Section 4:
Soils and foundations

## 4.1 Introduction

Under the Terms of Reference for the Inquiry into the representative sample of buildings the Royal Commission must inquire into and report on the nature of the land associated with the buildings considered. To fulfil this obligation we considered that we should develop a general understanding of the subsurface conditions in the Christchurch central business district (CBD), as well as considering the impacts of the soils on the performance of buildings on particular sites.

## 4.2 Expert advice

We commissioned the following reports:

1. “Foundations on Deep Alluvial Soils” by Associate Professor Misko Cubrinovski of the University of Canterbury and Ian McCahon1, a principal of Geotech Consulting Ltd, Christchurch, dated August 2011. This report gives a general overview of the alluvial soils underlying the Christchurch CBD and discusses liquefaction, lateral spreading and the consequential impact of the Canterbury earthquakes on foundations. The report also provides some general concepts that should be followed in designing foundations for buildings on deep alluvial soils.

2. A peer review of the Cubrinovski and McCahon report by Professor Jonathan Bray2 of the University of California at Berkeley, dated October 2011.

3. “Foundation Design Reliability Issues” by Dr Kevin McManus3, a civil engineer. This report provides a largely technical discussion on the practice of foundation design in New Zealand.

The Royal Commission conducted a public hearing on these issues on 25 October 2011. The expert advisers gave evidence about the issues addressed in their reports to the Royal Commission. Nine submissions were also received from interested parties, the content of which has been considered and analysed by the Royal Commission and addressed where appropriate in this Report. Those who made submissions are listed in Appendix 3 of this Volume.

In addition, a report entitled “Geotechnical Investigations and Assessment of Christchurch Central Business District” by Tonkin and Taylor Ltd4, commissioned by the Christchurch City Council (CCC), was provided to the Royal Commission.

## 4.3 Canterbury soils

Christchurch is located on deep alluvial soils of the Canterbury plains, except for its southern edge, which is on the slopes of the Port Hills, on the remains of the Lyttelton volcano.

As discussed in section 2 of this Volume, Canterbury is situated on land that is being deformed by the oblique collision between the Australian and Pacific tectonic plates. The rate of deformation decreases with distance from the Alpine Fault to the east and from major faults in North Canterbury that branch off the Alpine Fault.

The Canterbury region is underpinned by complex inter-layered soil formations to a depth of 500 metres or more, deposited by eastward-flowing rivers from the Southern Alps into Pegasus Bay and Canterbury Bight on the Pacific coast. The Canterbury plains consist of very thick soil deposits.

## 4.4 Soils in the Christchurch CBD

The soils are highly variable both horizontally and vertically over relatively short distances, with different composition and densities of soils across small distances, as shown in Figure 1 below:

Figure 1: Subsurface cross-section of Christchurch CBD along Hereford Street (source: Cubrinovski and McCahon, August 2011, reproduced and modified from Elder and McCahon, 1990)

In the top 20–25 metres, the soils are geologically young and contain mixtures of sands, silts and peat, together with some swamp and marine deposits. As a consequence they are relatively weak and poorly consolidated. Beneath this layer are 300–500 metres of thick gravelly deposits, which are older strata that have greater strength. The water table in the CBD sits at a depth of 1–1.5 metres, which increases to about five metres to the west of the CBD. Aquifers exist in the top 25 metres. This combination of soil characteristics, aquifers and high water table increases the risk of liquefaction.

The soils in the CBD are fluvial deposits from both the Avon and Heathcote Rivers and the Waimakariri River, which is known to have flooded the area and significantly contributed to the shape and the ground conditions of Christchurch over a long period of time. Early maps show that in the 1850s, around the Avon River there were many streams and a number of areas of surface water. The old river channels have very loose soils in conditions that have a high potential for liquefaction. Cubrinovski and McCahon1 note that soil behaviour and liquefaction can be influenced by previous land use and the presence of rivers and streams dating from well over 150 years ago.

