Guidance on Detailed Engineering Evaluation of Non-residential buildings

Part 3
Technical Guidance

Section 5 - Foundations

Draft Prepared by the Engineering Advisory Group
Read in Conjunction with Sections 1 to 3 (separate document)

Revision 3, 16 May, 2012

This draft document is NOT for distribution.
The contents do not represent government policy

Document Status

It is intended that this document will provide guidance to structural and geotechnical engineers and to Territorial Authorities in the assessment of earthquake-damaged buildings. The purpose of the assessment is primarily to assist in determining whether buildings should be occupied, noting that absolute safety can never be achieved.

Ideally, a document such as this should have been in existence prior to the Canterbury Earthquakes, as it is needed almost immediately. Consequently, this document has been prepared with considerable urgency, acknowledging that comprehensiveness and depth may be compromised as a result. This document is likely to require significant further revision in order to be applied more broadly than the Canterbury earthquake recovery.

This document is part of a series of documents, as follows:

Part 1	Background
Part 2	Evaluation Procedure
Part 3	Technical Guidance

The sequence of release of the documents is deliberately out of numerical order, recognising the need for engineers to begin the detailed evaluations as soon as possible.

Where errors are omissions are noted in the document, it is requested that users notify the Engineering Advisory Group through John Hare at johnh@holmesgroup.com.

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5 FOUNDATIONS

5.1 INTRODUCTION

The performance of foundations has been a significant learning from the earthquakes, with the impacts of liquefaction and lateral spread often outweighing the shaking damage.

5.2 NOTATION

(not yet used)

5.3 DESCRIPTION

The following foundation systems have been used in the Christchurch area. This list may not be comprehensive, but covers the majority of systems in common use over a period of many years.

5.3.1 Shallow Foundations

Foundation elements are considered to be shallow when the depth to breadth ratio is less than 5 (D/B <5), generally including the following:

- Isolated pads
  Isolated pads are seldom appropriate for building foundations subject to seismic actions, especially in Christchurch where the ground conditions are known to be variable. These could well have suffered from differential settlement and differential lateral movement, especially in areas of liquefaction.

- Strip/beam footings
  Continuity of foundation elements is important to ensure integrity of a structure subject to differential ground movements. Where differential movements are excessive, the footings should be checked for structural damage.

- Pad and Tie Beam foundations
  Similar to above. Most earlier shallow foundations comprise systems of pads and tie beams, as required by the Code of the day, or by convention at the time. This practice reduced from the 80's, although some engineers may have used a reinforced slab on grade to achieve the tie effect.

  Tie beams at the perimeter of a building may have been used in order to resist out-of balance actions due to eccentric perimeter foundations where buildings are built to the boundary, and do not necessarily indicate a full tie beam system.

- Mat foundations
  Mat foundations are continuous structural slabs spanning between columns and walls etc. Their resistance to differential ground movements will vary according to their strength and stiffness. The level of damage will also depend on the extent of differential movements both vertical and lateral.

- Raft foundations
  Raft foundations are similar to mat foundations but have sufficient strength and stiffness to behave essentially as a rigid body when accommodating differential ground movements. True rafts are rare as the required levels of strength and stiffness are prohibitive.

5.3.2 Deep Foundations

Foundation elements are considered to be deep when the depth to breadth ratio is greater than 5 (D/B >5). Generally, in Christchurch, the following deep foundation types are in use:

- Driven concrete piles
  Typically these are 6 m to 15 m long, with some as short as 2 – 3m and rare buildings with piles in excess of 20m, and are driven to found onto a dense sand or gravel stratum. Few buildings in Christchurch have been founded on driven piles larger than 150mm square section in the last 15 years due to resource consent issues to do with noise and vibration during driving. They are typically designed as end bearing, although a contribution from side friction may be included. Both compression and uplift capacity from side resistance may be lost with liquefaction. Lateral capacity may also be affected if adequate embedment has not been achieved into the dense soils.

- Driven steel pile
  Not widely used in Christchurch but may be driven to found onto the more dense gravel strata at depth. Uplift capacity from friction may be lost with liquefaction unless adequate embedment has been achieved into a dense (non liquefiable) soil.

