Part C: Assessing, repairing and rebuilding foundations in TC3

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11. Introduction to TC3

11.1 Overview

The guidance provided in Part C focuses on foundation repairs and reconstruction for houses in Foundation Technical Category 3 (TC3) areas within the Green Zone of the earthquake-affected parts of the Canterbury region. It does not apply to the Residential Red Zone where significantly poorer ground conditions exist and more severe land damage is expected in future earthquakes.

Land that has been classified as TC3 in the Green Zone has a higher probability of being at some risk of moderate to significant land damage from liquefaction in future large earthquakes. Specific geotechnical investigations are required to check the likely land performance. Where the TC3 classification is confirmed by investigation, specific engineering design will often be required for the repair or rebuilding of foundations in this technical category.

UPDATE: December 2012

Part C must be read in conjunction with Parts A and B of the guidance. Material from Parts A and B is only repeated where considered necessary.

UPDATE: December 2012

Intended audience

This guidance is intended for the engineering design, construction and insurance sectors, local authorities, and their professional advisors and contractors to clarify the technical and regulatory requirements for TC3 land. Given that most foundation repairs and reconstruction in TC3 require specific engineering input, the principal users of this document will be professional geotechnical and structural engineers.

Decisions regarding the scope of repairs and rebuilding residential dwellings in Technical Category 3 are complex, and are much more reliant on engineering judgement than the other technical categories. Specific input from Chartered Professional Engineers (geotechnical and structural, as appropriate) is therefore required.

As the solutions included in the guidance have not yet been fully prototyped, it is expected that the guidance will need refinement with experience. It is also likely that other solutions and analytical tools will be developed during the repair and rebuilding process that can be incorporated into future versions of this guidance. Future updates will be available online from the Ministry's website www.dbh.govt.nz/guidance-on-repairs-after-earthquake.

Repair and rebuilding strategies and decisions will be influenced by insurance contracts and the decisions made by the parties to those contracts. The engineering considerations and criteria outlined in this document are intended to provide input into those decisions.

11.2 General principles

Part C of the guidance has been prepared based on a series of general principles. These principles have guided the development of the document, and are set out below to assist engineers in the interpretation and implementation of the proposed solutions for individual TC3 sites, and for situations where other solutions are formulated.

Underlying principles

1. Guidance in the document is based on current knowledge, and represents best practice advice prepared by the Ministry, drawing on the expertise of a range of highly experienced New Zealand and international geotechnical and structural engineers.

   The guidance will be updated as new technical information, experience from built solutions, and field test results become available.

2. The potential for land damage from liquefaction on the plains in Canterbury represents a complex continuum - from residential Red Zone areas being vacated where there are saturated loose, unconsolidated silts and sands close to the surface (often in combination with proximity to unrestrained free edges), through areas of more moderate damage potential, to areas that are considered to be of relatively low damage potential designated as TC1.

3. Houses assigned a TC3 categorisation remain in the Canterbury Green Zone, thereby allowing individual repair and rebuild solutions to be developed and constructed. However, houses in this category are on land with a higher potential risk of liquefying than the remainder of the land in the Green Zone. The future performance of this land in a seismic event is the most difficult to predict. Part C of the guidance does, to a certain degree, differentiate those sites within TC3 where future expected land settlement and lateral movement is likely to be less damaging than the remainder of TC3.

4. Residential sites in TC3 with foundation damage require professional engineering input (investigation, assessment and design) to determine what is an appropriate repair or rebuild solution for each particular site (if in fact repair or rebuilding is required). It is noted that for some sites currently designated TC3, deep investigations will demonstrate that TC2 foundation solutions are appropriate.

UPDATE: December 2012

5. The guidance provides design solutions and methods that aim to substantially improve the performance of house foundations in future seismic events, while recognising that the land performance may still induce deformations and loads that could cause some damage.

6. It aims to improve the robustness of foundations to comply with life safety requirements in ultimate limit state (ULS) seismic events (and also provide a level of habitability and potential repairability in that design event) and to minimise damage and repair costs in serviceability limit state (SLS) events. Some damage may result in either design event. The future damage threshold under SLS is ‘readily repairable’; refer to the criteria in Part B, section 8.2.

7. Solutions included in the TC3 guidance attempt to balance the initial costs of improved robustness against the risk of future damage in a seismic event.

8. Following the methods and solutions provided in the document provides 'reasonable grounds' for designers and Building Consent Authorities that the resulting repairs or rebuild will meet the requirements of the Building Code. Refer section 1.3.

   However, given the potential variability of land performance in TC3, solutions provided are not 'Acceptable Solutions' that, if followed, are automatically deemed to comply with the Building Code (refer to section 1.3). Each house repair or rebuild requires close consideration and investigation by Chartered Professional Engineers to ensure that the different constraints and limits included in the guidance are observed, and that an appropriate repair or rebuild option is chosen, for the 'reasonable grounds' provision to be met.

9. Not all solutions are applicable in all areas, and designers need to be satisfied that adequate geotechnical information has been gathered to enable decisions to be made on appropriate designs.

10. Some new foundation solutions provided in the document can be applied without undertaking further detailed engineering analysis. However, others are provided as concepts that require further analysis and development of details, depending on the particular circumstances. It is expected that further solutions will be developed using specific design or testing as the Canterbury rebuild progresses.

Design principles

1. **Light-weight materials, particularly for roof and wall cladding, are preferred for all foundation types,** particularly in any location where liquefaction is possible, as these reduce the inertial loading on foundations and can reduce settlement in future seismic events. Heavier weight construction materials are however not precluded, and could still be used where supported by appropriate engineering advice and careful design of ground improvement or deep pile systems.

2. Removal of heavy materials and replacement using light-weight materials will sometimes allow existing foundations to be repaired rather than rebuilt.

3. Stiffened and tied together foundation solutions are required to improve resistance to lateral stretch and ground deformation. A slip layer beneath shallow foundations or foundation slabs will improve the performance against lateral spreading (stretch) at the surface.

4. Regular structural plan shapes are preferable to more complex plan shapes. A regular house plan is defined as meeting three basic criteria:
   - A base plan shape that is essentially rectangular. In the absence of specific design the guidance is applicable to those footprints with an aspect ratio no greater than 2:1.
   - One major projection (ie, greater than 2 m out from the base shape) is permitted. (This might result in an 'L', 'T' or 'V' shape base plan). The ratio of the projected dimension divided by the length of the side in common with the base shape must be no greater than 1 (in the absence of specific engineering design).
   - Any number of minor projections (ie, 2 m or less) are permitted off the base shape, or off the major projection. Again, the ratio of the projected dimension divided by the length of the side in common must be no greater than 1.

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December 2012

5. Minimising penetrations of the crust (the ground between the surface and the layer that is likely to liquefy) will reduce the likelihood of liquefaction ejection coming to the surface. This principle is followed particularly with the shallow surface solutions and for service trenches where possible. Liquefaction ejection results in soil loss and is a primary mechanism of ground deformation. It is, however, not currently possible to quantify the degree to which this might occur on a site or the resulting damage that may arise.

6. Providing a suspended timber ground floor facilitates simple repair of structures in future events.

7. Mixed foundation systems within the same structure are not recommended in TC3 (eg, Type 1 timber floor house and attached concrete slab garage).

8. The location and accessibility of services needs to be taken into account. It is preferable that new service connections and interfaces are appropriately flexible. Services should enter the building at few well-defined and well-recorded locations, through connections that are as flexible as possible. Should failure occur, this will be in well-defined locations outside the foundation system and services are then easy and quick to reconnect. Plumbing services in particular should be located near outside walls for access for repairability. Services located below floors must be properly restrained to move with the floor and minimise the risk of damage that is difficult to repair. Where slip layers are provided, services must not impede the ability of the foundation system to move laterally (this may require services to be fully enclosed within surface slabs, for example).

11.3 Scope

Canterbury focus

The options and recommendations in this Part of the document are specific to residential properties directly affected by the Canterbury earthquake sequence, **in particular, those properties that have been classified as being in the land Green Zone Technical Category 3 (TC3, sometimes referred to as 'Green-Blue'**). Although the guidance provides information on reducing the effects of future liquefaction on residential properties in the TC3 land category, this should not necessarily be taken as a best practice guide for addressing liquefaction in other parts of Canterbury or New Zealand.

National best practice guidance for the design of residential dwellings to take account of potential liquefaction will be prepared in due course, and will draw on information in this document.

Types of dwelling addressed

This document focuses principally on one- and two-storey timber or steel-framed dwellings, which are the dominant form of construction in the affected area. Accordingly, the document refers to the timber-framed buildings Standard, NZS 3604.

Technical scope

Part C provides guidance on foundation repairs and reconstruction within the TC3 land category. The document does not cover all situations, for example, sites where severe lateral movement is anticipated.

Information in Part A on foundation assessment criteria and approaches, retaining walls and superstructure assessment and repairs can be directly applied to TC3 properties.

Repairs for foundation damage

The extent and method of repairs requires careful consideration, including an understanding of what is practically achievable. In many cases where minor or moderate damage or settlement has occurred, it is considered that foundations and floors can be repaired and relevelled.

Repair approaches for the foundations of dwellings affected by settlement are described in section 14.

In some cases where the foundations have sustained significant damage and require replacement, only relatively minor damage has occurred to the house superstructure above (wall and roof framing, linings and cladding). In these cases, it may be appropriate to lift up and move the house and construct new foundations and floors. These situations are treated in the first instance as new foundations, covered in section 15.

New and rebuilt foundations

To mitigate the effects of liquefaction, as a guiding principle **it is preferable to build using light materials rather than heavy materials**. Light construction (roof, walls and floors) significantly reduces the imposed load on the subsoils, thereby reducing the settlement potential – for example, a light-weight dwelling imposes as little as 30% of the weight around the perimeter compared to that imposed by a heavy roof, masonry cladding and concrete slab dwelling. Recent research has also demonstrated that decreasing horizontal inertial loads decreases the propensity for vertical settlements during liquefaction events from soil-structure interaction "ratcheting".

UPDATE:
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It has been observed that houses of light-weight construction have suffered significantly less damage and are likely to be significantly less expensive to repair than houses constructed from heavier materials, especially in TC3 areas. This guidance provides some foundation solutions that enable other forms and weights of cladding material for some areas of TC3.

This document provides information on the relevant engineering principles and parameters to be adopted for a foundation and floor system that complies with the Building Code and is therefore capable of gaining a building consent. This should assist the engineers undertaking specific structural and geotechnical engineering design, and inform discussions with insurers as to whether the proposed solution falls within the scope of the insurance policy.

Approaches for the construction of new foundations for dwellings in TC3 are described in section 15.

For new foundations, the following three broad types are described:

- deep piles
- site ground improvement
- surface structures

It should be noted that some solutions will not be practical in all areas of TC3. Deep piles, for example, are not viable solutions in all parts of TC3 due to the potential for excessive lateral deformations from global lateral movement in some areas.

For each foundation type, possible options are indicated. Guidance as to the suitability and applicability of the new foundation options is outlined. Design parameters and specification and construction guidance are provided as appropriate. Some options involve standard solutions (eg, modified NZS 3604).

Although the level of guidance provided varies between the new foundation types, all require specific engineering design input. Selection guidance and key design parameters are provided to enable this design input to be undertaken.

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.

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 by a suitably detailed structure on a TC2 type foundation system. For these types of structures in TC3, the provisions of the guidance for TC2 areas can therefore be applied for rebuilds, repairs and relevelling. 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. For surface structure solutions (see section 15.4) this will put some limits on the type of foundation system selected in order to avoid differential movement.

11.4 Future guidance for TC3

The formulation of the TC3 guidance has been undertaken within a limited timeframe to allow solutions to be provided for TC3 sites that will allow repairs and rebuilding to get underway.

The guidance document will be updated and revised as greater understanding is gained of the earthquake sequence and its impact on the land and on structural performance, and improved or refined solutions are developed.

UPDATE: December 2012

On-going work is anticipated to result in updating of the guidance including:

- Resolution by EQC of land repair strategies for relevant affected properties. The Earthquake Commission will soon clarify details of EQC land insurance cover for TC3 areas. This will include damage thresholds for various land damage types. These thresholds may be different from the thresholds applicable for the TC3 building options set out in this Guidance Document. EQC insurance cover for land damage is separate from insurance cover for building damage.
- Liquefaction settlement analysis. Limits provided in the document are considered as 'indices' (ie, not exact calculations, which in practice are not achievable). Research work is underway to compare the actual performance of land to theoretical calculated settlements. Different assessment methods may be recommended as a result of this work.
- Further consideration of issues raised by practitioners and interested parties from the limited consultation period during the development of the guidance.
- Refinement of the foundation solutions as experience of the options is gained.
- Establishment of a suitable standard engineering sign-off statement for a range of repair and rebuild situations which require further dialogue between the BCAs and consulting engineers.
- Peak ground acceleration (PGA) values to use for general geotechnical design and for other soil classes, refer Appendix C2.

12. Future land performance in TC3

12.1 Background

"To clarify repair and reconstruction options, residential properties in the CERA Green Zone on the flat have been assigned (on an area-wide basis) one of three foundation technical categories (TC1, TC2 and TC3) that reflect both the liquefaction experienced to date and future performance expectations." (refer to Part A, section 3.1).

The basis for and description of the foundation technical categories is given in Part A, section 3.

The future land performance expectations for each of the technical categories are outlined in Table 3.1 in Part A.

12.2 Lateral spreading and other lateral ground movements in TC3

Significant lateral spreading and other lateral ground movements occurred to some properties in TC3 areas during the recent earthquake sequence. Most of the affected sites experienced the greatest lateral movements from the 4 September 2010 and 22 February 2011 earthquakes, with more moderate or no significant movements from the later aftershocks. Generally more significant and extensive movements occurred close to the larger rivers and streams, with more localised lateral movements occurring adjacent to smaller stream channels and sloping ground. The areas where the most severe and extensive lateral spreading occurred have since been red-zoned by CERA (ie, they are not within TC3 areas).

The potential for future lateral ground movements in TC3 areas can be reasonably inferred from land damage experienced in the Canterbury earthquake sequence, provided that the site has been "tested" by sufficiently high ground shaking during these earthquakes. These observations can be supplemented by applying well-known engineering principles of susceptibility to lateral spreading (eg, proximity to a rapid change in ground level, or free edge) when assessing future lateral spreading potential.

The focus of categorising global lateral movement is based on an ultimate limit state (ULS) design earthquake event. Structures which are designed in accordance with the TC3 guidance to tolerate the lateral ground movements possible in a ULS event would be expected to also tolerate the lateral ground movements possible in a SLS event.

The potential for future lateral ground movements is defined in the document to enable the design engineer to assess the effect from the earthquake sequence, given the passage of time since the liquefaction events. Caution must be exercised where figures for ground movement have been specified in the document.

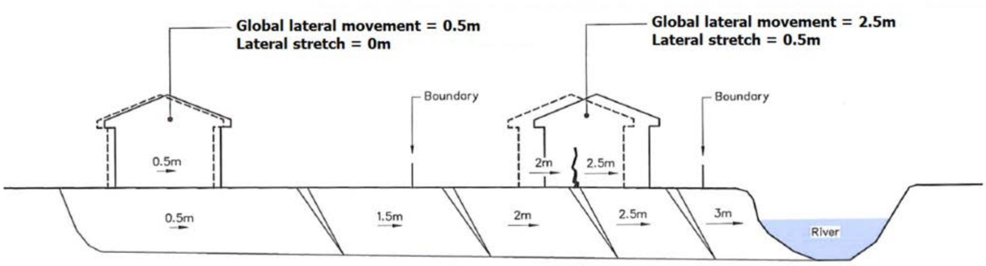
Two components of potential lateral movement need to be considered when designing repaired or rebuilt foundations in those areas with the potential for lateral ground movements. They are:

- Global lateral movement of a site
- Lateral stretch of the ground surface across a building footprint

UPDATE: December 2012

These two components of lateral ground movement are shown in the simplified cross-section in Figure 12.1. Lateral spreading in the majority of cases tends to result in blocks of land moving laterally towards a free edge. More lateral movement tends to occur in the blocks closest to the edge with progressively less movement of blocks further back. For dwellings which are located entirely within an intact block, the entire structure and the block of land beneath it move together as one (global lateral movement). In this case there has been global lateral movement, but no differential lateral movement (ie, stretching) between different parts of the superstructure. If the structure straddles adjacent blocks, then in addition to the global component of lateral movement, there can also be stretching and tearing of the ground beneath the structure. This stretching of the ground (lateral stretch) can introduce significant lateral forces into the foundation elements and superstructure.

Figure 12.1: Simplified cross-section showing components of lateral ground movement (values illustrative only)



12.2.1 Global lateral movement of a site

The global component of lateral ground movement does not greatly affect the design and performance of shallow foundations, such as footings, rafts or shallow piles which are founded within the surface blocks of land. The entire superstructure and foundation is able to move as one along with the global movement of the block.

For deep piles this global component of lateral ground movement has significance for design. While the superstructure and upper portion of the piles are moved sideways by the surface blocks, the lower portion of the piles will be designed to be embedded into non-liquefied ground at depth below the blocks where there is minimal lateral ground movement. The piles are therefore required to withstand the effects of displacement of the top of the pile relative to the toe. Accordingly, many common deep pile systems and foundation details may not be appropriate in areas with the potential for major global lateral movements in future earthquakes.

The following generalised global lateral movement categories have been developed to aid foundation design in Technical Category 3:

Table 12.1: Global lateral movement categories for TC3 (at ULS)

Minor to Moderate	Major	Severe
0 to 300 mm global lateral movement	300 to 500 mm global lateral movement	>500 mm global lateral movement *generally not expected in TC3 areas*

All the new foundation options outlined in section 15 for TC3 are applicable for sites in the **minor to moderate** global lateral movement category. For sites in the **major** global lateral movement category, deep pile foundations are unlikely to be suitable unless careful pile type selection and specific engineering design is undertaken (refer to sections 15.2.5 and 15.2.6). However, some of the ground improvement and surface structure options in section 15 are likely to be appropriate for sites in the **major** global lateral movement category.

For sites in the Severe global lateral movement category (expected to be rare in TC3), more substantial engineering works (for example more robust ground improvement schemes, beyond the scope of this document) are likely to be needed.

Procedure for assessing global lateral movement of a site

For the purposes of repair and rebuilding of foundations in TC3, the following procedure is recommended for assessing the **global lateral movement** category for the site (ie, the building footprint):

1. Undertake a desk study of available information, such as post-earthquake observations, results from regional-scale data analysis, geotechnical investigations, and ground-level profiles. Identify potential triggers for lateral ground movement.
2. Physically examine the site, immediate neighbourhood and any structures which remain on the site for evidence of lateral ground movements (eg, cracks in the ground or foundations, damage to kerbs and paths, deformation of fences, offset services etc). A lower-bound estimate of the global ground movement that has occurred can be made by summing observed crack and offset widths across the site and immediate surrounds and to the free edge.
3. Check whether the site is in an area of higher or gently-sloping ground which may be susceptible to suburb-scale lateral ground movements caused by elevation differences if the underlying soil liquefies. This type of large-scale movement has the potential to cause significant global lateral ground movements. However, as it causes only minor ground stretching, and thus little damage to surface structures, it may not be apparent from site observations that large global displacement has occurred. As a minimum, it is recommended that sites within the areas listed in Table 12.2 are assumed to be in the Major global lateral movement category. Deep piles are unlikely to be an appropriate foundation option in these areas without careful specific design. This is unlikely to be an issue for residential structures because the higher ground (and thus thicker crust) in these areas means that the shallower foundation solutions for TC3 properties outlined in section 15 are likely to be appropriate.

Table 12.2: Areas of major global lateral ground movements identified within TC3 to date

North New Brighton – All TC3 properties east of Anzac Drive, South of Queenspark Drive, and North of New Brighton Rd.
Wainoni – All TC3 properties within the area bounded by Wainoni Rd, Shortland St, Pages Rd, Kearneys Rd, Cypress St, Ruru Rd, McGregors Rd, Pages Rd and Cuffs Rd.

4. In some cases, if observation-based assessment is inconclusive, it may be beneficial to undertake geotechnical analysis to provide a theoretical prediction of lateral ground movements.
5. If the assessment undertaken in the previous steps provides insufficient evidence for a global lateral movement category to be assigned, then, as a fall-back option, the category may be selected based on a simplified criteria of distance from a free edge. If there is no evidence to the contrary, then sites may be assumed to be in the **minor to moderate** global lateral movement category if the distance to a free edge is greater than specified in Table 12.3. For sites closer to the free edge, the **major** global lateral movement category may be more appropriate.

Table 12.3: Distance from free edge beyond which minor to moderate global lateral movement can be assumed in TC3 (excluding areas in Table 12.2), in the absence of any evidence to the contrary

Location	Distance
Avon River, downstream of Banks Ave (including estuary)	200 m
Avon River, between Barbadoes St and Banks Ave	150 m
Avon River, between Mona Vale and Barbadoes St	100 m
Heathcote River, downstream of Colombo St	100 m
Dudley Creek and tributaries, east of Hills Rd	100 m
All other significant waterways and steep changes in ground level	50 m

12.2.2 Lateral stretch of the ground across a building footprint

The degree of lateral stretching of the ground which may occur across a building footprint in future earthquakes is typically significant when considering the design and performance of both deep and shallow residential foundation options. Stretching of the ground can introduce significant lateral forces into the foundation elements and superstructure. It is therefore crucial that the magnitude of possible future ground stretching is assessed when selecting and detailing a foundation system. If lateral stretch of the ground is possible, the foundation solution should have the capacity to prevent tearing of the structure, provide a low probability of structural collapse, and ideally also offer resilience and ease of repair.

Table 12.4 summarises the generalised lateral ground stretching for which categories have been developed to aid foundation design in TC3. It should be noted that there will be some sites which fall into different categories for global lateral movement than for lateral stretch (eg, some sites may have **major** global lateral movement, but only **minor to moderate** lateral stretch across the building footprint).

Table 12.4: Categories of lateral stretch of the ground across a building footprint for TC3 (at ULS)

Minor to Moderate	Major	Severe
0 to 200 mm lateral stretch across building footprint	200 to 500 mm lateral stretch across building footprint	>500 mm lateral stretch across building footprint *Generally not expected in TC3 areas*

All the new foundation options outlined for TC3 properties in section 15 are applicable for sites in the **minor to moderate** category of lateral stretch across the building footprint. For sites in the **major** lateral stretch category, several of these foundation options are considered suitable, refer to section 15 for further details. For sites in the **severe** lateral stretch category, which are expected to be rare in TC3, more substantial engineering works are likely to be needed. Such works are beyond the scope of this document.

Procedure for assessing lateral stretch across a building footprint

For the purposes of repair and rebuilding of foundations in TC3, the following procedure is recommended for assessing the **lateral stretch of the ground across a building footprint**:

1. Undertake a desk study of available information, such as post-earthquake observations, results from regional-scale data analysis, geotechnical investigations, and ground-level profiles. Identify potential triggers for lateral ground movement.
2. Physically examine the site, immediate neighbourhood and any structures which remain on the site for evidence of lateral ground movements (eg, cracks in the ground or foundations, damage to kerbs and paths, deformation of fences, offset services etc). An estimate of the lateral ground stretch which has occurred across a building during the earthquake sequence can be made by summing observed crack and offset widths across the footprint. When estimating the stretch across the footprint that may be possible in future earthquakes any stretching observed on the rest of the site and immediate surroundings should also be noted. An assessment should also be made of the potential for this type of stretching to occur under the building footprint in future. Observed patterns of ground cracking may provide useful information but might not reliably predict the exact location of future stretching. (A more complete engineering understanding of the mechanism of ground movement would be required to assess the potential for future ground stretching to affect the building).
3. Review information made available on CERA's Canterbury Geotechnical Database.
4. In some cases, if observation-based assessment is inconclusive, it may be beneficial to undertake geotechnical analysis to provide a theoretical prediction of lateral ground movement and lateral stretch.
5. If the assessment undertaken in the previous steps provides insufficient evidence for a lateral stretch category to be assigned, then as a fall-back option the category may be selected based on a simplified criteria of distance from a free edge. If there is no evidence to the contrary, then sites may be assumed to be in the **minor to moderate** lateral stretch category if the distance to a free edge is greater than specified in Table 12.3. For sites closer to the free edge, the **major** lateral stretch category may be more appropriate.

12.3 Vertical settlement in TC3

Significant vertical settlements occurred in the majority of properties in TC3 areas during the recent earthquake sequence. In some locations these settlements were damaging and obvious (ie, caused differential movement of foundations or were associated with surface cracking and ejection of liquefied soils) and in other cases the movement was uniform enough across a site to cause minor or no damage to foundation elements.

UPDATE: December 2012

The general objective of deep geotechnical investigations in TC3 is to establish the extent and potential for future liquefaction-induced ground settlement and if required for pile founding or design of ground improvement.

For the foundation repair options and most new foundation types, it is especially important to understand the potential level of vertical settlement from future liquefaction in SLS events, where it is desirable to limit damage as much as possible. It is also useful to understand the potential for deformations at ULS, where 'life safety' and 'repairability' is more the focus.

It is recognised that the calculation of liquefaction-induced settlements is an inexact process. The current calculation methods are the 'set of tools' available to engineers for routine analyses at this time. In order to characterise the potential behaviour of the site and to effectively subdivide the TC3 land into 'less' and 'more vulnerable' categories an 'index number' for TC3 properties has been developed. This index reflects the consequential effects of settlement, taking into account the behaviour of the shallower soils being more influential than that of deeper soils.

The calculation of vertical consolidation settlement of the upper 10 m of the soil profile under SLS loadings has been chosen as the basis for this 'index number'. The index value for the division has currently been set at 100 mm to help guide the selection of suitable repair and rebuild options.

Two categories of vertical land settlement from liquefaction at SLS are therefore established, as follows and detailed in Table 12.5:

(i) Less than 100 mm (calculated over the upper 10 m of the soil profile)

(ii) Greater than 100 mm (calculated over the upper 10 m of the soil profile)

Table 12.5: Categories of vertical land settlement (index values at SLS)

Minor to Moderate	Potentially Significant
<100 mm	>100 mm

Guidance for calculating liquefaction-induced settlements is provided in section 13.5. To ensure consistency in approach and outcome for homeowners, *for the purpose of this document* all practitioners will need to adopt a common calculation method for assessing settlements.

13 Geotechnical investigations in TC3 – general

13.1 General

The scope of a deep geotechnical investigation must be determined by the geotechnical professional responsible for giving advice on the property in question.

The geotechnical professional must be either:

- a CPEng. geotechnical engineer or
- for the purposes of this document, in relation to ground investigations for singular residential properties, a PEngGeol. engineering geologist with competence, suitable relevant training and experience in foundation investigations and liquefaction assessment.

UPDATE: December 2012

Professionals are reminded that they are bound by the IPENZ Code of Ethical Conduct, which states (Rule 46) that the professional must **undertake engineering activities only within his or her competence**. Practitioners who do not have the requisite competence and suitable geotechnical training, qualifications and experience must seek the oversight of a CPEng. geotechnical engineer.

UPDATE: December 2012

Residential sites in Technical Category 3 will require a greater scope of geotechnical investigations than those required in Technical Categories 1 and 2. These investigations are required to better understand local site conditions so that informed engineering judgements can be made on the appropriate foundation solution for the site. Suburb-wide geotechnical investigations have been undertaken in most areas within TC3 in the Christchurch area. Those investigations are typically spaced hundreds of metres apart. Due to the significant local variability in ground conditions in the TC3 areas more site specific information is considered necessary to enable specific design at a site and to make sound engineering judgements.

It is anticipated that there will be two general styles of investigations:

- **Single or isolated house site investigation** – House sites which have geotechnical investigations undertaken as stand-alone projects, generally in isolation from or in advance of other investigations
- **Area-wide investigations** – House sites which have geotechnical investigations undertaken in the same general location as multiple other sites (ie, 'area-wide' investigations)

In addition to these two general investigation strategies, investigation requirements vary for repaired and rebuilt foundations. Further details of these requirements are covered in section 13.4.

The general requirements for geotechnical investigations in TC3 are presented diagrammatically in Table 13.1 and Figure 13.1.

Table 13.1: Summary relationship between likely final investigation densities and foundation types

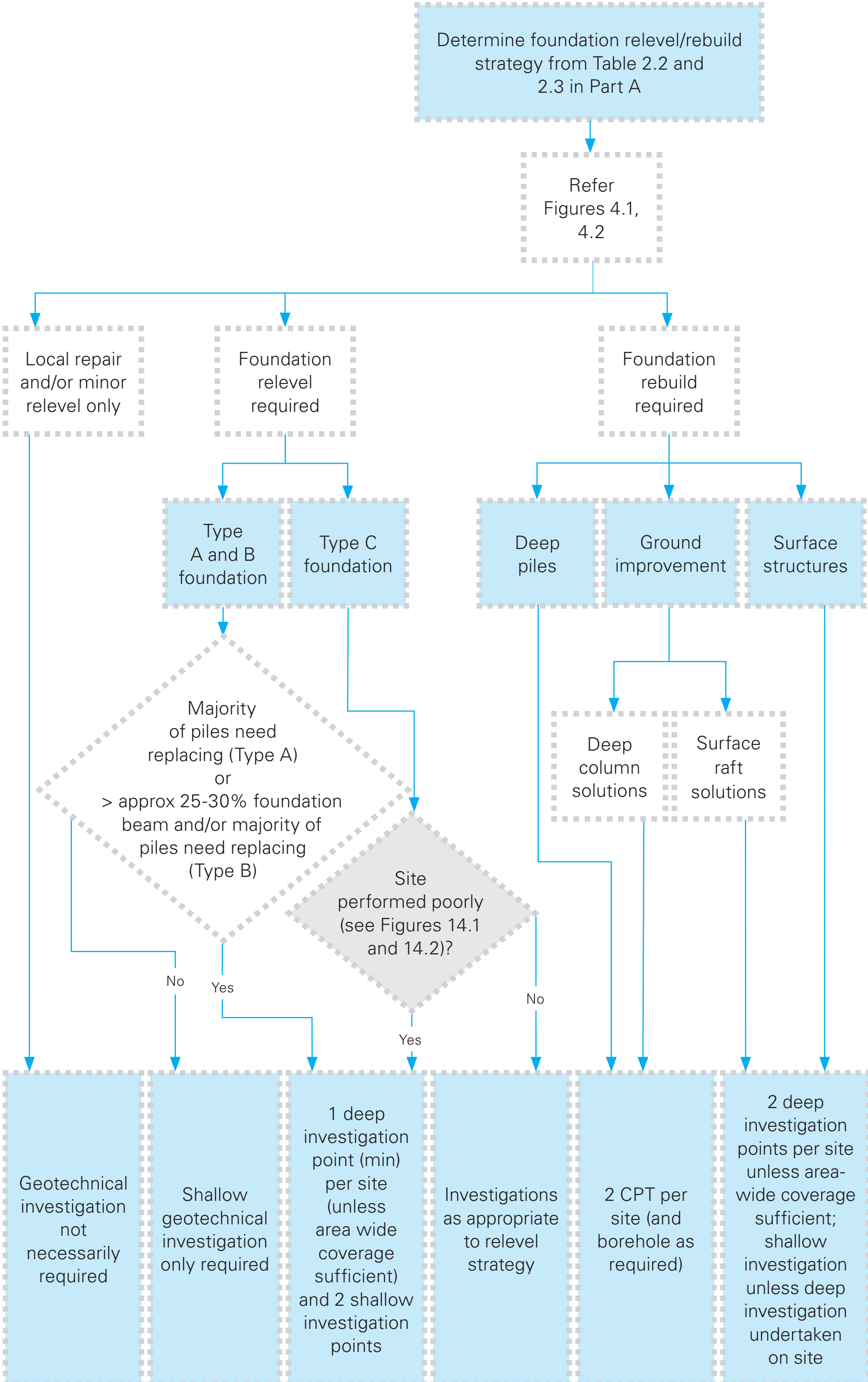
UPDATE: December 2012

Strategy	Foundation Solution	CPTs		Boreholes	Shallow Investigations
Repaired Foundations	**No foundation relevel required** (refer Table 2.3 in Part A and Figures 14.1 & 14.2)	Not required		Not required	Not required
	Foundation repair and/or minor (local) relevel required (refer Table 2.3 in Part A and Figures 14.1 & 14.2)	Not required		Not required	Not generally required
	Foundation relevel required (refer Table 2.3 in Part A and Figures 14.1 & 14.2)	Type A & B	Probably not required (at the discretion of the geotechnical professional)	Not required	2 per site
		Type C	As appropriate to relevel strategy or 1 per site on poor sites unless area-wide investigation adequate	Not required	2 per site

Strategy	Foundation Solution	CPTs	Boreholes	Shallow Investigations
Rebuilt Foundations	**Deep piles** (refer section 15.2)	2 per site where achievable	1 per site if CPT encounters a dense layer and does not prove adequate depth or consistency	Not generally required
	Ground improvement (refer section 15.3)	Subject to improvement option utilised: (refer Figure 13.1) 2 per site unless (at the sole discretion of the geotechnical professional) area-wide investigation results are considered adequate	Probably not required (at the sole discretion of the geotechnical professional)	2-4 per site (if deep investigations not undertaken on the site) or supplementary investigations to identify soil types in treated zone as specified by method statement (refer Appendix C4) or geotechnical professional
	Surface structures (refer section 15.4)	2 per site unless (at the sole discretion of the geotechnical professional) area-wide investigation results are considered adequate	Unlikely to be required (at the sole discretion of the geotechnical professional)	2-4 per site (if deep investigations not undertaken on the site)

Note: Site conditions and chosen solutions may dictate that more investigation is required than indicated above (see the following sections as appropriate 14.2.2, 15.2.4, 15.3.3, 15.4.7)

Figure 13.1: Overview of general geotechnical investigation required



Note: Site conditions may dictate additional investigations to those indicated above.