The results of an extensive ground investigation commissioned by the CCC and undertaken by Tonkin and Taylor4, to evaluate the nature and variability of subsurface conditions in the Christchurch CBD and adjacent commercial areas to the south and north-east, will be held in a database available to the public. The information may be used to assess the potential impact of future large earthquakes on structures and their foundations and to assist decision making regarding land-use planning by local authorities. It should also enable geotechnical specialists to prepare concept designs for foundations and ground improvement options for future development.

The investigations carried out by Tonkin and Taylor4 did not establish that there were areas in the CBD that could not be built on because of the ground conditions. However, more robust foundation design and/or ground improvement may be required4 than was previously understood to be necessary. Land within 30 metres of the Avon River is the most likely to be affected by lateral spreading.

## 4.5 Role of soils in an earthquake

Cubrinovski and McCahon1 identify two fundamental ways that seismic waves travelling through deep alluvial soils influence the performance of land, infrastructure and buildings during strong earthquakes.

High frequency seismic waves attenuate more rapidly with distance than low frequency waves. This causes the general shape of the response spectrum to change with the distance the seismic waves travel. The deep formations of sand, silt and gravel deposits below the Christchurch CBD amplify the long period vibrations in the seismic waves. The interaction of those waves with the relatively soft upper layers in the Christchurch CBD cause local variations in the vibrations at the ground surface.

Second, the soils are deformed by the seismic waves, both temporary displacements and permanent movements, and deformations (e.g., residual horizontal and vertical displacements, ground distortion, surface undulation, ground cracks and fissures). The soils are considered to have failed when ground deformation seriously affects the performance of land or structures.

In these ways, the composition of soils below the foundations can have a major influence on the behaviour of structures.

## 4.6 Soil liquefaction and lateral spreading

Soil can transform within seconds from its normal condition into a liquefied state as a result of strong ground shaking. Hydrostatic pressure on the liquefied material causes it to flow towards an area of lower pressure, which is generally upwards to the surface. The water flow brings fine particles such as sand and silt with it, and these eventually re-solidify and provide the ground with some stable structure.

The process of liquefaction and, in particular, the ejection of the excess water between the grains of sediment (pore water) results in a complete loss of shear strength, which in turn can result in heavy structures sinking into the ground and light structures floating to the surface. This often leads to localised collapse zones, sinkholes and vents.

New soils formed from sediment that has settled on top of the ground are relatively weak. Contrary to what some at first thought, Cubrinovski and McCahon1 state that repeated liquefaction of these areas can occur in further ground shaking events. This has been confirmed by Professor Bray2 in his report to the Royal Commission.

Many areas that liquefied in the 4 September 2010 earthquake have liquefied up to five times more in subsequent earthquakes. The extent and severity of liquefaction varied across the CBD (Figure 2),

Figure 2: Preliminary liquefaction map indicating areas within the CBD affected by liquefaction in the 22 February earthquake. Red = moderate to severe liquefaction; green = low to moderate liquefaction (source: Cubrinovski and McCahon, August 2011)

Lateral spreading is a particular form of land movement associated with liquefaction, which produces large lateral ground displacements that can range from tens of centimetres to several metres. It is potentially very damaging to buildings and infrastructure. It typically occurs in ground close to waterways or in backfills behind retaining walls. If the underlying soils liquefy, the soil mass tends to move down-slope. The preceding liquefaction results in the soil providing very little resistance to the ground movement. Even on a gentle slope (2–3º) the loss of strength of the soil, coupled with the cyclic motion of the earthquake, can cause a down-slope movement to occur.

The local resistance of a pile comes from friction and end-bearing. The friction forces resist the settlement of the pile. If sand and silt around the pile liquefy, the friction is lost; then, when liquefaction ceases, the sand and silt settle and drag down on the pile. Thus the direction of the friction force reverses, driving the pile down so that end-bearing increases.

In general, lateral spreading displacements within the CBD were up to 30cm, but at a few locations about 50–70cm. This was less than in the eastern suburbs, where displacements in the February 2011 earthquake were often up to two metres. Spreading displacements were sometimes seen up to 150 metres from the Avon River. Any building in those areas is likely to have been subjected to some sort of stretching of its foundations.