- Driven timber piles
  Typically these tend to be shallower than other pile types and may be vulnerable to both bearing and lateral capacity strength loss within or underneath the bearing stratum. Not common for commercial buildings

- Bored cast-in-place piles
  Usually 6 – 15m deep and 0.6 to 1.2m diameter, occasionally up to 1.5m diameter and up to 20m deep. Typically excavated in water filled steel casing which is withdrawn during concreting. Although often designed as end bearing with some contribution from side resistance, in reality, for many of them, the gravity loads will have been carried since construction by the side resistance mechanism. Loss of side resistance from pore water pressure effects during shaking may lead to settlement from gravity loads, (see discussion below).

  Uplift in bored piles in Christchurch is resisted by side resistance. There is no knowledge of belling or under-reaming of any piles in Christchurch, where the cohesionless sands and gravels below the water table do not allow undercutting or even any excavation outside a fully cased hole without bentonite slurry support.

- Bulb (Franki) piles
  Common on many buildings between about 1970 and late 1980s. Steel casings were bottom driven to depth, a cement-gravel plug driven out to form the bulb, and then casing withdrawn as shaft concreted. Typically 450mm – 600mm diameter shafts on

nominal 1m diameter bulbs and less than 10 – 12m depth. The bulbs are below the reinforcing cage and thus there is no reliable uplift capacity except on the shaft unless there is a second bulb driven out through the reinforcing cage above the compression bulb. Piles may have limited fixity at the base affecting lateral capacity.

- Screw piles
  Typically these are 10 m to 20 m long and are screwed into a dense stratum. Capacity comes from end bearing onto the screw flanges. Uplift capacity comes from "upside down" bearing which may fail if the overlying materials liquefy. There is minimal side resistance along the stem.

- Continuous Flight Auger (CFA) piles
  This is a relatively new technology in Canterbury so is included here for completeness only, as there are not known to be many in use yet. CFA piles are essentially bored piles installed without casing, so most of the notes relating to bored piles will apply. The maximum length and diameter is limited by available equipment but is in the order of 600mm diameter and 15m length. Using specially adapted equipment, an auger is screwed into the ground and then withdrawn as concrete is pumped down the centre of the flight under pressure, displacing the soil. Once withdrawn, a reinforcing cage is placed into the concrete. This technique is relatively quick, but is technically challenging and requires good QA procedures and experienced operators.

5.4 SEISMIC RESPONSE CHARACTERISTICS AND COMMON DEFICIENCIES

This section is intended to provide an overview of the behaviour of the different foundation types, but it is noted that specialist geotechnical advice should be sought in cases where excessive movement or damage has occurred. Guidance on when to seek such advice is given in Table 5-1 below.

Table 5-1: Soil and Foundation Damage Assessment Criteria (from Part 2)

	Level of geotechnical assessment		
Parameter	Desktop study	Geotechnical investigation (2)	Geotechnical investigation with intrusive foundation investigation (3)
Geotechnical engineering	Geotechnical engineering input to be considered	Involvement of appropriately qualified and experienced geotechnical engineer is essential	
Settlement (mm)	50	100	200
Differential Settlements	1:250	1:150	1:100
Liquefaction ($m^3/100m^2$)	2	5	10
Lateral Spreading total (mm)	50	250	500

	Level of geotechnical assessment		
Parameter	Desktop study	Geotechnical investigation (2)	Geotechnical investigation with intrusive foundation investigation (3)
Lateral stretch	1:400	1:100	1:50
Cracks (mm/20m)	20	100	200
Damage to superstructure	Cosmetic	Minor to Significant Structural	Severe to major structural
Damage in Area (Major remedial works)	Slight	Moderate to substantial (1 site in 5)	Widespread to major (1 site in 3, to most)

Note: (1) If any one parameter exceeds the limits set out in a column, then the scale of investigation is to be increased to the next level.

(2) New investigation required if existing good borehole or CPT data is not available. Consider limited exposure of most critical ground foundation elements.

(3) Recommend full exposure of typical foundation elements and/or further intrusive investigations of foundations as appropriate. This will typically require excavation alongside shallow foundations and/or pile caps, to expose pile/cap connection. Could consider drilling pile from above or other test methds if there are concerns regarding the remaining pile integrity.

5.4.1 Shallow Foundations

A key generic issue relevant to all types of shallow foundations is to decide whether or not shallow foundations remain appropriate for the structure or whether underpinning with deep foundations is required. This decision should not be based solely on the performance of the foundation to date, but on the risks of damaging settlement from future events, based on proper analysis of the ground conditions. While differential settlements as measured post February 2011 may be within tolerable limits for the structure, another earthquake could produce similar or greater differential movement, cumulative to the first, which could then lead to severe structural damage or failure.