13.2 Single or isolated house site investigation

The geotechnical investigation process in TC3 should broadly follow the subdivision investigation requirements set out in Part D, under the guidance of a CPEng. geotechnical engineer or suitably experienced PEngGeol. engineering geologist.

Where practical at least two deep investigation points (CPTs, boreholes with SPTs, etc) should be undertaken to enable site characterisation to 10–15 m depth. This might be achieved in conjunction with nearby existing deeper information where it is feasible on or immediately adjacent to the site.

UPDATE: December 2012

Given the relative cost of CPT data it is considered best practice to push CPTs to refusal, however where there are very deep deposits (for example in excess of 20 m) of penetrable materials some judgement is required regarding the usefulness of the deeper information. It must be recognised also that early termination of CPT investigation depths may result in loss of potentially useful information regarding possible pile founding depths, ground improvement options, overall site settlements and general site characterisation. Conversely, while a minimum target depth of 15 m is recommended (and early termination at this depth is not encouraged), if CPTs refuse at between 10 m and 15 m depth the cost of a physical borehole to gain additional information may not be warranted in the first instance, in all cases.

UPDATE: December 2012

It is recognised that CPT data is generally superior to SPT data in determining liquefaction susceptibility, and therefore CPTs will normally be carried out in preference to SPTs. CPT equipment should be calibrated, and procedures carried out, to ASTM D5778-12. Where ground conditions dictate the need for SPTs it is important that equipment that has been properly energy rated is used so that an appropriate energy ratio can be used to correct SPT 'N' values.

UPDATE: December 2012

In many cases only a single location will be initially feasible (due to access considerations and other constraints). In some cases where CPT testing is hampered by gravel layers, a single borehole with SPT testing may be appropriate, augmented by shallower investigations. It will then be up to the judgement of the CPEng. Geotechnical Engineer or PEngGeol. whether these may be supplemented by additional shallow investigations, geophysical testing and/or if further deep investigation points are necessary (either during the initial investigation phase, or possibly post-demolition where this occurs).

Groundwater measurements during the investigations should also be undertaken. Liquefaction assessments should be carried out following the guidelines in section 13.5, as well as further analyses appropriate to the particular foundation or ground remediation solutions being considered for the site.

In addition to the above deep investigations, shallow testing (in accordance with TC2 requirements) can be used to supplement the deep investigations as required.

13.3 Area-wide investigations

Where a large number of house sites are to be grouped together for an area-wide or suburb-by-suburb investigation, and the area-wide investigation shows ground conditions to be relatively consistent, the number of investigation points may be able to be reduced and still allow analyses of individual house sites based on the information from an area-wide investigation. The use or application of area-wide investigations can be applied by engineers whether they are working on multiple properties for a specific client (such as a PMO Engineer working for EQC or an insurer) or on an individual site for a property owner, where deemed appropriate by the engineer.

Such a reduction of investigation density will have to be at the discretion of the CPEng. geotechnical engineer or suitably trained and experienced PEngGeol. engineering geologist for each specific site. The density will need to be such that geotechnical professionals are comfortable with the likely quality of data and proximity of data points to the house sites they are working on. **The density of investigations is expected to be in the order of six to eight investigations per hectare**. Further investigation points may be required, depending on the consistency and quality of the data obtained, the type of foundation solution being considered for a particular site, and the underlying soil conditions. These factors may have considerable influence on the final amount of geotechnical investigation carried out. Where deep piles are opted for, more intense site-specific investigations, are likely to be necessary. In addition to the above deep investigations, shallow testing (in accordance with TC2 requirements) can be used to supplement the deep investigations as required.

13.4 Geotechnical investigation requirements for repaired and rebuilt foundations

Different geotechnical investigation requirements apply to dwellings with foundations that can be repaired compared to dwellings with foundations that will be replaced. To determine whether foundation repair or replacement is required, refer to Part A, Table 2.2 and Table 2.3 and Figures 14.1 and 14.2.

UPDATE:
December 2012

In general, foundations that require minor repair or relevelling only will not necessarily require geotechnical investigations. Those foundations with significant damage will require deep investigations so that a liquefaction analysis can be undertaken to determine likely future settlements. The foundation repair or replacement strategy for these dwellings will be determined by the outcomes of the liquefaction analysis.

13.5 Liquefaction assessment

In addition to standard geotechnical characterisation, the site data should be analysed using recognised methods as outlined below to determine liquefaction susceptibility, and in particular likely ground deformations under design serviceability limit state (SLS) and ultimate limit state (ULS) ground motions. (It is important to note that the methods outlined below must be employed when using these guidance documents).

13.5.1 Liquefaction analysis methodologies (minimum requirements)

A standard liquefaction analysis methodology outlined below, and repeated in Part D, shall be used in conjunction with specified input ground motions and, where appropriate, observations of land damage from recent seismic events. As discussed in section 12, it is recognised that the calculation of liquefaction-induced settlements is an inexact process. For the purposes of calculating consistent 'index numbers' to compare with nominal 'limits' set out in these guidance documents, a consistent methodology will need to be adopted by all users. These methodologies should only be applied by those with a strong background in geotechnical earthquake engineering. Other methods or adjustments that are not included in this document (for example 'thin layer' correction techniques) do not form part of this methodology.

DELETION: December 2012

For the purposes of this document, calculations of liquefaction potential (triggering) should be carried out using the methods of Idriss & Boulanger 2008, as outlined in the publication "Soil Liquefaction During Earthquakes" – EERI monograph MNO12. Only data obtained directly from CPT, SPT or seismic shear wave velocity measurements shall be used in carrying out liquefaction assessments. Where primary data has been obtained for the site using these methods, *and site access constrains the further use of these primary methods*, supplementary infill data can be considered from Swedish Weight Sounding or DPT using recognised correlations. For fines corrections where soil samples have not been retrieved and tested, the method of Robertson and Wride (1998) should be used. For the calculation of post-liquefaction induced settlements, the method of Zhang et al (2002) is to be used. **It should be noted that this does not imply that these methodologies are mandated for applications outside the scope of this document**.

UPDATE: December 2012

For comparison against 'index values' in these guidelines, calculations can generally be limited to the upper 10 m of the soil profile. (This does not however extend to section 15.3 - Site ground improvement). Potential issues do also need to be considered below 10 m depth (refer to section 13.6 for details).

UPDATE: December 2012

Ground input motions

Ground input motions for SLS and ULS liquefaction analysis are provided in Appendix C2. In summary, for deep soft soil (Class D) sites they are:

- SLS 0.13g
- ULS 0.35g

These figures are the result of extensive probabilistic modelling by GNS Science and observations of land and building damage caused during the Canterbury earthquake sequence, and are recommended by the Ministry as of April 2012 for liquefaction analyses on the flat land of Christchurch.

In response to new knowledge about the seismic risk in the Canterbury earthquake region, the former Department of Building and Housing (now the Ministry) made changes to the Verification Method B1/VM1, from 19 May 2011, to increase the seismic hazard factor Z (as described in AS/NZS 1170) for the region. The update to B1/VM1 states that the revised Z factor is intended only for use for the design and assessment of buildings and structures – **it is not applicable for use in geotechnical design**. The figures above are now provided to be used for liquefaction analysis.

UPDATE: December 2012

Liquefaction hazard, liquefaction-induced settlements and lateral spread

For design guidance refer to the following documents or methodologies (It should be noted that this does not imply that these methodologies are mandated for applications outside the scope of this document):

- For background information: refer to the latest edition of NZGS guidelines "*Geotechnical Earthquake Engineering Practice Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*" (current edition July 2010).
- For specific analysis methodology for liquefaction triggering: refer to Idriss & Boulanger 2008 "*Soil Liquefaction During Earthquakes*" – EERI monograph MNO12.
- 'For estimating apparent fines content (FC) for use in the CPT fines correction, set out in Idriss & Boulanger (2008) (equation 78), where soil samples are not being retrieved: refer to Robertson and Wride (1998) "*Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test*" Can. Geotech. J. 35(3), 442-459. ie, – (a) if Ic <1.26, apparent FC = 0%; (b) if 1.26 < Ic< 3.5, apparent FC (%) =1.75 $Ic^{3.25}$ - 3.7; and (c) if Ic > 3.5, apparent FC =100%.
- For estimation of post-liquefaction induced settlements in CPT analyses, refer to Zhang, Robertson & Brachman (2002) "*Estimating Liquefaction-Induced Ground Settlements from CPT for Level Ground*", Can. Geotech. J. (39), 1168-1180. In particular, Appendix A of that paper provides useful guidance on calculating volumetric strains. **Note:** the input parameters of FOS and $(q_{c}1_{n})_{cs}$ are to be derived from the method of Idriss & Boulanger (2008), as modified above.
- For surface crust assessment: refer to Ishihara (1985) "*Stability of Natural Deposits During Earthquakes*" Proc. of the 11th International Conference in Soil Mechanics and Foundation Engineering, pp 321-376 – Figure 88 p 362. (Reproduced as Figure 107 on p 157 of Idriss & Boulanger (2008) (optional).

- For refinement of SLS assessment: observations of damage or lack thereof in areas deemed to have been "sufficiently tested at SLS" by recent seismic events can be used to judge the applicability, or not, of settlements calculated at the design SLS level (optional). This can be achieved with reference to the PGA conditional median contours and associated conditional standard deviations contained in the paper (Bradley and Hughes 2012) and kmz file that can be found at the Canterbury Geotechnical Database canterburygeotechnicaldatabase.projectorbit.com.
  - As an initial screening tool, where a site has experienced at least 170% of design SLS (using the conditional median pga values from one of the three compiled events corrected to a M7.5 event; ie $PGA_{7.5} = PGA/MSF$), then the site can be regarded as having been 'sufficiently tested' for an SLS event.
  - If this screening test is not met, then the site can be evaluated by calculating the 10 percentile PGA from each of the three compiled events (i.e. the median value less 1.28 standard deviations, again magnitude scaled to M7.5). If one of these values equals or exceeds the design SLS event then the site can be regarded as having been 'sufficiently tested' for an SLS event. (At this level it is likely that most sites will have been tested to SLS or beyond by enough of a margin that in future SLS events the land damage will likely be no worse than already experienced at that site).
  - To calculate the 10 percentile PGA, use $PGA_{10} = PGA_{50}*\exp(-1.28*\sigma_{lnPGA})$, where PGA_{50} is the conditional median PGA and σ_{lnPGA} is the conditional standard deviation of PGA at a site. For consistency with the methodology used to analyse liquefaction triggering, the Magnitude Scaling Factor of Idriss & Boulanger (2008) should be used – i.e. $MSF = [6.9*\exp(-M/4)]-0.058 \leq 1.8$. Thus, $PGA_{10\_7.5} = PGA_{10}/MSF$.

Note: This does not imply that these methodologies are mandated for applications outside the scope of this document.

It is hoped that, with time, a modified methodology for liquefaction settlement/damage calculation that is depth-weighted will be derived from extensive site data and damage observations in the recent earthquake sequence. This may be incorporated in these requirements at an appropriate stage.

Modification by reference to soil deposit ageing is not considered appropriate in the Canterbury region.

Guidance on determining nominal lateral spread zonings is given in section 12.2 of this document.

13.6 Technical Category TC3 confirmation

If damage to the land or foundations is less than implied by the TC3 categorisation, then the deep geotechnical investigation and liquefaction analysis undertaken by a CPEng. geotechnical engineer or suitably qualified PEngGeol. engineering geologist may indicate that the site has TC2 rather than TC3 performance characteristics for that particular site.

UPDATE: December 2012

As part of this determination, liquefaction characteristics need to be assessed over the full depth of the soil profile investigated. However, when comparing calculated settlement values to the index values in Table 3.1 in Part A, calculations can be limited to the upper 10 m of the soil profile. This does not in any way imply that potential issues do not need to be considered below 10 m depth, this is simply a calculated 'index' number for comparison to the index values in Table 3.1 in Part A. Full depth settlement assessments should also be carried out, to allow consideration of (for example) differential settlements where deep liquefiable deposits vary significantly across a site. For this reason, CPTs should not be termiated short of refusal depth. Specific design based on the deep geotechnical investigation and TC2 solutions signed off by a suitably qualified CPEng. geotechnical engineer can then be undertaken.

As part of the building consent process, or in some cases independent from that process, the geotechnical information and the geotechnical report will be submitted to the Canterbury Geotechnical Database. The geotechnical report will contain the results of the liquefaction analyses and a reasoned justification from the CPEng. geotechnical engineer or suitably qualified and experienced PEngGeol. engineering geologist to support the opinion of TC2 – like site performance.

DELETION: December 2012

> *This will allow the use of TC2 foundation systems on those individual sites where such suitability has been determined by the CPEng. geotechnical engineer or suitably qualified and experienced PEngGeol. engineering geologist.*

The emphasis is on carrying out investigations to allow the design of a suitable foundation system for the site, whether that is a TC3 compliant system or a TC2 compliant system.

13.7 Longevity of factual and interpretative reports

It is considered in most cases that factual geotechnical investigation information (eg, CPT data, borehole data etc) would be appropriate for engineering use for at least five years and in many cases longer (at the discretion of the geotechnical engineer).

The predominant geotechnical issue that most properties in TC3 areas will be facing are liquefaction-related or bearing capacity issues. Some sites will also have compressible peat soils to consider. With regard to liquefaction, the underlying soils generally return to their pre-earthquake densities soon after seismic events.

The most likely change that might occur over time is a change in the groundwater profile. Engineers should consider this in their judgements and, if appropriate, undertake updated groundwater level investigations if historic information is being used. It is noted that **interpretive** methodologies are changing with time, and site usage can also vary. It is recommended that if an interpretive report is more than two years old, or the proposed building that the report originally applied to has changed significantly, (eg, layout, height, weight of building materials, foundation loads etc) and/or design loadings have changed (eg, design PGA levels), then the report is reviewed by the geotechnical engineer for current applicability.

Additionally, if the site has been altered by excavations or filling, the report will need to be reviewed.

13.8 Building consent information

For information on the Canterbury Geotechnical Database and the format for building consents, refer to sections 8.2.5 and 8.2.6.

UPDATE:
December 2012

14 Repairing house foundations in TC3

14.1 General

This section contains suggested approaches for the repair and reinstatement of house foundations where the level of damage does not require foundation replacement or complete rebuilding. It is emphasised that these approaches will not suit all houses that are considered repairable, and that each house will require careful consideration.

Situations involving the complete replacement of the foundations beneath an existing house, or the construction of a new dwelling, are addressed in section 15.

In general, the provisions in this section apply only to those sites in the 'Moderate' lateral stretch category (see section 12.2).

14.2 Assessment of foundation damage

The first step in assessing repair options for a damaged house in TC3 is to make a reasoned judgement on the severity of the damage that has occurred to the house structure.

Tables 2.2 and 2.3 in Part A give guidance on whether foundation damage requiring specific engineering input is present. As indicated in Part A, sections 2.2 and 2.3, **sound engineering judgement must be applied when using these tables**.

For example, criteria that need to be considered in a domestic house include:

- the intended use of the space
- construction materials of the floor surfacing
- practicality of the repair (ie, cost versus benefits)
- capacity to resist deformation
- effect of gradients on amenity of the space.

These considerations may trigger the need for relevelling or rebuilding in some situations where the guideline tables do not indicate such a situation, and conversely it is also expected that in other situations, despite being indicated by the guideline table, relevelling or rebuilding is not necessarily warranted.

DELETION:
December 2012

In applying the indicator criteria from Table 2.2 in Part A, due consideration must be given to the amount of damage that was likely to have been present before the earthquake events, and some guidance on this is given in Part A, section 2.2.

If more than just cosmetic repairs are necessary, then the indicator criteria in Table 2.3 in Part A should be used in conjunction with engineering judgement to determine the level of repairs necessary for the structure. This decision will be based on the criteria in Table 2.2 in Part A and sound engineering judgement. Again, reference must be made to Part A, section 2.3 when using these indicator criteria.

UPDATE:
December 2012

If no foundation damage is present that requires repair and specific engineering design, then superstructure repairs can proceed using guidance from Part A, section 7. In this case, minor cracks (<5 mm) in concrete floors and foundation beams can be repaired in accordance with Appendix A4 of those guidelines. No geotechnical investigation will be required in these cases.

Generally a decision will be made on whether a structure corresponds to one of the following cases:

- **Case 1:** Local repair and/or minor (local) relevel only required
- **Case 2:** Foundation relevel required (ie, widespread differential settlements)
- **Case 3:** Foundation rebuild required
- **Case 4:** Total demolition and rebuild required (ie, new structure)

For **Case 4**, if the house is to be demolished and a new structure built, the foundation solutions in section 15 of this document should be referred to. The extent of investigations will vary, and are described in sections 13 and 15.

For **Cases 1 to 4** above, where varying degrees of foundation repair and/or relevelling is required, reference should be made to the process flow charts in Figure 14.1 (foundation Types A and B) and Figure 14.2 (foundation Type C) to determine both investigation requirements and actions to be taken.

Figure 14.1: Overview of process for repairing foundations on TC3 sites for Foundation Types A and B

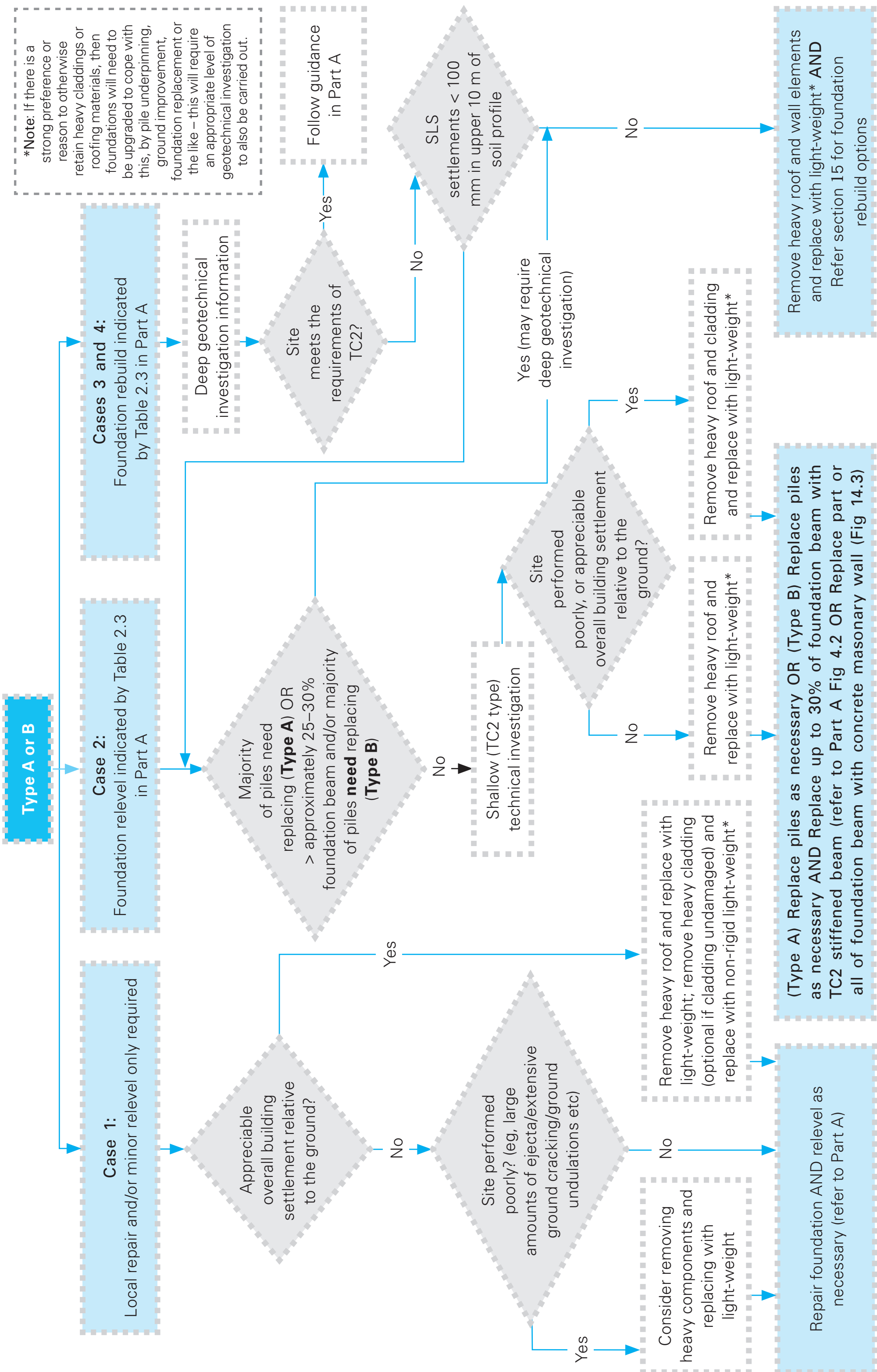
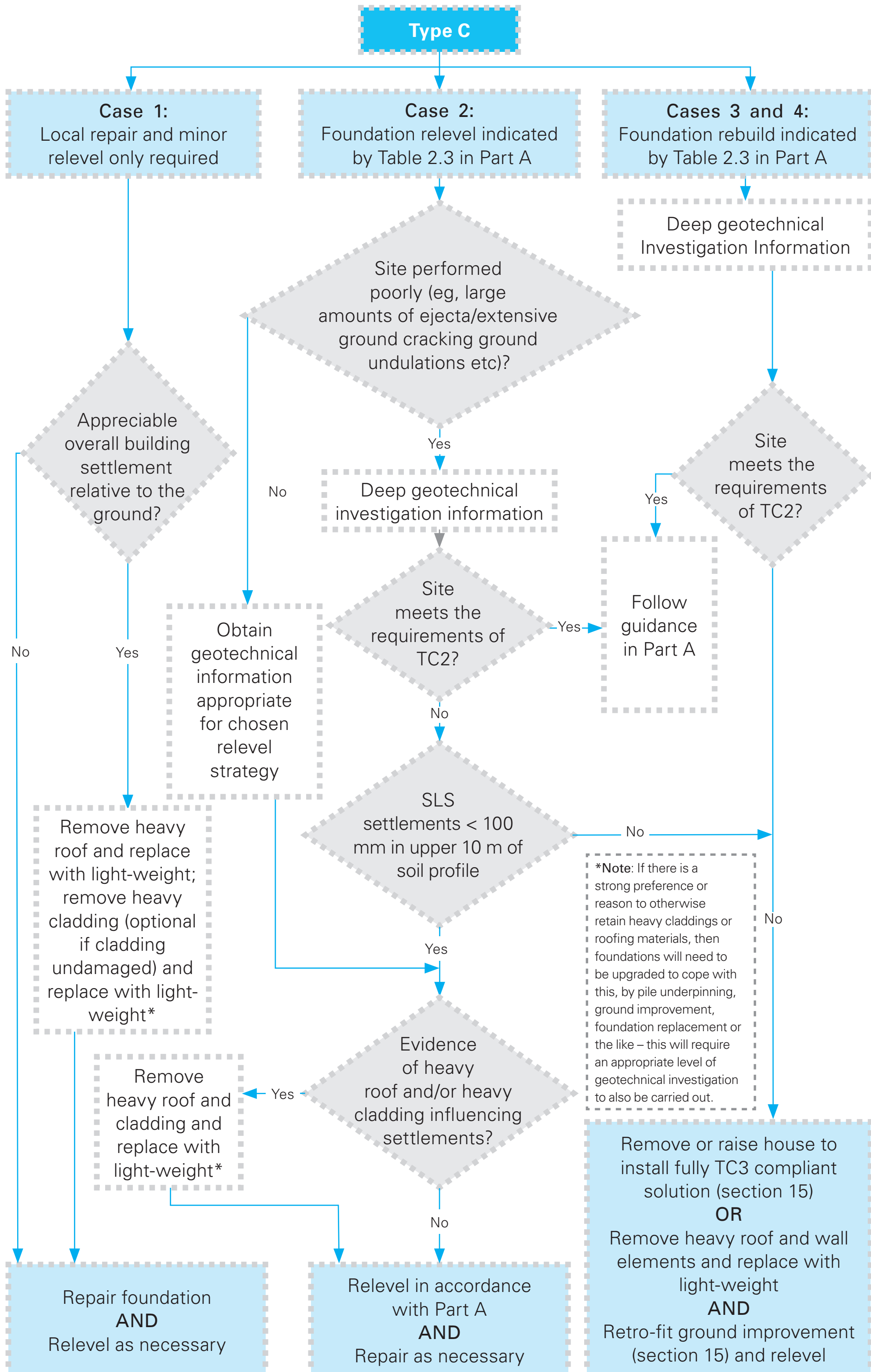


Figure 14.2: Overview of process for repairing foundations on TC3 sites for Foundation Type C



14.2.1 Case 1 – local repair (and local relevel)

UPDATE: December 2012

If a house has sustained only minor foundation damage (ie, substantial relevelling is not required), but local repairs are necessary, then a deep geotechnical investigation is not necessarily required. An assessment of whether the site and building have performed well or not should be made. In order to make a fully reasoned assessment on the extent or repairs or modifications necessary, engineering judgement will be required. Factors to consider include:

- Were there large amounts of liquefaction ejecta during the earthquake events?
- Was there extensive ground cracking of the site?
- Are there large ground undulations as a result of the earthquake events?
- Has the dwelling settled relative to the surrounding land?

If the site and building have performed well (and in the case of a Type A or B house with a heavy roof, there are no indications of significant damage to the ceiling or wall linings of the house), then localised foundation repairs and minor (local) relevelling can proceed. This might include replacement of short sections of a Type B foundation beam with an enhanced perimeter beam (refer Figure 4.2 and Figure 4.2a in Part A).

Load reduction strategies

The following load reduction strategies are recommended for heavily clad houses:

For a **Type A or B** house with a heavy roof, where there are signs of significant damage to the linings, indicating that the heavy roof has caused enhanced levels of damage, it is recommended that consideration be given to removal of the heavy roof and replacement with light-weight roofing materials (ie, corrugated steel, pressed steel tiles etc).

For **Type A or B** houses with heavy roofs and/or heavy claddings where:

a) the site has not performed well, or

b) there is evidence that the building has settled (albeit evenly) relative to the ground (this applies to all foundation types)

It is strongly recommended that the heavy roof is removed and replaced with light-weight materials. Scenario b) above indicates that the weight of the building is giving rise to undesired or adverse performance.

UPDATE: December 2012

For **Type C** homes with heavy roofs and/or heavy claddings where there has been appreciable building settlement relative to the ground, the roof should be removed and replaced with light-weight. Where heavy claddings are damaged, the cladding should be removed and replaced with light-weight.

Where a heavy cladding has been damaged to the extent that it requires removal then it is recommended that the cladding be replaced with light-weight (or medium-weight) materials. If claddings are to be altered or replaced, an appropriate level of professional advice should be sought to ensure the new claddings are suitable for the existing building.

UPDATE:
December 2012

Where foundation damage has occurred and there is a strong preference or reason to retain heavy claddings or roofing materials, then the foundations will need to be upgraded if poor house and/or ground performance is observed. Possible methods may include pile underpinning, ground improvement, foundation replacement or the like. This will require an appropriate level of geotechnical investigation to be carried out.

In all cases it is recommended that abandoned chimney bases or concrete foundations that are no longer required are removed. These structures have been observed to cause local differential settlement during liquefaction events. If a chimney is to remain then it is strongly recommended that any framing elements, subfloor elements and their supports are decoupled from the chimney base.

14.2.2 Case 2 – foundation relevel (and local repair)

If foundation relevelling is required and considered achievable, then the following factors need to be taken into account:

- the nature and extent of damage
- the lateral spreading (stretch) potential
- the liquefaction-induced vertical settlement potential
- whether the dwelling has settled relative to the surrounding land.

Repairs and relevelling can be considered if a site is assessed as having **moderate** (refer Table 12.1) lateral stretch potential (ie, <200 mm at ULS) (refer to the three lateral stretch categories outlined in 12.2).

If a site is assessed as having major or severe lateral stretch potential (ie, >200 mm at ULS), then neither repairs nor relevelling should be undertaken without careful engineering analysis and consideration.

In areas identified as having major global lateral movement potential, care will need to be taken with repairs to houses that are supported on deep piles.

Type A and B foundations can be relevelled if damage to the foundations is not too severe. The threshold of damage below which full foundation replacement is not required is:

- **for Type A** – majority of piles **not needing** replacement
- **for Type B** – less than approximately 25-30% of the foundation beam **needing** replacement and/or **the majority** of piles **not needing** replacement. (See the middle pathway of the flowchart in Figure 14.1).

If these damage levels are exceeded for Type A and B houses, then it becomes a foundation rebuild situation (ie, Case 3). If not, then relevelling and local repairs can proceed in accordance with Part A, section 4.3, following a shallow investigation to determine the shallow bearing capacity. With reference to Figures 4.1 and 4.3 in Part A , if the static geotechnical ULS bearing capacity is confirmed as being greater than 300 kPa, then the construction and engineering sign-off on a building consent application can be in accordance with NZS 3604 and this section. If the static geotechnical ULS bearing capacity is less than 300 kPa, then the engineering sign-off on a building consent application will be based on specific engineering design and this section may be used to support the building consent application. See Part A, section 3.4.1 for further guidance on specific engineering design calculations of bearing pressures.

UPDATE:
December 2012

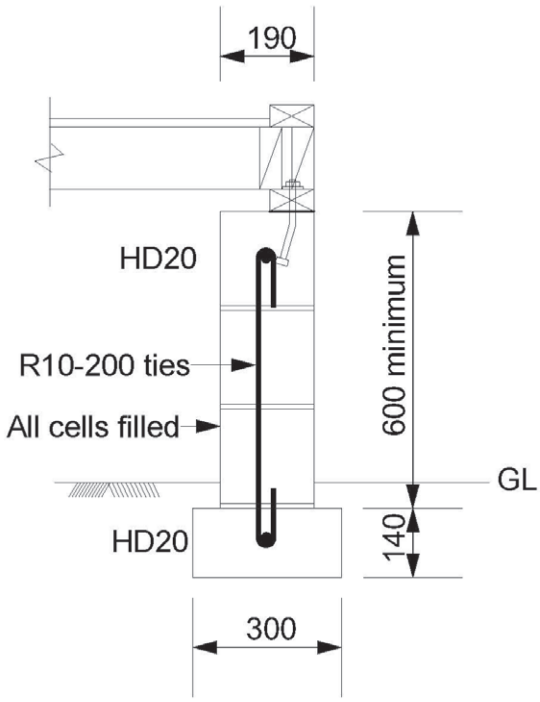
If relevelling is carried out using permanent deep piles then at least all perimeter foundation elements and load bearing walls should be supported on such piles (to prevent future gross differential movements). Internal non-loadbearing timber floors may require future relevelling or packing if supported on shallow piles in this case. The use of differential support systems is not recommended where significant peat deposits are present.

In addition, the performance of the site needs to be assessed. If the performance has been poor (eg, significant surface ejecta, extensive ground cracking, ground undulations etc), then it is strongly recommended that any heavy roofing materials and any heavy cladding materials are removed and replaced with light-weight materials before relevelling. If the site and building have performed relatively well, then the recommendation applies only to heavy roofing materials.

Where foundation damage has occurred and there is a strong preference or reason to retain heavy claddings or roofing materials, then the foundations will need to be upgraded. Possible methods include - pile underpinning, ground improvement, foundation replacement or the like. This will require an appropriate level of geotechnical investigation to be carried out.

The perimeter wall of a Type B dwelling with less than 25% to 30% damage can be fully replaced with an alternate concrete masonry wall as shown in Figure 14.3 where the resulting cladding is light or medium-weight and roof is light-weight.

Figure 14.3: Perimeter foundation wall detail for TC3



Mid-height subfloor vents should be provided in accordance with NZS 3604. The R10 ties can be in pairs either side of the vents

UPDATE: December 2012

In all cases it is recommended that abandoned chimney bases or concrete foundations that are no longer required are removed because these structures have been observed to cause local differential settlement during liquefaction events. If a chimney is to remain, then it is strongly recommended that any framing elements, subfloor elements and their supports are decoupled from the chimney base.