## 4.7 Damage to structures

Damage to piles can occur near the interface of different soil layers beneath buildings that are subject to cyclic (back and forth) movement. The movement can cause damage near the top of the pile structure and at the point where soft, deformable soils meet stiff soil layers. As the ground stretches beneath the building, large deformations may be imposed on the foundations. Substantial total settlements, differential settlements and tilting of buildings are common consequences of soil liquefaction.

All the features and modes of ground deformation discussed above are present and very pronounced in the case of lateral spreading. As the ground spreads laterally in one direction, it displaces the foundation permanently in this direction, in addition to imposing the cyclic temporary loadings. The biased loads associated with lateral spreading are particularly dangerous because they test the ductility (flexibility) of structures and their capacity to sustain large deformation without failure or collapse.

## 4.8 Impact of Alpine Fault rupture

Cubrinovski and McCahon1 noted that an earthquake of magnitude 8 from a rupture of the Alpine Fault could lead to a long period of shaking in the Christchurch CBD, largely because the Alpine Fault is 650km long. They estimate such a rupture could produce 20–22 major cycles of shaking, whereas a magnitude 6 earthquake might produce only five significant cycles. A long period of shaking provides more time for amplification in soils and contributes to the impact of the earthquake on structures.

In an Alpine Fault event, peak ground accelerations at Christchurch will be materially lower, estimated at around 0.06–0.17 times the acceleration due to gravity (g), than those experienced in the February earthquake, because of the distance they travel from the source (at its closest point the Alpine Fault is 125km from Christchurch). In comparison, for the magnitude 6.2 February earthquake the peak ground accelerations within the CBD were much higher: 0.4–0.8g.

The combination of distance and large magnitude of an Alpine Fault event would lead to a long period of shaking, which would cause soil movement to a greater depth, perhaps 15–20 metres, compared with the top 5–8 metres that moved back and forth as a body in the February earthquake. Excitement of soils at different depths has a flow-on effect on how structures respond.

Cubrinovski and McCahon1 made calculations to determine the peak ground accelerations that a magnitude 8 Alpine Fault earthquake would need to produce in the CBD in order to trigger the same level of liquefaction as the February earthquake. They concluded that it would be induced by peak ground accelerations that were less than half as strong as in the February earthquake.

In conclusion, liquefaction is likely to occur in an Alpine Fault earthquake but it is unlikely to be as great as in the February 2011 earthquake. There may, however, be cases where the soils perform poorly and liquefaction is worse owing to the long period of shaking that is likely to occur with a rupture of the Alpine Fault.

## 4.9 Canterbury earthquakes: performance of foundations

Table 1 shows a range of foundation types that have been used in the Christchurch CBD (Cubrinovski and McCahon1).

Table 1: Typical foundation types used within the CBD

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| --- | --- | --- |
| Foundation type | Building type | Foundation soils |
| Shallow foundations (isolated spread footings with tie beams) | • Multi-storey buildings• Low-rise apartment buildings | • Shallow alluvial gravel• Shallow sands, silty sands |
| Shallow foundations (raft foundations) | • Multi-storey buildings• Low-rise apartment buildings with basement | • Shallow alluvial gravel• Shallow sands, silty sands |
| Deep foundations (shallow piles) | • Low-rise apartment buildings | • Medium dense sands (soft silts and peat at shallow depths) |
| Deep foundations (deep piles) | • Multi-storey buildings | • Medium dense to dense sands (areas of deep soft soils or liquefiable sands underlain by dense sands) |
| Hybrid foundations (combined shallow and deep foundations or combined shallow and deep piles) | • Multi-storey buildings | • Highly variable foundation soils including shallow gravels and deep silty or sandy soils beneath the footprint of the building |

Liquefaction and the loss of strength of surface soils has had adverse effects on the building foundations in the CBD, including differential settlements, lateral movement of foundations, tilting of buildings and some bearing failures. Several buildings experienced serious consequences from the ground movement. We make the following observations based on the Cubrinovski and McCahon1 report and our own consideration of a representative sample of buildings discussed in Volume 2 of this Report:

4.9.1 Shallow foundations bearing into shallow, dense gravels in some parts of the CBD performed variably because these deposits themselves were so variable. Stiff raft foundations bearing onto these shallow gravels appear to have performed well (see the discussion of the Christchurch Central Police Station and the CCC civic offices).