Settled footings may be the result of liquefaction or soil response at depth, or simply have been overloaded by the earthquake induced axial loads. The Building Code VM4 document permits use of a generic geotechnical strength reduction factor of $\phi_g = 0.8 – 0.9$ for load combinations including earthquake ”overstrength”, which is much higher than factors typically used for other load combinations, resulting in a high risk that the ultimate capacity of the footing will be exceeded at the design load. In reality, the bearing capacity of shallow foundations is reduced by inertial effects during shaking as well as from increased pore water stresses, which in combination with high seismic loading from the structure can induce large deformation.

It is considered that generally, limited foundation movement may be acceptable, noting that this will rarely represent a life safety hazard, unless there are subsequent deformation compatibility issues in the superstructure. However it has been noted that in many cases, excessive foundation movement was observed without yielding of the primary structure. In effect, for ductile systems, this meant that the entire building system ductility was 'short-circuited' by the foundation failure. While this may have limited the extent of damage in the structure, in many cases the outcome has been less repairable, and there must be concerns that this may result in deformation induced failures in the gravity system. Conversely, if the foundation systems are stiffened, greater seismic actions will be transmitted to the superstructure.

These concerns are more applicable to vertical support than lateral support. Excessive differential settlements may have severe consequence for a building, but there is relatively little concern with limited base sliding, which has not been observed, so far, to have caused issues. It is considered that a strength reduction fact of ϕ=1 may be applicable to sliding resistance for this reason.

Some foundations have suffered from non-uniform aspects such as basements under only parts of the building, irregular footprints with changes in plan dimension with height (resulting in differential gravity loading, such as with a podium), or piles and tie-downs installed to provide tension capacity under parts of a shallow foundation only. Particular attention should be given to the areas around such features in looking for damage, differential movement etc. A number of buildings have suffered differential movement due to uplift of basements under part of the ground floor.

Basements can be exposed to high uplift pressures generated in liquefied sands or in loose gravels. This can result in vertical displacement as well as damage to the basement floor, depending on the construction as a raft or slab between footings or piles. Uplifted basements, particularly those on gravels rather than liquefied sands, may have large voids below them. Basement walls may have been subjected to lateral earth pressures much higher than normal static loading. Many basements were partially flooded after the earthquake, as the result of damage to walls, floor or tanking.

5.4.2 Deep Foundations

There are several key generic issues for deep foundations that need to be considered:

- Loss of side resistance (skin friction) in piles may occur from pore water pressure increase during shaking, even if full liquefaction does not trigger. Where full liquefaction is triggered at depth, all side resistance above may be effectively lost or reversed because of settlement of the overlying strata. In such cases so called "negative skin friction" may contribute to pile settlement.

- Bored cast-in-place piles are perhaps the most susceptible to settlement caused by pore water pressure rise and liquefaction above the base of the pile because the gravity loads are carried initially almost entirely by side resistance. If this mechanism is overloaded, the pile will settle until the end bearing mechanism is mobilised (which could be as much as 5 – 10 percent of the pile diameter). This can potentially be exacerbated if poor construction has left a zone of disturbed material at the base of the piles.

- Cyclic axial loading during the earthquake may cause loss of capacity and settlement, especially for piles that carry only light gravity loads and rely mainly on side resistance.

- Settled piles may simply have been overloaded by the earthquake induced axial loads, possibly including high vertical accelerations. The Building Code VM4 document permits use of a generic geotechnical strength reduction factor of $\phi_g = 0.8 – 0.9$ for load combinations including earthquake "overstrength" loads, which is much higher than factors typically used for other load combinations, resulting in a high risk that the pile capacity will be exceeded at the design load . Strength reduction factors for pile design, including earthquake load cases, should be selected based on a proper risk assessment procedure such as that given in AS2159-2009.

  It is considered that generally, limited foundation movement may be acceptable, noting that this will rarely represent a life safety hazard, unless there are subsequent deformation compatibility issues in the superstructure. However it has been noted that in many cases, excessive foundation movement was observed without yielding of the primary structure. In effect, for ductile systems, this meant that the entire building system ductility was 'short-circuited' by the foundation failure. While this may have limited the extent of damage in the structure, in many cases the outcome has been less repairable, and there must be concerns that this may result in deformation induced failures in the gravity system. Conversely, if the foundation systems are stiffened, greater seismic actions will be transmitted to the superstructure.