For **Type C** foundations (ie, concrete floor slabs with edge beams) the process is slightly more complicated. Type C foundations typically cannot sustain the same levels of deformation as Types A and B foundations without exhibiting damage. In this case, if the site appears to have performed poorly (eg significant surface ejecta, extensive ground cracking, ground undulations, settlement of the house relative to surrounding land, etc) the results of a deep geotechnical investigation are required in order to gauge the likely future performance of the site, particularly under SLS loadings. As discussed in section 12.3, the SLS settlements over the top 10 metres of the soil profile should be assessed. If this calculated value is more than 100 mm, then it becomes a foundation rebuild situation (ie, case 3). If not, then relevelling and local repairs can proceed in accordance with Part A, section 4.3. The building performance also needs to be assessed in terms of the influence of heavy roofing or cladding materials on settlements. If the performance has been poor (eg, hogging of the floor slab is evident), then it is strongly recommended that any heavy roofing materials and any heavy cladding materials are removed and replaced with light-weight materials before relevelling.

If the site has performed relatively well, but hogging is still evident, then this recommendation applies only to heavy roofing materials. If there is a strong preference or reason to retain heavy claddings or roofing materials contrary to these recommendations, then the foundations will need to be upgraded to cope with this. Possible methods include pile underpinning, ground improvement, foundation replacement or the like. This will require an appropriate level of geotechnical investigation to be carried out.

If both the site and building have performed well then relevelling can proceed without necessarily removing heavy materials. It is recommended that the removal of heavy materials is still considered in all cases.

14.2.3 Case 3 – foundation rebuild

If a foundation rebuild is required, in most cases the results of a deep geotechnical investigation will be required in accordance with section 13 requirements, and a rebuild will be determined in accordance with section 15.

For **Type A and B** houses in this situation, if the deep geotechnical investigation demonstrates that the assessed SLS settlements over the top 10 metres of the soil profile is less than 100 mm, then it is permissible to treat the situation as a relevel (ie, it can revert to case 2) if judged appropriate by the engineer. This could include use of the concrete masonry perimeter detail as shown in Figure 14.3.

For a foundation rebuild all heavy roof and cladding elements should be replaced with light-weight materials.

Any of these options may, but not necessarily, require the temporary removal or lifting of the house structure to allow construction to proceed.

For **Type C** houses, either the house will need to be removed temporarily or raised to allow the construction of one of the foundation options in section 15. It may be possible, in some cases, to install ground improvement with the house in place (eg, LMG piles or jet grouted columns) - in which case all heavy roofing elements and heavy wall claddings will need to be replaced with light-weight materials.

15 New foundations in TC3

15.1 Foundation types and selection considerations

This section covers foundations for new houses as well as situations where foundations are completely rebuilt for existing houses in TC3.

15.1.1 Foundation types

Three broad types of residential foundations have been established to meet the varying vertical settlement and lateral spreading requirements applying in TC3. These are:

- deep piles
- site ground improvements
- surface structures with shallow foundations

Each has different capabilities to accommodate various levels of vertical settlement and lateral spreading, and requires different constraints with respect to the configuration and weights of superstructure (eg, deep piles will not be suited to areas of TC3 where global lateral movement or lateral stretch is major or severe).

Table 15.1 summarises the principal objectives of each foundation type, and the main constraints.

Table 15.1: Overview of proposed TC3 foundation types

Type	Objectives	Dwelling Constraints	Land Constraints
Deep piles	Negligible settlement in both small and larger earthquakes	No height and/or material constraints likely	Not suitable where either *major or severe* global lateral movement likely or dense non-liquefiable bearing layer not present
Site ground improvement	Improving the ground to receive a TC2 foundation	Limits on some two storey/heavy wall types and plan configurations	Some ground improvements can be specified to accommodate *major* lateral stretch
Surface structures/ shallow foundations	Repairable damage in future moderate events	Only suitable for light and medium wall cladding combined with light roofs, regular in plan	In the absence of ground improvement, Type 1 & 2a options only suitable for minor to moderate vertical settlement and varying lateral stretch, Type 2b can accommodate up to 200 mm SLS settlement Type 3 (specific design) concepts can be designed for major lateral stretch and some for potentially significant vertical settlement

Note: Further elaboration of foundation types is summarised in Table 15.4 for site ground improvement and Table 15.5 for surface structures.

UPDATE:
December 2012

The subsequent sections 15.2 to 15.4 describe each of the foundation types and the options within them in more detail. Specific design parameters, specification and construction guidance are provided as appropriate.

Suitably experienced professional engineers may wish to use other foundation types or systems in TC3.

Guidance is given in each subsection on how the options relate to the categorisation for lateral movement and vertical settlement defined in sections 12.2 and 12.3 respectively.

Table 15.2 summarises the relationship between the commonly used floor and foundation types, and the lateral movement and vertical settlement categories, and compares them with the corresponding options and requirements for TC1 and TC2.

In reading this table it must be remembered that **the overall process of selecting and documenting foundation systems and details for houses in TC3 is a specific engineering design process that requires Chartered Professional Engineering input**. Depending on the assessed ground conditions and options selected by the Chartered Professional Engineer, some elements can be adopted and specified directly from these Guidelines without further engineering **design**. These include Types 1 and 2 Site Ground Improvement methods (section 15.3) and the Type 1 and 2 Surface Structures (section 15.4).

UPDATE:
December 2012

Table 15.2: Overview of floor and foundation types for new and rebuilt foundations
(a) Deep piles

	TC1	TC2	TC3	
Global Lateral Movement (ULS)	Nil	Minor <100 mm	Minor to Moderate < 300 mm	Major 300 to 500 mm
Lateral stretch (ULS)	Nil	Minor <100 mm	Minor to Moderate < 200 mm	Major 200 to 500 mm
Concrete Slab on Deep Piles	Deep pile options (section 15.2)	Deep pile options (section 15.2)	**Deep pile options from section 15.2**	Not suitable

Note: The use of deep piles in any location or Technical Category requires specific engineering design

UPDATE:
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Table 15.2: Overview of floor and foundation types for new and rebuilt foundations
(b) Site ground improvement and surface structures

	TC1	TC2	TC3			
Lateral stretch (ULS)	Nil	Minor <100 mm	Minor to Moderate < 200 mm		Major 200 to 500 mm	
Vertical Settlement (SLS)	0 – 15 mm	0 – 50 mm	<100 mm	>100 mm	<100 mm	>100 mm
Concrete Raft Slab	NZS 3604	Options 1 to 4 from Part A, section 5.3.1 of the guidance	Options 2 to 4 from Part A, section 5.3.1 of the guidance with **Site Ground Improvement (section 15.3)**		Options 2 and 4 from Part A, section 5.3.1 of the guidance with **Site Ground Improvement Types 2a and 3**	N/A
		Simple house plan shapes; refer to Table 7.2 for wall and roof cladding weight limits				
Timber Floor	NZS 3604	NZS 3604	**Type 1 and 2 Surface Structures (from section 15.4)**	NZS 3604 foundations with **Site Ground Improvement (or Type 3 Surface Structures)**	**Type 2A Surface Structures (from section 15.4)**	**Type 2B (up to 200 mm lateral stretch) and Type 3 Surface Structures (from section 15.4)**
			Simple house plan shapes with layout constraints	Simple house plan shapes	Simple house plan shapes with layout constraints	Specific Engineering Design of foundations
		Light or medium wall cladding combined with light roofs				

15.1.3 Other considerations in selecting foundations types and finished floor levels for TC3

In addition to the general constraints indicated in Table 15.1 and covered in more detail in subsequent sections, there are other considerations that need to be taken into account in selecting new and rebuilt foundations for TC3.

Building platform heights

The potential for future liquefaction-induced settlement in many properties in TC3 leads to the geotechnical requirement to limit the increase in mass added to the land. The recommended increase in height for building platforms is 250 mm. Greater increases may be allowable on a site-by-site basis subject to geotechnical engineering assessment.

DELETION: December 2012

Flood risk

General comments on the flood risk and relationship with floor levels in Christchurch City, Waimakariri District and Selwyn District are provided in Part B, section 8.4.

The current situation with regards to flood risk must be checked on a case-by-case basis with the relevant council.

Insurance contract provisions

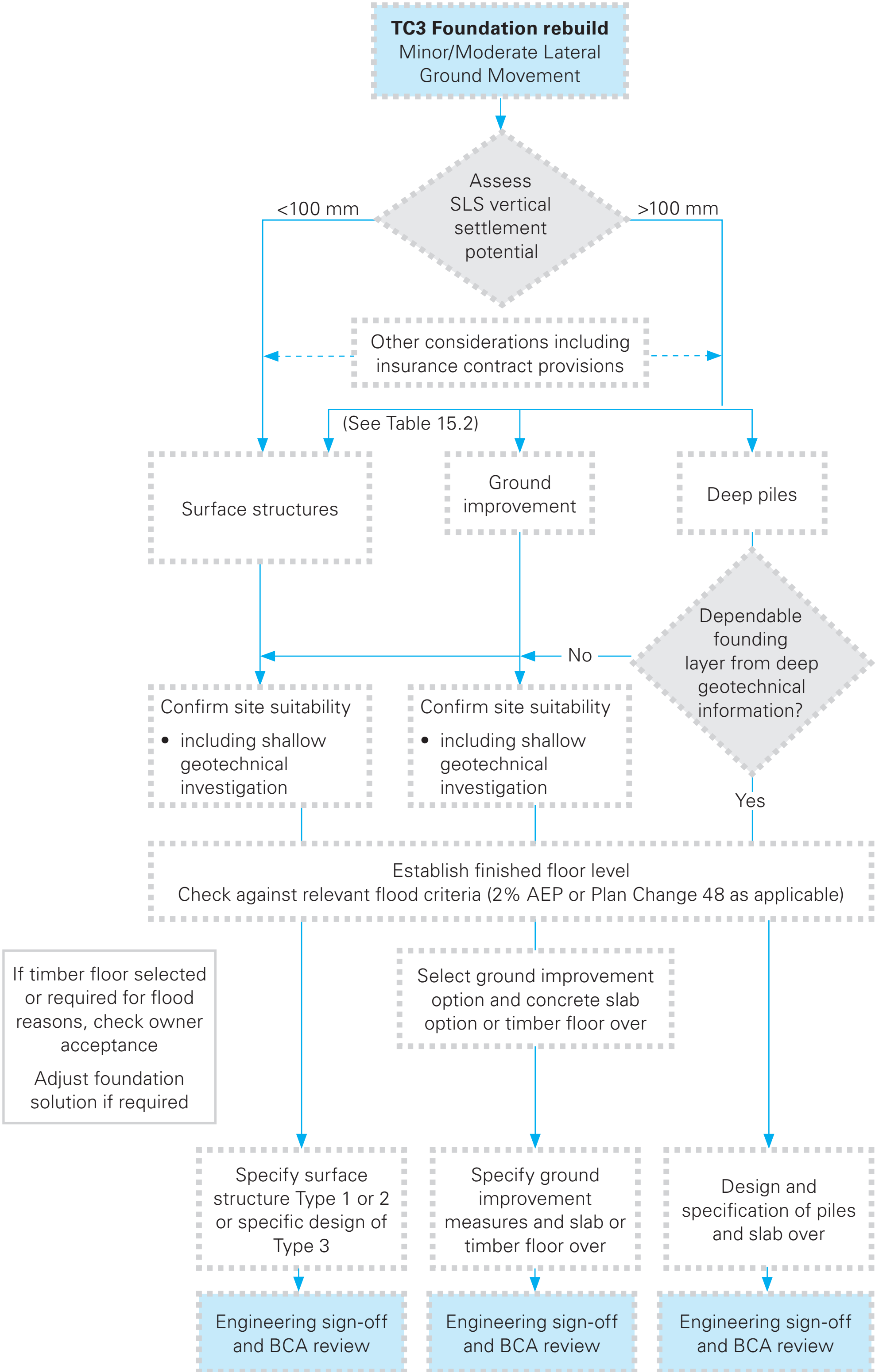
Selection of house types and configurations and hence foundation types may be guided by the provisions of insurance contracts as well as regulatory compliance requirements.

For example, some contracts may require that the existing house configuration and materials be incorporated in the rebuilt structure. This would impact on the foundation and superstructure selection process.

The general flowchart in Figure 15.1 provides an illustration of the overall process for the case of a new or rebuilt foundation in TC3 in areas of minor or moderate lateral movement. The flowchart indicates in broad terms the stages at which the above issues that extend beyond geotechnical and structural engineering should be taken into account.

UPDATE:
December 2012

Figure 15.1: General process flowchart for new and rebuilt foundations in TC3 (for sites with Minor to Moderate lateral ground movement)



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15.2 Deep piles

15.2.1 Objective and scope

The objective of using deep piles is to obtain dependable vertical load capacity at both SLS and ULS levels of earthquake. Where deep piles are appropriately selected, designed and constructed, they provide the greatest flexibility for the superstructure configuration and weight.

Deep piles are not considered suitable for major or severe global lateral spreading situations, and require careful detailing for ductility to accommodate lesser levels of lateral spreading.

This section provides guidance for deciding whether or not a particular site in TC3 is suitable for a deep pile foundation, and for selecting a suitable pile type for the site. The most suitable types for residential construction in liquefaction-prone areas are identified and described. Guidance is also given for suitable design methodologies and parameters. Additional design information is included in Appendix C3.

15.2.2 General requirements

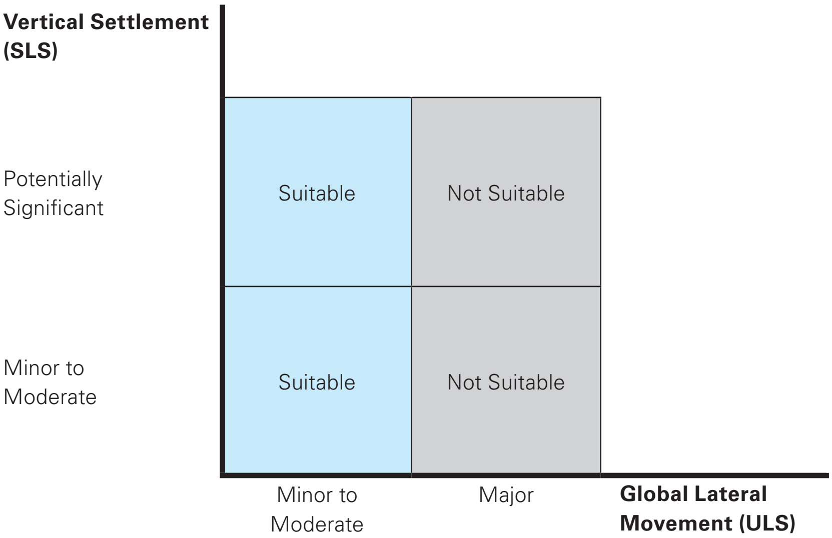
The following general requirements are necessary for a site to be considered suitable for deep pile foundations in TC3:

1. There must be a clearly identifiable bearing stratum that will provide adequate support for the pile type being considered. (For example, dense sand or gravel with corrected SPT N_{60} > 25 or CPT q_c >15 MPa).
2. There must be confidence that the bearing stratum is sufficiently thick to provide adequate support for the piles and to bridge over any underlying liquefiable layers.

   A minimum proven thickness of 3 m, or 4 m for two-storey heavy construction (roof or cladding), will provide this confidence.
3. The bearing stratum must be extensive enough across the site to provide uniform support to the entire footprint of the dwelling.
4. The piles must be capable of transferring the weight of the building to the bearing stratum, reliably, and meeting settlement requirements, even with liquefaction of overlying soils.
5. Pile foundations should be capable of withstanding lateral movement at the ground surface relative to the bearing stratum without suffering a brittle shear failure. A minimum lateral movement of 300 mm shall be considered even for sites with no surface evidence of lateral movement.
6. Pile foundations are not considered suitable (without special engineering) for sites where major or severe global lateral movement (>300 mm) has occurred (refer section 12.2).

A summary of the suitability of deep piles with respect to the different levels of global lateral movement and vertical settlement is shown in Figure 15.2.

Figure 15.2: Deep pile suitability summary (concrete or timber floor)



15.2.3 Pile types and options

The following pile types are considered the most suitable types for residential construction in TC3. Typical sizes and indicative capacities for these pile types are given in Table 15.3.

Screw piles

UPDATE: December 2012

Screw piles consist typically of one or more steel plate helixes welded to a steel tube. The pile is screwed into the ground and then the tube is filled with concrete. Torque measurements are used to identify penetration into the target-bearing stratum. These piles have the advantage that almost all of the load is transferred to end bearing on the steel helixes embedded into the target-bearing stratum, with minimal side resistance along the shaft. With liquefaction of overlying materials, there will be little down-drag. For this reason, multi-helix piles must not have helixes within the liquefiable deposits, or in any deposits above the bearing stratum that are underlaid with liquefiable deposits. The concrete-filled steel tube stems are very ductile providing good ability to cope with global lateral movement.

Design of these piles for axial capacity is usually by proprietary methods, and these should be supported by documentary evidence such as field load tests of relevant-sized piles in local conditions.

Alternatively, calculations may be made using standard bearing capacity equations, but taking account of the following issues:

- depth of embedment into the bearing stratum, and
- load-displacement response.

Driven timber piles

Suitably treated timber poles can be driven to bear into the target-bearing stratum. Timber piles are easily handled on site and are resilient to driving stresses and to lateral ground movements. Where driven at reasonably close spacing, they have the added benefit of densifying loose sandy soils.

It is important that the piles are driven to a target depth within the bearing stratum, as determined by the site investigation. It is not acceptable to simply drive to refusal or to a set.

Design of these piles will need to be by calculation using standard procedures to evaluate bearing capacity within the bearing stratum, neglecting all contributions from side resistance above the bearing stratum.

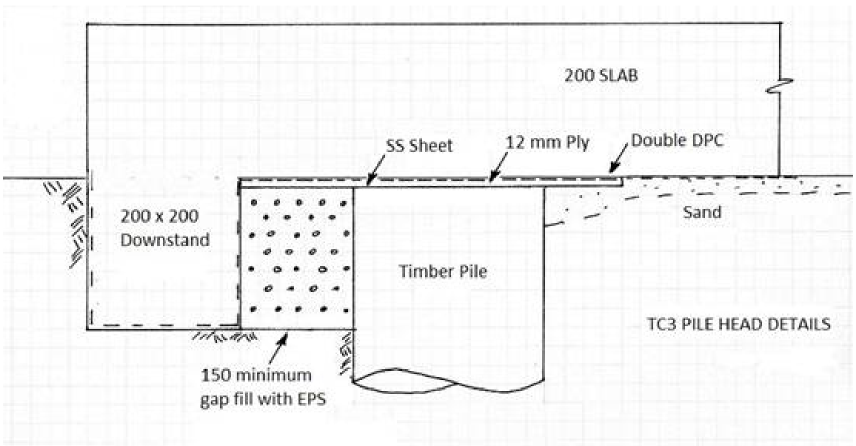
At some sites it may not be possible to drive timber piles to the target depth because of excessive resistance through intermediate strata causing premature refusal to driving. In such cases, jetting or pre-drilling may be necessary or other foundation types will need to be used.

If driving vibrations are excessive, options to reduce vibrations include pre-drilled holes and/or vibrating piles to an appropriate depth and completing driving with a hammer.

In all cases, jetting or pre-drilling should not be continued into the bearing stratum and the piles should be driven to the target depth within the bearing stratum using a suitable hammer.

UPDATE:
December 2012

Figure 15.3: Pile head detail – timber



Note: 12 mm ply plate to be CCA treated.

Driven steel H-piles

Steel H-Piles are readily available in a range of stock lengths (9 m – 18 m). They have the advantage of being relatively easy to drive through intermediate stiff soil layers compared to other pile types. Also, they have less side resistance to other pile types meaning that they will pick up less down-drag from the overlying soil crust.

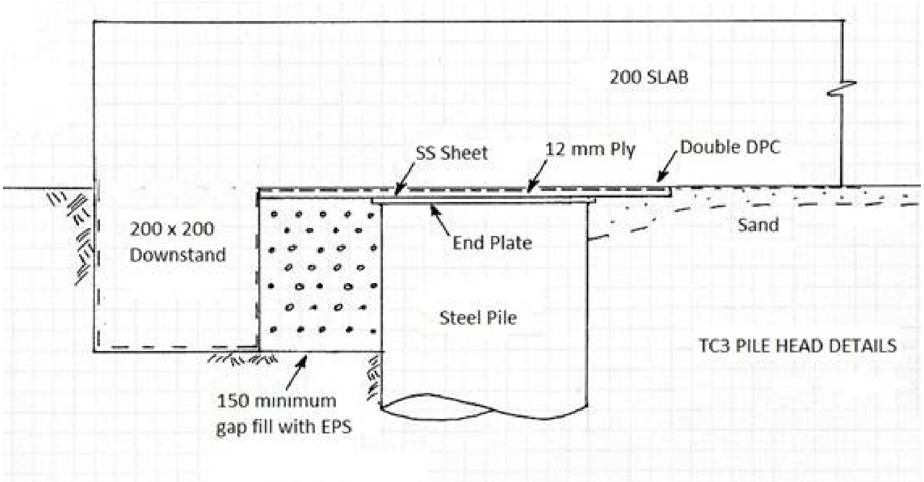
These piles are also highly ductile and able to withstand more lateral spreading than other pile types. However, they have less end-bearing resistance than other pile types and will be more suited to sites with a very dense or thick gravel bearing layer.

It is important that the piles are driven to a target depth within the bearing stratum, as determined by the site investigation. It is not acceptable to simply drive to refusal or to a set.

Design of these piles will need to be by calculation using standard procedures to evaluate bearing capacity within the bearing stratum, neglecting all contributions from side resistance above the bearing stratum.

UPDATE: December 2012

Figure 15.4: Pile head detail – steel



Note: 12 mm ply plate to be CCA treated.

UPDATE: December 2012

Driven steel tubes

Steel tubes are available in a wide range of sizes and stock lengths. Suitable sections should have sufficient wall thickness to be able to withstand driving stresses and structural loads. Tubes may be driven either closed-ended with welded base plates or open-ended. Open-ended piles may be easier to drive through intermediate hard layers but are more susceptible to damage if obstacles are encountered.

Steel tube piles should be concrete filled after installation making them highly ductile and able to withstand more lateral spreading than other pile types.

It is important that the piles are driven to a target depth within the bearing stratum, as determined by the site investigation. It is not acceptable to simply drive to refusal or to a set.

Design of these piles will need to be by calculation using standard procedures to evaluate bearing capacity within the bearing stratum, neglecting all contributions from side resistance above the bearing stratum, and including the effect of down-drag from non-liquefied soils.

Driven precast concrete piles

Concrete piles can be manufactured to desired length and driven to bear in the target-bearing stratum. Where driven at reasonably close spacing, they have the added benefit of densifying loose sandy soils. The main limitation of precast concrete piles is limited ductility to withstand lateral ground movements, and piles will need to be specially detailed for ductility.

It is important that the piles are driven to a target depth within the bearing stratum, as determined by the site investigation. It is not acceptable to simply drive to refusal or to a set.

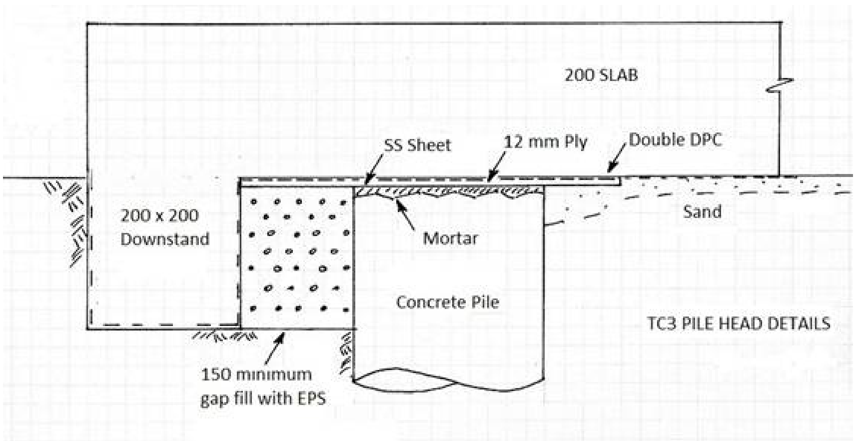
Design of these piles will need to be by calculation using standard procedures to evaluate bearing capacity within the bearing stratum, neglecting all contributions from side resistance above the bearing stratum.

At some sites it may not be possible to drive precast concrete piles to the target depth because of excessive resistance through intermediate strata causing premature refusal to driving. In such cases, jetting or predrilling may be necessary or other foundation types will need to be used.

In all cases, jetting or pre-drilling should not be continued into the bearing stratum and the piles should be driven to the target depth within the bearing stratum.

UPDATE: December 2012

Figure 15.5: Pile head detail – concrete



Note: 12 mm ply plate to be CCA treated.

The following pile types are considered **less suitable** for residential construction in TC3. These are not precluded from use, but will require additional engineering input to ensure satisfactory performance.

Continuous flight augur piles (CFA)

CFA piles are formed by first screwing a hollow-stemmed augur into the ground to the target depth, then slowly withdrawing the augur while high-slump concrete is pumped down the hollow stem to form the pile. Special monitoring equipment is required to ensure that the concrete flow rate matches the withdrawal rate of the augur to prevent formation of voids. A steel reinforcing cage is inserted immediately after the final withdrawal of the augur.

These piles are considered less suitable because they typically have a high side-resistance capacity, and initial load transfer after construction will be mostly by side-resistance, including through the liquefiable strata. During liquefaction, most side resistance will be lost and will have to be transferred to end bearing – a relatively soft mechanism that can induce settlements as the load is transferred to the base of the pile.

The settlement in this case needs to be checked carefully, including the added load from down-drag. Settlement may be controlled by embedding the piles deeper into the bearing stratum.

Design of these piles will need to be by calculation using standard procedures to evaluate bearing capacity within the bearing stratum, neglecting all contributions from side resistance above the bearing stratum, and including the effect of down-drag from non-liquefied soils.

These piles will also need to be specially detailed for ductility to prevent brittle shear failure from lateral soil movements.

Bored piles

Bored holes for cast-in-place concrete piles will generally be unstable in TC3 areas and will require temporary support using steel casings or drilling slurries and will need to be poured using a tremie. These techniques are unlikely to be economical for residential construction.

These piles are considered less-suitable because they typically have a high side-resistance capacity, and initial load transfer after construction will be mostly by side-resistance, including through the liquefiable strata. During liquefaction, most side resistance will be lost and will have to be transferred to end bearing – a relatively soft mechanism which can induce settlements as the load is transferred to the base of the pile.

The settlement in this case needs to be checked carefully, including the added load from down-drag. Settlement may be controlled by embedding the piles deeper into the bearing stratum.

Design of these piles will need to be by calculation using standard procedures to evaluate bearing capacity within the bearing stratum, neglecting all contributions from side resistance above the bearing stratum, and including the effect of down-drag from non-liquefied soils.

Micropiles

Micropiles are small diameter piles and include both driven and bored varieties. The main drawback of micropiles in this situation is that they typically achieve most of their load capacity from side resistance with relatively small end-bearing capacity. Therefore, they will need to penetrate well into the target-bearing stratum to achieve sufficient capacity after neglecting the side resistance through the liquefiable strata and taking into accouint the effects of down-drag.

15.2.4 Particular geotechnical investigation requirements

Where deep pile foundations are being considered at a site, it will be necessary to carry out a deep site investigation. The objective is to identify a suitable bearing stratum with the minimum characteristics identified above. In addition, it is necessary to identify the thickness of the surface crust and other non-liquefying layers to be able to assess the most suitable pile type and any issues with driving and down-drag.

The following investigation strategy is recommended:

1. Carry out CPT at site (refer section 13).
2. If the profile appears to meet the general requirements for deep pile foundations, continue with a second CPT to provide confidence that bearing stratum extends across the footprint.
3. If the CPT is unable to prove the minimum thickness required for a bearing layer, then a machine borehole with SPTs at 1 m or 1.2 m centres is required to prove the minimum thickness of the bearing layer.

15.2.5 Design approaches and parameters

Deep pile foundations will need specific engineering design in all cases, given the complexities of identifying a suitable bearing layer, calculation of bearing capacity, and lateral loading. The design of the floor slab supported by the pile system also requires careful consideration.

Piles

The objective of using pile foundations is to limit settlement of the building independent from settlement and deformation of the ground above the bearing stratum. Calculation of pile load-deformation response in each individual situation is complex and generally excessive for a residential building. Building weights are likely to be low and pile sizes small, so a simplified procedure is recommended based on standard limiting equilibrium strength calculations and a conservative strength reduction factor. This procedure is:

1. Sum the ULS-factored gravity building loads.
2. Calculate ideal vertical capacity of pile embedded in target bearing stratum (from only that part of pile embedded within the target bearing stratum) using standard limiting equilibrium procedures.
3. Apply $\Phi_g = 0.4$ (intended to both provide reliable capacity and also limit settlements)

The design equation becomes:

$$\Phi_g R_u \text{ \{in bearing stratum\} } \geq 1.2G + 1.5Q$$

For this simplified design procedure (for driven piles for residential buildings only) the down-drag forces acting on the pile above the bearing stratum may be ignored. If bored piles or CFA piles are being considered, then the down-drag forces should be added to the factored gravity loads and the effects of loss of side resistance with liquefaction should be carefully considered.

It is assumed that if the above design procedure is followed for the ULS case, then it will not be necessary to separately consider the SLS case. If liquefaction is triggered for the SLS case, then the above design procedure should limit settlement to 25 mm or less.

Kinematic effects (lateral soil-pile interactions) do not need to be explicitly considered in each case for the pile types indicated as being most suitable. Analysis of these pile types has shown that they should be able to withstand lateral surface movement of up to 300 mm for typical situations (see Table 15.3 for details). If the less suitable pile types are to be used, designers will need to demonstrate their ability to withstand a lateral surface movement of 300 mm while maintaining an ability to continue to support the building and be reusable for a repaired structure. The results shown in Table 15.3 are based on an assumed thickness for the liquefied layer of 6 m. If the liquefied layer at a site is significantly thinner than 6 m then the ability of piles to accommodate global lateral movement will be reduced and designers should make their own assessment of kinematic effects.

Pile buckling within liquefied soil layers does not need to be considered explicitly for the most suitable pile types for the typical conditions considered in Table 15.3. Pile buckling may be an issue for heavily loaded, slender piles within very thick liquefied layers. Additional guidance is given in Bhattacharya et.al. (2004).

Simplified design procedures for driven piles based on SPT and CPT results are given in Appendix C3.

Table 15.3 summarises typical available pile sizes and corresponding indicative capacities. Figures 15.6 and 15.7 show layouts and sample detailing for a flat concrete slab on deep piles and Figures 15.8 and 15.9 show layouts and sample detailing for a waffle slab on deep piles.

Table 15.3: Typical pile sizes and indicative capacities

UPDATE: December 2012

Pile Type	Screw Pile	Driven Timber	Driven H-Pile	Driven Steel Tube (Concrete filled, closed end)	Driven Concrete	Driven Concrete
Typical size	300 Helix x 150 NB	250 SED	200 UC	200 CHS	150 x 150	200 x 200
Load capacity[1]		95 KN	90 KN	75 KN	70 KN	95 KN
Lateral displacement[2]	300 mm	300 mm	300 mm	300 mm	300 mm[3]	300 mm[3]
Advantages	Minimal down drag, very high ductility	Cheap, light, readily available	Good ductility, penetrate hard layers, reduced down-drag	Very high ductility	Cast to required length	Cast to required length
Disadvantages	Limited contractor capacity	Difficulty penetrating dense layers	Relatively expensive	Relatively expensive	Limited ductility; need length certain prior to fabrication; difficult to splice	Limited ductility; need length certain prior to fabrication; difficult to splice

Note:

1. Dependable capacity embedded 1 m into N_{60} = 25 sand or gravel. Higher capacities will be obtainable in denser soil or deeper embedment.
2. Ability to withstand global lateral movement assuming 2 m thick stiff crust and 6 m thick liquefied layer.
3. Special detailing for ductility required. Assessment based on proprietary design of Hi-Stress Concrete Ltd. Other pile designs will require specific analysis.

Slab on piles

Slabs should be designed to span over the piles, ie, not requiring support from the soils beneath the slabs.

Two sample slab options have been designed to be supported on the deep piles. The first is a solid 200 mm thick slab and the second is a waffle slab (refer Figures 15.6 and 15.8). These options are adapted from TC2 foundation options 2 and 4 respectively in Part A, section 5.3.1.