4.9.2 Shallow foundations on sites where ground improvement was carried out before construction also performed variably. While bearing failures were prevented by the ground improvement, there were some excessive differential settlements and tilting. As a result, some buildings had to be demolished.

4.9.3 Buildings on deep pile foundations generally fared better where the piles penetrated to competent soils at sufficient depth and were not underlain by liquefaction. However, a significant number of piled buildings suffered differential settlement where the bearing layer was too thin or underlain with liquefiable layers, or where there was a loss of side-resistance with liquefaction so the load was transferred to a soft end-bearing mechanism.

4.9.4 Even in areas of severe liquefaction, pile-supported structures generally showed less differential and residual movement, provided that the piles reached competent soils at depth.

4.9.5 Multi-storey and high-rise buildings supported on shallow foundations sitting on shallow gravels showed variable performance. Thickness of gravel and underlying soil layers resulted in some differential settlements, tilting and permanent lateral displacements. These adverse effects were especially pronounced in transition zones where ground conditions changed substantially over a short distance.

4.9.6 Hybrid foundations (where part of the building was on deep piles and part on shallow foundations) performed poorly because of differential movements between the two systems (see the discussion of the Victoria Square apartments building).

4.9.7 Other significant foundation damage included the failure of ground floor and basement slabs in uplift under the very high pore-water pressures associated with soil liquefaction and ground shaking (see the discussion of the Westpac Tower building).

4.9.8 Within the CBD, zones of ground weakness (whether localised or continuous over several blocks) showed pronounced ground distortion and liquefaction that adversely affected a number of buildings. Buildings as little as 20 metres apart behaved differently according to the condition of the ground. This was seen, for example, in Armagh Street.

4.9.9 The effects of lateral spreading in the CBD were localised but quite damaging to buildings, causing sliding and stretching of the foundations and structures (see the discussion of the Town Hall).

In addition, the performance of building foundations in the CBD was adversely affected by the interactions of adjacent buildings with the underlying soils, which further deformed the soils and exacerbated damage to neighbouring buildings.

Although pile-supported structures typically suffered less damage, piles can lose support when supported in or above soils that liquefy.

## 4.10 Seismic design and construction of building foundations in CBDs of New Zealand cities

This part of the Report addresses geotechnical considerations relevant to the design and construction of new buildings in New Zealand CBDs. Although many of the issues raised are general, the discussion is of particular relevance to the rebuild of the Christchurch CBD. It is obviously important that new development there should be robust, and constructed with foundations designed to ensure resilience of the above-ground structure.

### 4.10.1 Site geotechnical model

A thorough and detailed geotechnical investigation of each building site leading to development of a full site model is a key requirement for good foundation performance. The objective of the investigation should be to gain a good understanding of the geological history of the site (including the various soil strata), the future behaviour of the site and any variability across the site. The extent of the investigations should be sufficient to give designers confidence in predicting satisfactory performance of the site and the building foundations.

An individual site cannot be considered in isolation, but only in context with surrounding sites and the geomorphology of the area. Context is especially important when considering the risk of soil liquefaction and damaging lateral ground movements during earthquakes and other geological events. The limitations of the sub-surface information and the uncertainties inherent with a model should also be recognised and alternative interpretations of the data considered.

Better access to sub-surface data from neighbouring sites would assist the understanding the site geology, stratigraphy and context. Relevant data is often limited: often there are no records of previous geotechnical investigations and foundations. This creates an unacceptable hindrance to better understanding of adjacent site conditions and evaluation of the safety of existing buildings in New Zealand.

Sub-surface investigations, especially borings, are expensive and there is always a tension between the desire of the geotechnical engineer to obtain sufficient data and the desire of the developer to minimise cost. In Christchurch, the extent of sub-surface exploration has been variable, and often less than accepted practice in other centres in New Zealand and internationally. While it is difficult and dangerous to set exact norms, a geotechnical investigation (which can be up to two per cent of the whole construction cost) would seem a modest investment compared to the risk of unsatisfactory performance. Given the extent of unsatisfactory seismic performance of foundations in Christchurch, a greater expenditure on geotechnical site investigations in future is warranted.

