the connections beyond the outside face of the foundation.

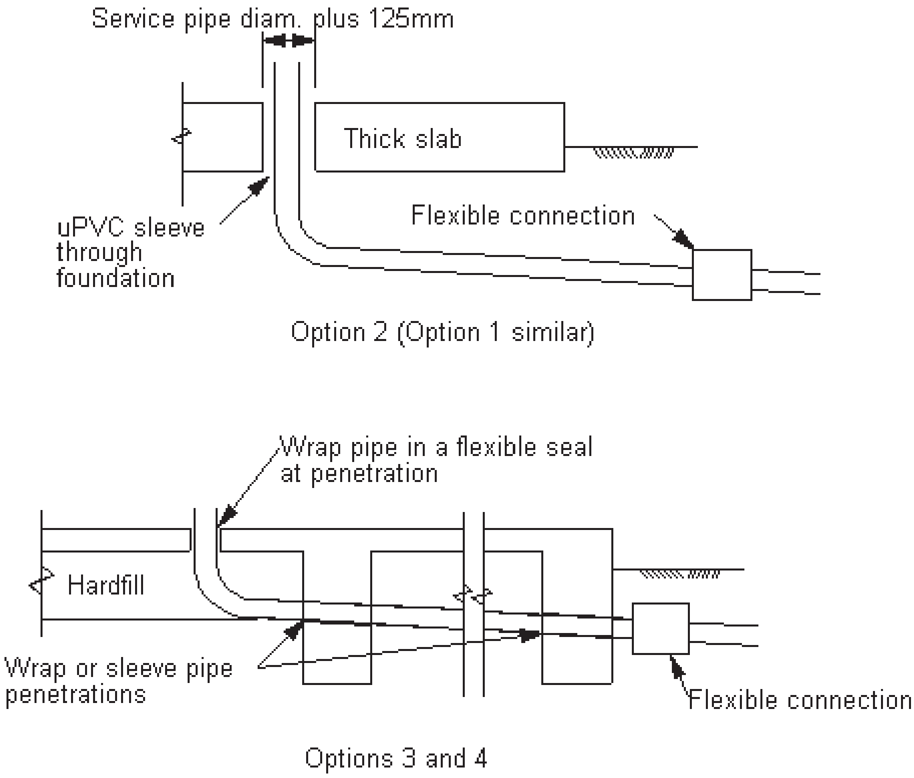
Flexible connections should be considered between the straight lengths of pipe, and located outside the building footprint. Greater pipeline flexibility is achieved by using rubber ring joint pipes.

Consideration should also be given to the provision of greater falls in the lines than the minimums required by the Standards. This will make the continued operation of the system more viable should tilting of the ground occur during any future liquefaction event.

Where the pipes pass through the slab, a duct or sleeve is recommended (see Figure 5.14). Ideally, the duct should have a diameter 125 mm greater than the service pipe. Otherwise, a flexible seal should be employed to allow some movement between the pipe and the floor.

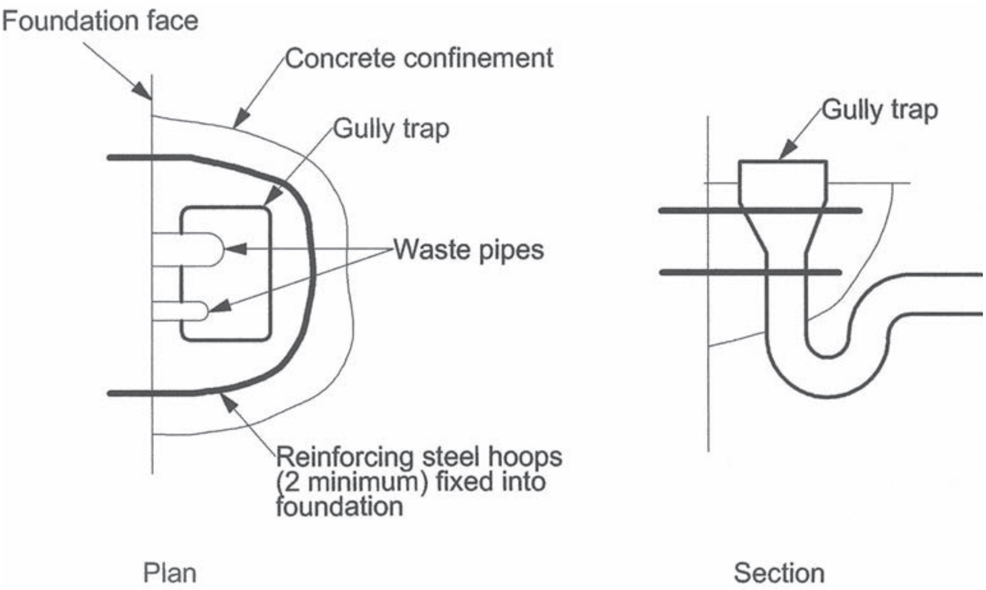
Where sewer pipes are installed in a trench parallel to the foundation, the branch drains, such as those connecting to gully traps, should contain a flexible connection adjacent to the foundation.

Figure 5.14: Waste water pipe routing



Plumbing codes require at least one gully trap on the perimeter of a house. Invariably, waste pipes pass through the foundation slab and discharge into the gully trap from above it. Sometimes the waste pipes enter via the side wall of the gully trap. It is recommended that the gully traps be encapsulated in concrete which is tied to the house foundation (hooped reinforcing bars), preventing differential movement should there be ground spreading or settlement adjacent to the foundation (see Figure 5.15).

Figure 5.15: Restraint of gully trap



Stormwater pipes

Where storm water pipes are installed in a trench parallel to the foundation, the branch connections to the downpipes should contain a flexible connection.

Underground power and communications cables

Fortunately, these cables are quite flexible. Underground power cables may be ducted or buried directly in a trench. In either case there is scope for accommodating unexpected extensions by 'snaking' the cables or looping within access chambers. Consideration should be given to accommodating the cables in oversize ducts where they pass through the floor.

Appendix B

Extracts from the *MBIE Guidance: Repairing and rebuilding house affected by the Canterbury earthquakes*, for design of shallow foundations in the TC3 zone

Extract from: Ministry of Business, Innovation and Employment (April 2015, updated May 2018), Repairing and rebuilding houses affected by the Canterbury earthquakes, Version 3a, Section 15.4: New Foundations in TC3, pg. 15.47 to 15.62.

15.4 Surface structures with shallow foundations

15.4.1 Objective and scope

This section provides surface foundation options and design criteria that can be used on most TC3 sites without ground improvement or deep foundation works. These options can be relevelled in the event of future differential settlements caused by earthquakes, and can accommodate varying levels of lateral spreading without causing rupture of the superstructure.

It is considered that any damage experienced in SLS level earthquakes would be readily repairable and is not likely to prevent continued occupation of the dwelling.

The surface structure types outlined in this section are only applicable for timber or steel-framed structures with light roofing materials and light-weight and medium-weight wall cladding, and with regular plan layouts.

Due to the range and different combinations of future vertical land settlement and lateral spreading (stretch) on TC3 sites, careful consideration needs to be given to the selection of surface structure options.

15.4.2 Types and options

Three types of surface structure are proposed in this section.

UPDATE: December 2012

The **Type 1 surface structure** is a modified NZS 3604 light-weight platform which is capable of withstanding moderate differential vertical settlement from liquefaction at SLS levels (ie, corresponding to **minor** land settlement of less than the index value of 100 mm or sites where ground improvement has been carried out in accordance with section 15.3.4), and **minor to moderate** lateral strain across the building footprint at ULS levels (ie, up to 200 mm). In both situations, only minor repairs are likely to be required. However, if it is found that there is evidence of previous lateral spread at the site then the preference is to use a Type 2 surface structure.