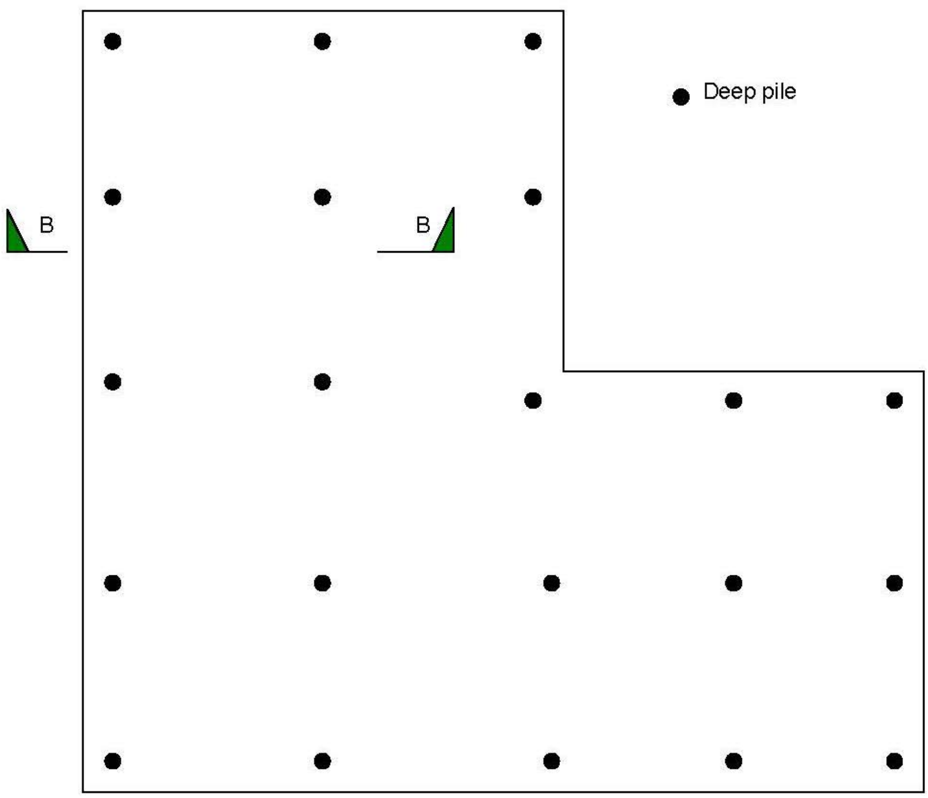
The beams of the waffle slab are 500 mm wide to provide space for pile head details.

For situations where significant lateral stretch of up to 200 mm has occurred across the footprint or is considered likely to occur, the special sliding pile head details shown in Figures 15.3, 15.4 and 15.5 should be used.

UPDATE: December 2012

Figure 15.6: Illustrative pile layout for a flat concrete slab

TC3 Flat Slab Foundation on Piles



Example Foundation Plan

Slab steel (based on maximum pile spacing of 3.75m each way):

Single storey light roof/light walls - D12-250 top and bottom each way
Single storey heavy roof/heavy walls - D16-300 top and bottom each way

Two storey light roof/light walls - D16-300 top and bottom each way
Two storey heavy roof/heavy walls - D16-250 top and bottom each way

UPDATE: December 2012

Figure 15.7: Section A-A – Illustrative pile layout for a flat concrete slab

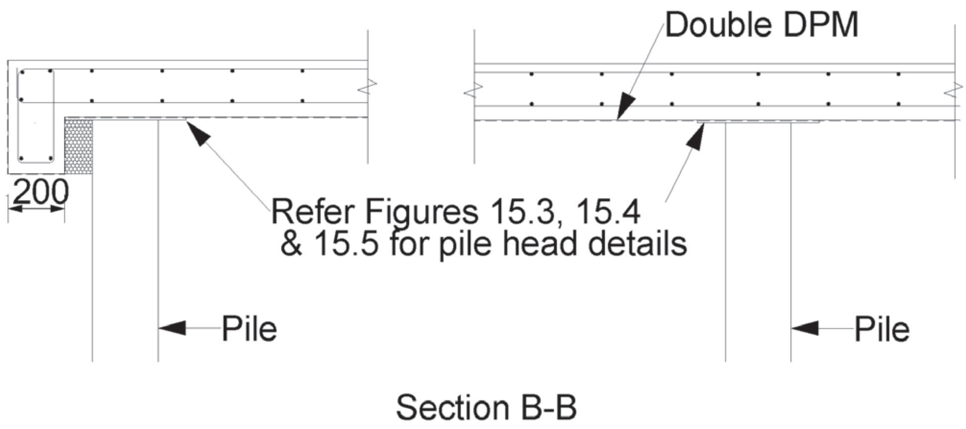
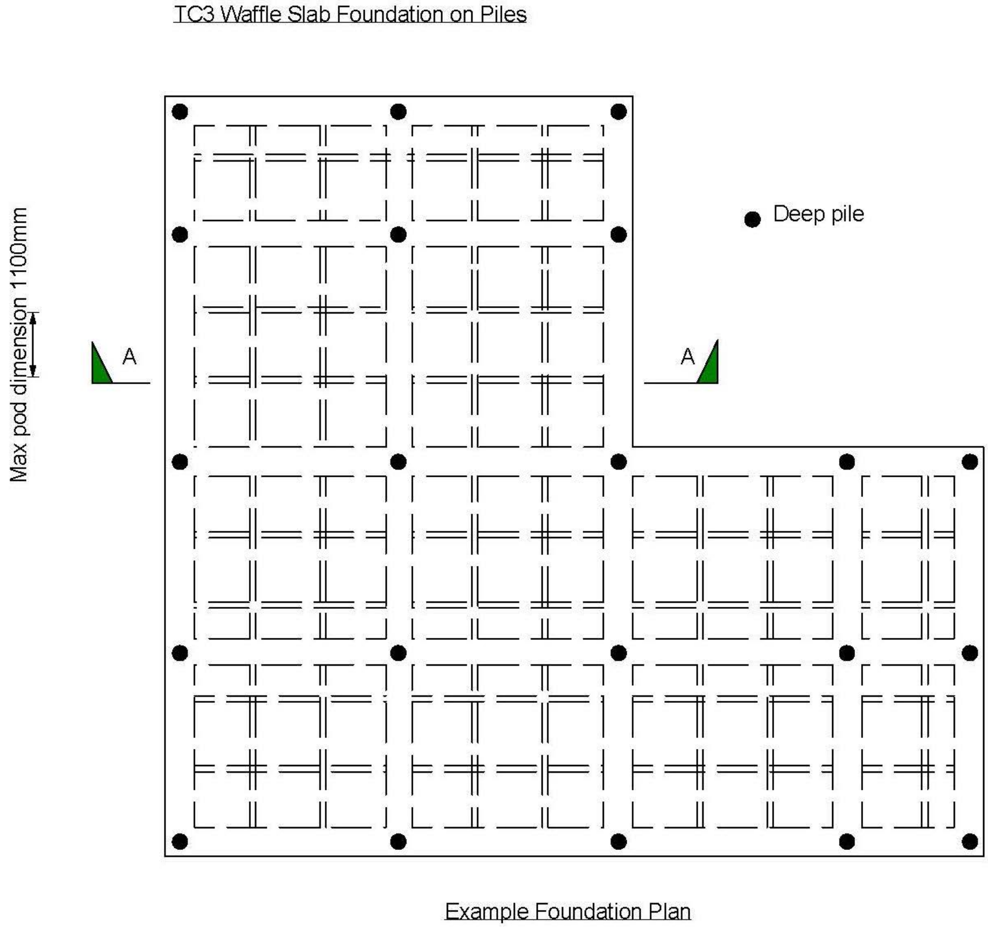
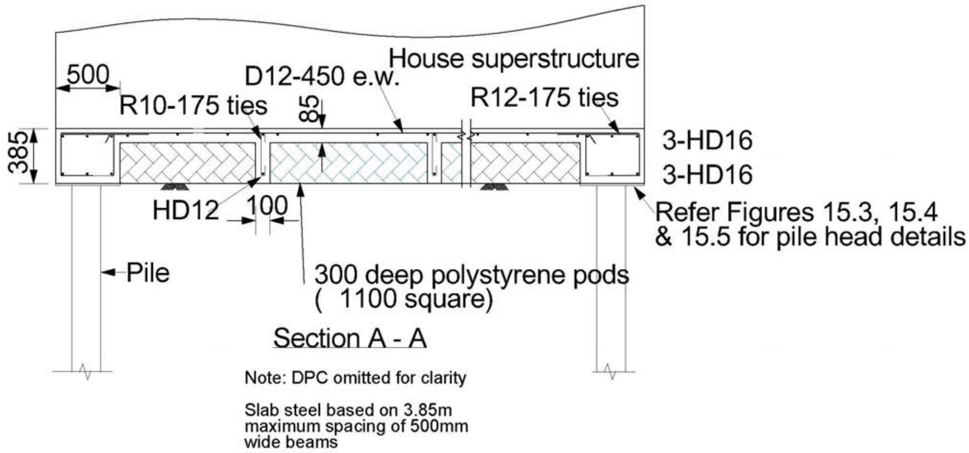


Figure 15.8: Illustrative layout and sample details for a waffle slab on deep pile



UPDATE:
December 2012

Figure 15.9: Sample detail for a waffle slab on deep piles



15.2.6 Specification and construction issues

Deep piles require a good level of resilience (timber) or ductility (steel and concrete) to be able to cope with the required minimum level of global lateral movement. Timber piles and screw piles (concrete-filled steel tubes) and steel H-piles and tubes are considered to have sufficient resilience or ductility for sites with moderate potential for global lateral movement. Concrete piles, either driven precast or cast insitu (eg, CFA), will require special detailing for ductility.

At sites with major potential for lateral movement, all pile types will require specific design and detailing to ensure that they can withstand the expected lateral movements without suffering a brittle shear failure.

Piles must be specified to be installed to the target depth established from the site investigation by the engineer. For driven piles, the required driving energy to achieve the necessary penetration should be estimated and suitable pile-driving equipment should be specified accordingly.

Difficulties may arise during installation from intermediate hard layers that are difficult to penetrate. These hard layers should be identified during the investigation and taken into account when assessing the suitability of any particular pile type and driving equipment. Predrilling through such layers should generally be acceptable and may be beneficial in reducing the amount of down-drag on the piles. However, pre-drilling should not extend into the bearing layer, and the pile should be driven to target depth in the bearing layer using a suitable hammer.

Leaving piles bearing on to intermediate hard layers because of an inability to penetrate to the target layer is not acceptable.

It is likely that after an earthquake event, the ground surface will settle relative to the piled building. Service connections will require special detailing to ensure that they are able to cope with the expected relative movement.

15.3 Site ground improvement

The guidance in section 15.3 provides information about the use of ground improvement to mitigate liquefaction-induced foundation damage. The design philosophy and objectives, types of improvement methods currently recommended, and general design and construction considerations are presented in this section. In addition, Appendix C4 provides specific construction and quality control requirements and example method statements for each ground improvement type. **Appendix C4 must be read in conjunction with this section**.

UPDATE:
Replaced all of 15.3
June 2015

Section 15.3 contains a number of recommended ground improvement methods for mitigating the effects of liquefaction induced by seismic shaking. The types of ground improvement and different options are summarised in section 15.3.5. In some cases these methods offer benefits in managing other geotechnical issues which also need to be considered for each site.

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

When following this guidance, it is expected that the CPEng geotechnical engineer responsible for the specification of the ground improvement works takes into consideration all aspects of the site when selecting a suitable method, and provides all normal documentation such as design drawings, specifications, Producer Statements or statements of professional opinion, and Design Features Reports. It is also important that construction and post installation quality control records are kept, and as as-built locations are recorded.

15.3.1 Objectives and scope

The intention is that an **integrated foundation solution** is constructed, consisting of:

- ground improvement carried out in accordance with the recommendations set out in this section of guidelines, combined with:
- stiff foundation elements or relevellable timber subfloors from section 5 or 15.4.

An integrated foundation solution is expected to provide a building platform that mitigates liquefaction-induced **differential** settlement to the degree that acceptable structure/foundation performance is maintained. In some cases, for example flood zones, **total** settlement might also be a factor in deciding both the final depth of treatment and the form of the foundation system. (It is recognised in the latter case that in many cases this may not be economic, and also goes beyond the basic performance requirements of the Building Code – however these issues should still be considered. In some instances homeowners may wish to contribute more to the costs of the project in order to gain additional protection).

As described in section 8.2, the desired outcome at SLS levels of shaking is a low level of damage that is readily repairable. At ULS, a low probability of rupture of the structure is a requirement of the Building Code. During the Canterbury earthquake sequence, house superstructures designed to NZS 3604 generally met these performance requirements. However it was often liquefaction induced land deformations that resulted in high levels of foundation damage, particularly for houses with concrete floors. An integrated foundation solution selected from the guidance will result in a foundation system that is unlikely to be the weak link in the building system. Therefore, given the cost of constructing foundations and the difficulties that can be involved in repairing them, a higher degree of resilience for the housing stock at ULS levels of shaking will be provided.

These objectives can be achieved by the careful selection of one of the options outlined in this section.

The ground improvement methods in this section are applicable to conventional one- to two-storey residential construction (see 1.4.3) on confirmed TC3 sites (ie sites verified by site investigation as requiring TC3 foundation solutions). For buildings that fall outside this scope, the provisions of this document do not necessarily apply, and specific engineering design will be needed.

15.3.2 Field testing programmes

In 2011 the Department of Building and Housing (now the Ministry of Business, Innovation and Employment) commissioned a field trial of a number of ground improvement options to see if infrastructure-scale methodologies could be adapted to residential-scale developments. During the field trial the selected options were subjected to blasting induced liquefaction, and the performance of each of the mitigation methods was assessed by reference to measured settlements, ground vibration and pore pressure response.

In addition to the 2011 trials, in 2013 the Earthquake Commission (EQC), in conjunction with MBIE and other parties, investigated several shallow ground improvement methods designed to strengthen or build a non-liquefiable 'crust'. The 2013 trials are referred to as the EQC Ground Improvement Trials and the resulting Science Report is currently in the process of being published.

The testing programmes were internationally overviewed. The programmes have produced data that has been analysed and reviewed relative to current international practice, to provide a measure of the expected performance of these mitigation options in typical Christchurch liquefaction-prone soils.

15.3.3 Design approach

15.3.3.1 Typical soil types

In Christchurch, liquefiable soil generally falls into three broad categories – namely:

- relatively clean sand sites (this generally means having a Soil Behaviour Type Index (I_c) of less than 1.8 or fines content (FC) < 15%)
- silty sand (1.8 < I_c < 2.3 approx. or FC > 15%), or sandy silt sites (2.3 < I_c < 2.6 approx. or FC > 35%)
- sites where clean sands are interbedded with silty materials.

Predominantly silty soils may also be susceptible to liquefaction if the silt is non-plastic or of low plasticity. Soils with a high fines content and exhibiting some plasticity (I_c > 2.6), are generally regarded as non-liquefiable (but may still be subject to cyclic softening).

Critical liquefiable layers which might affect foundation performance can be found at varying depths in the soil profile. Shallower or thicker liquefiable deposits will have a greater effect on foundation performance than deeper or thinner deposits. Other aspects, both technical and practical, will also vary from site to site. For these reasons, the ground improvement methods in this section are not intended to be universal solutions – each site must be considered on its own merits when selecting the most suitable method for that site.

Note:

1. Where 'clean sands' are referred to in this section, this generally means soils with an I_c < 1.8 (approx.) or a fines content < 15%. Where silty sands are referred to, this generally means soils with 1.8 < I_c < 2.3 (approx.) or a fines content between 15 and 35%.
2. The FC and I_c delineations (for varying degrees of 'siltiness') discussed in this document should be read only in the context of soil behaviour with respect to liquefaction triggering.

15.3.3.2 Liquefaction mitigation strategy

It is important to note that the overall liquefaction mitigation strategy comprises an integrated foundation solution, not ground improvement alone. The role of the ground improvement component of the works is to reduce, not eliminate, future ground deformations to the extent that the surface foundation component (either a stiff foundation element or in some cases a relevellable timber subfloor) can meet the performance objectives outlined in section 15.3.1.

The liquefaction mitigation strategy associated with the improvement methods comprises either:

- shallow ground improvement - Accept that liquefaction will occur and reduce the potential for damaging differential settlement and flexure of the house superstructure by constructing a non-liquefiable surface ‘crust’ in combination with a robust, stiffened foundation system; or
- deep ground improvement - Eliminate or greatly reduce the liquefaction potential (at design levels of shaking) throughout the depth of the soil profile expected to contribute to ground surface settlement (eg 8-10m for lightweight residential structures). This is the traditional approach to ground improvement. Again, this would be in combination with a suitable surface stiff foundation system.

Each of the methods contained in this document will behave differently in response to an earthquake and their inclusion does not imply equivalency of performance between them. The methods in this document have been selected to provide suitable performance, but will not in all cases completely remove the risk of liquefaction or liquefaction-induced damage. However, it is expected that the overall integrated foundation system will control differential movements such that each method will meet the performance objectives defined in section 15.3.1, and therefore will comply with the requirements of the Building Code.

Shallow ground improvement in combination with a robust, stiffened foundation system to control liquefaction-induced differential settlement is expected to be suitable for many TC3 sites in Christchurch. Shallow ground improvement will mitigate the effects of liquefaction of soils within the depth of improvement and also mitigate the surficial effects of deeper liquefiable layers. However, where the liquefiable soils extend well below the depth of improvement, there will not be a reduction in liquefaction potential or related settlement in those materials, and therefore total settlements may still be large.

On some sites, it may be desirable to control total as well as differential settlement; for example where such settlement would result in the building floor level falling significantly below design flood levels and raising the house back above the flood level would be difficult or costly. There are also some limitations on the applicability of the ground improvement methods outlined in this document, based on calculated index settlements for a site (see section 15.3.8).

The principal purpose of the index settlement calculations is to provide a convenient method for broadly classifying sites. When selecting ground improvement methods it is also important to consider the location in the soil profile of the critical liquefiable layers. As an example, if ground improvement is being adopted, and the bulk of the liquefaction-induced settlement occurs in a sandy layer between 4.5m depth and 7m depth that is overlaid with siltier materials, it may not be advisable to select a 4m deep composite crust solution. Selecting either a deep ground improvement solution, or a shallow raft solution, could be a better option in this case.

Site performance, and hence overall resilience, will generally improve with increasing depth of ground improvement, with maximum improvement occurring when treatment extends through the full depth of potentially liquefiable soils. Deep ground improvement methods are included here primarily for control of total settlements; however, they also can be useful for mitigation of the effects of lateral spread, depending on the location of the critical liquefiable layers. Although nominally excluded from 'major' lateral stretch areas in terms of the scope of this document, these methods can be successfully utilised in lateral spread zones with specific engineering design input.

15.3.3.3 Mechanisms of improvement

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

- densification of the in situ soil to eliminate or reduce triggering of liquefaction at design levels of ground shaking. Most effective in clean or low fines content sands. Methods include:
  - rapid impact compaction (RIC)
  - dynamic compaction (DC)
  - columns of highly compacted aggregate, (eg RAP – Rammed Aggregate Piers™)
  - stone columns (ie conventional stone columns) also known as vibro-replacement stone columns.
- replacement of near surface weak soils with a stronger non-liquefiable soil to form a stiff crust. Effective in both sandy and silty soils. Methods include:
  - ex situ: excavate, backfill and recompact – use compacted native soil, cement-stabilised soil or imported gravel to construct an engineered fill raft.
  - in situ stabilisation – mixing cement into the soil to construct a cement-stabilised raft.
- stiffening of the liquefiable soils to improve the integrated foundation system performance through a reduction of cyclic strains; sometimes in combination with increasing liquefaction resistance through densification. This can be effective in both sandy and silty soils. However in sandy soils densification is typically more effective than stiffening. In silty soils the stiffening effects may be primarily due to increases in lateral stresses (which can be lost if large lateral strains occur, for example during a lateral spread event). Methods include:
  - columns of highly compacted aggregate (eg RAP)
  - deep soil mixing (DSM)
  - driven timber piles.

An example of soil stiffening is the use of RAP columns. During the 2013 EQC ground improvement trials, RAP columns were found to perform better than most other methods tested in eliminating or reducing the onset of liquefaction in sandier materials at design levels of ground shaking. As the fines content of the soil increased, the effectiveness of this method to densify the soil decreased. However, it was noted that the installation of the columns still acted to stiffen the overall soil mass which resulted in a reduction in triggering of liquefaction up to moderate levels of ground shaking.

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

15.3.4 Geotechnical investigation requirements

The selection of any ground improvement options in TC3 should be made on the basis of adequate site-specific geotechnical investigations.

The site assessment and ground investigation should include good quality information on the soil types, geotechnical properties and the depth to groundwater. Supplementary testing may be required for detailed design.

There will be limitations on the use of some methods where conditions are highly variable, where peat or silty or clayey soil layers are present, and where steep interfaces occur between subsurface layers (ie highly variable depths of liquefiable deposits across a building footprint resulting in the potential for accentuated differential settlements).

The following general requirements are necessary for investigation of sites which are being considered for ground improvement:

1. Collection and assessment of geotechnical information should be undertaken as outlined in section 13 of this guidance to support the remediation design.
2. Investigations should determine that soil types will respond to the selected improvement method, and that treated zones are sufficiently uniform (or the ground improvement design is suitably robust) that the design will not be compromised by spatial variability within the soil layers.
3. Investigation depths should be adequate to enable the assessment of total settlements for the site.
4. Laboratory testing to determine fines contents and plasticity of soils can be carried out as part of a liquefaction investigation, but for routine house investigations this is often not done (instead relying on simple correlations with in situ testing). This generally errs on the side of conservatism (if any) for Christchurch soils. For design of ground improvement works however, it is recommended that more consideration be given to sample retrieval and lab testing. This will in many cases enable refinement of the ground improvement design. In particular, on silty sand sites the results of lab testing may result in less conservative (ie less expensive) design outcomes. Reference should be made to report UCD/GCM-14/01 'CPT and SPT Based Liquefaction Triggering Procedures' by R Boulanger and I Idriss (2014), particularly in relation to C_{FC} correlations.
5. Calculation of liquefaction triggering and ground settlements should be carried out in accordance with section 13.5 and with particular reference to technical guidance Q&A's 50 & 51.

15.3.5 Improvement types and options

The following is a list of the more common methods or types of ground improvement systems used internationally. There are many variants, but they can be generally grouped by their principal mechanism of mitigating liquefaction effects as follows:

- densification of either the crust layer and/or the deeper liquefiable soils. This includes methods such as compaction, excavation and replacement/recompaction, vibroflotation, preloading, dynamic compaction (DC), and rapid impact compaction
- crust strengthening/stabilisation by permeation grouting, stabilisation mixing or replacement
- reinforcement using deep soil-cement mix piles, jet grouting, stone columns, close spaced timber or precast piles
- containment by ground reinforcement or curtain walls
- drainage using stone columns or earthquake drains.

Most of these methods require clear access to the treated zone ie greenfield site, demolition or temporary removal of the existing dwelling.

Based on the outcomes of the 2011 and 2013 MBIE/EQC field trials, the following methods or types are currently included in these guidelines (grouped by construction methodology rather than mitigation mechanism):

- **Type G1** – Shallow densified crust (ie excavated and recompacted soil or replacement fill (sometimes reinforced); also dynamic compaction or rapid impact compaction).
- **Type G2** – Shallow cement stabilised crust (ie cement-mixed soils, either by excavate and recompact or in situ mixing).
- **Type G3** – Deep soil mixing (ie soil mixed or jet-grouted columns).
- **Type G4** – Deep stone columns.
- **Type G5** – Crust reinforced with inclusions – (ie intermediate depth highly compacted aggregate columns, stone columns or driven timber piles).

These methods are further divided into 10 sub-types, which are listed in Table 15.4 below. This table also summarises advantages and disadvantages of each method, as well as applicability criteria that are discussed later in this section.

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
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]
	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.
	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.
	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.
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]
	G2b	Stabilised crust (In situ mixing)	2m	15.3.10.2(b)	• Can be used in all soil conditions.[5] • No dewatering required.
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]
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]
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]
	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]

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 I_c 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).

	Disadvantages	Applicable surface foundation components[7]	
		TC2 Type Foundations[8,9]	
		Concrete slab Type 2 or 4	Type B (ring foundation)
	• 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)
	• Relatively large equipment required, high mobilisation costs. • Vibrations may negatively impact nearby properties • Not effective in silty soils (FC > 15-25% or I_c > 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.		
	• Same as for Type G1b.		
	• Likely to require dewatering where groundwater table (GWT) < 1.5m deep. • Requires select import materials. • Stockpile area required.		
	• Likely to require dewatering where groundwater table (GWT) < 1.5m deep. • Some specialist contractor knowledge required. • Requires select import materials. • Stockpile area 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.		
	• 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)
	• Not as effective in siltier soils (FC > 15-25% or I_c > 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.		
	• Not as effective in siltier soils (FC > 15-25% or I_c > 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)	
	• Not as effective as shallow stone columns. • Annulus may form around piles during intense ground shaking, allowing ejection of sediment. • Stock pile area required.		

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.

15.3.6 Selection of improvement type

When selecting a ground improvement option for a particular site technical considerations include:

- SLS settlements – whether the site is in the better performing part of TC3 or not (defined as having predicted index SLS settlement of ≤ 100mm in the top 10m of the soil profile).
- where the liquefiable materials appear in the soil column – eg if it is necessary to mitigate the effects of liquefaction occurring from materials below 5m depth, a deep ground improvement method may be more suitable for the site than a shallow method. Alternatively, it might be desirable to instead select an approach such as a surface structure from section 15.4, particularly if there is a relatively intact surface crust that might otherwise be compromised by installing inclusions through it.
- the location of the untreated liquefiable deposits in relation to the proposed finished depth of treatment - also how the behaviour of untreated liquefiable materials might affect future foundation performance.
- post-treatment settlements (SLS and ULS) – eg in a large event, whether settlements will cause undue differential settlements, or possibly flooding issues for the house.
- risk of lateral spread.
- soil type – whether the site is predominantly sandy (and thus amenable to densification methods), or silty, and if there are significant organics present.
- water table depth.

Additionally there are construction issues to consider. On greenfield sites there will be fewer construction issues to consider than for existing sites. These construction issues include:

- within existing housing areas there will be a need to consider proximity issues such as noise, vibration, stability of excavation batters and drawdown effects from dewatering.
- where ground improvement is being installed for an existing house that is undergoing repair, the house will be likely to require temporary removal, or the use of horizontally mixed soil beams might be considered (see section 15.3.12).
- access for the relevant plant and machinery should be carefully considered, especially for houses on rear sections, with narrow accessways or overhead services.
- a building consent will be required when undertaking ground improvement works, if the works are to be part of the intended integrated foundation solution. A resource consent may also be required - requirements should be confirmed for each project. Even if resource consent is not required for ground improvement works, it is important to note that it will be necessary to comply with various performance standards relating to hours of work, erosion and sediment control, construction noise and vibration. Contaminated sites (HAIL or other), historic places, and sites of archaeological interest will need to be carefully managed.

15.3.7 Surface foundation component

As outlined in section 15.3.1 the integrated foundation solution will consist of ground improvement *combined with* a surface foundation component.
The surface foundation component will depend on:

- the ground improvement method selected (ie whether it comprises a continuous 'raft' element, or discrete 'inclusions');
- the ground conditions at the site (ie the expected future deformation performance of the site); and
- other site characteristics (eg flood zones etc).

The basic requirements for constructing the surface portion of the integrated foundation solutions are set out in the following parts of the guidance:

- TC2 concrete slabs Type 2 or 4: Part A, section 5.3.1
- TC2 Type B suspended floors: Part A, section 5.3.2
- TC3 Type 1 (suspended floor) surface structure: Part C section 15.4.3
- TC3 Type 2 (suspended floor) surface structure: Part C section 15.4.4
- TC3 relevellable slab: Part C section 15.4.8 (requires specific design).

Building weight considerations

For the shallower ground improvement methods Types G1 and G2 (refer section 15.3.5), TC2 concrete slab Types 2 or 4, or timber floors from section 5.3 of the guidance can accommodate single-level houses with heavy cladding and two storey houses with light and medium cladding (refer also to Table 7.2). For the deep ground improvement Types G3 and G4, this limitation does not apply (subject to specific engineering input). For Types G5, TC2 concrete slab options 2 or 4 (from section 5.3 of the guidance) can also accommodate single-level houses with heavy cladding and two storey houses with light and medium cladding.

TC3 surface structures Type 1 and 2 are limited to houses with light or medium weight wall claddings and light weight roofs.

15.3.8 Applicability limitations

15.3.8.1 General

Each method is limited to some extent in the scope of its applicability, and the surface foundation components that are suitable for use in conjunction with that method. In some instances these limitations may be able to be overcome by using specific engineering design to formulate a scheme that is equally as robust as those described in this guidance.

Some methods are suitable only for better-performing TC3 sites but their applicability can be extended to other TC3 sites by modifications to the construction specification.

15.3.8.2 Shallow surface crust treatment options

Shallow surface crust treatments (Types G1 and G2) are applicable to sites where the index SLS settlements calculated over the upper 10m of the soil profile ≤ 100mm (or 50mm post-treatment). This is because these methods control liquefaction over more limited depths than the deeper solutions and therefore are limited to the better-performing parts of TC3. (ie those areas having predicted index SLS settlement of ≤ 100mm in the top 10m of the soil profile).

Where these conditions are met, these methods can be used in conjunction with the following surface foundation components:

- TC2 concrete slab Type 2 or 4
- TC2 Type B (ring foundation) with suspended timber floor
- TC3 Type 1 (suspended floor) surface structure
- TC3 Type 2 (suspended floor) surface structure
- TC3 relevellable concrete surface structure.

Alternatively, where calculated settlements exceed these limits, shallow method Types G1 and G2 can still be used under the following circumstances:

- where treatment extends to 2m outside the foundation line, AND
- where in situ methods (cement mixing or compaction) are used then geogrid should be installed at a depth of 0.5m (noting that the excavate and replace/recompact options already include geogrids at the base); AND
- where the following surface foundation components are used:
  - TC3 Type 1 (suspended floor) surface structure
  - TC3 Type 2 (suspended floor) surface structure
  - TC3 relevellable concrete surface structure.

Where a TC3 Type 2 surface structure is constructed on an extended Type G1 or G2 ground improvement, the compacted hardfill layer from section 15.4 can be omitted as long as geogrid reinforcing is still incorporated in the upper 500mm of the improved crust (unless already present in the base of the raft).

Some of these methods are applicable to areas of ‘major’ lateral stretch, where additional geogrid reinforcing can be placed in the base of the densified crust (to enhance tensile capacity).

Note:

Ground improvement methods requiring excavation are unlikely to be economic if sheetpiling or extensive dewatering is required.

15.3.8.3 Deep treatment options

Deep treatment options (ie Types G3 and G4, deep soil-mixed columns, jet grouted columns and deep stone columns to 8m depth) do not have limitations on their use in terms of calculated vertical settlements, other than a requirement to check post-treatment total settlements as outlined in section 15.3.9

Given the discrete nature of the inclusions used in these methods (and also the methods below that involve a crust reinforced with inclusions), slab-type solutions are preferred. These methods can therefore be used in conjunction with the following surface foundation components:

- TC2 concrete slab Type 2 or 4
- TC3 Type 2 (suspended floor) surface structure
- TC3 relevellable concrete surface structure

It may also be possible to use a TC2 Type B (ring foundation) or a TC3 Type 1 surface structure, but only if the surface components are aligned accurately with the discrete subsurface elements, as a specifically engineered design solution.

These methods could be applied in areas of 'major' lateral stretch with specific engineering design.

15.3.8.4 Crust reinforced with inclusions

The application of Type G5 crusts reinforced with inclusions (as specified in this guidance), which focus more on controlling differential settlements while accepting some degree of total settlement, is restricted to sites where SLS settlements calculated over the top 10m of the soil column are less than 100mm (or 50mm after ground improvement).

Where these conditions are met, these methods can be used in conjunction with the following surface foundation components:

- TC2 concrete slab Type 2 or 4
- TC3 Type 2 (suspended floor) surface structure
- TC3 relevellable concrete surface structure.

It may also be possible to use a TC2 Type B (ring foundation) or a TC3 Type 1 surface structure, but only if the surface components are aligned exactly with the discrete subsurface elements, as a specifically engineered design solution.

These methods are generally not applicable to areas of 'major' lateral stretch, without specific engineering design.

15.3.9 Additional requirements and considerations

Where total settlements might affect the future viability of the house (eg in flood zones) a deep ground improvement option would be preferable, if this would mitigate the liquefaction risk to the point where post-improvement liquefaction settlements are no longer an issue (As discussed in section 15.3.1, it is recognised that in many cases this may not be economic). In other instances, the timber floor options in conjunction with the chosen ground improvement method may be more suitable than concrete slab options as they provide more freeboard protection against flooding and also can be significantly easier to re-level.

Accordingly, residual total post-liquefaction ground settlements (ie post-ground improvement) should be assessed in all cases. If post–improvement ULS total settlements calculated over the upper 10m of the soil profile exceed 150mm, a suspended floor solution, or relevellable concrete surface structure (see section 15.4.8), should be opted for. This will allow easier and therefore more economic recovery options. Additionally, if significant issues arise from potential future total settlements, such as minimum flood levels or large differential settlements, they should also be considered. This could be addressed (if chosen to do so) by using deep ground improvement (preferably). Alternatively, if a shallower treatment is chosen, a timber floor substructure or a relevellable concrete surface structure could be used (or by otherwise providing additional freeboard).

Using a TC2-type stiffened slab to allow the building to be reasonably easily re-levelled and raised by exterior jacks, combined with G1, G2 or G5 ground improvement options may be an alternative to deep ground improvement where total settlement is an issue. Ground conditions, the building weight and its shape would need to be considered. The stiffened slab would need to be designed to span across its full width without undue deformation. If a slab option from Part A section 5.3.1 is used then a span of 8m (sometimes greater) is possible. Refer to section 20.4 Part E of the guidance for further information regarding specific reinforcement details. Careful attention to the detailing of services (see section 5.7) is required to prevent damage during future re-levelling/raising efforts. This option is possibly less desirable as it potentially provides less post-settlement freeboard than a suspended floor foundation.