We note that the substantial Tonkin and Taylor4 report is to be made publicly available and should be very helpful when designing foundations for new buildings in the Christchurch CBD, though it will not obviate the need for site-specific investigations. We assume that as more investigative work is carried out in the context of new developments, the results of that work will also be made publicly available. In time, a more detailed database will be available to guide the designer. We consider that there will be a clear public benefit if that process is followed.

Dr McManus3 recommended that greater use should be made of the cone penetrometer test (CPT) for sub-surface exploration. This provides a standardised and cost-effective continuous profiling of ground conditions. By comparison, the standard penetration test (SPT) is carried out at fixed intervals, usually of 1.5 metres or more. Where the alluvial environment is highly variable this interval is far too coarse to properly characterise the sub-surface soils. CPT testing should be carried out to the depth of refusal (i.e., the maximum depth to which the pile can be driven without damage), at close enough separations across the site to be able to characterise the variability of the various strata.

The depth of penetration of the CPT test is often limited because the penetrometer cannot penetrate cobbly gravels or other very dense layers at depth. It will almost always be necessary to continue the sub-surface exploration to a greater depth using drilling equipment with SPT tests at intervals. If a significant thickness of weak soil continues beneath the dense layer causing refusal, it is possible to continue with the CPT if a temporary casing is left in place through the gravel after drilling.

The necessary depth of the sub-surface exploration, by whatever means, requires careful judgement by the geotechnical engineer. Frequently explorations are terminated at too shallow a depth. The depth of exploration should extend through all soil strata considered able to affect the behaviour of the site and the building foundations, and then continue to a sufficient additional depth to ensure all potential problem soils have been identified. Where deep pile foundations are being considered, the exploration should continue well into the bearing stratum and at least 10 diameters below the intended founding depth.

Detailed guidelines for evaluation of soil liquefaction are provided by the New Zealand Geotechnical Society (NZGS). These should now be updated to include new information and experience from Christchurch. The preferred exploratory tool for liquefaction assessments is the CPT, supplemented by laboratory testing of soil samples recovered from layers identified as being at risk.

The potential for softening of granular soils under strong seismic shaking needs to be considered. It appears that in the Canterbury earthquakes, high pore-water pressures affected many gravel deposits as well as causing the more obvious liquefaction in sand. This resulted in upward heave of some basements founded directly on gravel, and the short-term loss of shear strength may have contributed to poor pile performance on some sites.

Recommendations

We recommend that:

3. A thorough and detailed geotechnical investigation of each building site, leading to development of a full site model, should be recognised as a key requirement for achieving good foundation performance.

4. There should be greater focus on geotechnical investigations to reduce the risk of unsatisfactory foundation performance. The Department of Building and Housing should lead the development of guidelines to ensure a more uniform standard for future investigations, and as an aid to engineers and owners.

5. Geotechnical site reports and foundation design details should be kept on each property file by the territorial authority and made available for neighbouring site assessments by geotechnical engineers.

6. The Christchurch City Council should develop and maintain a publicly available database of information about the sub-surface conditions in the Christchurch CBD, building on the information provided in the Tonkin and Taylor4 report. Other territorial authorities should consider developing and maintaining similar databases of their own.

7. Greater use should be made of in situ testing of soil properties by the CPT, SPT or other appropriate methods.

8. The Department of Building and Housing should work with the New Zealand Geotechnical Society to update the existing guidelines for assessing liquefaction hazard to include new information and draw on experience from the Christchurch earthquakes.

9. Further research should be conducted into the performance of building foundations in the Christchurch CBD, including sub-surface investigations as necessary, to better inform future practice.

### 4.10.2 Foundation loadings and design philosophy

The principal loads to be resisted by the foundations are determined by the structural engineer after analysing the proposed building specifications and using structural design actions and combinations of actions specified in NZS 1170.55. This Standard covers most design actions including self-weight, live load, wind, snow, earthquake, static liquid pressure, groundwater, rainwater ponding and earth pressure. The resulting loads to be resisted by the foundations include vertical and horizontal components and sometimes moments.