The Type 1 surface structure is likely to differentially settle in response to future liquefaction-induced land settlement. However because of the light-weight nature and regular shape of the superstructure, it can rely on the stiffness of the superstructure to redistribute loads to remaining bearing points beneath the foundation. Sand ejecta may accumulate in the underfloor space because there is no "seal" of the ground surface beneath the floor, but access for sand removal is relatively simple.

UPDATE: December 2012

> *This surface structure type is presented in section 15.4.3 as a standard solution that can be directly applied without further specific design on sites that are considered to meet the above geotechnical criteria (with the exception of determining static bearing capacities – see section 15.4.8).*

UPDATE:
December 2012

The **Type 2 surface structures** provide platforms that are capable of resisting **major** lateral strain (ie, between 200 and 500 mm) at ULS and different levels of differential vertical settlement at SLS levels, and also suitable on other sites where ground improvement has been carried out in accordance with section 15.3.4.

Type 2A is a timber floor constructed over a 150 mm thick concrete 'underslab' on a gravel raft, and is capable of resisting vertical liquefaction-induced settlement of the land of up to 100 mm at SLS. Type 2B features a 300 mm thick concrete 'underslab', and is capable of resisting vertical land settlement of up to 200 mm at SLS. Both Types 2A and 2B should experience manageable curvature in response to settlement, allowing them to be relevelled, having sustained minimal superstructure damage.

This surface structure type is presented in section 15.4.4 as a standard solution that can be directly applied without further specific design on sites that are considered to meet the above geotechnical criteria. It is suggested that initial applications of this solution type may be reviewed by the Ministry in conjunction with the consenting process (review process to be defined).

The **Type 3 surface structures** comprise a mix of relevellable and stiff platforms that are also capable of resisting **major** lateral strain (ie, between 200 and 500 mm) in a ULS event. It is intended that they be designed to either bridge loss of support or be light-weight flexible platforms that are capable of being simply relevelled.

Two options within this type are presented in section 15.4.5 as concepts only, and require specific engineering design and specification. Each remains essentially in a flat plane or with a manageable curvature after an earthquake, allowing it to be relevelled, having sustained minimal superstructure damage in the process.

The sample concepts for this surface structure type require specific design for all sites where they are used. It is suggested that initial applications of this solution type are discussed with the Ministry (process to be defined).

A summary of the suitability of the different types of surface structures with respect to the different levels of lateral stretch and vertical settlement is shown in Table 15.5.

UPDATE:
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Table 15.5: Surface structure capability summary

	Vertical Land Settlement (SLS)		Lateral Stretch (ULS)	
	<100 mm (Moderate)	>100 mm (Potentially Significant)	<200 mm (Moderate)	<500 mm (Major)
Type 1 – light-weight platform (standard solution) Enhanced NZS 3604 subfloor	Yes	No[1]	Yes	No
Type 2 – underslab platform (standard solution) **Type 2A** – 150 mm underslab on gravel	Yes	No[1]	Yes	Yes
Type 2B – 300 mm underslab on gravel		Up to 200 mm[1]		
Type 3 – concepts for specific design **Type 3A** – Re-levellable platform	Yes	Subject to design	Yes	Yes
Type 3B – Stiff platform				

(1) Unless ground has been improved (refer to section 15.3.4)

15.4.3 Type 1 surface structure foundations – light-weight relevellable platform

This concept utilises normal NZS 3604 piled construction with the exception that the bearers are bolt laminated to ensure continuity along the bearer (Figure 15.15). All the piles are 125 mm square NZS 3604 ordinary timber piles, each fixed to the bearers with four wire dogs and two skew nails. In the event of a lateral spread beneath the floor of up to 200 mm, the outer piles are expected to remain upright, stabilised by the plywood perimeter bracing, and the soil is expected to deform around the pile foundations. The inner piles are expected to rotate about the connection to the bearer and may require replacement or straightening after a significant lateral spreading event. The plywood bracing system is capable of resisting the ULS shaking expected in the Canterbury Earthquake Region.

While the performance under spreading is expected to be better when the spreading is in the direction of the bearers, there is also sufficient bracing in combination with a floor diaphragm to resist spreading in the orthogonal direction.

Depending on the degree of tilt on the inner piles after the earthquake, some piles may need to be replaced. However, the extension of the ground beneath the foundation will cause the piles to tilt in opposing directions, providing a degree of triangulation, which will serve to brace the floor against translation.

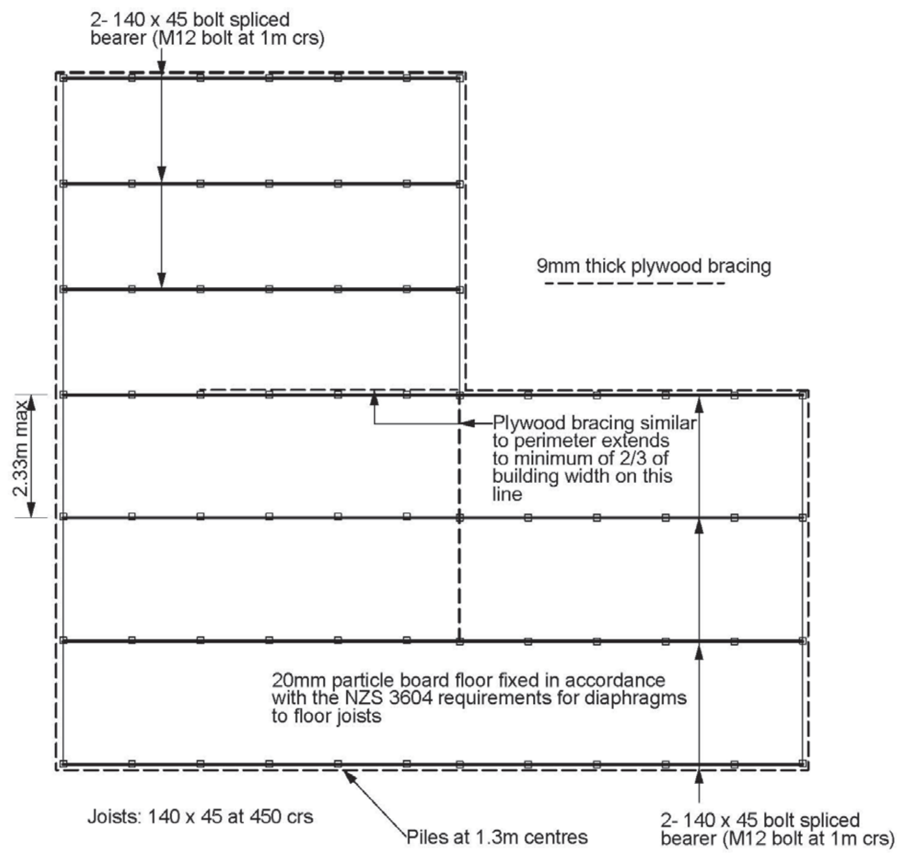
Fibre-cement products may be used in lieu of plywood and further information on substitution in this foundation type should be sought from the manufacturers' websites.