  These concerns are more applicable to vertical support than lateral support. Excessive differential settlements may have severe consequence for a building, but there is relatively little concern with limited base sliding, which has not been observed, so far, to have caused issues. It is considered that a strength reduction factor of ϕ=1 may be applicable to sliding resistance for this reason, noting that attention should however be paid to the likelihood of settlement of soils below the pile caps and foundation beams, which may reduce the amount of lateral resistance available. Although shear or flexural failure of piles has not been observed, consideration should be given to the impact of settlement on the lateral capacity of piled systems.

- Pile settlement may also be from liquefaction of sand layers below the founding layer. Many parts of Christchurch have dense gravel or sand layers that may be several metres thick but underlain with much looser sands. Deeper liquefaction may not have been considered in the pile design, particularly of older buildings.

- Similarly to piles that have settled excessively, the same concerns apply to piles or anchors that have been used in tension mode, to resist overturning actions. Premature failure of such systems may have occurred due to overestimation of the capacity of the system, which may have reduced with seismic loading. Use of the lower strength reduction factors in accordance with the above would be equally applicable in such cases. Another concern with tensions piles is that they will also have an appreciable compression capacity. Differential settlement may result where tension piles have provided supplemental support to shallow foundations at one end of a mixed system, or where different tension requirements (for opposite directions of loading, or oblique angles of attack) have resulted in different numbers of piles at opposing ends.

- The buckling of slender piles in liquefied soil has not been observed, but should be considered. However, it is thought that the interbedded nature of most Christchurch subsoils generally means that there are enough lenses of non-liquefiable soils to provide restraint to slender piles.

Damage to foundations may not always be evident from the surface, particularly where a large area has been subject to lateral displacements. Where there is evidence of relative motion between the structure and the ground, pile heads and the connection to the structure should be checked for overload in shear. Shear transfer from the ground to the building is typically assumed to be carried by friction underneath the building and by passive resistance of the soil against buried foundation beams and walls etc. The friction mechanism will typically fail quickly with any settlement of the ground and the passive mechanism degrades rapidly with development of gapping. For this reason, and because the earthquake shaking was stronger than design levels, it is likely that the piles may have carried far more shear than the designer ever intended.

Kinematic interactions between the ground and the piles need to be carefully considered. Ground deformations are known to have been significant around many parts of Christchurch, including both dynamic and permanent deformations. These ground deformations may impose significant strains within piles resulting in pile damage and permanent deformation well below the ground surface. Physical investigation of such damage is difficult and expensive and may be impractical. Analytical procedures are available as a first step to try and estimate the pile strain levels and therefore likelihood of damage. Guidance for selecting the appropriate level of investigation is given in Table 5-1.

5.5 ASSESSMENT AND ANALYSIS

Assessment and analysis must consider both the geotechnical and structural engineering aspects of the foundation system and the ability of the foundations to support the building through both gravity and seismic actions in future events.

Key questions to be addressed may be as follows:

- If there has been damage from liquefaction or lateral spread, what will the future performance in SLS level events be? Although this may be difficult to quantify, it is important to consider this in respect of the need to implement either site remediation or foundation upgrade, whether or not there is considered to be a life safety hazard from the movement.

- What will the future performance in ULS events be, given observations of the performance to date? This question should take into account the impact of further movement, if there is no change (site remediation or foundation upgrade) to the foundations apart from required repairs; and should take into account the impact of aggregated movement and/or damage.

- If site remediation or foundation upgrade is to be undertaken, what will the consequent impact on the superstructure be? Note that in some cases, where soil damage has occurred, it may have effectively limited shaking transmission to the superstructure. Therefore engineers should consider future shaking with respect to the improved site.

Note that all of the above are initially geotechnical engineering matters and the need to address these at a detailed level will be informed by Table 5-1 above. Where the assessment criteria are not triggered, it may reasonably be inferred that founding conditions are adequate. Care should be taken where the site is reasonably remote from the earthquake epicentres and is known to have soft subsoils. It is recommended at the very least that reviewers verify the soil type to an adequate depth below the building and check the proximity to any known existing or former watercourse. Guidance on what might be an adequate depth is given below. If in doubt, consult a geotechnical engineer.