In all cases, if the chosen depth of treatment comes to within a metre or less of the full depth of a liquefiable layer, it is recommended to extend the treatment to the base of the liquefiable deposit. This is because it has been shown that shear strains can be concentrated into thin untreated layers at the base of ground-improved blocks. This will result in shear-induced deformations that are larger than conventionally predicted volumetric strains (and also results in material migrating laterally from under the improved block).

Such behaviour can cause more settlement than would otherwise be expected to occur in the untreated layer. There are documented cases where this increase in strain has been in excess of 100% of the normally calculated volumetric strain. This is a complex issue subject to ongoing research.

In most cases like this (ie where the chosen depth of treatment comes to within a metre or less of the full depth of a liquefiable layer), it is prudent to fully treat the layer; particularly if the layer is very loose, or if the layer is one of the more critical soil layers contributing to the liquefaction hazard. If the thin liquefiable layer is however left in place, the calculated settlement of that layer should be increased by at least 100% when assessing post-improvement liquefaction settlements of the treated site. Careful consideration should also be given to the potential for localised deformation and differential movements developing in the liquefied layer which are not accounted for in the simplified liquefaction evaluation procedures.

Although settlements are assessed at the normal design levels of shaking of SLS and ULS (ie a 25 year return period and a 500 year return period for a house), it is recommended also that a sensitivity check is carried out at an intermediate level of shaking – nominally a 100 year return period. This will often be useful when making the correct choice of ground improvement method and extent of treatment. Depending on the location of critical liquefiable layers, this sensitivity check may show for example that one method will give better performance at this intermediate level of shaking than another method, and therefore provide superior long term resilience. This is despite both methods potentially having similar outcomes at SLS and ULS levels of shaking.

A further consideration is that the plan shape of the site ground improvements should be sufficiently regular (refer to Part A section 11.2 and Supplementary Guidance 'Regular Structural Plan Shapes in TC3' dated September 2013 for guidance).

15.3.10 Specification, construction, and verification requirements

General requirements for each of the options are set out in this section. **It is important to refer to Appendix C4 in each case for detailed construction requirements and method statements**.

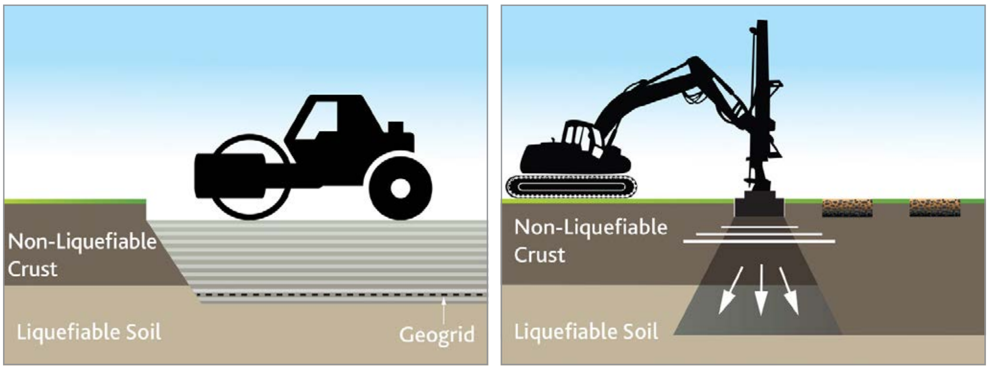
15.3.10.1 Type G1 – Shallow Densified Crust Treatments

These methods involve the formation of a densified block of soil beneath the foundation elements. This is achieved either by densifying the soils that already exist at the site, or by densifying imported materials. Where the imported materials are relatively strong (ie gravels) the depth of the treated zone is reduced.

15.3.10.1(a) Types G1a, G1b, G1c – Densified Raft of Recompacted Soil or Replacement Fill

These methods require the formation of a densified block of soil to a depth of 2m or more to be formed **beneath the foundation elements**. This will generally be achieved by either excavation and recompaction of the subsoils; or by Dynamic Compaction (DC), or Rapid Impact Compaction (RIC).

Figure 15.10: Densified Raft – excavate and recompact (Type G1a) (left) and Rapid Impact Compaction (Type G1c) (right)



DC is normally a commercial-scale methodology where cranes drop weights onto the ground in order to compact the soils. RIC is a type of dynamic compaction, downscaled from the traditional methodology. RIC utilises smaller scale plant that is more appropriate to the residential setting. Both DC and RIC increase the density and therefore the stiffness and bearing capacity of soils through the controlled and repeated impact loading. The depth of influence for DC and RIC is expected to be 3 - 4m. (However it is noted that, like most ground improvement methods, the technique can be varied to treat soils to greater depths).

Excavation and recompaction is best suited to sites where excavation and temporary drawdown of the water table is possible and there is sufficient space to stockpile and manage the materials. In addition, the following requirements generally apply for methods involving excavation and recompaction (ie Type G1a):

1. The construction of a dense raft of engineered fill is required to a minimum depth of **2m** beneath the foundation elements.
2. The excavation base should extend at least **1m** beyond the footprint of the proposed structure.
3. **Two** layers of geogrid are required near the base of the raft. In areas of 'major' lateral stretch, three layers of geogrid are required.

The above requirements also generally apply to the other methods of densification, but they have other specific requirements to achieve an equivalent dense raft (see Appendix C4). When utilising dynamic compaction methodologies (in particular) the potential impacts on neighbouring properties and services need to be very carefully considered.

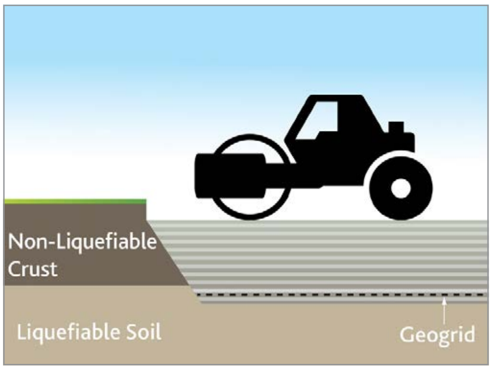
Those methods (DC and RIC) where geogrid layers cannot be placed in the base of the improved zone are not considered applicable to areas of 'major' lateral stretch.

With methods that involve excavation and recompaction, where the excavated materials are unsuitable for recompaction, another possibility is replacement of the excavated materials with imported materials. This is however unlikely to be economic, and a Type G1d (reinforced crushed gravel raft) would be preferable in that situation (being only 1.2m thick instead of 2m).

15.3.10.1(b) Type G1d – Reinforced Crushed Gravel Raft

This method provides a geogrid reinforced gravel raft to a depth of 1.2m **beneath the foundation elements**.

Figure 15.11a: Reinforced Crushed Gravel Raft (Type G1d)



The following requirements generally apply:

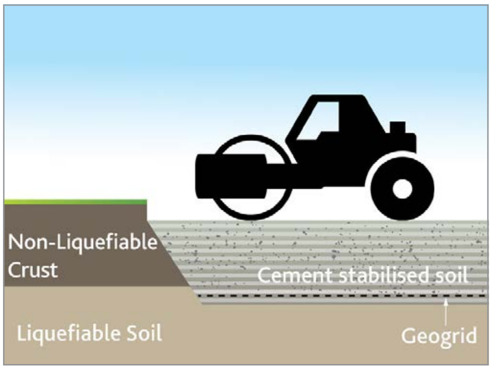
1. The construction of a dense raft of engineered crushed gravel fill is required to a minimum depth of **1.2m** beneath the foundation elements.
2. The excavation base should extend at least **1m** beyond the footprint of the proposed structure.
3. **Two** layers of geogrid are required in the base of the raft. In areas of 'major' lateral stretch, three layers of geogrid are required.

15.3.10.2 Type G2 – Shallow Cement Stabilised Crust treatments

15.3.10.2(a) Type G2a – Reinforced Cement Stabilised Crust

This method will provide a cement-stabilised block of soil to a depth of 1.2m beneath the foundation elements and includes a geogrid reinforcement layer. This will generally be achieved by excavation of the subsoils, mixing with cement and in situ recompaction in layers with a geogrid layer placed above the first layer. On sites which contain high organic or excessively fine-grained soils, an alternative is to dispose of the excavated subsoils and replace with sandy soil, stabilised with cement.

Figure 15.11b: Reinforced Cement Stabilised Crust (Type G2a)



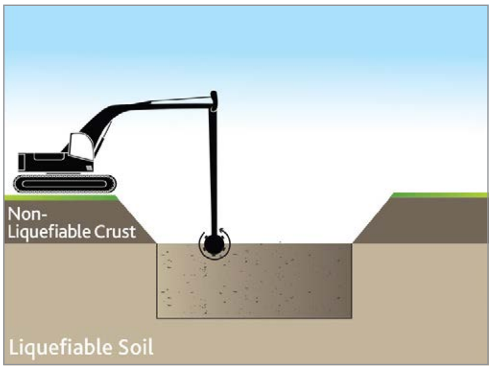
The following requirements generally apply:

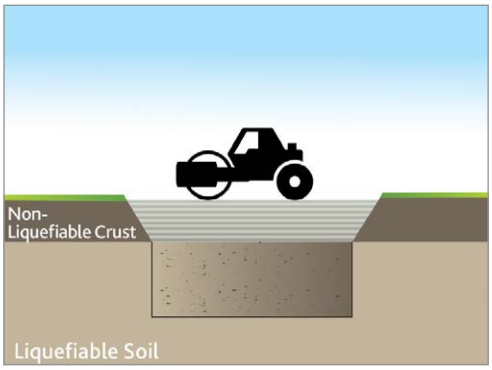
1. The construction of a raft of stabilised fill is required to a minimum depth of **1.2m** beneath the foundation elements, dosed with cement and compacted.
2. The excavation base should extend at least **1m** beyond the footprint of the proposed structure.
3. **One layer** of geogrid is required in the base of the raft. In areas of 'major' lateral stretch, **two layers** of geogrid are required.
4. Other additives, in additional to cement, can be considered if difficulties are being experienced with compaction. For example, lime can be useful in soils with a high clay content.

15.3.10.2(b) Type G2b – Cement Stabilised Crust – In Situ Mixing

This method will provide a cement-stabilised block of soil to a depth of 2m beneath the foundation elements. This will generally be achieved by mechanical mixing in situ the cement with the soil using a panel mixer or rotary cutter machine from the surface.

Figure 15.11c: In situ Cement Stabilisation (Type G2b)





The construction of a stabilised crust may be undertaken by in situ stabilisation with cement, and surface compaction with a heavy static roller (where organic content is less than 5% by volume); groundwater lowering is not necessarily required. This method is more likely to be successful on cleaner sand sites. Where soils are organic or are predominantly fine grained, then replacement with stabilised sandy soils will be required

In situ mixing is expected to be undertaken with a panel mixer or rotary cutter equipment.

The following requirements generally apply:

1. The construction of a raft of stabilised fill is required to a minimum depth of **2m** beneath the foundation elements, dosed with cement and compacted.
2. The stabilised area should extend at least **1.5m** beyond the footprint of the proposed structure.
3. The method of mixing should ensure uniform distribution and mixing of the cement. Overlaps of treated strips must be adequate to ensure there are no untreated zones.
4. The quantity of water added to facilitate mixing should be minimised to the extent possible. In silty soils the addition of water may be needed to facilitate mixing, however very careful control is required to avoid loss of strength.
5. Final cement dosage rates will vary and trial panels and/or laboratory testing is recommended.
6. Testing is required to ensure the strength of the stabilised layer exceeds the measures defined in Appendix C4.
7. This method is not considered applicable in areas of 'major' lateral stretch without specific engineering design – for example where the extent and depth of treatment can be extended to mitigate liquefaction potential in all soils that might contribute to a lateral spreading problem for the site.

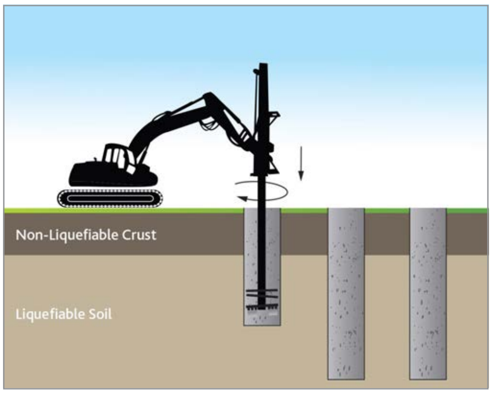
15.3.10.3 Types G3 & G4 – Deep Foundation Treatments

These methods provide a significantly deeper zone of treated materials. They will be less cost effective than the shallower methods, but are included as an option where, for example, a heavier than normal house is to be constructed (see section 15.3.7) or total settlements might otherwise present flooding issues for a house.

15.3.10.3(a) Type G3 – Deep Soil Mixing

This method will provide a relatively deep zone of ground improvement that will reduce soil shear strains during seismic events and therefore reduce the severity of liquefaction.

Figure 15.12a: Deep Soil Mixed Columns (Type G3)



This method is generally applicable for all soil types provided there are no peat zones or organic materials that exceed 5% by volume. They are normally constructed by jet grouting or are injected with cement grout by a rotary auger rig.

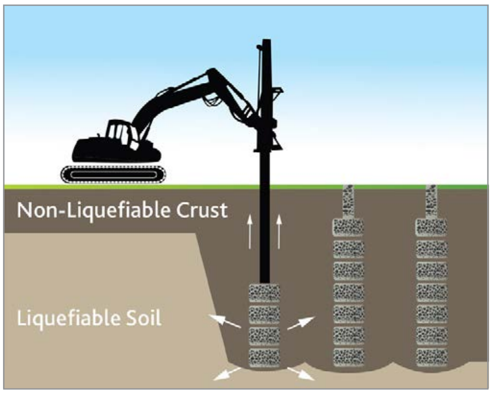
The following requirements generally apply:

1. The treated area must extend to a minimum of **1.5m** outside the building footprint.
2. The jet grouting or deep soil mix columns layout must be targeted to achieve ground treatment as specified in Appendix C4.
3. The columns must extend to a minimum depth of **8m** below ground level or be founded in dense sands or gravels which are proven to be continuous for at least **2m**.

15.3.10.3(b) Type G4 – Deep Stone Columns

Deep stone columns were not included in the 2011 and 2013 MBIE/EQC ground improvement trials because this fairly common method had been used at a number of sites in Christchurch prior to the earthquakes, from which the performance can be assessed. A shallower stone column solution is also available (Type G5a). However deep stone columns are also included as a method that might be employed on sites where Type G5a cannot be used due to excessive calculated settlements (refer section 15.3.9).

Figure 15.12b: Stone Columns (Type G4)



The following requirements generally apply:

1. Stone columns may be used on sites with less than **5%** by volume of peat and organic soils.
2. The treated area must extend to a minimum of **1.5m** outside the building footprint.
3. **Columns must be installed with a displacement procedure** - installation procedures that remove the native soils are not permissible.
4. Depth of columns should be determined by the engineer but the depth is expected to be a minimum of **8m** below ground level or as specified in Appendix C4.
5. In sandy soils, the columns are to be installed in such a manner that minimum mid-point testing provides a density or strength profile as set out in Appendix C4, or in clean sands a minimum column area replacement ratio (ARR) must be achieved, as set out in Appendix C4. In soils with a higher fines content the target density as specified in Appendix C4 must be achieved, or specific engineering analyses performed to demonstrate that the liquefaction potential is adequately mitigated.

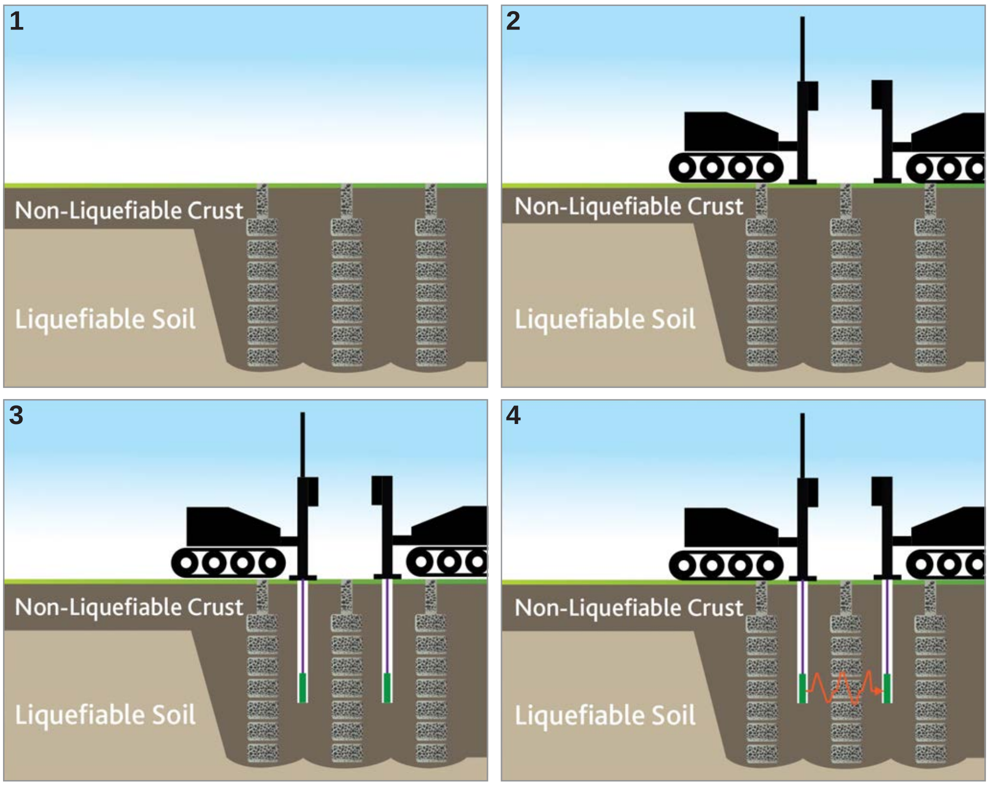
15.3.10.4 Type G5 – Crusts Reinforced with Inclusions

The methods in this section provide a 'crust' of adequate composite stiffness to reduce damaging differential settlements. These methods were tested during the EQC ground improvement trials in 2013. The trials for these methods provided the following results:

- **Type 5a (stone columns)** reduce the liquefaction vulnerability either by densification of the soils (when the soils are relatively sandy), or by providing 'reinforcement' (when the soils are siltier). The 'reinforcement' effect is primarily achieving a stiffening of the improved soil zone. In cleaner sandy soils (generally where the CPT-derived $I_c < 1.8$ approx.) the improvement (densification) achieved can be verified by performing CPT tests at the midpoints between inclusion locations.
- **Type 5b (timber piles)** redistribute foundation loads and therefore reduce differential surface settlement even if liquefaction occurs between the piles. (This may also occur with the Type 5a ground improvement if the inclusions are sufficiently stiff.

For siltier soils (generally where $I_c > 1.8$ approx.), CPT cone resistance may not indicate an appreciable improvement. For highly compacted aggregate columns only, the EQC ground improvement trials showed that, as an alternative to CPT testing measuring the composite (cross-hole) shear wave velocity (V_s) of the improved soil (ie the cross-hole shear wave velocity taken through both the native soil and the constructed inclusion) will give a good indication of the relative stiffness of the block. For highly compacted aggregate columns, where the composite cross-hole shear wave velocity (V_s) is shown to be between 190m/s and 215m/s as outlined in section C4.5 of Appendix C4, the composite soil block is considered to have sufficient stiffness to act as part of an integrated foundation solution that meets the performance criteria as outlined in section 15.3.1.

Figure 15.13a: Composite Shear Wave Velocity Measurement



While measuring cross-hole V_s is relatively simple in concept, it does require experience and must be carried out by highly trained specialists, using dual-receiver techniques. Interpretation of cross-hole velocity measurements is complex, especially for composite sections, and requires specialist technical expertise. At present, there is only limited capability of the required standard within New Zealand, but with time and demand this is likely to improve. Surface methods are not considered adequate as they are subject to an unacceptably high degree of variability in interpretation.

When using crusts reinforced with inclusions the primary goal is to control differential settlements so that a surface foundation can be constructed. Total vertical settlements should also be considered. Where such settlements might lead to other difficulties, flooding issues for example, then it may be preferable to use subfloor systems that take this into account. Examples of these subfloor systems are suspended timber substructures or relevellable concrete surface structures.

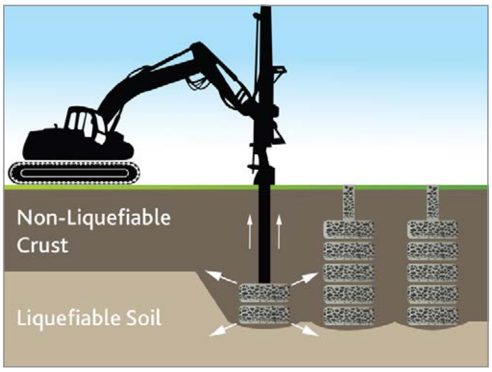
For crust reinforced with inclusions in siltier sands, experience has shown that systems that use highly compacted aggregate (eg 'Rammed Aggregate Piers™) are likely to be more effective than conventional stone columns or driven timber piles. Such systems are well suited for shallow deposits and are effective methods for densifying sandy soils. They also have the ability to reduce liquefaction susceptibility of sands interbedded with silty sediments and thus improve shallow ground performance in proportion to the amount of sand in the soil. All methods however will meet the basic performance requirements as set out in section 15.3.1. It should be noted that timber piles may not be as effective as the other methods on silty sand sites - but can be useful in situations where access for larger machinery is an issue.

15.3.10.4(a) Type G5a – Shallow Stone Columns / Columns of Highly Compacted Aggregate

These methods, which are best suited to clean sand sites, will provide a zone of improved ground (below the foundation elements) at least 4m deep. The improvement is achieved by the installation of stone columns by methods which also induce a proven level of ground strengthening.

When a system that uses highly compacted aggregate is employed (eg 'Rammed Aggregate Piers™) the spacing of the inclusions to achieve a given density will typically be wider (or in general the area replacement ratio (ARR) will be smaller) than for traditionally installed (ie vibro replacement) stone columns.

Figure 15.13b: Shallow Stone Columns (Type G5a)



The following requirements generally apply:

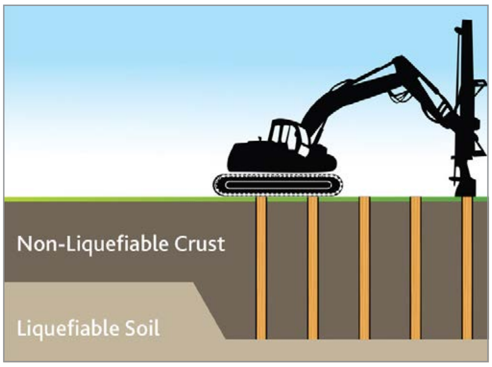
1. Shallow stone columns may be used on sites with less than **5%** by volume of peat and organic soils.
2. Columns must be installed with a displacement procedure - installation procedures that remove or heave the native soils are not permissible.
3. The treated area must extend to a minimum of **2m** outside the building footprint. This could be reduced through specific engineering design, by increasing the ARR around the perimeter of the treated area in combination with further stiffening up the supported foundation component.
4. Depth of columns should be determined by the engineer but shall be a minimum of **4m** (ie probe or mandrel depth) below founding depth or as specified in Appendix C4.
5. In sandy soils the columns are to be installed in such a manner that minimum mid-point testing provides a density or strength profile as set out in Appendix C4, or in clean sands a minimum column area replacement ratio (ARR) must be achieved, as set out in Appendix C4.
6. In soils with a higher fines content, the target density as specified in Appendix C4 must be achieved, or specific engineering analyses performed to demonstrate that the liquefaction potential is adequately mitigated. For highly compacted aggregate columns cross-hole shear wave velocity (V_S) testing can be utilised to assess the composite stiffness of the improvement zone.

15.3.10.4(b) Type G5b – Driven Timber Displacement Piles

This method will provide a 4m deep zone of improved ground beneath the foundation elements. The improvement is achieved by the installation of driven timber **displacement** piles (note that jetted piles are not considered acceptable for this form of ground improvement). The piles will tend to densify sandier soils, and in siltier soils will provide a reinforcing effect. This method was shown in the EQC trials to reduce liquefaction triggering at lower levels of ground shaking (ie somewhat higher than SLS level). When liquefaction does occur however, the piles will still tend to act somewhat as a raft-like system and redistribute foundation loads. This will help reduce differential settlements to an acceptable level when combined with a stiff surface foundation component (see section 15.3.8.2).

The EQC ground improvement research demonstrated that, while providing an acceptable minimum performance, other methods provided more resilience against the effects of liquefaction on siltier sites.

Figure 15.13c: Driven Timber Piles (Type G5b)



The following requirements generally apply:

1. Driven timber piles as a liquefaction ground improvement method may be used on sites without significant deposits of peat and organic soils. Where there are significant deposits of such materials, ground improvement may not be the optimal solution and instead piles would need to be driven to a foundation load-bearing layer and a suitable load-transfer mechanism for the surface foundation designed for gravity loads (see section 15.2).
2. The treated area must extend to a minimum of **2m** outside the building footprint.
3. Depth of driving should be determined by the engineer but shall be a minimum of **4m** (average) below surface founding depth (unless a continuous non-liquefiable layer exists at a shallower depth).
4. In sandy soils the piles are to be installed in such a manner that minimum mid-point testing provides a density or strength profile as set out in Appendix C4. In the absence of post-installation testing, piles shall be installed to provide an area replacement ratio as set out in Appendix C4.
5. A **200mm** layer of compacted gravel should be placed over the pile heads before construction of the surface foundation component. The piles should not be directly connected to the surface foundations to allow for easy future relevelling.

15.3.11 Service trenches and pavements

Where services cross the interface between the treated and untreated ground, detailing should consider the potential for differential movements by including flexible or piped sections and extension of the cement-treated backfill to form a transition zone.

In general, penetrations of the non-liquefiable crust should be minimised where practicable because they may form a zone of weakness that provides a release path for surface expulsion of liquefied soil.

Where services or other excavations are required in the treated (densified or stabilised) zone, care should be taken to minimise disturbance to the surrounding materials. Granular backfill is to be placed in 200mm thick layers with the addition of minimum 3% cement by weight and the materials are to be well compacted to achieve a dense surface as least as compact as the original improved ground. If excavations extend to within 500mm of the edge of the treated zone, the excavation should extend to the edge and the ground be made good as with the trench backfill.

Where pavements are to be constructed beyond the treated zone, a transition may be provided by treating a 300-500mm deep subgrade, by the addition of 3% cement by weight, and including construction joints at any interfaces that are formed.

15.3.12 Ground repairs

Quite separate to ground **improvements** that are carried out under a building footprint in order to form part of an integrated foundation solution, other forms of ground **repair** may be undertaken to return the liquefaction performance of the ground back to (or beyond) the performance of the ground prior to the Canterbury Earthquake Sequence.

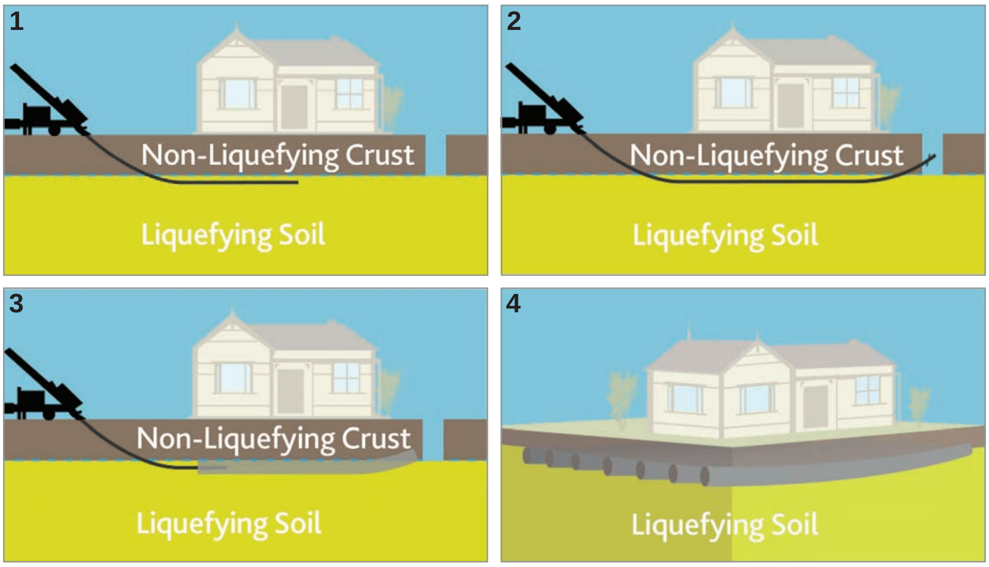
If a homeowner elects to carry out such ground repairs, any of the ground improvement methods as specified in section 15.3.10 and Appendix C4 could be used.

For treatment of the amenity areas outside the building platform, the level of ground improvement should be designed to limit differential deformations to acceptable levels. Consideration must also be given to future development (eg house extensions) that may require upgrading of the ground repair to ground improvement. Examples of this are shallow densified or stabilised crust methods where, outside the building platform, the top 400mm of the crust might be replaced by up to 400mm of topsoil. This could later be replaced by engineered hardfill if the building is later extended.

As the Earthquake Commission (EQC) has determined that a number of houses in Christchurch have suffered a form of land damage they refer to as 'Increased Liquefaction Vulnerability' (ILV), they have carried out an extensive testing programme for a number of different methods that might be used to undertake such repairs.

As a result of the EQC testing programme a new type of ground repair methodology has been found to be effective for ground repairs - Horizontal Soil Mixed beams ('HSM'). These consist of a series, usually two rows deep, of in situ soil cement mixed columns constructed on a horizontal plane, using a modified directional drilling procedure.

Figure 15.14: Horizontal Soil Mixed Beams



In this method, directional drilling equipment is used to pilot a horizontal borehole beneath an existing residential house, daylighting in a receiving trench on the opposite side of the house. A mixing head is then attached to the end of the drill string, which is then progressively reversed back along the alignment of the drill string. Grout is pumped through the drill rods to the mixing head, which mixes the grout into the surrounding soil leaving a horizontal beam of stabilised soil in the ground. This process is then repeated to make a double row of HSM beams below the house.

Considerable additional resilience can be added to the system by including steel reinforcing elements in the horizontal beams, and also by providing horizontal 'capping' beams across the ends of the rows of the beams.

HSM beams require Specific Engineering Design. It is noted that it is very important that the top layer of these horizontal beams is keyed well into the overlying non-liquefiable crust. Construction of these beams under existing houses can provide improved ground performance in future moderate seismic events.

The EQC Ground Improvement Trials Report, currently being finalised for publication, provides additional information relating to horizontal soil mixed beams.

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:
December 2012

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

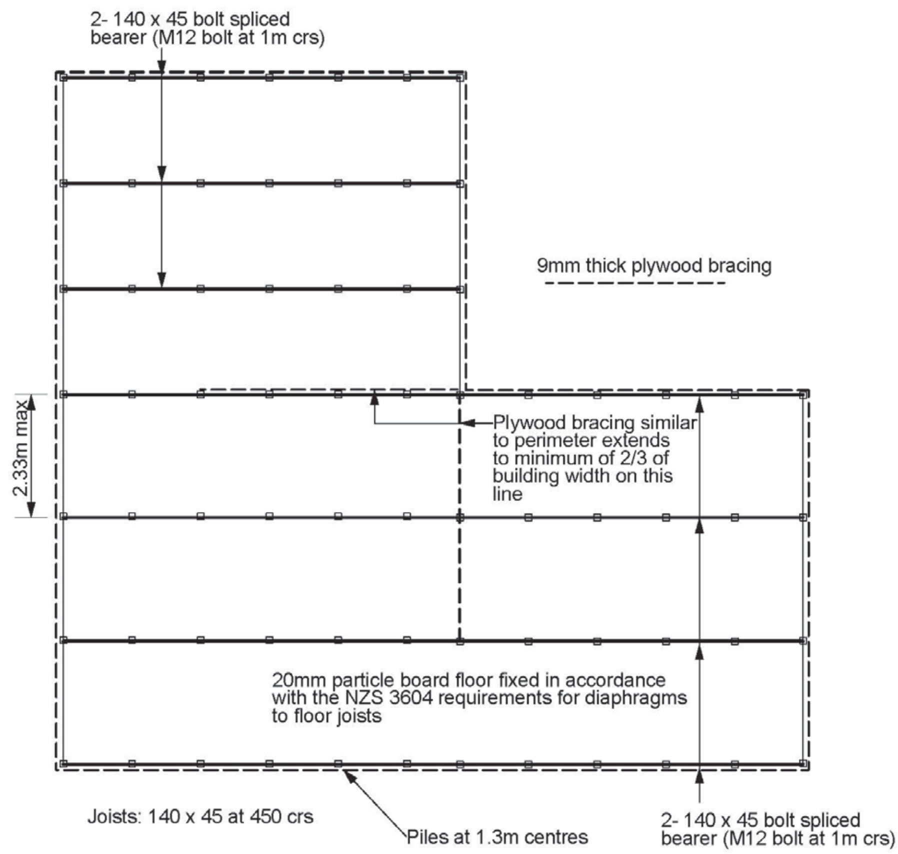


Figure 15.16: Perimeter foundation details for Type 1 surface structure

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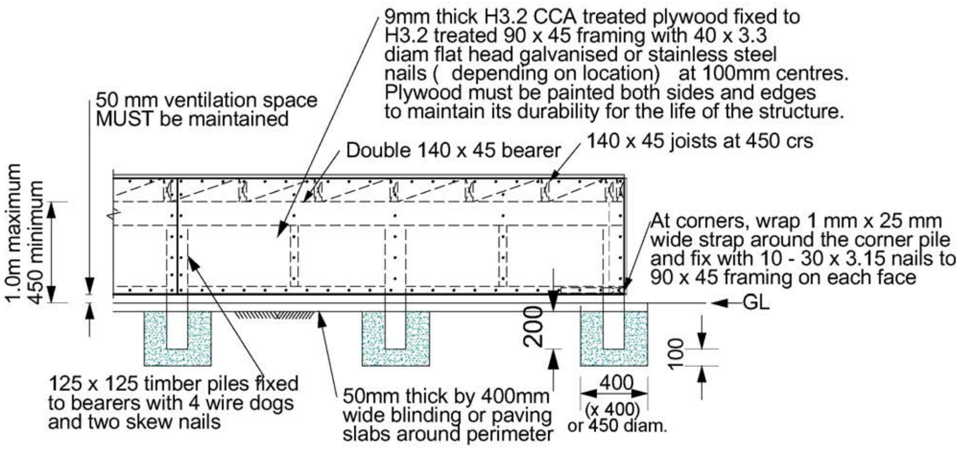
December 2012



Side Elevation (End Elevation similar)

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.