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

(a) Loading arises from ground accelerations that are impossible to predict in advance. Instead, design accelerations based on probabilistic analysis are used. The actual accelerations in any earthquake event will always be either greater or less than the design acceleration.

(b) The ground accelerations must be transmitted into the building by the foundations. Compliance of the foundations may reduce the acceleration of the building but the resulting relative displacements may damage the foundations.

(c) Earthquake shaking changes the strength and stiffness of the founding soils and reduces the capacity of the foundations. In an extreme situation some soils may liquefy (i.e., lose almost all of their strength and stiffness).

(d) Ground deformation during earthquake shaking induces indirect loads in deep pile foundations, including both instantaneous and permanent (kinematic) loads as well as the building loads.

Two limit states for the building are required to be considered separately by the designers under NZS 1170.55. These are the serviceability limit state (SLS), corresponding to specified service criteria for a building (often deformation limits), and the ultimate limit state (ULS), corresponding to specified strength and stability criteria together with a requirement for robustness (ability to withstand overload without sudden collapse). These concepts are addressed in section 3 of this Volume.

### 4.10.3 Serviceability limit state (SLS)

The main requirement of the foundations at the SLS is to minimise deformations (especially settlements), to limit damage and enable uninterrupted use of the building.

Dr McManus3 observed that in cases where liquefaction or significant softening is expected at the site during an SLS-level earthquake it will be very difficult to meet the settlement criteria unless the building is founded on well-engineered deep piles, or on shallow foundations where well-engineered ground improvement is carried out.

Load factors and strength-reduction factors are not applied under NZS 1170.55 when considering the SLS load case. Instead, given the uncertainty in estimating foundation settlements, designers should use conservative assumptions for soil parameters.

Recommendations

We recommend that:

10. Where liquefaction or significant softening may occur at a site for the SLS earthquake, buildings should be founded on well-engineered, deep piles or on shallow foundations after well-engineered ground improvement is carried out.

11. Conservative assumptions should be made for soil parameters when assessing settlements for the SLS.

### 4.10.4 Ultimate limit state

ULS actions and combinations are much less likely to occur during the lifetime of the building but need to be resisted with a very low risk of structural collapse or failure of parts relevant to life safety. The cost of damage repair after a ULS event may be substantial and repair may be uneconomic, resulting in demolition of the building.

The foundations form a key component of the overall building structure and their performance is critical to the safe and satisfactory performance of the building. Failure or excessive deformation of the foundations may threaten the stability of the building, prevent the intended lateral resistance mechanisms from developing, and cause excessive ductility demands on building elements, increasing the risk of collapse.

Buried foundation elements such as deep piles are difficult or even impossible to repair after an earthquake and preferably it should be the building superstructure that yields rather than the foundations. Excessive foundation deformations and tilting of buildings in Christchurch have required demolition of many buildings that were otherwise not badly damaged.

The foundations of a building should not fail or deform excessively before the building develops its full intended structural response, including member over-strengths. To ensure a sufficient level of reliability for the foundations under ULS loading, strength-reduction factors must be applied to the calculated capacities.

Deformation of the foundations under ULS loads (including overstrength actions) should also be considered. While there is no need to achieve the same low level of deformation as in the SLS case, the deformations must not be so great that they add appreciably to the ductility demand placed on the structure or prevent the intended structural response.

The deformations required to fully mobilise the calculated strength capacity of shallow foundations may be very large and are likely to govern design. Deep foundations may also suffer significant axial deformations, with soil liquefaction and other cyclic softening effects during earthquake shaking causing redistribution of loads along the pile length. Realistic assessments should be made of likely settlements, which should remain within acceptable limits.

The load path for transmitting horizontal ground accelerations into the building must be carefully considered. Yielding of the foundation soils may reduce the accelerations transmitted into the building, but the resulting relative deformations may still damage foundation elements and need to be carefully considered.