Table 5-2: Recommended depth of geotechnical investigations

Soil Profile to be identified to a depth below existing foundations of at least the greater of:	
Shallow foundations	Deep foundations
10m, or twice the footing width	10m, or 10 times the pile width below the pile tip

In general, from a structural engineering perspective, the assessment and analysis of foundation systems follows the same principles as may be applied to the remaining structure. Factors to consider from a structural engineering perspective include:

- How is the lateral load taken out (transmitted to, in reality) the superstructure? If the existing system is insufficient to transmit the load, what will the impact of the implied movement be? Note that this is not necessarily a significant or negative factor as movement in the foundations may reduce demand on the superstructure, provided that brittle foundation systems (such as slender non-ductile piles) are able to withstand the displacement. In cases where there is concern over this, the affected area of the foundation should be exposed in order to verify their condition and/or need for repair.

- What is the impact of super-structure behaviour on the foundations? Structural systems that impose significant actions on the foundations may have caused damage that is not visible, but needs repair. This should be evaluated as above.

- If there is likely to be further movement in future events, what is the aggregate impact on the superstructure of the cumulative movement?

5.6 REPAIR AND STRENGTHENING STRATEGIES

Repair techniques for foundation issues may vary considerably according to situation. For the structural repair, conventional techniques will be appropriate in many cases. However it is important that the reasons for the structural damage have been identified and the solution integrated with any geotechnical repair that may be required. The following sections contain some generic guidance on repairs that may be feasible. For further specific guidance, refer to the appendices.

Generally, in cases where there has been damage due to foundation failure, it will be necessary to work closely with the geotechnical engineer to develop a specific solution.

Typical methods include the following:

5.6.1 Geotechnical Repairs

Where gapping has occurred adjacent to footings, the gaps should be filled with sand-bentonite grout to restore the full passive resistance of the soil.

Where rocking of foundations has occurred (or suspected to have occurred) gaps may exist underneath foundation elements or under the edges of elements. Locate and fill such gaps.

Shallow foundations that have settled might be re-levelled or at least secured against further movement in future earthquakes by low mobility compaction grouting or underpinning with piles.

Deep foundation systems (piles) which have settled are much more difficult to re-level. In some cases it may be possible to grout below the pile tips and lift the entire pile and building. In other instances it may be necessary to cut off the existing piles and re-level from new piles.

5.6.2 Ground Improvement

Mitigation of liquefaction potential may be achieved by strengthening of the ground and/or limiting the potential for excess pore pressures to develop. A number of methods are available to achieve this including:

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), rapid impact compaction (RIC)

Crust Strengthening/ stabilisation

- permeation grouting
- stabilisation mixing or replacement

Deep Strengthening

- deep soil-cement mix piles, jet grouting, low mobility grouting
- stone columns

- close spaced timber or precast piles

Containment

- reinforcement
- curtain walls

Drainage

- stone columns
- earthquake drains

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

Recent trials undertaken by DBH using several of these methods are described by Bowen, Millar and Traylen (2012). The crust strengthening/ stabilisation method is expected to provide the most cost effective method for a site with clear for light weight single level buildings. There may be limitations for buildings extending close to the boundaries unless work is undertaken cooperatively with adjacent properties. Larger structures will probably require deep ground improvement. Any ground improvement that that does not extend to below the base of the liquefiable layers at a site, and that therefore is underlain with untreated liquefiable soils is potentially at some risk of settlement in future earthquakes.

The containment method, low mobility grouting and permeation grouting options provide opportunities to achieve improved ground performance without removal of the existing structure but further testing may be required to assess the performance of the former method.

There are relatively few sites where the methods have been applied in Christchurch, and these should generally be well known to geotechnical engineers. Reviewing engineers must seek specialist geotechnical advice in assessing the performance and suitability for ongoing use of such sites.