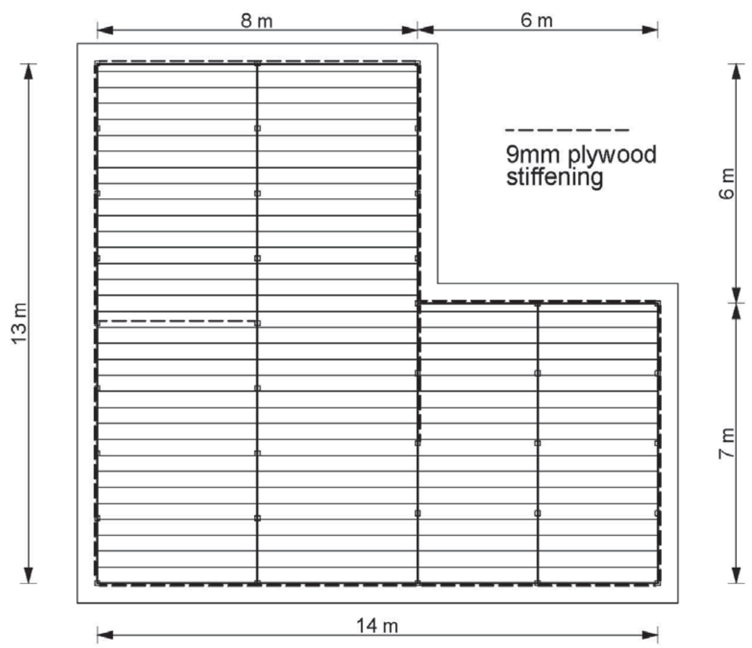
The Type 2A option is a 150 mm thick concrete 'underslab' on a gravel raft, and is capable of resisting vertical liquefaction-induced settlement of the land of up to 100 mm at SLS. Type 2B has a 300 mm thick concrete 'underslab' and is capable of resisting vertical settlement of up to 200 mm at SLS.

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

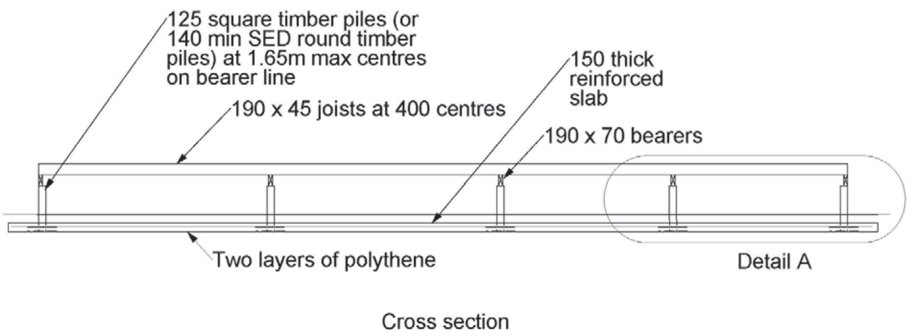


Figure 15.19: Detail of Type 2A surface structure at the timber piles (including gravel raft)

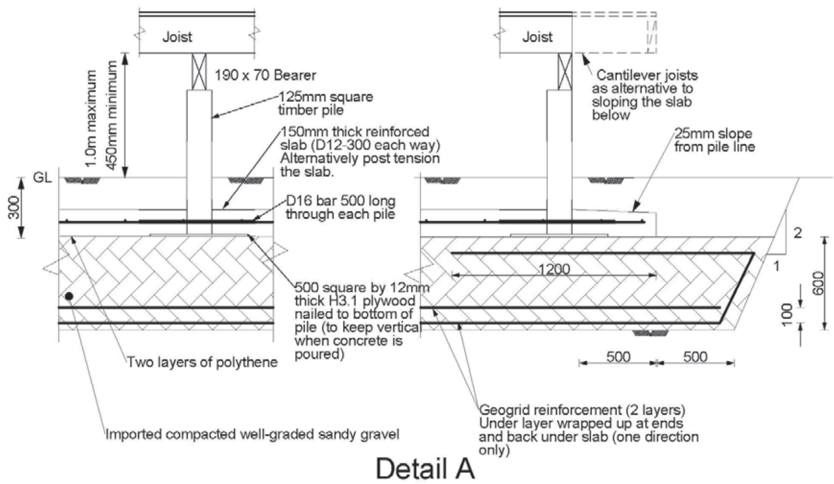


Figure 15.20: Section through Type 2B surface structure at the timber piles (including gravel raft)

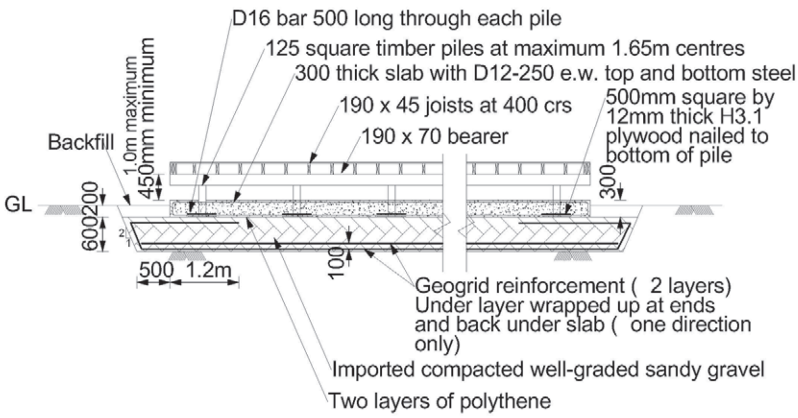
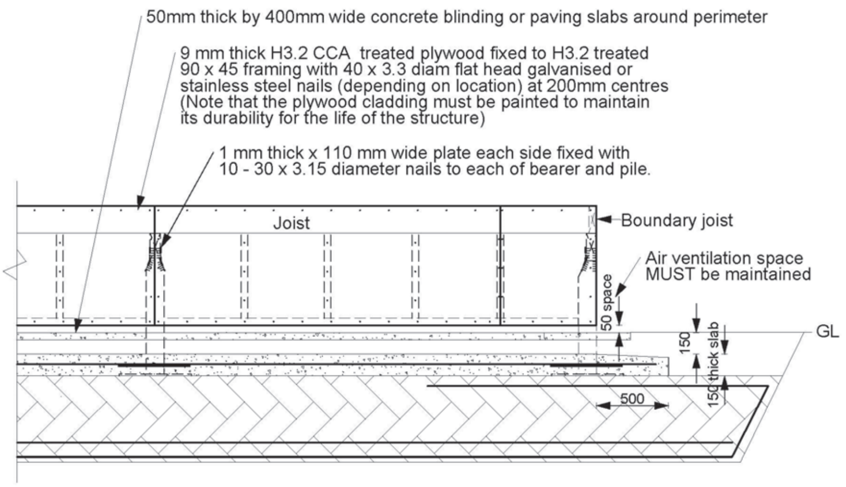


Figure 15.21: Detail of plywood stiffening to Type 2 surface structure (Type 2A illustrated)



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

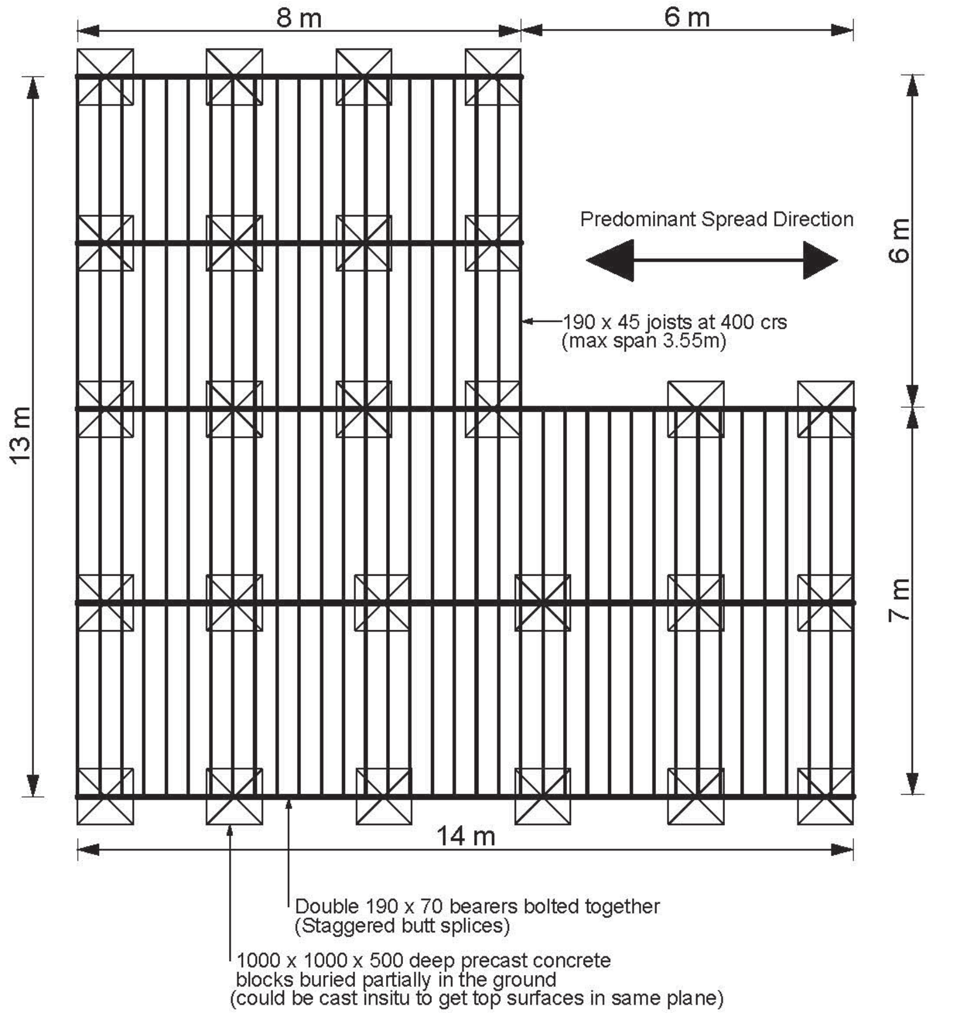
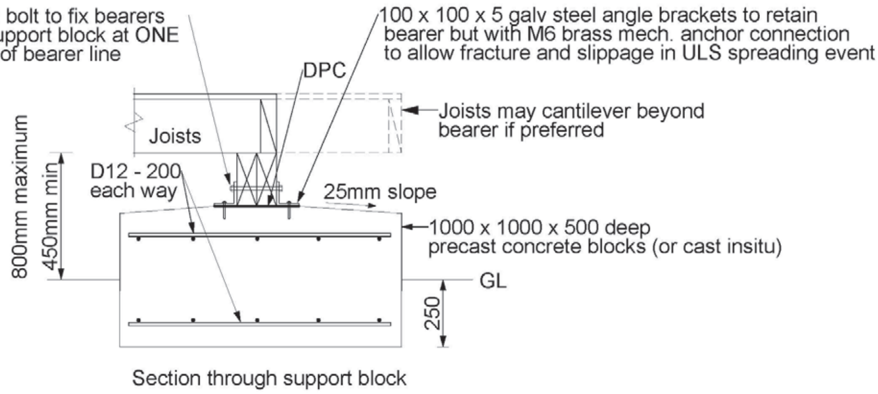


Figure 15.23: Type 3A surface structure - Detail at supporting blocks



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.

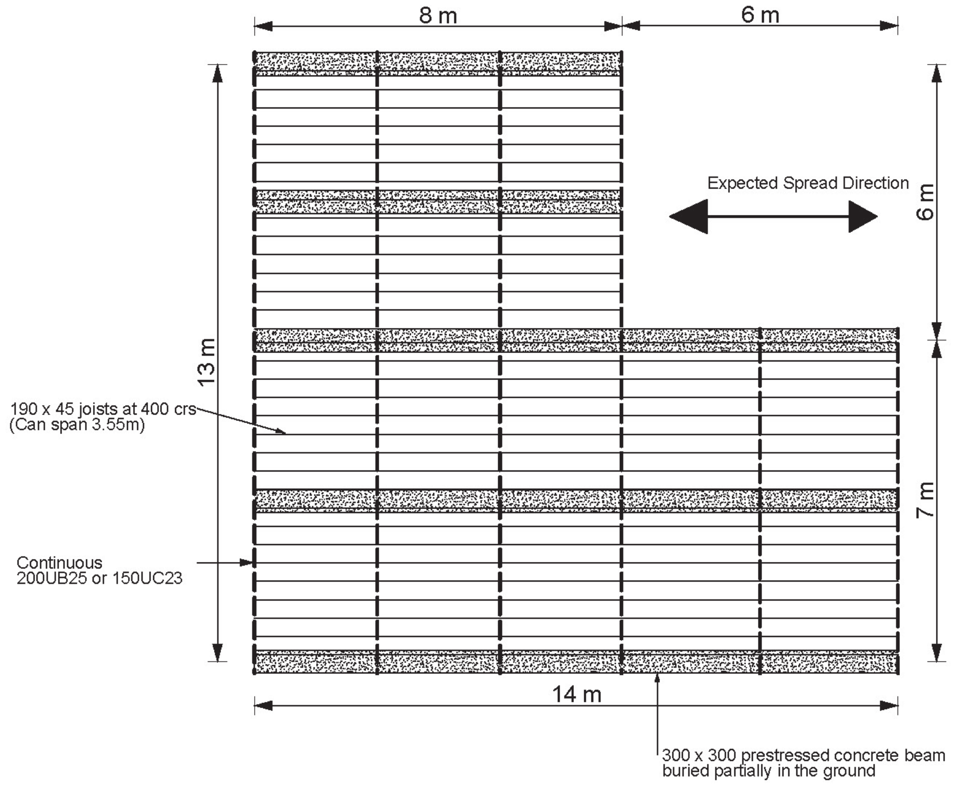
A conventional timber floor and superstructure can be built on the steel beams.

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.

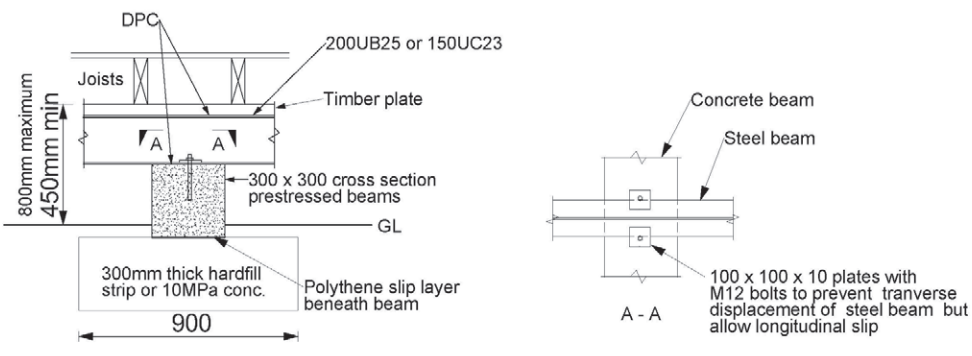
Lateral spreading in the direction of the prestressed beams of up to 250 mm SLS and 500 mm ULS spreading can be accommodated by this example concept.

Figure 15.24: Plan of Type 3B surface structure



Steel beams will tie the concrete pencil beams in position.

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

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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

<table>
<tr><th></th><th>TC2 Foundations</th><th>Hybrid TC2/ TC3 Foundations</th><th colspan="3">TC3 Foundations</th></tr>
<tr><td>Land Settlement Demand</td><td>SLS <50 mm
ULS<100 mm</td><td>SLS <50 mm
ULS>100 mm</td><td>SLS <100 mm</td><td>SLS <200 mm</td><td>SLS >200 mm</td></tr>
<tr><td rowspan="2">Construction</td><td rowspan="2">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)</td><td rowspan="2">Timber: TC3 Surface Structures
Concrete: 300 mm flat slab (Option 2) with gravel raft (Option 1)</td><td rowspan="2">Timber floor on enhanced NZS 3604 subfloor (Type 1 surface structure)
Or Timber floor over concrete underslab on gravel raft (Type 2A surface structure)</td><td>Timber floor over concrete underslab on gravel raft
(Type 2B surface structure)</td><td>Specifically designed subfloor grid
(Type 3 surface structure)</td></tr>
<tr><td colspan="2">Or ground improvement and Type 1 or 2 timber-floored surface structure – refer to section 15.3.4</td></tr>
<tr><td>Structure Performance Outcome Anticipated</td><td>Minor/ slight differential settlement
(ie <25 mm SLS, <50 mm ULS)</td><td>Minor/ slight differential settlement (ie. <25 mm SLS)
Limited damage to foundations at ULS</td><td colspan="3">Readily repairable damage may well occur at SLS
Limited damage to foundations at ULS</td></tr>
<tr><td>Design Considerations</td><td colspan="2">Provision has been made in standard solutions to accommodate effects of minor differential settlement at SLS and ULS should it occur</td><td colspan="2">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)</td><td>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)</td></tr>
<tr><td>Superstructure Constraints</td><td colspan="2">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</td><td colspan="3">Light or medium wall cladding combined with light roofs, regular superstructures only</td></tr>
</table>

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 C1: Basis for confirming compliance with the Building Code for new and repaired house foundations in TC3

This appendix, referred to in section 11.3, Regulatory Context, provides more detailed regulatory guidance as a basis to demonstrate compliance with the Building Code for foundation repairs and rebuilds for TC3 properties. Refer also to the guidance in Part B, section 8.2.

C1.1 Background and principal issues

Under section 17 of the Building Act, all building work must comply with the Building Code. For foundation design, Building Code clause B1 structure is the most relevant clause. Building Code clause B1.3.2 contains a requirement to limit the loss of amenity, commonly known as the Serviceability Limit State (SLS).

Understanding the performance requirements for SLS in TC3 will present a challenge for engineers. Even with the level of information that can be obtained from deep geotechnical investigations, there will be considerable variability and uncertainty for engineers in attempting to quantify future building settlement performance for TC3 properties.

While site conditions and the nature of liquefaction-induced settlement can damage buildings, they rarely affect life safety (ie, exceed the Ultimate Limit State (ULS)).

C1.2 Guidance for demonstrating Building Code compliance – foundation repairs and rebuilds

The following steps set out a consistent basis for engineers and building consent authorities to approach the consenting process.

Step 1: Consider legislative requirements (Building Act 2004)

a) All building work must comply with the Building Code to the extent required by the Act (section 17).
   - This requirement stands regardless of whether a building consent is required. It also doesn't matter whether the building work involved is the construction of new foundations or the alteration or repair of foundations to an existing building.

     Note: it is only the building work that is being undertaken that must comply with the Code; this does not mean the building as a whole needs to comply with the latest Code after the foundations have been repaired.
   - The inclusion of 'to the extent required by this Act' covers Building Act provisions such as the building consent authority being able to grant modifications and waivers to Building Code requirements (section 67).

b) Building work must be carried out in accordance with a building consent (section 40), however there are some exceptions where a building consent is not required (section 41).

Where a building consent is required, an owner must apply for the consent to the Building Consent Authority before the building work begins (section 44).

Building Consent Authorities must grant a building consent if they are satisfied on reasonable grounds that the building work, if properly completed in accordance with the submitted plans and specifications, complies with the Building Code (section 49).

Where the building work includes an alteration to an existing building, the building must continue to comply with the other Building Code provisions at least to the extent that it complied before the alteration (section 112).

Step 2: Consider Building Code requirements – Building Code clause B1 (Structure)

Buildings, building elements and sitework must:

- (Clause B.1.3.1). Have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during construction or alteration and throughout their lives. (Generally referred to as the Ultimate Limit State, ULS)
- (Clause B1.3.2). Have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation, or other physical characteristics throughout their lives, or during construction or alteration when the building is in use (Generally referred to as the Serviceability Limit State, SLS)

Engineers and Building Consent Authorities will also need to consider other Building Code clauses that may be relevant to the foundations, for example B2, Durability, or E2, External Moisture.

Step 3: Consider Building Code Verification Methods – B1/VM1 and B1/VM4 and incorporated Standard AS/NZS 1170 (as amended by B1/VM1)

If B1/VM1 is followed, which incorporates AS/NZS 1170 with some amendments, then engineers will be regarded as complying with Clause B1 of the Building Code.

ULS and SLS criteria are defined in AS/NZS 1170.

SLS seismic loads for residential properties are based on a one in 25 year earthquake (AS/NZS 1170.0).

B1/VM4 is the Verification Method for foundations. B1/VM4 excludes the design of foundations on loose sands or saturated dense sands (1.05 of B1/VM4). This means that for most TC3 properties, B1/VM4 is not applicable for demonstrating compliance with the Building Code.

Step 4: Consider the guidance in Part A and Part B

Part B, section 8.2.3 provides greater clarity on the performance requirements for SLS, as follows:

- 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 structural damage when it is subjected to an SLS load. All parts of the building should remain accessible and safe to occupy.

  Services should remain functional at the perimeter of and 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 one in 25 year earthquake (refer to AS/NZS 1170.0). (refer to Part B, section 8.2.3 of Guidance, emphasis added)

The emphasis added above indicates the types of criteria relevant for assessing SLS for residential construction.

- Table 8.1 in Part B provides criteria for the nature of future damage that corresponds to 'repairability' for both timber-framed/light-clad dwellings and concrete-slab dwellings of any cladding type.
- Part A, section 4 discusses repairing house foundations in Canterbury. It includes guidance for properties where only minor to moderate liquefaction-induced settlement is likely to occur in future SLS earthquakes (ie, TC2 land). Recommendations provided are also relevant for foundation repairs and construction, new foundations on TC3 land.

Examples include:

- the preference to build using light materials rather than heavy materials (refer Part A, section 1.4 Technical Scope, and Part A, section 5.1) to mitigate the potential for liquefaction-induced settlement
- for new construction of foundations, some constraints on plan regularity and light cladding apply (refer Part A, section 5).

Step 5: Consider the TC3 guidance for repairing and rebuilding foundations on TC3 Land

Lightening the load on the foundations will improve the performance of the structure in future SLS earthquakes and provides a way for engineers to have confidence in the future SLS performance without the need to undertake complex quantitative assessments (refer Figures 14.1 and 14.2 and Table 15.2). For example, removing heavy roof tiles to reduce the weight on the soil layers provides scope for foundation repair rather than rebuild in some locations. In addition, masonry veneers can be removed and replaced with light-weight alternatives where damaged.

If the indicator criteria for foundation repair are not exceeded (see Table 2.3 in Part A), repair options demonstrating compliance with the Building Code will depend on whether the calculated consequential liquefaction-induced SLS earthquake settlement (using data from a deep geotechnical investigation (top 10 m only)) is less than 100 mm (refer Figures 14.1 and 14.2). If it exceeds 100 mm, more stringent requirements apply.

If the indicator criteria for foundation repair are exceeded (see Table 2.3 in Part A), rebuild options demonstrating compliance with the Building Code will depend on whether the calculated liquefaction-induced SLS earthquake settlement (using data from a deep geotechnical investigation (top 10 m only)) is less than 100 mm and whether lateral stretch exceeds 200 mm in a ULS earthquake (refer Table 15.2). If these limits are exceeded, more stringent requirements apply.

The more stringent requirements referred to above may include:

- removal of heavy building elements, viz roof and cladding, and replacement with lighter building elements; and
- incorporation of specifically designed features within the supporting structure that are intended to facilitate repairs after a SLS or ULS event causing damage.

C1.3 General

Following the methods and solutions provided in this document, including the considerations and criteria listed above, provides 'reasonable grounds' for designers and Building Consent Authorities that the resulting repairs or rebuild will meet the relevant requirements of the Building Act and Building Code.

Given the uncertainty about the future performance of some of the most liquefaction-prone land, there may be cases where designers, after proper enquiry, are not able to satisfy themselves that a Code Compliant solution is reasonably feasible. The Building Act provides for the Building Consent Authority to issue waivers or modifications to the Building Code (s 67). With the consent of the homeowner and their insurer, this may be an option to consider.

C1.4 Engineering sign-off

An important part of the overall compliance process is the engineering sign-off statement submitted by the engineer.

Residential foundation work is now Restricted Building Work, and must be signed off appropriately by a Licensed Building Practitioner.

It is intended that the next version of this document will include acceptable standard wording for the engineering sign-off for both repairs and the different forms of rebuilt foundations.

Appendix C2: Guidance on PGA values for geotechnical design in Canterbury

C2.1 Purpose

This guidance is issued to provide preliminary guidance on peak ground acceleration (PGA) values for use in geotechnical design, pending further research. This guidance applies to the Canterbury earthquake region only. This is the area covered by the Christchurch City Council, the Selwyn District Council and the Waimakariri District Council.

C2.2 Background

On 19 May 2011 in response to new knowledge about the seismic risk in the Canterbury earthquake region, the Department of Building and Housing (now the Ministry of Business, Innovation and Employment) made changes to the Verification Method B1/VM1, to increase the hazard factor Z (as described in AS/NZS 1170) for the region. The update to B1/VM1 states that the revised Z factor is intended only for use for the design and assessment of buildings and structures – **it is not applicable for use in geotechnical design**. This is because the seismic modelling assumptions and outputs for structural design are different to those required for geotechnical design, and this is particularly significant for the Christchurch region where the seismic hazard is dominated by earthquakes of lower magnitude.

GNS Science have been updating their probabilistic seismic hazard model for the Canterbury earthquake region. This model incorporates the anticipated decline in seismic activity in the region with time over the next 50 years. Preliminary results have been produced from the updated model.

In preparing this guidance the Ministry has considered the preliminary results of the GNS Science hazard model, the effects of ongoing model-refinement, and a range of practical engineering issues. Once the GNS Science reporting is complete, the Ministry will issue more comprehensive guidance as necessary.

C2.3 Interim guidance on PGA values for geotechnical design

Table C2.1 summarises interim recommendations for PGA values for geotechnical design for the Canterbury earthquake region. In accordance with recommended practice for geotechnical analysis, these PGA values are based on a geometric-mean definition (in contrast to the larger-component definition used in AS/NZS 1170 for structural analysis).

The recommended values apply only to deep or soft soil (Class D) sites. This site class is likely to apply over most of the plains in the greater Christchurch area. Non-linearities in the hazard model for Canterbury mean the constant multipliers between PGA values for different site classes used in NZS 1170 are not applicable for the Canterbury earthquake region. Further advice will be provided in future for other site classes.

Recommended PGA values are shown for two types of applications – for liquefaction-triggering analysis and for general geotechnical analysis (excluding rockfall). The main difference between the seismic hazard analyses undertaken for these two types of applications is in the magnitude-weighting factors applied. For liquefaction-triggering analysis, which is very sensitive to earthquake magnitude (ie, duration), a magnitude-weighting factor of $(M/7.5)^{2.5}$ was used in the seismic hazard analysis. For general earthquake geotechnical analysis (such as seismic displacement estimation or embankment or retaining wall design), which is less sensitive to earthquake magnitude, a weighting factor of $(M/7.5)^{1.285}$ was used. For some types of analysis (such as rockfall), magnitude-weighting may not be relevant, so these magnitude-weighted PGA design values are not applicable. Further advice will be provided in future regarding non-weighted seismic hazard analysis.

For geotechnical design for Class D sites on the plains in the Canterbury earthquake region, it is recommended that design PGA values are taken as the greater of either those in Table 1 or those derived from AS/NZS 1170. Note that the latter value only becomes the critical case closer to the Southern Alps, where the seismic hazard is made up more of larger-magnitude earthquakes so the difference between the hazard models for structural and geotechnical purposes is less significant.

The seismic hazard model uses magnitude-weighting to derive an aggregated estimate of peak ground acceleration. Therefore for geotechnical analyses that require an earthquake magnitude to be specified (eg, most liquefaction analyses), a magnitude of 7.5 should be adopted in conjunction with these PGA values.

The current hazard model indicates a slight reduction in predicted PGA levels with greater distance away from the Greendale and Port Hills faults. However at this preliminary stage of model development, it is not appropriate to make recommendations regarding reduced values in more distant locations.

Table CA2.1 provides interim PGA recommendations only for annual exceedance probabilities of 1/25 and 1/500 (ie, SLS and ULS for typical Importance Level 2 structures with a 50-year design life). Structures with a greater importance level or design life are likely to require more detailed and project-specific consideration of seismic hazard than this generalised guidance can provide.

UPDATE:
December 2012

Table C2.1: Interim recommendations for PGA values for geotechnical design in Canterbury (for a M7.5 design event)

Annual probability of exceedance (average over next 50 years)	Peak ground acceleration (g) for deep or soft soil (Class D) site	
	Liquefaction-triggering analysis only [1]	General geotechnical analysis [2] (excluding rockfall)
1/25	0.13	TBA
1/500	0.35	TBA

(1) Corresponds to a magnitude-weighting factor of $(M/7.5)^{-2.5}$
(2) Corresponds to a magnitude-weighting factor of $(M/7.5)^{-1.285}$

Appendix C3: Recommended procedure for calculating capacity for single driven piles in cohesionless soils

The following procedure is based on FHWA (2006) and Meyerhof (1976) with modifications to account for the presence of liquefiable strata above the bearing stratum. The method is empirical and based on SPT blow counts.

For sites where CPT data is available to the full minimum thickness of the bearing layer, alternative procedures based on direct correlation with CPT results may be used. The procedure of Elsami and Fellenius (1997) is recommended and is given below (adapted for the specific purpose of designing residential piles in TC3 sites).

In using these methods, it should be remembered that they are normally used only for preliminary capacity and pile length estimates. However, they are considered adequate for the present purpose of designing pile foundations for residential dwellings in TC3 areas.

C3.1 Procedure for using method based on SPT Data

STEP 1 Correct SPT field N values for field equipment.

The following corrections assume that a standard SPT split spoon sampler is being used. The split spoon sampler should be used in all soils except gravely soils where a 2 inch diameter conical tip may be used.

Correction for hammer efficiency:

$$C_E = \frac{ER_M}{60}$$

ER_M = the measured energy ratio as a percentage of the theoretical maximum.

UPDATE: December 2012

SPT equipment can differ markedly and should be verified with PDA testing and certification.

Correction for rod length:

3 – 4 m	C_R =	0.75
4 – 6 m		0.85
6 – 10 m		0.95
> 10 m		1.0

Corrected N value is given by $N_{60} = N\, C_E C_R$

STEP 2 Compute the average corrected SPT N value, N*, for each soil layer along the embedded length of pile.

STEP 3 Compute unit shaft resistance, f_s (KPa) for each soil layer for driven, displacement piles from:

$$f_s\ (\text{KPa}) = 2\ N^* \qquad f_s < 200\ \text{KPa}$$

for driven, non-displacement piles such as H-piles, use:

$$f_s\ (\text{KPa}) = N^* \qquad f_s < 100\ \text{KPa}$$

STEP 4 Compute ultimate shaft resistance, R_{si} (KN), for each soil layer *i* below the lowest liquefiable soil layer.

$$R_{si} = f_{si}\ A_{si}$$

In which A_{si} = Pile shaft surface area in layer = (pile perimeter) x (embedded length).

For H-piles in cohesionless soils, the "box" area should generally be used for shaft resistance calculations.

STEP 5 Compute average corrected SPT N values, N^*_O and N^*_B, near pile toe.