DELETION:
December 2012

To provide the best performance in the event of significant liquefaction and/or spreading, several principles are recommended in the layout of the superstructure, where practicable. These include:

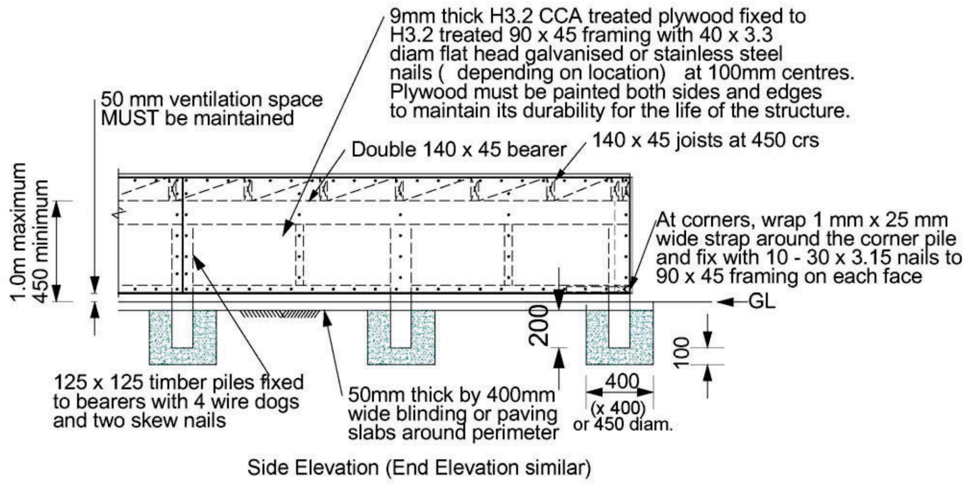
- A simple rectangular floor plan is preferred. When the floor length-to-width ratio is greater than two, a central bracing wall should be included.
- If the floor is less than 12 m long and the other (shorter) direction is greater than 6 m then the central plywood bracing wall could be omitted.
- 'L' or 'T'-shaped floors may be constructed (as in Figure 15.15) but the plywood bracing must continue beneath the floor at re-entrant corners for at least 2/3 of the building width along these lines.
- Total floor area limited to approximately 150 m^2.
- Sheet claddings and sheet linings (as opposed to strip linings such as weatherboards, unless underlaid with sheet lining).
- Rooms with an upper size limit (maximum wall spacing of no more than 7 m in the long direction of the room).
- Long wall elements between windows and walls continuous above and beneath windows (ie, a deep beam with holes in it rather than a series of discrete elements).
- Internal cross walls continuous from one side to the other with doorway openings kept to a minimum size.
- A pitched truss roof with the ridge running in the long direction of the house (likely to be the most normal roof construction on a rectangular floor plan).
- Solid connections between the tops of internal walls and the roof framing (helps to mobilise the stiffness of the triangulated roof).
- A 2.7 m stud in lieu of a 2.4 m stud (provides deeper wall panels over doorways and above and below windows).

Figure 15.15: Plan of Type 1 surface structure



UPDATE:
December 2012

Figure 15.16: Perimeter foundation details for Type 1 surface structure



15.4.4 Type 2 surface structure foundations – flexible relevellable platform

The structures in this category are built in a conventional NZS 3604 fashion with timber support piles. However the short piles are supported by a reinforced concrete ground slab rather than the ground itself (Figures 15.17 to 15.22). Termed an 'underslab', these slabs are sufficiently reinforced to resist lateral spreading in any direction.

The piles do not penetrate the soil surface, but are instead encapsulated in the reinforced concrete slab, with vertical loads from the superstructure being transferred to the reinforced slab via dowels passing through the piles.

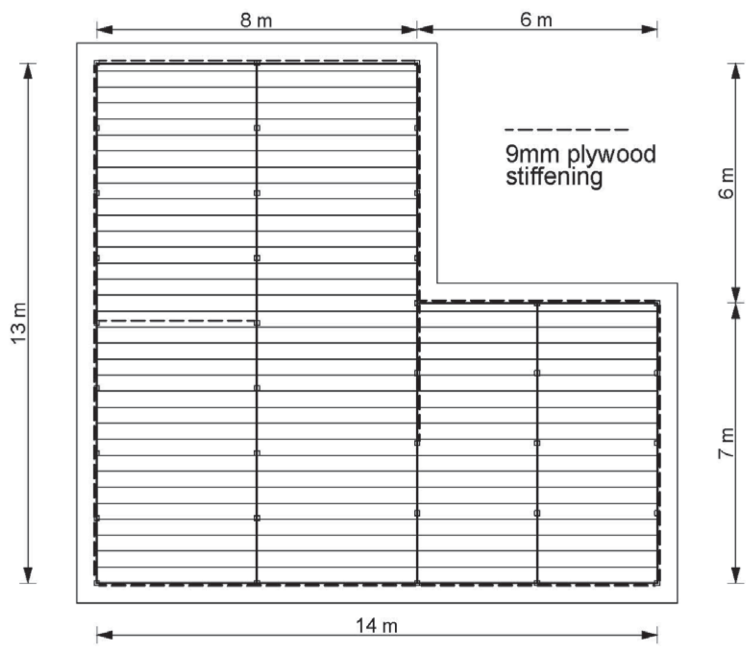
The Type 2A option is a 150 mm thick concrete 'underslab' on a gravel raft, and is capable of resisting vertical liquefaction-induced settlement of the land of up to 100 mm at SLS. Type 2B has a 300 mm thick concrete 'underslab' and is capable of resisting vertical settlement of up to 200 mm at SLS.

These slabs could be post-tensioned in order to improve the out-of-plane stiffness compared to the reinforced slab option, noting that stressing a slab is a specialised process.

Both options can accommodate lateral spreading in excess of 250 mm in a future SLS event and up to 500 mm in a future ULS event in any direction. As the slab is set into in the soil, lateral displacement of the slab under earthquake shaking will be restrained.

The underside of the joists may be up to 1 m above the slab with no need for diagonal bracing, providing a clear working space beneath the floor. While vertical differential settlement beneath the slab will result in a deformed floor profile as the piles settle, it is expected that relevelling of the floor can be achieved by packing the tops of the settled piles.

Figure 15.17: Plan of Type 2 surface structure



UPDATE:
December 2012

Figure 15.18: Section through Type 2A surface structure at the timber piles

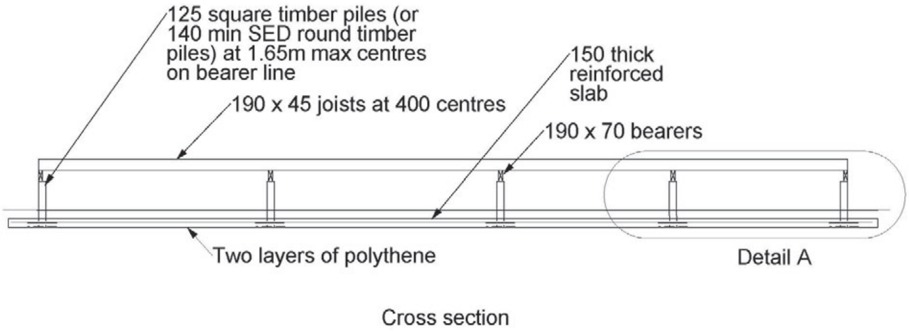


Figure 15.19: Detail of Type 2A surface structure at the timber piles (including gravel raft)