5.6.3 Structural Repairs

Table 5-3: Repair and Strengthening methods

Repair type	Advantages	Disadvantages	SLS performance	ULS Performance
Epoxy injection	Simple to implement, relatively unintrusive May be carried out with limited occupation, provided screening for fumes is possible	Epoxy fumes may be HASE hazard Need to access all sides of structure to get full coverage Limited penetration for narrow cracks and/or wide elements	Will restore some of initial (pre-cracked) stiffness, but not all.	Provided that there are no significant issues with strain hardening, will restore most of initial strength

Repair type	Advantages	Disadvantages	SLS performance	ULS Performance
Partial break-out and rebuild	Restores more of strength in damaged areas that epoxy; less intrusive than full demolition and rebuild. May compromise gravity capacity in the short-term, requiring temporary shoring.	Still has significant noise and vibration, may not be possible to even partially occupy, or must be done out of hours.	Will restore close to full stiffness, therefore SLS performance as good as prior to earthquakes	Provided that damaged reinforcement is replaced, will restore to capacity that existed prior to earthquakes
Full break-out and rebuild	Allows strength to be added with no increase in dimensions	Highly intrusive	Restores SLS condition to at least that which existed pre-earthquake, subject to consideration of soil capacity	Allows improvement to ULS performance to whatever level is required, subject to consideration of soil capacity
External reinforcing of foundation with reinforced concrete skin	No demolition of existing required, therefore minimises propping and shoring requirements	Can be difficult to achieve required bond of old to new. May change relative stiffness of elements, changing load distributions. May require epoxy injection first	Restores SLS condition to at least that which existed pre-earthquake, subject to consideration of soil capacity	Allows improvement to ULS performance to whatever level is required, subject to consideration of soil capacity

5.7 REFERENCES

Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury Part 2 Evaluation Procedure, Draft prepared by the Engineering Advisory Group, Rev 5, 19 July 2011.

New Zealand Society for Earthquake Engineering, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*, June 2006.

Bowen, Millar & Traylen. *Full Scale Testing of Ground Remediation Options for Residential Repair Following the Canterbury Earthquakes*, Proceedings of the NZSEE Conference, April 2012.

Revised Guidance on Repairing and Rebuilding Houses affected by the Canterbury Earthquake Sequence, Department of Building and Housing, November 2011.

Interim Guidance for Repairing and Rebuilding Foundations in Technical Category 3. Department of Building and Housing, 27 April 2012.

Appendix 5A – Light Industrial/Commercial Buildings

5A.1 INTRODUCTION

There are many light industrial/commercial buildings in Christchurch which have suffered varying degrees of damage from the recent earthquakes. Typically these are long span portal or truss roofs over large footprint concrete floors with light weight cladding, block or concrete tilt panel walls, and often incorporating small 2 storey or mezzanine office spaces. Such buildings in the west of the city from north of the airport through to Hornby and Sockburn, have generally not suffered significant foundation damage, but other commercial / industrial zones in the city including Sydenham, Waltham, Linwood, Woolston, Ferrymead and Bromley include areas with significant liquefaction damage.

The issue arises as to how to treat this type of building with such ground damage. The underlying soil profiles vary, but often there are liquefiable sands to depths of 10-15m and in some parts of Linwood and Bromley it can be as deep as 20-25m. On some of these sites the existing damaged buildings have been demolished because of the liquefaction damage.

Readers are also referred to the DBH *Interim Guidance for Repairing and Rebuilding Foundations in Technical Category 3*. Although this document was prepared primarily for the repair and rebuilding of residential buildings, there may often be sufficient similarity to light industrial or commercial buildings that the same approaches may be used are adapted, with engineering judgement.

5A.2 DESIGN CRITERIA

All building work must comply with the Building Code regardless of whether a building consent is required. This includes work for, or in connection with, the construction, alteration, repair, or demolition or removal of a building. Building Code clause B1 Structure requires new building work to have a low probability of rupture, becoming unstable or collapsing (Clause B1.3.1). AS/NZS 1170 is referenced in Verification Method B1/VM1, which if followed, is treated as complying with building code clause B1. Buildings which are designed using AS/NZS1170 are required to satisfy the following primary design cases:

1. The Serviceability Limit State (SLS) design case, and,
2. The Ultimate Limit State (ULS) design case.

The Ultimate Limit State case is not commented on further here, except that foundation behaviour at this level of seismic loading and with liquefaction must not precipitate building collapse. The Serviceability Limit State is more problematic with the currently increased seismic hazard for Christchurch for liquefiable sites as liquefaction is predicted for many sites at a SLS level of shaking

Serviceability Limit State

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. SLS seismic loads for importance level 2 properties are based on a one in 25 year earthquake. AS/NZS 1170.0 defines serviceability limit states as: *states that correspond to conditions beyond which specified service criteria* *for a structure or structural element are no longer met. (Note: The criteria are based on the intended use and may include limits on deformation, vibratory response, degradation or other physical aspects.)* (Clause 1.4.9)

A DBH elaboration of this states that 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 (refer Table 5A.1 below), possibly including minor cracking, deflection and settlement that do not affect the structural, fire, weather-tightness performance or use of the building.