The three main available load paths are:

• sliding friction between the supporting soils and the underside of the building;

• passive resistance of the soil against downstand beams and other vertical faces such as basement walls and lift pits; and

• lateral resistance of piles (where present).

The allocation of load among these three mechanisms will depend on the relative stiffness of each load path. Typically, sliding friction comes first, then passive resistance of vertical faces, then lateral resistance of deep piles.

Where no piles are present there is a complex interaction between sliding friction and passive resistance against downstand beams (McManus6). Where deep piles are supporting the weight of the building, friction is likely to vanish rapidly because of soil settlement and should be discounted.

Deep pile foundations are also subject to indirect loading from soil deformation during and after earthquake shaking (kinematic loading). Ground shaking results in shear deformation in the soil column and deep piles are stressed as they try to resist these deformations. The most damaging kinematic effects on piles are from lateral spreading of the stiff surface crust relative to the base of the piles, which are usually embedded in a strong, non-moving bearing layer.

Recommendations

We recommend that:

12. Foundation deformations should be assessed for the ULS load cases and overstrength actions, not just foundation strength (capacity). Deformations should not add unduly to the ductility demand of the structure or prevent the intended structural response.

13. Guidelines for acceptable levels of foundation deformation for the ULS and overstrength load cases should be developed. The Department of Building and Housing should lead this process.

### 4.10.5 Strength-reduction factors

Designing building foundations in New Zealand may be described as a load and resistance factor design (LRFD) procedure, in which the uncertainty and variability in the loads and design actions on foundation elements are considered separately from the uncertainty and variability in the resistance of the foundation elements.

The appropriate load factors are given in AS/NZS 11707. For earthquake loads, the load factor is 1 because the uncertainty in earthquake loads is accounted for directly within the probabilistic analysis underlying the code.

Strength-reduction factors for foundations are given in the New Zealand Building Code (NZBC) Verification Method 4 (B1/VM4) and are typically quite low (down to 0.4) because of the low reliability of foundation capacity assessments. The reasons for this low reliability include the inherent variability of soil deposits, uncertainty in measuring soil properties, complex behaviour of soil materials and uncertainty in modelling foundation behaviour.

For earthquake load cases involving earthquake overstrength, B1/VM4 permits the use of a strength-reduction factor of 0.8–0.9, irrespective of the level of uncertainty in the assessment of foundation capacity (which may be very large). The use of such high factors (equivalent to a safety factor of 1.1–1.25) is inappropriate in most cases and implies a significant risk that individual foundation elements will be loaded beyond their capacity, resulting in excessive plastic deformations. The high variability in capacity among individual foundation elements means that the structure may not behave as the designer intended.

Strength-reduction factors for the gravity load resisting elements of the foundation should be based on a risk-based procedure such as AS 2159–20098. The objective should be to ensure reliable foundation performance under all load combinations before, during and after an earthquake. B1/VM4 should be revised accordingly.

Recommendations

We recommend that:

14. The concessional strength-reduction factors in B1/VM4 for load cases involving earthquake load combinations and overstrength actions (g = 0.8–0.9) should be reassessed.

15. The strength-reduction factors in B1/VM4 should be revised to reflect international best practice including considerations of risk and reliability.

The development of full ULS lateral load resistance of foundation elements is not always critical to the safe performance of buildings. Lateral loads are transmitted from the ground into the building, causing it to accelerate in sympathy with the moving ground. Premature yielding of shallow soils forming the lateral load path may have the beneficial effect of reducing that acceleration. The trade-off is relative lateral displacements between the foundation elements and the ground surface.

For buildings on shallow foundations, any relative lateral displacement (sliding) may contribute to bearing failure under footings, differential settlements and tilting. This should be avoided by applying appropriate strength-reduction factors in design.

For buildings on deep pile foundations, soil yielding may be beneficial, both in reducing building accelerations and in reducing pile kinematic effects. Pile axial capacity should not be affected.

Recommendations

We recommend that:

16. For shallow foundations, soil yielding should be avoided under lateral loading by applying appropriate strength-reduction factors.