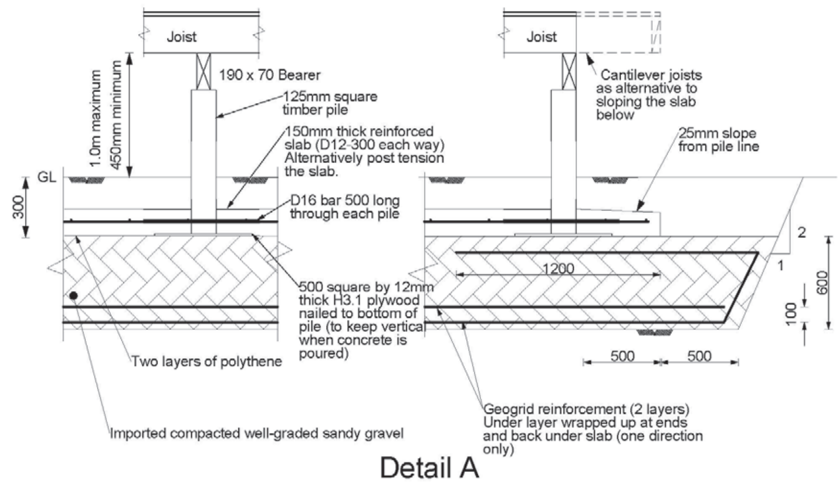


Figure 15.20: Section through Type 2B surface structure at the timber piles (including gravel raft)

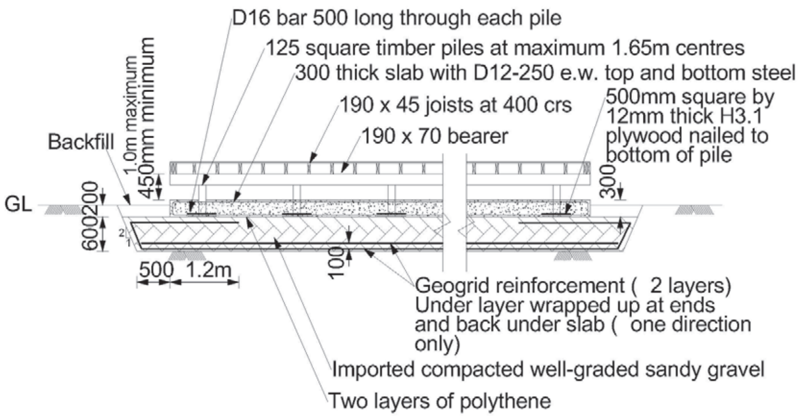
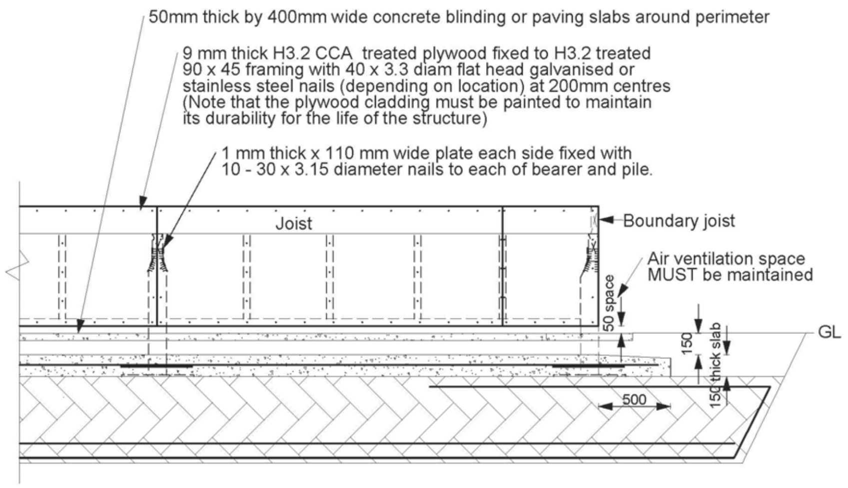


Figure 15.21: Detail of plywood stiffening to Type 2 surface structure (Type 2A illustrated)



15.4.5 Type 3 surface structure foundations – concepts for specific design

Type 3A – Isolated concrete pads beneath stiff continuous bearers (relevellable platform)

This concept has been developed to accommodate lateral spreading beneath the bearers in any direction, and consists of a system of 1 m square surface concrete blocks which support 190 x 140 bearers (laid in the line of expected lateral spread) and 190 x 45 floor joists (refer Figures 15.22,and 15.23). The design philosophy is to maintain a resilient floor plate that can slide on the concrete pads in a ground-spreading event but which will remain in place when subjected to wind loads and earthquake shaking.

This system can accommodate lateral spreading in excess of 250 mm in a future SLS event and greater than 500 mm in a future ULS event.

The concrete blocks can be cast offsite and installed on a prepared base. However, it may be less difficult to achieve a consistently level surface across the blocks if they are cast insitu.

The use of two 190 x 70 members, bolt-spliced together with staggered splices, ensures that adequate tensile strength of the bearer is maintained. The connections between the joists and the bearers will need to be designed to ensure that the bearers will slide on the concrete blocks before the connections fail. Connections (consisting of steel angle brackets connected to the concrete blocks with M6 “frangible” brass anchors) between the bearers and the concrete blocks are expected to lock the floor in position under service-level seismic loads and all wind loads. However, under the more severe ground-spreading loads, the bolts securing the brackets are expected to shear off, allowing the bearers to slide freely on the concrete blocks. The bearers are fixed to the concrete block at one end of their length but allowed to slide over the blocks at other crossings.

This concept does not offer significant resistance to differential vertical displacements of the ground beneath the blocks, and some superstructure damage is expected to occur in ULS events. However, any relevelling of the dwelling is expected to be possible by packing the space between the concrete blocks and the bearers. Good access is provided beneath the floor for this operation. New retaining bolts could then be installed. Calculations have indicated that should the vertical support from one concrete block be lost, the bearer will span between adjacent blocks, but the floor will feel springy until packing is installed to regain the support.

If the potential spreading is clearly going to be in one direction only, the alignment of the dwelling could be oriented so that the bearers run in the direction of the spreading. Then the concrete block size could be reduced in the direction orthogonal to the spreading.