DBH continues by noting that Building Code clause B1 also requires new building work to have a low probability of causing loss of amenity through undue deformation, vibratory response, degradation or other physical characteristics throughout its life (clause B1.3.2).

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

Current Acceptable Solutions, Verification Methods and Standards do not provide an explanation of what is meant by "loss of amenity". However, loss of amenity might include loss of services including sewer and water connections, damage to sanitary fixtures, parts of the building being no longer available, significant cracking and deformation of flooring, or the building envelope not being weather tight. Measures should be taken when designing and building foundations on land with the potential for liquefaction to minimise the possibility of loss of amenity should a SLS earthquake event occur.

For the DBH guidelines for residential buildings, loss of amenity is taken as the exceedance of the following tolerable impact:

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

Table 5A.1 provides criteria for the nature of damage that corresponds to "repairability".

The regulations do not quantitatively define any SLS limits, but Appendix B (Informative) of B1/VM4 states that "foundation design should limit the probable maximum differential settlement over a horizontal distance of 6m to no more than 25mm under serviceability limit state load combinations of AS/NZS 1170 Part 0, unless the structure is specifically designed to prevent damage under a greater settlement."

It is clear then that the deflections and foundation deformation which is acceptable at an SLS event may well be different for different structures depending on its use and construction, provided that the there is no loss of function of the structure or its parts, or loss of amenity.

With the increased Z and R factors there are few sites on liquefiable ground in Christchurch which will not undergo some liquefaction at an SLS level of shaking. For residential buildings a consensus is that differential settlements across the building should be no more than 50mm and hence the building remains functional and without damage that would impact on the amenity of the structure. However, for some light industrial buildings these criteria may be unnecessarily severe and a greater tolerance may be possible to accept for some buildings, provided that the structure is remains functional. On the other hand, there are also buildings where even small settlements may impact on the functioning of the housed activity (egg forklift access for high racking, machine alignment etc).

The interpretation of SLS criteria needs to be looked at carefully for the particular use of each structure. On liquefiable sites, this will also require geotechnical testing of the site to ascertain likely ground behaviour and deformation in a SLS event.

However, while some buildings are purpose built for a particular use, most light commercial buildings are for general use and may be expected to have several changes of use during their design life. Tolerable deformation for an initial use may be unacceptable for a later use. To provide some level of protection for later owners, but allow flexibility in design the design criteria must be explicitly and prominently stated in consent documentation if the design allows greater than normal deformation.

The following are provided as guidelines for acceptable performance for light commercial buildings with general types of occupancy at SLS.

Tolerable Impact Limits

1. No injuries or loss of life. Demonstrably not dangerous in terms of definitions within BA121/CERA7, or readily able to be made so.

2. Building is accessible, functional, and safe to occupy and work in.

3. The structure shall remain functional so that the building can continue to perform its intended purpose without excessive difficulty, cost, or loss of economy.

4. Some loss of amenity, e.g. gradients on floors, but must still be able to function in terms of process or operations, e.g. materials handling.

5. Minor damage to structure, but should be repairable.

6. Criteria for reparability should be as per Table 5A.1: SLS Performance Expectations for Light Industrial Buildings.

Table 5A.1: SLS Performance Expectations for Light Industrial Buildings

Key Terms	Element	Interpretation	Comment
1. Performance attributes 1.1 Safety 1.2 Serviceability 1.3 Functionality 1.4 Other Aspects, egg repairability, reinstatement	Building	Ability to be functional in terms of general commercial use. If design allows greater than normal deformation as being tolerable for a specified intended use, this information must be explicitly and prominently stated in consent documentation such that future purchasers can be informed.	Use may be function-driven and user-specified. Default standards may apply.