In most cases, the pile toe will be located within a dense bearing stratum with an overlying stratum of loose and possibly liquefiable soil. N^*_B is the average corrected N value for the bearing stratum extending 3 diameters below the toe, and N^*_O is the average corrected N value within the overlying stratum. For cases where the overlying stratum is expected to liquefy, set $N^*_O = 0$.

STEP 6 Compute unit base resistance, q_b (KPa).

For the case of a weaker layer overlying the bearing layer compute q_b from:

$$q_b = 400N^*_O + 40(N^*_B - N^*_O)\ \frac{D_B}{b} \leq 400N^*_B$$

in which D_B = depth of embedment of toe into bearing layer, and b = pile diameter.
For cases where the overlying layer liquefies, set $N^*_O = 0$.

For piles driven into non-plastic silts, the unit toe resistance, q_b, should be limited to 300 N^*_B instead of 400 N^*_B.

STEP 7 Compute ultimate base resistance, R_b (KN).

$$R_b = q_b\ A_b$$

In which A_b = Pile base area.

For steel H and unfilled open end pipe piles assume that the pile will plug and use the 'box' area of the pile, provided the depth to diameter ratio is greater than 30, otherwise use only the steel cross-section area. q_b should be limited to 5000 KPa for open piles.

STEP 8 Compute ultimate pile capacity, after liquefaction, R_{uliq} (KN).

$$R_{uliq} = \sum R_{si} + R_b$$

in which shaft resistance, R_{si} is only counted from soil layers, including the bearing layer, that are below any liquefiable soils. Down drag from non-liquefying layers **above** liquefiable soils may be neglected for driven piles for residential purposes in TC3 sites. Shaft resistance through liquefiable layers is assumed to be zero.

STEP 9 Apply the design inequality:

$$\Phi_g R_{uliq} \geq 1.2\ G + 1.5\ Q$$

Φ_g = 0.4 is recommended to provide reliable capacity and also to limit settlements.

C3.2 Procedure for using method based on CPT data

STEP 1 Correct q_c for pore water pressure acting on the shoulder of the cone according to the equation:

$$q_t = q_c + u_2\ (1 - a)$$

in which a = the area ratio for the cone (value to be supplied by the CPT contractor)

STEP 2 Calculate the 'effective' cone resistance according to the equation:

$$q_E = q_t - u_2$$

STEP 3 The unit shaft resistance for a pile is correlated with the effective cone resistance based on the soil profile, according to the equation:

$$f_s = C_{sc}\ q_E$$

in which C_{sc}= the shaft correlation coefficient, given as follows
= 0.05 for clay
= 0.025 for stiff clay and clay-silt mixtures
= 0.01 for mixtures of sand and silt
= 0.004 for sand

STEP 4 Compute ultimate shaft resistance, R_{si}, for each soil layer $_i$ below the lowest liquefiable soil layer.

$$R_{si} = f_{si}\ A_{si}$$

in which A_{si} = pile shaft surface area in layer = (pile perimeter) x (embedded length)

For H-piles in cohesionless soil, the 'box' area should generally be used for shaft resistance calculations.

STEP 5 The unit base resistance is computed using a geometric averaging of the effective cone tip resistance over the influence zone at the pile base which, for piles driven into a dense bearing layer, is taken to be a range from 4 pile diameters below the base to 8 pile diameters above the base. The unit base resistance is then given by:

$$q_b = C_{tc}\, q_{Eg}$$

in which C_{tc} is the toe correction coefficient and may be taken as 1.0 for piles less than 400 mm in diameter and q_{Eg} is the geometric average of the effective cone tip resistance over the influence zone.

STEP 6 The total pile end bearing resistance is then given by:

$$R_b = q_b\, A_b$$

in which A_b = area of the base of the pile.

STEP 7 Compute ultimate pile capacity, after liquefaction, R_{uliq} (KN).

$$R_{uliq} = \sum R_{si} + R_b$$

in which shaft resistance, R_{si} is only counted from soil layers, including the bearing layer, which are below any liquefiable soils. Down drag from non-liquefying layers **above** liquefiable soils may be neglected for driven piles for residential purposes in TC3 sites. Shaft resistance through liquefiable layers is assumed to be zero.

STEP 8 Apply the design inequality:

$$\Phi_g R_{uliq} \geq 1.2\ G + 1.5\ Q$$

Φ_g = 0.4 is recommended to provide reliable capacity and also to limit settlements.

Appendix C4: Method statements for site ground improvement

Appendix C4 provides the basic technical details and procedures for the construction of the ground improvement methods discussed in section 15.3. The selection and design of the ground improvement, and overall integrated foundation solution, must also be carried out in accordance with section 15.3.

UPDATE:
Replaced all of Appendix C4
June 2015

Guideline construction specifications for carrying out ground improvement works are also being developed by the New Zealand Geotechnical Society. When published that document should also be referred to. For the rebuilding of TC3 residential houses in Canterbury where the MBIE guidelines are being used as a means of demonstrating compliance, the MBIE guidelines should take precedence in the event there are conflicts between the two documents.

The following method statements are for residential sites, generally either rebuilds or new builds, where the work will be executed by an appropriately qualified and experienced earthworks subcontractor, completing the work as a standalone operation. Ground remediation beneath existing houses (eg Horizontal Soil Mixed Beams) is not addressed in this Appendix.

Demolition or removal of all structures including buildings, foundations, paths, drives and fences will need to be carried out in the area of the proposed ground improvement work, where such impediments exist. Additionally all topsoil, waste or unsuitable fill materials, trees and vegetation (including stumps and root balls) in the works area will also need to be completely removed prior to starting ground improvement construction.

All underground services should be clear of the works area before commencing any ground improvement work. This requirement is not anticipated to be necessary for construction on new sections or subdivisions; however, verification of any underground services should be sought from service providers.

C4.1 Construction quality and quality control

As demonstrated during the 2010-2011 Canterbury earthquakes and the 2013 EQC ground improvement trials, the effectiveness of ground improvement schemes in resisting damaging differential settlement due to liquefaction is highly dependent on the works being well constructed, and the constructed works meeting the design intent. This is particularly true for the shallow foundation treatment and crust reinforced with inclusions methods, which rely on forming a relatively thin dense or stiff crust capable of resisting differential settlement from liquefaction beneath the improvement zone.

For any project, it is recommended that the Design Engineer or their representative performs periodic site inspections during the course of the ground improvement works. The purpose of such inspections is to allow the Engineer to confirm that the ground conditions match those upon which the design was based (particularly for excavate and replace methods), and that the works are being constructed according to the design plans and specifications.

A detailed discussion of construction quality and quality control (QC) is beyond the scope of these guidelines; however, some of the more important aspects are highlighted below:

Excavate and Replace Methods (Types G1a, G1d, G2a)

For these methods, achieving the specified compaction of the backfill, and correct placement of the geogrids is particularly important. The backfill materials should be at, or slightly above the optimum moisture content for achieving the maximum dry density. In sandy soils, vibratory compaction methods are most efficient. In soil subgrade near to or below the water table however, vibration can fluidize or heave the soil, thereby preventing adequate compaction.

The presence of a firm subgrade at the base of the excavation is important to facilitating adequate compaction of the lower lifts of backfill. A layer of high strength woven geotextile fabric or geogrid placed across the base of the excavation can be used to help stabilise soft or 'pumping' subgrade. However, simply spreading and compacting rock or concrete rubble (up to 150mm in size) into the base of the excavation will often stabilise it sufficiently to allow compaction of the subsequent fill layers. This can be a cost-effective solution if there is a nearby source of the appropriate material.

In order to maximise the ability of geogrid (where used) to resist future soil movements, it is important that the grid is tensioned prior to backfill placement to remove wrinkles and folds.

For method Type G2a (cement stabilised crust - excavate, mix and replace), in addition to the specified compaction, applying the correct amount of cement, suitable moisture control and achieving thorough mixing of the soil/cement are critical to forming a strong, stiff improved zone.

Cement Stabilised Crust – in situ mixing (Type G2b)

Use of the correct cement content for the soil conditions present on site is critical for this method. Typically, higher cement contents are required for silty and clayey soils than for clean sands. In all cases, higher soil moisture contents will require higher cement contents in order to achieve complete hydration and 'setting' of the stabilised material.

Thorough mixing of the cement into the soil is very important. Cement needs to be mixed into the soil in a manner that creates a homogenous mixture, with uniform cement and moisture contents throughout the improvement zone. In some cases the addition of some water may help promote better mixing and blending of the cement and soil (especially in silty and clayey soils), as well as prevent segregation of cement in the soil mass. However recent experience in Christchurch has shown that for sites with shallow ground water or wet soils, the addition of water to aid mixing should be carried out very carefully. If water is added it should be only to the extent necessary, to avoid problems with achieving adequate hydration of the soil-cement mix. In particular, care must be taken in sandy soils and non-plastic silts to avoid increasing the water-cement ratio beyond the point where proper hydration can be achieved.

Shallow Stone Columns / Columns of Highly Compacted Aggregate (Type G5a)

Both conventional stone columns (ie 'vibro replacement') and highly compacted aggregate columns (such as rammed aggregate piers - RAP™) are viable ground improvement methods where soil conditions suit. Where the method requires that the stone or aggregate column comprise a relatively stiff element (in addition to displacing and densifying the soil surrounding the column) the material forming the column must be compacted to a greater degree than would be typical for a vibro replacement stone column. Therefore, the construction methodology will be different for a highly compacted column than for a conventional stone column installation.

Geogrid for fill reinforcement

Where geogrid is specified in this document, a geogrid with the following geogrid characteristic performance parameters is preferred.

- Radial Secant Stiffness at 0.5% strain of 390kN/m (within a tolerance of -75kN/m)[1]
- Radial Stiffness Ratio shall be 0.8 (within a tolerance of -0.15)[1]
- The junction efficiency shall be 100% (within a tolerance of -10%)[2]
- The hexagon pitch of the geogrid shall be 80mm (within a tolerance of ±4mm)[3]

As an alternative, a biaxial polypropylene geogrid with a minimum ultimate tensile strength of 40kN/m and retaining a minimum of 28kN/m at 5.0% strain may be substituted.

Verification of target soil density / Stiffness / Composite stiffness

Verification of the constructed ground improvement works may include relative compaction, target soil density, stiffness, or some combination of these. There are several test methods available for assessing whether the target ground improvement has been met. It is important that test locations are uniformly distributed throughout the area of the ground improvement works, to ensure that the required level of improvement is uniformly achieved.

A statistical approach to soil testing and laboratory sample testing is acceptable, with 95% of tests exceeding the strength criteria provided that:

- this is calculated from at least 20 measurements
- that no two results which fail to exceed the criteria are adjacent (vertically or in plane) and
- no single result is less than 80% of the target strength.

Relative compaction of fill

In Canterbury, common methods used to measure relative compaction of fill include the scala penetrometer (sometimes referred to as 'DCP') and the nuclear densometer (NDM). Each of these methods has advantages and limitations. The limitations of the various tests and their applicability to a particular material should be understood by those performing construction quality control.

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1 measured in accordance with EOTA technical report TR41 – B1
2 measured in accordance with EOTA technical report TR41 – B2
3 measured in accordance with EOTA technical report TR41 – B4

Scala penetrometer tests, for example, are not appropriate for soils that contain significant amounts of gravel or oversized material, due to the potential for these larger particles to influence the blow count. In addition, for anything other than very shallow depths, friction on the sides of the penetrometer will influence blow counts (without pre-augering).

If NDM testing is used, care must be taken to confirm that the referenced maximum dry density is applicable to the material being tested. Even relatively small changes in soil classification (eg sand to silty sand or gravelly sand) may require additional laboratory testing of representative samples to confirm the maximum dry density and optimum moisture content for compaction.

For well-graded sandy gravel backfill only

A comparison of the compacted dry density to the solid density of soil particles (specific gravity) of the fill material can be used. This is an alternative to the use of the dry density and optimum moisture content of the soil to determine relative compaction. The solid density of soil particles is unlikely to change significantly if the parent material does not change, and will therefore only be affected by changes in grading (eg particle size) and level of compaction. Provided the fill material is from a consistent source and is well graded, a '% solid density' of 82% is expected to provide adequate compaction. A typical solid density of soil particles for greywacke-derived soils such as those found in the Canterbury area is 2.65 t/m3. Hence, upon confirmation of this value, and at the discretion of the Design Engineer, a target dry density of 2.18 t/m3 may be used as the sole criteria for relative compaction control, without the need for laboratory testing to determine the maximum dry density of the fill material. Note: It will be difficult to achieve satisfactory results using this approach unless the fill materials are optimally graded.

Recommended minimum testing frequencies for compaction testing are provided in this document. Typically used test methods, particularly NDM, are very quick and easy to perform. Therefore more frequent testing may be specified at very little, if any, additional cost. This might be particularly useful early on in a project when the contractor is determining the optimal compaction equipment, level of compactive effort and optimum moisture content to achieve the target dry density. The Design Engineer or their representative is encouraged to perform sufficient testing to comfortably satisfy themselves that the desired level of compaction has been achieved throughout the fill.

Post-improvement in situ testing

The cone penetration test (CPT), standard penetration test (SPT) and heavy dynamic penetrometer test (DPT – heavy) are quite commonly used to measure the effectiveness of post-improvement in situ densification; particularly on larger projects. Where in situ density testing of improved ground is performed to confirm whether triggering of liquefaction will be prevented, the CPT or SPT should be used. This is because the empirical databases, upon which the generally accepted simplified liquefaction triggering analysis methodologies are based, comprise of data only from these two types of tests.

Of these two, the CPT is the preferred method for measuring the effectiveness of in situ densification. This is because it is a standardised ground characterisation tool that provides results that are far less variable than SPT results. Additionally, unlike the SPT, the CPT provides a nearly continuous profile of penetration resistance.

However, the SPT is still acceptable for assessing ground improvement when ground conditions preclude the use of CPT (eg where surficial or interbedded thick or dense gravel layers are present).

Shear wave velocity (V_S) testing has also been used to assess liquefaction potential. However, the results of the 2013 EQC ground improvement trials indicate that for Christchurch sandy and silty soils, simplified triggering methods based on V_S testing (Andrus and Stokoe, 2000[4]; Kayen et al, 2013[2]) do not correlate well with liquefaction triggering, when compared with SPT or CPT based methods. Therefore for typical projects, the use of V_S-based simplified triggering methods alone for confirming liquefaction resistance of improved sandy or silty soils in Canterbury is not recommended. However, cross-hole V_S testing is useful as a proxy for the overall effects of ground improvement and as a measure for the increase in composite soil stiffness in relation to the stiffness of the soil prior to improvement, as discussed in the following section.

It is also recognised that V_S is potentially useful for evaluating the liquefaction potential of gravelly soils where CPT and SPT cannot be performed, or the results of such tests are questionable. V_S-based triggering assessment of gravelly soils is beyond the scope of this document. The Design Engineer may however wish to consider the use of V_S to assess triggering potential in such soils.

Assessing composite soil stiffness using cross-hole shear wave velocity testing (Type G5a)

As discussed in section 15.3, the Type G5a shallow ground improvement method primarily relies on increasing the soil density and/or composite stiffness of the improved soil, in combination with an overlying stiff surface foundation element, to mitigate the effects of liquefaction to the point where the performance of the integrated foundation solution meets the objectives outlined in section 15.3.1. Stone columns or columns of highly compacted aggregate constructed in relatively clean sands ($I_C < 1.8$ approx.) are capable of densifying the ground sufficiently to prevent triggering of liquefaction (depending on the level of shaking) or reduce the effects of liquefaction sufficiently so that the performance objectives of section 15.3.1 are met. Conversely, achieving such densification in silty sands/ sandy silts ($1.8 < I_C < 2.6$ approx.) is unlikely.

Where densification is not being achieved (ie in siltier soils), verification that the design objective of the Type G5a ground improvement has been met can also be carried out by assessing the composite stiffness of the improved zone. The composite stiffness, as used in this document, is defined as the combined stiffness of the ground improvement elements (ie aggregate columns) and the soil matrix between the elements. If a minimum composite stiffness is achieved, liquefaction triggering is either prevented or (depending on the level of shaking) reduced such that damaging differential ground surface settlement is unlikely to occur.

The 2013 EQC trials (refer to section 15.3) identified a composite cross-hole V_S of 220m/s as a value above which triggering of liquefaction between aggregate columns is unlikely to occur under ground shaking up to and including ULS-level ground shaking. However, as discussed in section C4.6, it is not considered necessary to achieve a factor of safety, FoS,

4 Liquefaction Resistance of Soils from Shear-Wave Velocity. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 126, No. 11, November, 2000, pp 1015-1025.

2 Shear-Wave Velocity–Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 139, No. 03, March, 2013, pp 407-419.

against liquefaction of 1 or more at ULS levels of shaking. A composite V_S profile has been developed to provide similar levels of robustness as the q_{c1Ncs} CPT target profiles set out in C4.6. The required profile is a V_S of 190m/s at 1m depth, increasing to 200m/s at 2m depth and 210m/s at 4m depth.

Conducting and interpreting cross-hole V_S testing is a relatively specialised procedure and should only be undertaken by appropriately qualified personnel using purpose-built equipment.

Laboratory testing for design and quality control

All laboratory testing should be performed according to the procedures outlined in the latest applicable New Zealand Standards testing standard(s) with the following exceptions when assessing the liquefaction potential of a site:

6. For determination of the plasticity index (PI), Atterberg limits testing should be performed in accordance with ASTM D 4318 test method.
7. For determination of fines content, a 75μ sieve should be used.

The majority of the international liquefaction case history database that forms the basis for the simplified liquefaction assessment methods is based on lab data using these methods. Therefore the use of these two tests maintains consistency with that. The ASTM Atterberg limits test in theory will result in a given sample having a higher plasticity index, PI, than will be measured by the NZS method because the sample preparation results in a wetter sample. The use of the ASTM method for determining plasticity also maintains consistency with the commonly used PI-based methods for assessing liquefaction susceptibility of fine-grained soil.

If no NZ standard exists for a particular test, the relevant ASTM standard should be substituted.

C4.2 Area replacement ratio (ARR)

The area replacement ratio (ARR) is a common parameter for specifying the ‘amount’ of ground improvement needed when using methods that involve the construction of discreet inclusions (such as stone columns, deep soil mix columns or timber piles). The formula for computing ARR values in the guidance is listed below, and should be used by the Design Engineer for consistency when applying these guidelines:

$$ARR = c_1\left(\frac{D}{S}\right)^2$$

Where:
ARR = area replacement ratio
D = average diameter of inclusion
S = centre to centre spacing of inclusions
c_1 = a constant, depending upon pattern of inclusions
For a square pattern: $c_1 = \pi/4$
For a triangular pattern: $c_1 = \pi/(2\sqrt{3})$

C4.3 Shallow foundation treatments

Densified Crust Method Statement (excavate and recompact) (Type G1a)

This method is generally suited to sand and silty sand sites (~$I_c \leq 2.6$) and where the water table is at least 1.5m below ground level. However, it may also be applicable to predominantly silty sites if the silt can be adequately stabilised by blending with sufficient quantities of angular gravel or crushed concrete.

The densified crust is to be constructed to a minimum of 2.0m deep (below the underside of foundation elements) over the entire house footprint, and extend a minimum of 1.0m beyond the perimeter foundation line. Two layers of geogrid are incorporated into the densified crust to add resilience and improve the ability of the crust to resist differential settlement, and (in the case of lateral stretch) fracturing/pulling apart. In areas of 'major' lateral stretch as defined within these guidelines, a third layer of geogrid is incorporated in the base of the raft.

It may be necessary to batter the sides of the excavation, and provide a drainage sump to remove ground water for the duration of the excavation, filling and compaction work. This method may have limited application where the groundwater level is high, preventing a 'dry' and stable excavation.

A resource consent for dewatering may be required if the site is potentially contaminated. The potential effects on settlement of neighbouring properties needs to be assessed when designing the dewatering system.

Step	Type G1a – Typical Activity Sequence for Densified Crust (excavate and recompact)
1a.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
1a.2	During excavation any organic material is to be removed from site and reported to the Design Engineer.
1a.3	Any physical obstructions encountered during excavation shall be reported to the Design Engineer for further direction.
1a.4	Excavation in strips or sections may be necessary due to site constraints such as adjacent properties or the physical shape of the house. In this case additional care is required at the vertical edge joins by cutting into the previous compacted zone at 1.5h:1v to enable compaction integrity across the joins.
1a.5	Commence excavation to 2.0m (below the underside of foundation elements) and if water is present, construct dewatering sump adjacent to work area. Install pump in the sump and pipe to sediment control.
1a.6	Level and compact the base of the excavation. Where the base of the excavation is stable, the excavation may terminate at 1.8m and the base 200mm compacted in situ. Static compaction is likely to be required in wet or saturated subgrade to avoid fluidizing and/or heaving the ground.

1a.7	The base of the excavation should be stable (not yielding) prior to backfilling. In the event that soft areas are present in the base layer and the target compaction is not achieved, the soft materials should be removed and replaced with suitable material placed and compacted as described in step 1a.9. The base can also be stabilised by placing a layer of compacted rock or crushed concrete (dia. ≤ 150mm) over the soft area to create a 'working platform'. A nonwoven geotextile fabric separation layer comprising Bidim A19 or equivalent should be placed under and over the 'platform' to prevent potential migration of soil into voids within the rock/concrete. The top of any stabilising layer should be kept at a depth of at least 1.2m below foundation level. Alternatively, cement can to be added and mixed into the first 200mm of the subgrade layer to stabilise it. The amount of cement required to stabilise moist (not saturated) soil will be in the order of 10% by weight. The mixed layer should be compacted to the extent practicable and allowed to harden prior to placing any additional fill.
1a.8	Place the first 200mm layer (loose thickness) of fill and compact as described in step 1a.9, then install two layers of geogrid (refer the preferred performance characteristics above – refer to section C4.1 for further information) separated by a 200mm thick layer of compacted fill. The grid should extend neatly to the sides of the excavation, and be lapped at joints as specified by the manufacturer. **Prior to placing fill on top of the geogrid, it is important that the grid is sufficiently tensioned to remove any wrinkles, bulges, etc**. Note that three layers of geogrid, each separated by 200mm of compacted fill, are required in areas of 'major' lateral stretch as defined in this document. Static compaction is likely to be required in wet or saturated ground.
1a.9	Backfill the excavation by placing fill in horizontal loose layers not exceeding 200mm in thickness, moisture conditioned as necessary, and compacting to achieve a minimum of: • 95% standard or 92% of vibrating hammer compaction (NZS 4402:1988 – Test 4.1.1 or Test 4.1.3); • 82% of the solid density of the fill material – (well-graded sandy gravel only, refer to section 4.1); or, • (for non-gravelly soils only), a Scala penetration resistance of 7 blows per 100mm. Perform compaction testing at 600mm vertical intervals within the fill at a minimum frequency of 1 test for each 50m^2 of treatment area or a minimum of 3 tests per interval.
1a.10	Remove dewatering pump and sump once clear of the water table. Backfill and compact as for the foundation treatment work area.
1a.11	Import fill as required to make up for shrinkage due to compaction. The fill can be sand or well-graded sandy gravel to be compatible with required final finished layer.
1a.12	Provide the Design Engineer with complete records of: 1) the material used to construct the raft; 2) results of laboratory MDD/moisture content or solid density tests of backfill materials; 3) results of field compaction testing of backfill; and 4) an 'as-built' plan. Documentation of other relevant details (ie stabilisation of the excavation subgrade with cement or rock) should also be provided. Field compaction test results should include depth below ground level, and horizontal locations relative to a fix point such as a corner of the excavation, and the depth below the top of the raft.

Densified Crust Method Statements – Dynamic Compaction (DC) (Type G1b) and Rapid Impact Compaction (RIC) (Type G1c)

These methods are most suited to clean sand sites (generally I_c < 1.8 / fines content (FC) ≤ 15% approx.) where the depth to the water table is at least 1m below ground level. However, the suitability of a specific site will need to be confirmed before these methods are used.

Due to the size of the plant, noise and vibrations, dynamic compaction and rapid impact compaction will be best suited to open areas away from existing development – such as new subdivisions or areas where several repair sites are located adjacent to one another. In particular, the potential effect of vibrations on nearby structures and occupants needs to be considered.

Vibration monitoring should be conducted on adjacent properties. The recommended maximum vibration level is a peak particle velocity (PPV) < 5mm/s at the nearest structure. However, this should be confirmed by monitoring adjacent buildings for signs of damage, and adjusting site practices as necessary.

These ground improvement methods are intended to form a densified raft of soil at least 2m thick over the house footprint, by targeting a depth of influence of up to 4m (to compensate for the lower level of control that this method has compared to others). The treatment must extend at least 2m outside the perimeter foundation line. The expected minimum energy requirement to achieve the target ground improvement in clean sands is 50 tonne-metres (t-m) for Dynamic Compaction, and 8 tonne-metres (t-m) for Rapid Impact Compaction. However a higher level may be required and this is to would be determined by testing.

The following steps are typical of the dynamic compaction process, in this case, assuming a 1.2m diameter, 8 tonne weight falling 6m. The optimum number of weight drops is to be determined by field trial. Other pattern options arising from economy and individual site constraints are acceptable but the total energy at each node must be achieved.

Step	Type G1b – Typical Activity Sequence for Dynamic Compaction
1b.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
1b.2	Set out position of primary pass nodes based on a 2.5m regular right-angle grid pattern.
1b.3	Any physical obstructions encountered during compaction shall be reported to the Design Engineer for further direction.
1b.4	Undertake a trial set of 8 drops and record the depth of penetration (set) after each drop. Finalise the optimum number of drops based on the total 'set' versus blows.
1b.5	Perform primary pass compaction with at least 4 drops per node location (or greater number based on trial). Record the total set at each node, and the set for each drop at every 10th node.

1b.6	On completion of the primary pass, relevel the site with compacted imported clean fill. The fill can be sand or well graded sandy gravel to be compatible with required final finished layer.
1b.7	Set out position of secondary pass nodes at 2.0m centres off-set 50% in both directions relative to the primary pass nodes.
1b.8	Perform secondary pass compaction with at least 4 drops per node location (or greater number based on trial). Record the total set at each node.
1b.9	On completion of the secondary pass relevel site with imported fill as for step 1b.6.
1b.10	Set out position of ironing pass nodes at 1.5m centres.
1b.11	Perform ironing pass compaction with 2 drops per node location. Record the set at each node.
1b.12	On completion of the ironing pass relevel site with imported compacted clean fill.
1b.13	Undertake verification testing to a depth of at least 3m on a 10m grid to confirm that the required level of soil improvement has been achieved (refer to discussion below).
1b.14	After verifying that the target improvement has been achieved, the entire improvement area should be sub-excavated to a depth of 400mm and recompacted as engineered fill (refer to Step 1a.9 of the typical activity for improvement method G1a above.
1b.15	At the completion of work, provide the Design Engineer with: 1) records of the all node drop and set data; 2) records of additional fill placed; 3) an 'as-built' plan showing the DC points relative to the structure footprint; 4) results of field verification testing; and 5) documentation of any relevant construction issues (ie obstacles encountered, changes in construction sequence).

The following steps are typical of the RIC process, in this case, assuming a 1.5m diameter, 7.5 tonne weight falling 1.3m. The optimum number of weight drops is to be determined by field trial. The literature suggests a terminal set of 5mm/blow should be used for control. Other pattern options arising from economy and individual site constraints are acceptable but the total energy at each node must be achieved.

Step	Type G1c – Typical Activity Sequence for Rapid Impact Compaction
1c.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
1c.2	Set out position of primary pass nodes at 2.5m regular right angle grid.
1c.3	Any physical obstructions encountered during compaction shall be reported to the Design Engineer for further direction.
1c.4	Undertake a test of 40 drops and record the depth of penetration (set) after each drop. Finalise the optimum number of drops from the set versus blows.
1c.5	Commence primary pass compaction with at least 12 drops per node location (or greater number based on trial). Record the total number of drops and total set at each node.

1c.6	On completion of the primary pass relevel site with compacted imported clean fill. The fill can be sand or well graded sandy gravel to be compatible with required final finished layer.
1c.7	Set out position of secondary pass nodes at 2.5m centres off-set 50% in both directions relative to the primary pass nodes.
1c.8	Commence secondary pass compaction with at least 12 drops per node location. Record the total number of drops and total set at each node.
1c.9	On completion of the secondary pass relevel site with compacted imported clean fill.
1c.10	Undertake verification testing to a depth of at least 3m on a 10m grid to confirm that the required level of soil improvement has been achieved (refer to discussion below).
1c.11	After verifying that the target improvement has been achieved, the entire improvement area should be sub-excavated to a depth of 400mm and recompacted as engineered fill (refer to Step 1a.9 of the typical activity for improvement method G1a above.
1c.12	At the completion of work, provide the Design Engineer with: 1) records of the all node drop and set data; 2) records of additional fill placed; 3) an 'as-built' plan showing the RIC points relative to the structure footprint; 4) locations and results of field verification testing; and 5) documentation of any relevant construction issues (ie obstacles encountered, changes in construction sequence).

To confirm that the required level of improvement has been achieved for either method, CPT testing should be used. The testing should be conducted at a frequency of 1 test per 100m^2 of ground treatment area, with a minimum of 3 tests per house site. Target post-improvement CPT tip resistance profiles are presented in section C4.5. Note that for soils with an appreciable fines content (ie I_C > 1.8 / FC > 15% approx.), the fines correction to tip resistance can be significant, hence, it is more appropriate to use the equivalent clean sand tip resistance, q_{c1Ncs}. As discussed in section C4.5, soils with an I_C > 2.6 or plasticity index, PI, of greater than 12 do not require improvement.

Densified Crust Method Statement (reinforced crushed gravel raft) (Type G1d)

This method is generally suitable for most sites where the water table is at least 1.0m below ground level.

The crushed gravel raft is to be a minimum of 1.2m deep (below the underside of foundation elements) over the entire house footprint, and extend a minimum of 1.0m beyond the perimeter foundation line. The raft is to be constructed of crushed gravels comprising TNZ M/4 40mm or equivalent (eg crushed AP40 with at least 70% stone having 2 or more broken faces. Outside reinforced grid zones, crushed AP65 can be used).

Two layers of geogrid are incorporated into the raft to add resilience and improve the ability of the crust to resist differential settlement and (in the case of lateral stretch) fracturing/pulling apart. In areas of 'major' lateral stretch as defined within these guidelines, a third layer of geogrid is incorporated.

It may be necessary to batter the sides of the excavation, and provide a drainage sump to remove ground water for the duration of the excavation, filling and compaction work. This method may have limited application where the groundwater level is high and a 'dry' and stable excavation cannot be practically formed.

A resource consent for dewatering may be required, particularly if the site is potentially contaminated. The potential effects on settlement of neighbouring properties needs to be assessed when designing the dewatering system.

Step	Type G1d – Typical Activity Sequence for Densified Crust (reinforced crushed gravel raft)
1d.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
1d.2	During excavation any organic material is to be removed from site and reported to the Design Engineer.
1d.3	Any physical obstructions encountered during excavation shall be reported to the Design Engineer for further direction.
1d.4	Excavation in strips or sections may be necessary due to site constraints such as adjacent properties or the physical shape of the house. In this case additional care is required at the vertical edge joins by cutting into the previous compacted zone at 2h:1v to ensure compaction integrity is attained across the joins.
1d.5	Commence excavation to 1.2m (below the underside of foundation elements) and if water is present, construct dewatering sump adjacent to work area. Install pump in the sump and pipe to sediment control.
1d.6	Level and compact the base of the excavation. Static compaction is likely to be required in wet or saturated subgrade to avoid fluidizing and/or heaving the ground.
1d.7	The base of the excavation should be stable (not yielding) prior to backfilling. In the event that soft areas are present in the base layer and the target compaction is not achieved, the soft materials should be removed and replaced with suitable material placed and compacted as described in step 1a.9. The base can also be stabilised by placing a layer of compacted rock or crushed concrete (dia. ≤ 150mm) over the soft area to create a 'working platform'. A nonwoven geotextile fabric separation layer comprising Bidim A19 or equivalent should be placed both under and over the 'platform' to prevent potential migration of soil into voids within the rock/concrete. Alternatively, cement can to be added and mixed into the first 200mm of the subgrade layer to stabilise it. The amount of cement required to stabilise moist (not saturated) soil will be in the order of 8% by weight. The mixed layer should be compacted to the extent practicable and allowed to harden prior to placing any additional fill.
1d.8	Place the first 200mm layer (loose thickness) of crushed gravel and compact as described in step 1a.9, then install two layers of geogrid (refer the preferred performance characteristics above – refer to section C4.1 for further information) separated by a 200mm thick layer of compacted fill. The grid should extend neatly to the sides of the excavation, and be lapped at joints as specified by the manufacturer. **Prior to placing fill on top of the geogrid, it is important that the grid is sufficiently tensioned to remove any wrinkles, bulges, etc**. Note that three layers of geogrid, each separated by 200mm of compacted crushed gravel, are required in areas of 'major' lateral stretch as defined in this document.