Figure 15.22: Plan of Type 3A surface structure

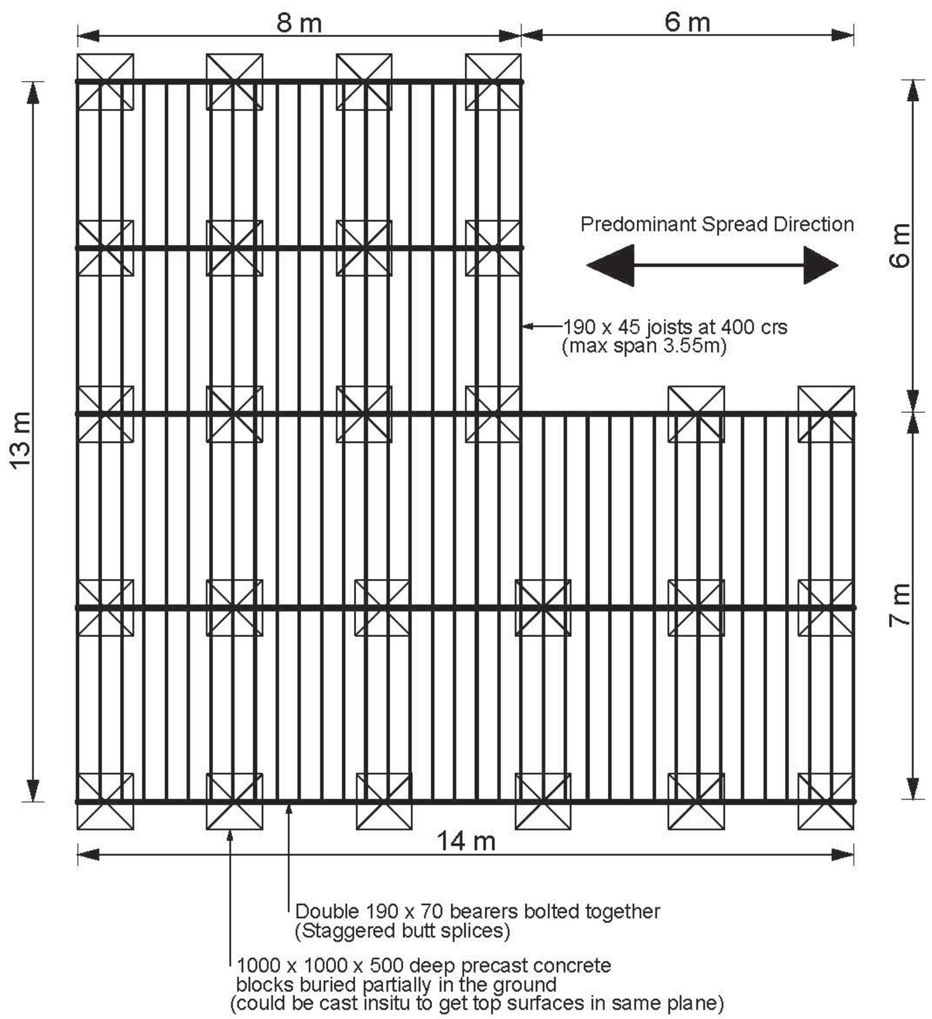
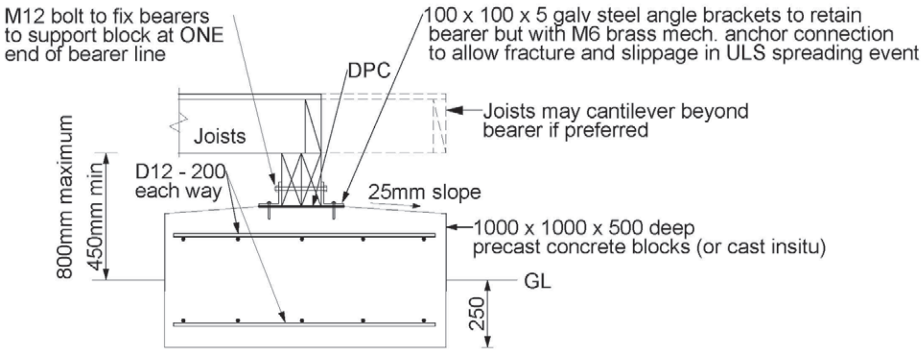


Figure 15.23: Type 3A surface structure - Detail at supporting blocks



Type 3B – Steel beams over prestressed concrete beams (stiff platform)

This concept consists of prestressed concrete 300 mm square 'pencil ground beams' running the full length of the house and laid in the direction of expected lateral spread. A grid of steel beams (250 UB25 or 150 UC 23) is placed over these and they support the floor joists. This combination offers a reasonably stiff floor grid against vertical differential displacements (refer Figures 15.24 and 15.25). The steel beams run orthogonal to the pencil beams and are lightly clamped to the pencil beams. However, in the event of greater than anticipated spreading parallel to the steel beams, the clamped connections are expected allow the steel beams to slide over the concrete beams.

The steel beams could be increased in size to improve the out-of-plane stiffness in the direction parallel to their axis. Similarly, the prestressed concrete beams could be increased in size to improve stiffness. However, the 300 mm x 300 mm beams are light enough (3 tonne) to lift with small cranes.

A conventional timber floor and superstructure can be built on the steel beams.

Large differential vertical displacements beneath the concrete beams will be partially reflected in the deflection of the floor plate but good access is provided for relevelling if required.

The concept is directional in that lateral spreading of the ground beneath the concrete beams can be accommodated, with the aid of a polythene slip layer, in the direction of the beams. In the direction orthogonal to the beams the passive pressure of the spreading soil could pull the pencil beams apart, hence the clamped as opposed to rigid joints with the steel beams.

Lateral spreading in the direction of the prestressed beams of up to 250 mm SLS and 500 mm ULS spreading can be accommodated by this example concept.

Figure 15.24: Plan of Type 3B surface structure

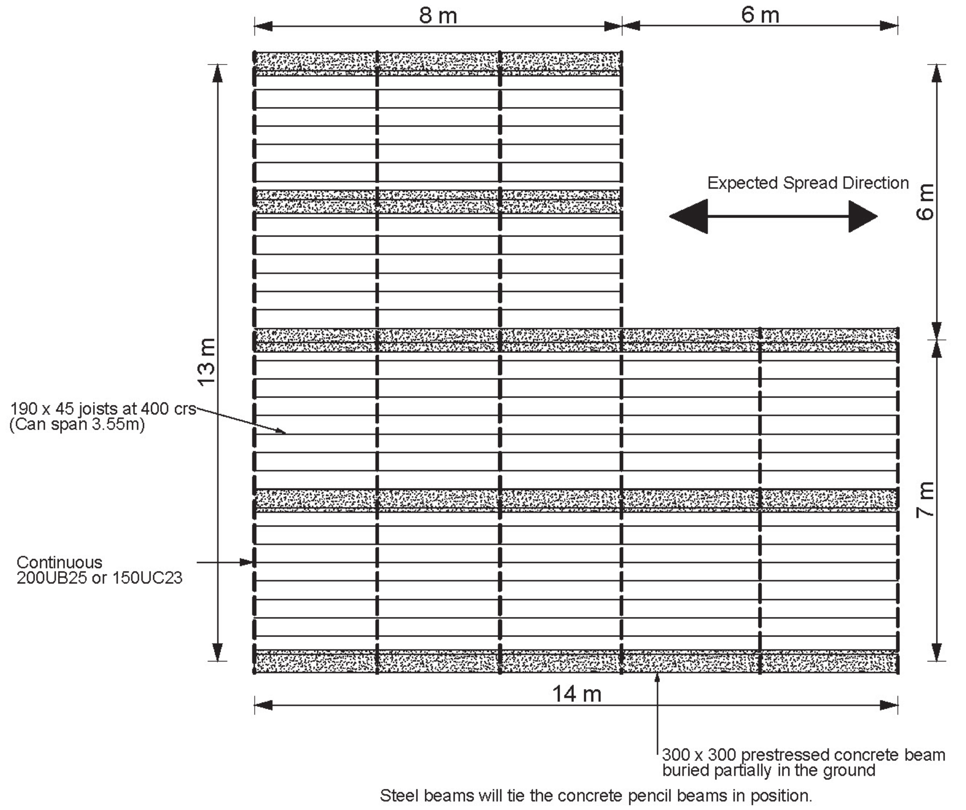
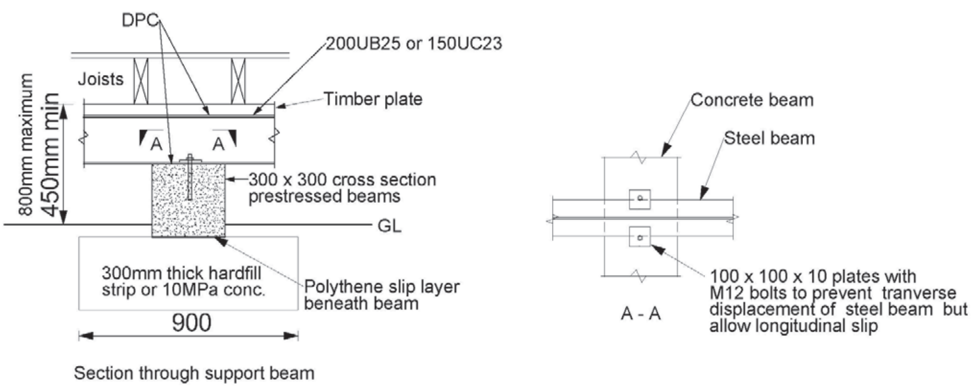


Figure 15.25: Type 3B surface structure – Section through pre-stressed concrete support beam and beam connection



15.4.6 Hybrid TC2/TC3 foundations

Some TC3 sites will 'straddle' the liquefaction settlement limits of TC2 and TC3, where the SLS settlements are assessed as being less than 50 mm, but the ULS settlements are assessed at greater than 100 mm.