Key Terms	Element	Interpretation	Comment
2. Damage to structure (minor and repairable)	Foundation structure and floor surfaces	(a) No rupture, but minor curvature acceptable. (b) Defined thresholds in terms of absolute levels (50 mm)(1) and gradient (L/240). (c) Adequate strength maintained in the interim. (d) Able to be re-levelled simply to defined thresholds.	Some materials handling functions in industrial buildings will be more tolerant, e.g. normal pneumatic forklift operations. Structure gradient and level criteria will apply for specialist materials handling function, e.g. automated forklift access to pallet racking. NB: Gradient thresholds for different applications to be developed.
	Walls - exterior	Able to resist impact or distortion. Minor cracking to precast concrete panels at joints and in applied coatings.	Remains essentially watertight. Lateral structural integrity maintained.
	Walls - interior	Minor cracking at lining joints.	Lateral structural integrity maintained.
3. Damage to building fabric and lining (minor)	Cladding external joinery	Some cracking to cladding panels due to in-plane distortion. Some cracking of linings above openings, e.g. doors, windows.	-
	Roof	Roof claddings sound, intact, and securely attached.	Capable of remaining weather-tight.
4. Repairability	All elements	Repairable without relocation for more than six weeks or loss of function (25%) for more than three months.	Total cost of repairs that is able to be covered by normal insurance.
5. Building remains safe to occupy, accessible, and functional	Access doors (external)	Capable of daily operation and being secured (may need special maintenance - i.e. runner/catch adjustment, easing. Safe egress in emergency situation.	Requires ability to operate and maintain in this state for several months.
	Doors, windows	Minor jamming - i.e. may need to ease.	
6. Other aspects	Services	No significant damage to water, gas, and electrical service connections that cannot be rectified by normal maintenance. Readily repairable damage to waste and stormwater pipes.	Design of utility connectors to include provision for movement. Any loss of service will be due to malfunction of network utility system. Remedy will include use of temporary services, e.g. chemical toilets.
	Health and Safety	Owner needs to be able to demonstrate healthy and safe environment for workers. Employee health and safety must not be compromised.	
	Damp-Proofing	Floor, cladding, fabric	Requirement to prevent ground moisture from entering internal spaces.

Notes: (1) The 50mm limitation may be ignored in larger buildings where functionality is not compromised, provided that the gradient is not exceeded.

5A.3 REPAIR

Foundation repair of this type of structure, where ground damage has occurred, will generally be confined to re-levelling or underpinning of existing shallow foundations. For floors, if badly damaged, it may be possible to re-level them with injection grouting under the slab or else they may have to be removed and re-laid. Options to mitigate future liquefaction hazard is limited in such cases, and hence in many instances, repair will not be acceptable if the SLS criteria are to be met. Construction of a compacted gravel base under a replacement floor slab could go some way in meeting SLS criteria on some sites. Other methods which should allow SLS compliance include compaction grouting or piling to underpin the walls and to support the floor.

5A.4 REPLACEMENT

On many sites where liquefaction has occurred, conventional shallow foundations are no longer appropriate to meet SLS criteria given the degree of liquefaction hazard. The issue is complicated by buildings extending to the property boundaries, the very large floor areas and, on many sites, the large depth of liquefiable soils. The reality is that it may no longer be economic to construct light commercial buildings on these sites which comply with the Building Code.

Foundation options include:

Piling: Piling allows support to the building even when close to a boundary, however there can be issues of pile integrity if a long pile length required to penetrate to below liquefiable soils, both in terms of potential buckling and the degree of lateral deformation that the piles would be subjected to. The large floor areas for most light commercial buildings raise the questions of what to do with these, as it is frequently uneconomic to pile the entire floor.

Ground Improvement. Methods range from surface treatments such as reinforced gravel raft to deep methods such as soil mixing, stone columns etc. One of the issues with ground improvement is that it is usually required to extend beyond the building footprint as there are issues in terms of the edge performance of any improved block of ground. Ground improvement is therefore problematic for sites where the structure is to be built to the boundary, and the lot size is such that bringing the walls in from the boundaries is impracticable. This could be overcome if adjacent property owners co-operate and several properties are treated together. In some locations it is conceivable that a change of use may be necessary to allow for the greater expenditure of ground improvement or piling, and to allow space around the building.

Raft foundations

Raft foundations may be a viable option on sites with limited liquefaction potential at a SLS level earthquake. Rafts can be effective in limiting differential settlement and internal deformation, but are still subject to global settlement and potential tilting. As well as simple reinforced concrete raft foundation, mitigation may be enhanced with a reinforced gravel raft, or ground treatment to a limited depth below the building. Edge effects may make these solutions difficult to apply on sites with buildings constructed to the boundary.