1d.9	Backfill the excavation by placing crushed gravel fill in horizontal loose layers not exceeding 200mm in thickness, moisture conditioned as necessary, and compacting to achieve a minimum of: • 95% standard or 92% of vibrating hammer compaction (NZS 4402:1988 – Test 4.1.1 or Test 4.1.3); or • 82% of the solid density of the fill material – (well-graded sandy gravel only, refer to section 4.1). Perform compaction testing at 600mm vertical intervals within the fill at a minimum frequency of 1 test for each 50m^2 of treatment area or a minimum of 3 tests per layer.
1d.10	Remove dewatering pump and sump once clear of the water table. Backfill and compact as for the foundation treatment work area.
1d.11	Provide the Design Engineer with complete records of: 1) the material used to construct the raft; 2) results of laboratory MDD/moisture content or solid density tests of backfill materials; 3) results of field compaction testing of backfill; and 4) an 'as-built' plan. Documentation of other relevant details (ie stabilisation of the excavation subgrade with cement or rock) should also be provided. Field compaction test results should include depth below ground level, and horizontal locations relative to a fix point such as a corner of the excavation, and the depth below the top of the raft.

Reinforced Cement Stabilised Crust Method Statement (excavate, mix and replace) (Type G2a)

This method is generally suited to clean sand to sandy silt sites where the water table is at least 1.0m below ground level. For sites with a higher water table, temporary dewatering may be required.

The cement stabilised crust is to be a minimum of 1.2m deep (below foundation elements) over the house footprint and extend at least 1m beyond the house perimeter foundation line. It may be necessary to batter the sides of the excavation, and provide a drainage sump to remove ground water for the duration of the excavation, filling and compaction work. This method may not be used where water inflows cannot be controlled to prepare a 'dry' base of excavation.

The minimum cement content required for stabilisation is 8% (by dry unit weight). Alternatively, laboratory testing can be used to determine the required minimum cement content. Following are the minimum laboratory strength/stiffness to be achieved at 7 days (or 28 days at the discretion of the Design Engineer):

- UCS > 1 MPa and initial tangent Young's Modulus of 250 MPa; or
- CBR > 25.

Where the water table is close to the base of the excavation and it is difficult to fully compact the first layer of fill, an increase in cement content to 10% is recommended and compaction should be undertaken using a static roller.

A resource consent for dewatering may be required if the site is potentially contaminated. The potential effects on settlement of neighbouring properties needs to be assessed when designing the dewatering system.

Step	Type G2a – Typical Activity Sequence for Reinforced Cement Stabilised Crust (excavate, mix and replace)
2a.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
2a.2	During excavation any organic material is to be removed from site and reported to the Design Engineer.
2a.3	Any physical obstructions encountered during excavation shall be reported to the Design Engineer for further direction.
2a.4	Excavation in strips or sections may be necessary due to site constraints such as adjacent properties or the physical shape of the house. In this case, additional care is required at the vertical edge joins to ensure compaction integrity is attained across the joins, ie excavate on a batter and bench into the previously treated strip.
2a.5	Commence excavation to 1.2m below foundation depth and if water is present construct dewatering sump adjacent to work area. Install pump in the sump and pipe to sediment control. The depth of excavation can be reduced to 1.0m if suitable plant is used to dose, mix and compact the base 200mm layer of stabilised soil in situ.
2a.6	Level the base of the excavation, and compact it sufficiently to allow proper compaction of subsequent layers of backfill material. Static compaction is likely to be required in wet or saturated subgrade to avoid fluidizing and/or heaving the ground.
2a.7	Excavated soil and cement should be passed through a rotary mixer fitted with a currently certified weighing device to ensure the cement is added at the target dosage rate. Note the time taken to mix the cement uniformly throughout the batch and apply to all subsequent batches. As an alternative to ex situ mixing, the soil may be placed in 200mm lifts and cement spread and **uniformly** mixed in situ with a rotovator with blades that extend 50mm into the underlying layer. In situ mixing of the first layer of fill over the geogrid (refer to step 2a.8) may not be possible due to the risk of damaging the grid.
2a.8	Place the first 200mm layer (loose thickness) of stabilised soil and compact as described in step 2a.9, then install two layers of geogrid (refer the preferred performance characteristics above – refer to section C4.1 for further information) separated by a 200mm thick layer of compacted stabilised fill. The grid should extend neatly to the sides of the excavation, and be lapped at joints as specified by the manufacturer. **Prior to placing fill on top of the geogrid, it is important that the grid is sufficiently tensioned to remove any wrinkles, bulges, etc**. Note that three layers of geogrid, each separated by 200mm of compacted stabilised soil, are required in areas of 'major' lateral stretch as defined in this document.
2a.9	Backfill the excavation by placing the soil-cement mix in horizontal loose layers not exceeding 200mm in thickness and compacting to achieve a minimum of 95% standard compaction (NZS 4402:1988 – Test 4.1.1). Perform compaction testing at 600mm vertical intervals within the fill at a frequency of 1 test for each 25m^2 of treatment area.
2a.10	Remove dewatering pump and sump once clear of the water table. Back fill and compact as for the foundation treatment work area.

2a.11	If using less than 8% cement by weight, obtain QC test samples of the stabilised soil by sampling mixed material at a rate of 1 sample per 100m^3 of material placed. Each sample should include sufficient material to make 4 100mm diameter test cylinders. The samples should be taken from the placed material prior to compaction, and compacted into 100mm diameter moulds within 1 hour of cement mixing. The samples should be carefully stored and transported to a testing laboratory (see note below table), and cured for 7 days (or 28 days at the discretion of the Design Engineer). To confirm that the target strength is achieved, the samples should be tested and meet the following criteria: • UCS > 1 MPa; or, • CBR > 25. Alternatively, in situ QC testing can be conducted in lieu of laboratory testing as follows (1 test/50m^2, minimum 3 test locations per residential site to just short of base of raft to avoid perforating base): • Uncorrected CPT q_c > 6 MPa; • Uncorrected SPT > 20; or, • Scala > 10 blows/100mm
2a.12	Provide the Design Engineer with complete records of: 1) results of laboratory testing conducted to confirm cement dosing rate, if done; 2) cement dosing rates applied during mixing; 3) results of laboratory MDD/moisture content tests; 4) results of field compaction testing of stabilised backfill; 5) results of laboratory QC tests; and 6) an 'as-built' plan. Field compaction test results/laboratory QC test results should include the depth below ground level, and horizontal test/sample locations relative to a fix point such as a corner of the excavation, and the depth below the top of the raft.

Note:
If samples are not **immediately** carefully stored, and then transported, experience has shown that degradation will almost certainly occur. This will result in samples that are not cured and will not be able to be tested, thus rendering the QC process abortive (in which case the alternative in situ measurements outlined above could be used, or it will be up to the works contractor and engineer to find alternative means of demonstrating compliance, as a specific engineering design process). Refer ASTM D 4220 for guidance.

Unreinforced Cement Stabilised Crust Method Statement (in situ mixing) (Type G2b)

This method is best suited to sand and silty sand soils. There are a number of proprietary techniques available for in situ cement – soil mixing. Two known to be locally available are:

- Tracked-panel stabilisation mixer.
- Rotary cutter and stabilisation mixer.

Both are coupled to either a grout or dry-cement batching plant, and are considered suitable for this method if they are operated to produce a homogeneous block of stabilised soil to the required strength and dimensions.

The stabilised crust should be at least 2m deep (below foundation elements) over the house footprint, to at least 1.5m outside the house perimeter foundation line.

The soil to be stabilised is to be uniformly treated with a minimum target dose rate of 10% of cement added to the soil (by dry unit weight). Alternatively, laboratory testing can be used to determine the minimum cement content. Following are the minimum laboratory strength/ stiffness to be achieved at 7 days (or 28 days at the discretion of the Design Engineer):

- UCS > 1 MPa and initial tangent Young's Modulus of 250 MPa; or
- CBR > 25.

Step	Type G2b – Typical Activity Sequence for Unreinforced Cement Stabilised Crust (in situ mixing)
2b.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
2b.2	During treatment any organic material encountered is to be reported to the Design Engineer.
2b.3	Any physical obstructions encountered during treatment shall be reported to the Design Engineer for further direction.
2b.4	Set out appropriate pattern and sequence to suit equipment type used, ensuring entire area receives a uniform distribution of stabilised mixed soil.
2b.5	Commence soil-mixing process and ensure entire treatment area is completed in one continuous operation. Ensure there is a minimum overlap of 500mm with each mixing pass to ensure a continuous stabilised area.
2b.6	If using less than 10% cement by weight, obtain QC test samples of the stabilised soil by sampling mixed material at a rate of 1 sample per 100m^3 of material placed. Each sample should include sufficient material to make 4 100mm diameter test cylinders. The samples should be taken from the placed material prior to compaction, and compacted into 100mm diameter moulds within 1 hour of cement mixing. The samples should be carefully stored and transported to a testing laboratory (see note below table), and cured for 7 days (or 28 days at the discretion of the Design Engineer). To confirm that the target strength is achieved, the samples should be tested and meet the following criteria: • UCS > 1 MPa; or, • CBR > 25. In situ QC testing should also be conducted as follows (1 test/50m^2, minimum 3 test locations per residential site to just short of base of raft to avoid perforating base): • Uncorrected CPT q_c > 6 MPa; • Uncorrected SPT > 20; or, • Scala > 10 blows/100mm
2b.7	Provide the Design Engineer with complete records of: 1) results of laboratory testing conducted to confirm cement dosing rate, if done; 2) cement dosing rates applied during mixing; 3) description of plant and mixing process used; 4) locations and results of field verification testing; 5) documentation of additional relevant construction issues such as addition of water or cement to compensate for unexpected conditions; and 6) an 'as-built' plan.

C4.4 Deep foundation treatments

Deep Soil Mix (DSM) Columns Method Statement (Type G3)

This method is generally suited to most soils provided there are no layers of peat or organic materials that exceed 5% of the treatment zone by volume.

Soil-mixed columns are constructed using either a jet-grouting rig and grout-batching plant, or a rotary auger drilling rig and dry-cement dispenser or grout-batching plant.

The drill has a rotary head fitted with grout jet nozzles to produce a (typical) nominal 800mm diameter column of grout-strengthened soil. The rotary auger rig introduces dry cement or grout through the base of the augers.

Ground improvement is required across the entire house footprint, and at least 1.5m beyond the perimeter foundation line. The minimum depth of the columns should be a minimum of 8m below ground level, or into a layer of dense non-liquefiable soil proven to a minimum 2m thickness, whichever is deeper, unless a shallower depth is demonstrated to be adequate based on specific design.

A minimum area replacement ratio (ARR) of 18% should be achieved.

The cement dosing rate is nominally 10% by dry weight, but must achieve a minimum 7-day strength of 2 MPa and an initial tangent Young's modulus of 400 MPa.

Step	Type G3 – Typical Activity Sequence for Deep Soil Mix (DSM) Columns
3.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
3.2	During treatment any organic material encountered is to be reported to the Design Engineer.
3.3	Any physical obstructions encountered during treatment shall be reported to the Design Engineer for further direction.
3.4	Set out the design grid pattern across the work area.
3.5	Commence drilling of first column to confirm ground conditions at design depth– advise Design Engineer and confirm target column depth.
3.6	Complete drilling and soil mixing column process to the entire work area.
3.7	Sample the jet grout mix at a rate of 1 sample per 50m^3 of column for laboratory testing. Each sample should comprise a minimum of 4 100mm diameter test cylinders. The samples should be taken from the placed material prior to compaction, and compacted into 100mm diameter moulds within 1 hour of cement mixing. The samples should be carefully stored and transported to a testing laboratory (see note below table), and cured for 7 days (or 28 days at the discretion of the Design Engineer).
3.8	Conduct laboratory unconfined compressive strength testing to confirm that the samples meet the required 7 day UCS of 2 MPa and initial tangent Young's Modulus of 400 MPa.

3.9	After verifying that the target improvement has been achieved, the entire improvement area should be sub-excavated to a depth of 400mm and recompacted as engineered fill (refer to Step 1a.9 of the typical activity for improvement method G1a above.
3.10	Provide the Design Engineer with records of: 1) cement dosing rates; 2) the samples collected; 3) results of strength and stiffness tests; 4) an 'as-built plan' showing the columns relative to the structure footprint; and, 5) documentation of any relevant construction issues (ie obstacles encountered, changes in construction sequence).

Note:
If samples are not immediately carefully stored, and then transported, experience has shown that degradation will almost certainly occur. This will result in samples that are not cured and will not be able to be tested, thus rendering the QC process abortive. It would then be up to the works contractor and engineer to find alternative means of demonstrating compliance, as a specific engineering design process). Refer ASTM D 4220 for guidance.

Deep Stone Columns Method Statement (Type G4)

Stone columns (often referred to as 'vibro replacement') are typically constructed using a suspended vibrating probe and follower tube using either a 'wet top feed' or 'dry bottom feed' process. The follower tube is used to tremie graded aggregate to the tip of the probe during extraction. The probe is also used during extraction for aggregate compaction. This method applies only to methods which displace and densify the soil, not ones that only replace the soil.

This method is typically effective densifying relatively clean sands (generally $I_c < 1.8$ / FC < 15% approx.). However, as the fines content of the sand increases, achieving significant densification becomes more difficult. In soils with FC greater than about 20-25% (or $I_c > 1.8 - 2.3$ approx.), international experience suggests that little densification may be achieved.

The ground improvement is required to be applied to the house floor plan, and at least 1.5m beyond the house perimeter foundation line. The minimum depth of the columns should be a minimum of 8m below ground level, or into a layer of dense non-liquefiable soil proven to a minimum 2m thickness, whichever is deeper, unless a shallower depth is demonstrated to be adequate based on specific design.

Stone materials should be uniformly graded free-draining aggregate or crushed concrete with at least two broken faces.

The following steps are typical of the stone column/vibro replacement process. The initial column diameter, spacing and layout is to be determined based on design. It is common practice to verify the effectiveness of the design layout with a field trial.

Step	Type G4- Typical Activity Sequence for Stone Columns (vibro replacement)
4.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
4.2	During treatment any organic material encountered is to be reported to the Design Engineer.
4.3	Any physical obstructions encountered during treatment shall be reported to the Design Engineer for further direction.
4.4	Set out the design grid pattern across the work area.
4.5	Commence installing first column to confirm soil conditions at design depth – advise the Design Engineer and confirm target column depth.

4.6	Complete stone column installations to entire work area.
4.7	Undertake verification testing to confirm that the required level of soil improvement has been achieved (refer to discussion below).
4.8	After verifying that the target improvement has been achieved, the entire improvement area should be sub-excavated to a depth of 400mm (or base of any disturbed materials, but no less than 300mm) and recompacted as engineered fill (refer to Step 1a.9 of the typical activity for improvement method G1a above).
4.9	Trim surface and provide 100mm drainage (aggregate) layer (as part of the reworked layer in step 4.8) comprising well-graded sandy gravel to prevent migration of fines.
4.10	Provide the Design Engineer with: 1) records of quantity of aggregate added to each column location; 2) results of field density (verification) tests; 3) an 'as-built' plan showing column locations relative to the structure footprint; and, 4) documentation of any relevant construction issues (ie obstacles encountered, changes in construction sequence).

To confirm that the required level of improvement has been achieved CPT testing should be used. The testing should be conducted at a frequency of 1 test per $100m^2$ of ground treatment area, with a minimum of 3 tests per house site. Target post-improvement CPT tip resistance profiles are presented in section C4.6. Note that for soils with an appreciable fines content (ie $I_c > 1.8$ / FC > 15% approx.), the fines correction to tip resistance can be significant, hence, it is more appropriate to use the equivalent clean sand tip resistance, qc1Ncs.

In lieu of CPT testing in soils with $I_c < 1.8$ or FC < 15% approx. a minimum column area replacement ratio (ARR) of 18% can be used. In soils with a higher I_c value/fines content, the target CPT resistances must be achieved, or specific engineering analyses performed to demonstrate that the liquefaction potential is adequately mitigated. As discussed in section C4.6, soils with an $I_c > 2.6$ or PI of greater than 12 do not require improvement.

C4.5 Crust reinforced with inclusions

Shallow Stone Columns / Columns of Highly Compacted Aggregate Method Statement (Type G5a)

This method includes conventional stone columns, typically constructed as described for ground improvement method Type G4, or highly compacted aggregate piers. As for the Method G4 ground improvement, Method G5a columns must be constructed using methods that displace and densify the soil; not replace the soil. The highly compacted aggregate piers are constructed by applying a high compaction effort (often a combination of downward pressure and vibration) to the aggregate to form stiff, high density columns. One example is the Geopier Rammed Aggregate Pier™ System (RAP). This is a patented/proprietary ground improvement system, but is similar in principle to various other methods including Terrapiers, Geo Piers and Impact Piers.

Both types of columns are most suited for densifying relatively clean sands ($I_c < 1.8$ / FC < 15% approx.). The amount of densification that can be achieved will decrease with increasing silt content, to the point where meaningful densification cannot be achieved. However, RAP or equivalent stiff aggregate columns can still have a beneficial mitigation effect in potentially liquefiable silty soils through stiffening effects. The highly compacted aggregate piers will generally result in a stiffer column than conventional vibro replacement.

The ground improvement works are to extend over the house floor plan, and at least 2m beyond the house perimeter foundation. The depth of the improvement (ie probe or mandrel depth) should be a minimum of 4m below the underside of foundation elements, or to a depth that results in a total non-liquefiable crust thickness of at least 4m under the foundation elements. For example, a 2m deep improvement combined with a non-liquefiable layer proven to extend from a depth of 2m to 4m below the underside of foundation elements.

The typical construction activities for shallow conventional stone columns or columns of highly compacted aggregate are the same or similar to the methodology for deep stone columns (Type G4) described above and therefore are not repeated here.

One difference between the two column types is that the area replacement ratio (ARR) required to achieve the required level of ground improvement is expected to be less for columns of highly compacted aggregate. To achieve densification of relatively clean sands with conventional stone columns typically requires an area replacement ratio in the order of 16 to 20%. The results of the 2013 EQC ground improvement trials (EQC Ground Improvement Trials report (currently being finalised for publishing) indicated that for 4m deep columns of highly compacted aggregate, an ARR as low as 8 to 12% was sufficient to adequately mitigate liquefaction effects at the ground surface up to the ULS level of ground shaking, in terms of the objectives outlined in section 15.3.1.

To confirm that the required level of improvement has been achieved CPT testing should be used. The testing should be conducted at a frequency of 1 test per 100m^2 of ground treatment area, with a minimum of 3 tests per house site. Target post-improvement CPT tip resistance profiles are presented in section C4.6. Note that for soils with an appreciable fines content (ie $I_c > 1.8$ / FC > 15% approx.), the fines correction to tip resistance can be significant, hence, it is more appropriate to use the equivalent clean sand tip resistance, q_{c1Ncs}.

In lieu of CPT testing in soils with $I_c < 1.8$ or FC < 15%, a minimum column area replacement ratio (ARR) of 12% for columns of highly compacted aggregate can be used. The minimum ARR should be increased to 18% for conventional vibro-replacement columns.

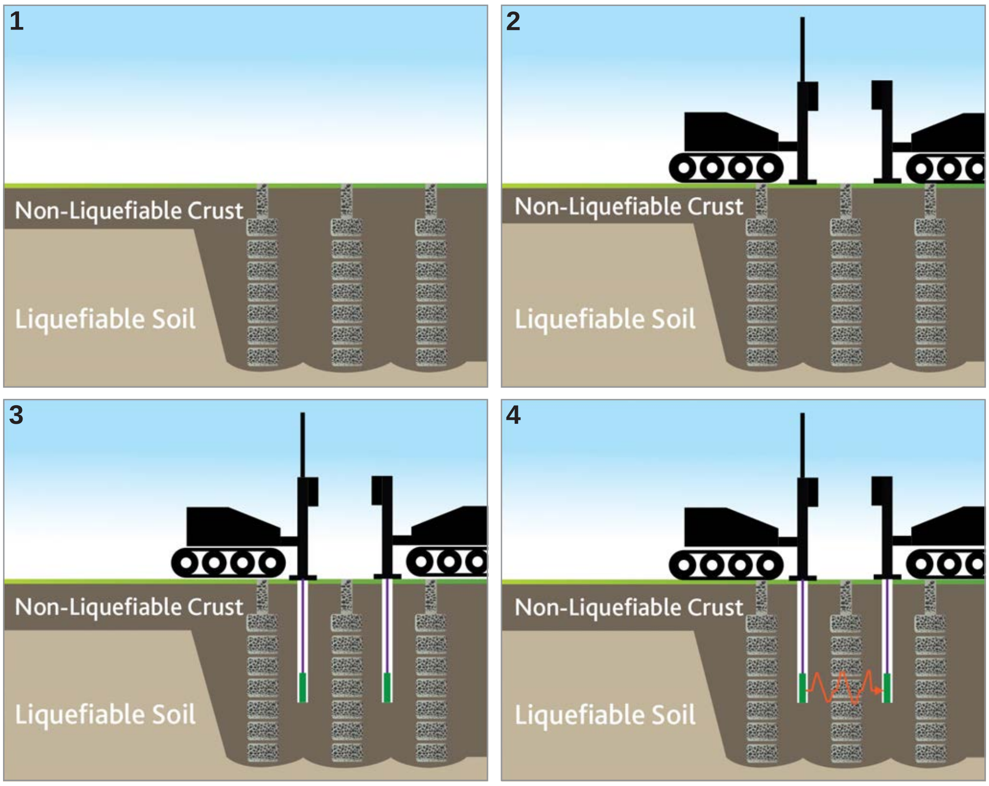
In soils with a higher I_c value/fines content, the target CPT resistances must be achieved, or specific engineering analyses performed to demonstrate that the liquefaction potential is adequately mitigated. Alternatively, cross-hole shear wave velocity (V_s) testing can be used to assess the composite stiffness of the improvement zone (ie the combined stiffness of the column and surrounding soil). The target improvement is considered to have been met if a composite cross-hole V_s profile within the improvement zone is achieved as follows:

Ground Improvement Types G5a Target Composite Shear Wave Velocity Criteria	
Depth below ground level (m)	**V_s (m/s) (only required to base of treated layer)**
1	190
2	200
4	210
5	215

Alternatively, a lower composite V_S value may still be acceptable if specific analysis/design demonstrates that the ground improvements will still reduce liquefaction and/or distribute foundation loads such that damage to the foundation system is likely to meet the design requirements.

The cross-hole pairs should be located in-line with, and halfway between, any two ground improvement points so that the composite V_S is representative of the soil/improvement point (refer to the figure below). The V_S should be measured at 0.5m vertical intervals throughout the depth of the improved zone, beginning at a depth of 1m below ground level. Cross-hole V_S tests should be conducted at a frequency of 1 test per 100m^2 of ground treatment area, with a minimum of 3 tests per house site.

Figure C4.1: Cross-hole shear wave velocity testing of Type G5 ground improvement



Driven Timber Displacement Piles Method Statement (Type G5b)

As discussed in section 15.3, driven timber piles may be used to densify relatively clean sands (I_C < 1.8 / FC < 15% approx.) although vibro replacement stone columns or columns of highly compacted aggregate may be preferable if there is sufficient site access. In silty soils, driven timber piles are not expected to provide significant improvement through soil densification, but they may still reduce differential ground surface settlement through redistribution of foundation loads.

The piles should be driven without the use of jetting. The pile depth should be a minimum of 4m below the underside of foundation elements (average depth with an allowable variation from this average of +/- 0.4m to allow for efficient use of available lengths).

Any variations from the average depth must be evenly distributed across the site.

The piles should have a minimum diameter of 200mm (for tapered piles this can be the average diameter over the length of the pile). Piles with a minimum diameter of 200mm that have not been shaved (ie 'uglies') are permissible. For the determination of the ARR, the average as-driven diameter of the piles may be used (with no more than 50mm variation from this average, a minimum average diameter of 200mm, and a maximum taper of 10mm per metre). Any variation of pile diameters must be evenly distributed throughout the pile grid. The pile grid spacing should be determined by the Design Engineer based on meeting the CPT tip resistances specified below, or in the absence of post-installation CPT testing, the minimum ARR specified below. The pile grid should extend across the entire house footprint, and at least 2m beyond the house perimeter foundation line.

The piles should be ground treated to the equivalent of H5, and cut ends of piles should be re-treated to the same level of protection. Re-treated ends shall not be placed at the lower end of the pile.

If timber piles are to be used solely as improvement through soil densification, the target CPT tip resistance of the soil between piles should be the same as specified for method G5a above. The minimum frequency of CPT testing should be 1 test per 100m^2 of improvement area with a minimum of 3 tests per house site. Alternatively, a minimum ARR of 5% can be used for tapered piles, or 5.5% for piles of uniform diameter.

The following steps are typical of the driven timber pile process:

Step	Type G5b- Typical Activity Sequence for Driven Timber Pile Grid
5.1	Set out perimeter of foundation treatment area and locate marker pegs clear of all workings. Remove all topsoil and other unsuitable materials.
5.2	During treatment any organic material encountered is to be reported to the Design Engineer.
5.3	Any physical obstructions or noticeably soft ground encountered during driving shall be reported to the Design Engineer for further direction.
5.4	Set out suitable grid pattern to the work area.
5.5	Commence installing a H5 treated pile to verify the target depth – advise Design Engineer and confirm depth.
5.6	Complete grid of pile installation across entire work area.
5.7	If using soil densification for verification of improvement, undertake verification testing at a rate of 1 test/100m^2 at points equidistant between the nearest piles to confirm that the target density of the soil has been achieved as specified for Type G4 above.
5.8	Over drive piles to allow for placement of a 200mm layer of compacted gravel over the pile heads.
5.9	Provide the Design Engineer with an 'as-built' plan of the pile grid, as well as material supplier certificates and documentation of any construction issues such as driving difficulty or broken piles.

C4.6 Target CPT tip resistances for ground improvement

The design philosophy for these target soil densification criteria is based around the primary objective that the integrated foundation solution should provide a building platform that controls liquefaction-induced differential settlement to the degree that acceptable foundation performance is maintained. This performance objective is discussed further in section 15.3.1.

It is recognised that at ULS levels of shaking it is not necessary to achieve a factor of safety (FOS) against liquefaction of 1 or more throughout the soil profile. Instead, by controlling the onset and severity of consequential effects following liquefaction triggering (without seeking to eliminate triggering altogether) it is possible to achieve the design objective (ie controlling differential settlement).

One of the key advantages of undertaking ground improvement to increase the relative density of the soil is that as well as increasing the FOS against liquefaction triggering for a given level of shaking, it also decreases the severity of post-triggering effects (eg volumetric/shear strain and excess pore pressure) for a given FOS.

Accordingly, the target soil densification criteria have been selected with the aim of limiting strains at ULS levels of shaking, while preventing triggering at SLS and intermediate levels of shaking (assumed to be the 100 year return period level of shaking as discussed in section 15.3.9).

It is recognised that different minimum target densities are appropriate for the shallow and deep treatment options. For the shallow treatment options, the foundation performance relies on forming a robust and stiff non-liquefiable surface crust to mitigate the surface effects from liquefaction of the underlying soils. Therefore a higher target density is required than for the deep treatment options, where foundation performance is achieved by reducing the potential for liquefaction over the entire depth (or at least the majority) of the liquefiable soil deposits.

The depositional environment in Canterbury is such that soil types are often layered or interbedded, and may either abruptly or gradually transition from one soil type to another (for example, from sand to silt or from sand to silty sand to silt). For this reason, the response to ground improvement may vary with depth (ie the target densities specified below may not be achieved in some layers). This does not necessarily mean the overall result is unacceptable. However, the situation would need to be addressed by demonstrating one of the following:

- that the non-responding soil layer does not actually need to be treated (ie it is already non-liquefiable due to its fines content or it is above groundwater level); or:
- through detailed analysis, the overall result is still acceptable.

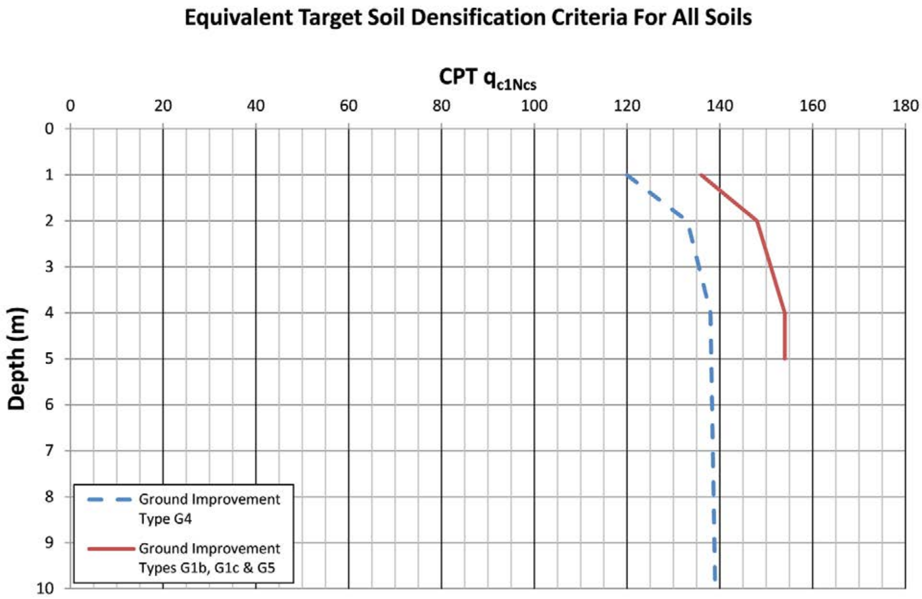
If neither of the above are applicable, re-working or intensifying the ground improvement will be required.

In considering the above, for methods G1b, G1c, G4 and G5, silty soils within the improvement zone may be exempted from these target strength criteria if it can be demonstrated using other accepted assessment techniques that they are not susceptible to liquefaction. For the purpose of this document, silty soils may be exempted if they possess a plasticity index greater than 12, or CPT Soil Behaviour Type Index (I_c) value greater than 2.6.

For methods G3 and G4 (deep foundation treatments), if relatively thin layers within the soil profile do not meet the criteria specified below, site-specific engineering analysis may be undertaken to assess whether possible liquefaction of these layers can be accepted without a significant reduction in expected performance of the foundation system.

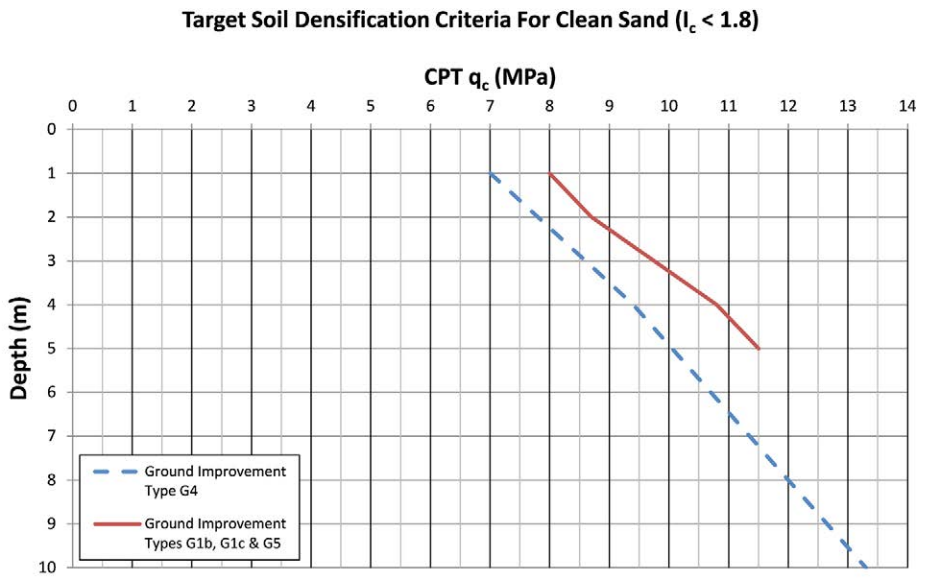
It should also be noted that post-treatment cone friction (fs) values may be influenced (ie increased) by horizontal stresses imparted by the treatment works, thus giving misleadingly low IC values and an unrealistic decrease in apparent fines content. For the purposes of this document it is therefore acceptable to use pre-improvement I_c values. (For further information refer to Nguyen, T., Shao, L., Gingery, J., and Robertson, P. (2014). 'Proposed modification to CPT-based liquefaction method for post-vibratory ground improvement.' Geo-Congress 2014).

Figure C4.2: Equivalent target soil densification criteria for all soils



Following are the CPT tip resistance profiles to be used to confirm whether the minimum level of ground improvement has been achieved for methods G1b, G1c, G4 and G5.

Figure C4.3: Target soil densification criteria for clean sand



Ground Improvement Types G1b, G1c & G5 Target Soil Densification Criteria		
Depth (m)	Target For Clean Sand (I_c < 1.8) CPT q_c (MPa)	Equivalent CPT q_{c1Ncs} Target For All Soils
1	8.0	136
2	8.7	148
4	10.8	154
5	11.5	154

Ground Improvement Type G4: Deep stone columns Target Soil Densification Criteria		
Depth (m)	Target For Clean Sand (I_c < 1.8) CPT q_c (MPa)	Equivalent CPT q_{c1Ncs} Target For All Soils
1	7.0	120
2	7.8	133
4	9.4	138
10	13.3	139