In these cases the amenity requirements at SLS under liquefaction conditions would be met by installing a TC2 foundation from Part A of the guidance, but damage might be at unacceptable levels at ULS. A foundation solution that is more robust than normal TC2 foundations is required, but the full requirements of a TC3 foundation solution from section 15.2 (deep piles) or 15.3 (ground improvement) might be unnecessary.

UPDATE: December 2012

In these cases, a combination of the TC2 Option 1 geogrid reinforced gravel raft with either an overlying Option 2 enhanced foundation slab (300 or 400 mm thick) or Option 4 (waffle slab) is recommended. This will provide a foundation system that is robust, and will be repairable (by grout injection) in the event of differential settlements following a ULS event. This is termed a Hybrid TC2/ TC3 foundation.

For a timber-floored house, one of the Type 1 or 2 surface structure options outlined earlier in this section is recommended.

In order to have determined that a site fits into this category, a deep geotechnical investigation must be carried out on the site in question (ie, if an area-wide investigation is being relied on, at least one deep CPT is still required on the site). However, where no significant liquefaction damage has occurred on the site (and this is the basis of ruling out SLS damaging settlements in areas that have been well tested beyond SLS levels of shaking), the area-wide investigation can be relied on, with only a shallow investigation being carried out on the site.

15.4.7 Particular geotechnical investigation requirements

UPDATE: December 2012

All concepts are surface structures that accept the possibility of (readily repairable) future liquefaction-induced deformations, and the key criterion is that they are adequately supported under dead and live loads. Therefore, once appropriately selected following consideration of deep geotechnical information (ie, either a site-specific investigation or appropriate area-wide information), a shallow soil investigation in accordance with the requirements for soil investigation for NZS 3604 structures is suitable. The proviso that where practical the hand auger should be taken down to 3 to 4 m (in other words a shallow investigation as described in Part A, section 3.4.1) applies.

A further engineering assessment of suitability is required, based on observations of foundation damage to any structure that is or was on the site. If the structure has or had undergone an obvious severe punching-mode failure of the foundations (or if the non-liquefiable surface crust appears to be less than a metre thick), then Type 2 surface structures (short timber piles retained in a reinforced concrete ground slab) are the preferred surface solution (or otherwise revert to a ground improvement or piling option if appropriate).

15.4.8 Design approaches and parameters

Table 15.6 summarises the alignment of the surface structure types with the range of shallow foundation options across Technical Categories 2 and 3, including the 'hybrid' TC2/TC3 foundation category described in section 15.4.6.

The corresponding performance expectations, design considerations and superstructure constraints across these technical categories are indicated.

Geotechnical considerations

UPDATE: December 2012

The Type 1 and 2 surface structure options can be specified as a standard solution when the established soil bearing capacity equals or exceeds 200 kPa geotechnical ultimate bearing capacity (or a specific assessment carried out in accordance with Part A, section 3.4.1), and the superstructure is constructed within the constraints specified in section 15.4.2. A 200 kPa geotechnical ultimate bearing capacity can be established (or specific engineering assessment carried out) in accordance with Part A, section 3.4.1. An engineering assessment is also required to establish whether or not SLS settlements (assessed over the upper 10 m of the soil profile) are less than 100 mm and whether or not the site is subject to only 'minor to moderate' lateral stretch (refer section 12.2).

The Type 3 surface structure concepts require that the foundations are sized in accordance with the assessed design loads and the soil bearing capacity (as assessed from a shallow investigation). An assessment is required to ensure that the site is not in a 'severe' lateral stretch area (refer section 12.2).

Shear stresses, and therefore tension forces, transferred from the ground to the foundation system can be calculated for Type 3 structures by assuming that lateral movement occurs under half of the structure, and applying a suitable soil/structure interface friction angle. For Type 3A structures particularly, account will also need to be taken of passive pressures on the 'upslope' side of any foundation elements that extend below ground level.

Where expected future lateral spread movements cannot be confidently determined to be strongly uni-directional, movements orthogonal (or a component of such) to the foundation system may also need to be considered (ie, shear and moment may also be induced in the foundation system as well as pure tensional forces). These concepts have the capacity to accommodate spreading in all directions, although some are likely to perform better than others.

UPDATE: December 2012

Relevellable concrete surface structures

Alternative concrete foundation options are likely to be developed by engineers for particular situations.

Requirements and performance expectations for TC3 specifically designed concrete foundations as follows:

Requirements/scope of application:

1. The application of such systems is limited to sites where less than 100 mm SLS settlement is expected (calculated over the upper 10 m of the soil profile).
2. A geotechnical engineer should assess deep geotechnical information (either site-specific or area-wide information) as per the current requirements for surface structures in section 15.4.7 (as amended).
3. The finished floor level is to be a minimum of 300 mm above adjacent ground or on sloping sites a minimum of 250 mm and an average of 300 mm above adjacent ground. Note that flood-level requirements may result in greater heights above adjacent ground. NZS 3604 clearances above adjacent ground and E1/AS1 clearances must also be complied with.
4. Foundations to support an NZS 3604 superstructure with light-weight roof claddings and limited to light or medium-weight wall claddings.
5. Relevelling can be carried out with non-specialist equipment, techniques or materials.

Key performance expectations:

1. A stiff foundation plate that can span between any temporary point load support during the relevelling process. This will typically involve the use of a suitably designed and detailed underslab to jack against during relevelling.
2. Floor plate curvatures under differential ground settlement in the load condition of G + 0.3Q should be less than 1 in 400 (ie, 5 mm hog or sag at the centre of a 4 m length) for the case of no support over 4 m, and no more than 1 in 200 for the case of no support of a 2 m cantilever at the extremes of the floor.
3. Foundation is readily relevellable – can be lifted after any settlement event and again in subsequent events.
4. The relevelling and repair (including any associated superstructure damage) can be completed within a 4-week period during which the occupants may have to be relocated.
5. No damage to services within the floor plate and readily repairable at the outside of the foundation following the earthquake and during the relevelling process.
6. The relevellable system should provide sufficient resistance to lateral displacement of the foundation under earthquake ground shaking expected in an ultimate limit state design event.

UPDATE:
December 2012

Table 15.6: Shallow foundation solution alignment – Vertical settlement

	TC2 Foundations	Hybrid TC2/ TC3 Foundations	TC3 Foundations		
Land Settlement Demand	SLS <50 mm ULS<100 mm	SLS <50 mm ULS>100 mm	SLS <100 mm	SLS <200 mm	SLS >200 mm
Construction	Timber: NZS 3604 timber floor and shallow piles Concrete: NZS 3604 slab and 800 mm gravel raft (Option 1) or flat slab (Option 2), ribbed slab (Option 3) or waffle slab (Option 4)	Timber: TC3 Surface Structures Concrete: 300 mm flat slab (Option 2) with gravel raft (Option 1)	Timber floor on enhanced NZS 3604 subfloor (**Type 1 surface structure**) Or Timber floor over concrete underslab on gravel raft (**Type 2A surface structure**)	Timber floor over concrete underslab on gravel raft (**Type 2B surface structure**)	Specifically designed subfloor grid (**Type 3 surface structure**)
				Or ground improvement and Type 1 or 2 timber-floored surface structure – refer to section 15.3.4	
Structure Performance Outcome Anticipated	Minor/ slight differential settlement (ie <25 mm SLS, <50 mm ULS)	Minor/ slight differential settlement (ie. <25 mm SLS) Limited damage to foundations at ULS	Readily repairable damage may well occur at SLS Limited damage to foundations at ULS		
Design Considerations	Provision has been made in standard solutions to accommodate effects of **minor** differential settlement at SLS and ULS should it occur		Provision has been made in standard solutions for Type 1 & 2 surface structures to accommodate effects of **minor to moderate** differential settlement at SLS (ready repairability) and at ULS (life safety and some repairability)		Provision must be made in specific engineering design solution Type 3 surface structures to accommodate effects of **significant** vertical settlement at both SLS and ULS (as determined from deep geotechnical information)
Superstructure Constraints	Timber ground floor: Light or medium wall cladding combined with light roofs Concrete ground floor: Refer to table 7.2 for wall and roof cladding weight limits		Light or medium wall cladding combined with light roofs, regular superstructures only		

UPDATE: December 2012

Table 15.6: Shallow foundation solution alignment – Lateral stretch

	TC2 Foundations	Hybrid TC2/ TC3 Foundations	TC3 Foundations	
Lateral Stretch Demand	To resist **minor** lateral spreading ie. <50 mm at SLS <100 mm at ULS		Up to 200 mm at ULS (**minor to moderate**) No expectation of significant lateral spread at SLS	Up to 500 mm at ULS (**major**) Potential for lateral spread at SLS that needs to be addressed in foundation design
Construction	Timber: NZS 3604 timber floor and shallow piles Concrete: NZS 3604 slab and 800 mm gravel raft (Option 1) Or flat slab (Option 2), ribbed slab (Option 3) or waffle slab (Option 4)	Timber: TC3 surface structure Concrete: 300 mm flat slab (Option 2) with gravel raft (Option 1)	Timber floor on enhanced NZS 3604 subfloor (**Type 1 surface structure**)	Timber floor over concrete underslab on gravel raft (**Type 2 surface structure**) or specifically designed subfloor grid (**Type 3 surface structure**)
Structure Performance Outcome Anticipated	No damage to foundation structure associated with lateral spreading is anticipated at SLS Limited damage to foundations at ULS		Repairable damage to foundation, but no superstructure damage from lateral spread at SLS Limited damage to foundations at ULS	Minor damage to superstructure at SLS Limited damage to foundations at ULS
Design Considerations	Provision has been made in standard solutions to accommodate effects of **minor** lateral spreading at SLS and ULS should it occur		Provision has been made in standard solution to accommodate effects of **minor to moderate** lateral stretch should it occur at SLS and to cover life safety aspects and some repairability at ULS	Provision must be made in specific engineering design solution to accommodate effects of **major** lateral stretch at SLS and to cover life safety aspects at ULS. Repairablility at ULS should be considered.
Superstructure Constraints	Timber ground floor: Light or medium wall cladding combined with light roofs Concrete ground floor: Refer to table 7.2 for wall and roof cladding weight limits		Light or medium wall cladding combined with light roofs, regular superstructures only	Light or medium wall cladding combined with light roofs, simple house plan shape

Appendix C

Extracts from the *MBIE Guidance: Repairing and rebuilding house affected by the Canterbury earthquakes*, relating to ground improvement

Extract from: Ministry of Business, Innovation and Employment (2015). "Repairing and rebuilding houses affected by the Canterbury earthquakes", version 3a, Ministry of Business, Innovation and Employment, April 2015.

C 15. NEW FOUNDATIONS

Table 15.4: Summary of ground improvement types covered by this guidance document[1] (grouped by construction methodology)

Group	Type	Description	Nominal depth of treatment below base of foundation	Refer Section	Advantages	Disadvantages	Applicable surface foundation components[7]	
							TC2 Type Foundations[8,9]	
							Concrete slab Type 2 or 4	Type B (ring foundation)
G1 Shallow densified crust	G1a	Excavate and recompact	2m	15.3.10.1(a)	• Can be used in all soil conditions.[2] • Simple construction using typical earth works plant. • Can do on a single, small section (eg compared with G1b). • May be suitable for 'major' lateral stretch zones with additional geogrid.[3]	• Likely to require dewatering where groundwater table (GWT) < 2.3m deep. • Stockpile area required.	Yes Only if pre-treatment SLS < 100mm (or 50mm post treatment)[10] (Otherwise refer to section 15.3.8 for other surface foundation component options)	Yes Only if pre-treatment SLS < 100mm (or 50mm post treatment)[10] (Otherwise refer to section 15.3.8 for other surface foundation component options)
	G1b	Dynamic compaction	4m	15.3.10.1(a)	• Highly effective in clean sands.[4] • Results in thicker improvement zone than some other 'shallow' methods. • No dewatering required. • No stockpile area required.	• Relatively large equipment required, high mobilisation costs. • Vibrations may negatively impact nearby properties • Not effective in silty soils (FC > 15-25% or Ic > 1.8 – 2.3 approx.) • Not suitable in soils with > 5% organics. • Not suitable in 'major' lateral stretch zones. • Not good for small sites/sites with restricted access. • Potentially high mobilisation costs.		
	G1c	Rapid impact compaction	4m	15.3.10.1(a)	• Same as for Type G1b; and, • Faster and more efficient than dynamic compaction for shallow (≤ 4m deep) applications.	• Same as for Type G1b.		
	G1d	Reinforced crushed gravel raft	1.2m	15.3.10.1(b)	• Same as for Type G1a; and, • Shallower excavation and less material handling. • Suitable for use in 'major' lateral stretch zones with additional geogrid.	• Likely to require dewatering where groundwater table (GWT) < 1.5m deep. • Requires select import materials. • Stockpile area required.		
G2 Shallow cement stabilised crust	G2a	Reinforced stabilised crust	1.2m	15.3.10.2(a)	• Can be used in all soil conditions.[5] • Simple construction using typical earth works plant. • Can do on a single, small section (eg compared with G1b). • Stiffer, stronger raft than Types G1a and G1d. • May be suitable for use in 'major' lateral stretch zones with additional geogrid.[3]	• Likely to require dewatering where groundwater table (GWT) < 1.5m deep. • Some specialist contractor knowledge required. • Requires select import materials. • Stockpile area required.		
	G2b	Stabilised crust (In situ mixing)	2m	15.3.10.2(b)	• Can be used in all soil conditions.[5] • No dewatering required.	• Specialist contractor knowledge and equipment required. • Potentially difficult to verify whether target improvement consistently achieved. • Not suitable in 'major' lateral stretch zones without specific engineering design. • Potentially high mobilisation costs.		
G3 Deep soil mixed columns	G3	Deep soil mixed columns	8m	15.3.10.3(a)	• Can be used in all soil conditions. • No dewatering required. • Good for reducing total settlement. • Outside the scope of this guidance in 'major' lateral stretch zones.[3]	• Specialist contractor knowledge and equipment required. • High mobilisation costs. • Not good for small sites/sites with restricted access.	Yes	No (Unless surface components align accurately with discrete subsurface elements as a specific engineering design solution)
G4 Deep stone columns[11]	G4	Deep stone columns	8m	15.3.10.3(b)	• Highly effective in clean sands.[4] • No dewatering required. • Good for reducing total settlement. • Outside the scope of this guidance in 'major' lateral stretch zones.[3]	• Not as effective in siltier soils (FC > 15-25% or Ic > 1.8 – 2.3 approx.)[6] • Specialist contractor knowledge and equipment required. • High mobilisation costs. • Vibrations may negatively impact nearby properties • Not good for small sites/sites with restricted access.		
G5 Crust reinforced with inclusions	G5a	Shallow stone columns[11]	4m	15.3.10.4(a)	• Highly effective in clean sands.[4] • No dewatering required. • Can access relatively small sites. • Outside the scope of this guidance in 'major' lateral stretch zones.[3]	• Not as effective in siltier soils (FC > 15-25% or Ic > 1.8 – 2.3 approx.)[6] • Specialist contractor knowledge and equipment required. • High mobilisation costs. • Stockpile area required.	Yes Only if SLS < 100mm (or 50mm post treatment)	
	G5b	Driven Timber Piles	4m	15.3.10.4(b)	• No dewatering required. • Uses conventional equipment • Can be used on sites with restricted access. • Outside the scope of this guidance in 'major' lateral stretch zones.[3]	• Not as effective as shallow stone columns. • Annulus may form around piles during intense ground shaking, allowing ejection of sediment. • Stock pile area required.		

1 This is only a general summary table. The text of section 15.3 as well as Appendix C4 must be referred to for important details.
2 Silts/clays likely to require blending with imported granular materials. Unsuitable soils such as peat, high plasticity/organic clay/silt must be removed and replaced with imported granular material.
3 Outside the scope of application of this guidance document but may be applicable with specific engineering design. In 'major' lateral stretch areas some restrictions on foundation types apply (refer to Table 15.2).
4 Clean sands generally means having a CPT Ic 1.8 or fines content < 15% approx.
5 Silty/clayey soils will require higher cement contents and careful moisture control; highly organic/peat soils should be removed from backfill material prior to treatment (Type G2a).
6 May still provide acceptable level of improvement in combination with a higher than typical area replacement ratio.
7 Further constraints may be imposed by the consideration of ULS settlements (see section 15.3.8).
8 In some cases TC3 surface foundations may also be applicable (see 15.3.8.2).
9 See Part A, sections 5.3.1 and 5.3.2.
10 Refer sections 15.3.8.2, 13.5 and Q&A's 50&51.
11 Includes columns of highly compacted aggregate.

Extract from: Ministry of Business, Innovation and Employment (May 2016, updated May 2018), Repairing and rebuilding houses affected by the Canterbury earthquakes, Version 0.

Table 5.1: **Summary of results for the Pilot ground improvement construction methods**

Site	Generic name	MBIE guidance ground improvement type	Description	Nominal thickness of treatment below base of foundation	Site characteristics: Suitable for residential areas	Site characteristics: Effective at greenfield subdivision sites (i.e. prior to public sale and release)	Site characteristics: Good method to manage contaminated sites	Construction considerations: Standard construction equipment	Small site access	Approximate construction duration per property (excluding pre-construction preparation and any necessary curing time)	Low risk of weather delays	Low construction monitoring requirements	Low requirement for verification testing	No dewatering or excavation support required	Minimal disposal of materials offsite	Minimal imported materials	Minimal on-site stockpile or working area requirements	Low dust nuisance potential	Low noise and vibration potential	Suitable to construct close to boundary	Low volumes of construction traffic movement to and from site	Liquefaction mitigation: Effective in sandy soil layers	Effective in silty soil layers	Effective in organic and peat layers	Effective in moderate lateral spread	Additional information
CLEARED OR GREENFIELD SITE	**Shallow densified crust (MBIE type G1)**	**G1c**	**Rapid impact compaction**	4.0m	✘	✔	✔	✘	✘	2 to 3 days	Y/N	Y/N	✘	✔	✔	Y/N	✔	Y/N	✘	✘	✔	✔	✘	✘	✘	Fill material required to infill depressions in the ground caused by compaction. Up to 0.5m of fill needed to reinstate.
		G1d	**Reinforced gravel raft**	1.2m	✔	Y/N	✘	✔	Y/N	2 to 3 weeks	✘	Y/N	✘	Y/N	✘	✘	Y/N	✔	Y/N	✔	✘	✔	✔	✔	✔	Gravel needs to be crushed in accordance with the MBIE guidance. Pre ordering of gravel maybe required. Correct grid placement at bottom of excavation can be difficult.
	Shallow cement stabilised crust (MBIE type G2)	**G2a**	**Ex-situ soil-cement mixing**	1.2m	✔	Y/N	✘	✘	Y/N	1 to 2 weeks	✘	✘	✘	Y/N	Y/N	Y/N	✘	✘	Y/N	✔	Y/N	✔	✔	Y/N	✔	Soil can be imported to bring the stabilised crust up to finished level if increased densification achieved during recompaction is high. Increased traffic movement.
		G2a	**Rotovated* soil mixed**	1.2m	✔	Y/N	✘	✔	✔	1 week	✘	✘	✘	Y/N	Y/N	Y/N	Y/N	✘	✔	✔	✔	Y/N	✔	Y/N	✔	An aggregate working platform may be required at base of excavation.
		G2b	**In-situ soil mixing**	2.0m	✔	Y/N	Y/N	✘	✘	1 to 2 weeks	Y/N	✘	✘	✔	Y/N	Y/N	Y/N	Y/N	Y/N	Y/N	✔	✔	✔	✘	✘	Minimum strength requirements and mixing consistency can be difficult to achieve.
	Composite strengthened crust (MBIE type G5)	**G5a**	**Shallow stone columns**	4.0m	Y/N	✔	✔	✘	✘	1 week	Y/N	Y/N	✘** ✔***	✔	✔	✘	Y/N	✔	Y/N	Y/N	Y/N	✔	✘	✘	✘	Ground movement should be monitored at adjoining properties.
		-	**Rammed aggregate piers**	4.0m	✔	✔	✔	✘	✘	1 week	Y/N	Y/N	✘** ✔***	✔	✔	✘	Y/N	✔	Y/N	Y/N	Y/N	✔	Y/N	✘	✘	Ground movement should be monitored at adjoining properties.
		G5b	**Driven timber poles**	4.0	Y/N	Y/N	✔	✔	✔	1 to 2 weeks	✔	✔	✔	✔	✔	✘	Y/N	✔	Y/N	Y/N	✔	Y/N	Y/N	Y/N	✘	No significant implications for properties with contaminated land.
UNDER EXISTING BUILDINGS		**-**	**Horizontal Soil Mixed beams******	2 x 0.5m	✔	✘	Y/N	✘	Y/N	2 weeks	Y/N	Y/N	✘	✔	✔	Y/N	Y/N	✔	Y/N	✘	✔	✔	✔	✘	Y/N	Relatively expensive. Reinstatement works may be required to repair damaged surfaces. Minor ground movement may result in minor changes to internal floors.

Note:

✔ = Yes ✘ = No Y/N = Maybe (potential limitations)

* A variation on ex-situ soil cement mixing.

** Specific design so requires verification testing. A paper highlighting the importance of verification testing by Wotherspoon et al. (2015) is provided in Appendix B.

*** Standard design that is conservative and does not require verification testing.

**** Horizontal soil mixed beams does not achieve the same level of performance as the cleared site ground improvement solutions and is therefore only suitable underneath existing buildings.

Extract from: Earthquake Commission (2015). "Residential Ground Improvement – Findings from trials to manage liquefaction vulnerability", report prepared by Tonkin + Taylor for the Earthquake Commission, October 2015, http://www.eqc.govt.nz/canterbury/ground-improvement-programme