17. For deep pile foundations, soil yielding should be permitted under lateral loading, provided that piles have sufficient flexibility and ductility to accommodate the resulting displacements. In such cases, strength-reduction factors need not be applied.

### 4.10.6 Shallow foundation design

Many buildings in the Christchurch CBD were on shallow foundations, including a number of quite large and tall buildings. While their foundations performed well under gravity loading before the earthquakes, many performed poorly during the earthquakes, with large settlements and tilting, mainly where there was soil liquefaction and lateral spreading. However, most raft foundations supported by dense gravel strata at shallow depth (which occur intermittently in the CBD) performed well.

Overseas experience also indicates that strong, well-engineered, shallow foundations in dense, strong soil or robust improved ground can perform well during strong shaking.

For many smaller buildings the cost of deep foundations may be prohibitive but such foundations may be unnecessary if a sufficient thickness of strong soil underlies the site at a shallow depth. With poorer, weaker soils, it may be economic to apply well-engineered ground improvement so that shallow foundations can be used instead of deep pile foundations.

If a natural gravel raft is to be used to support a building, a realistic assessment needs to be made of the intended founding stratum to ensure that is sufficiently strong, thick and consistent across the site to provide reliable foundations. Allowance should be made for the effects of raised pore-water pressures penetrating the gravel raft from liquefaction of adjacent or underlying loose soils. For this reason, continuous concrete raft foundations should be preferred over isolated pad footings.

The following requirements for shallow foundations need to be carefully addressed by designers:

(a) There must be a clearly identified bearing stratum at shallow depth that will provide adequate support for the building loads. Alternatively, well-engineered ground improvement must be carried out.

(b) The near-surface bearing stratum must be thick and strong enough to bridge any underlying liquefiable or weak soils. The necessary thickness is relative to the weight of the building and building form, as well as the properties of the bearing stratum and underlying soils.

(c) The bearing stratum must be proven to be continuous across the site in order to uniformly support the entire footprint of the building. No building should be supported partly on shallow foundations and partly on deep piles.

(d) Where the bearing stratum overlies liquefiable soils, the foundation system should be well tied together and able to span any pockets where support may be lost as a result of pore-water penetration into the stratum. Multi-storey buildings should have raft foundations or deep pile foundations.

(e) Where the bearing stratum overlies liquefiable soils the ground floor slab (or basement slab or raft) should be able to resist the very high pore-water pressures resulting from soil liquefaction at depth. Such high pressures have been found in Christchurch to penetrate even dense overlying gravels and cause heaving failure of floor slabs in contact with the ground surface.

Shallow foundations have the potential to function very well for buildings on sites with strong soils, natural gravel rafts overlying weaker soils, or where robust, well-engineered ground improvement is carried out. However, not all shallow foundations performed satisfactorily in Christchurch during the earthquakes. A range of complex issues needs to be addressed for satisfactory performance during strong earthquakes.

Recommendation

We recommend that:

18. The Department of Building and Housing should lead the development of detailed guidelines to address the design and use of shallow foundations.

Settlement governs the design of shallow foundations for the SLS. Significant surface settlement is likely where liquefaction is liable to be triggered during an SLS-level earthquake in underlying soil strata. A conservative assessment should be made of the extent of differential settlements that may occur within the building, which should remain within the guidelines provided by NZS 1170.5. If any risk of tilting of multi-storey buildings is identified for the SLS (which is likely where lateral spreading occurs), then the building should be founded on deep piles.

Settlement also is likely to govern the design of shallow foundations for the ULS. Strength (capacity) calculations for shallow foundations are based on considerations of limiting equilibrium and require very large soil deformation to become fully mobilised. Little guidance exists regarding acceptable foundation settlement for the ULS. Foundation deformations should not be so great as to increase the ductility demands on key elements of the structure, prevent the desired structural response during strong shaking, or otherwise increase the risk of collapse of the building.

Foundation settlements must not be so great as to add appreciably to the ductility demands placed on the structure or prevent the intended structural response.

Foundation (strength) capacity calculations should always be carried out (in addition to settlement calculations) for the ULS and appropriate strength-reduction factors applied. Allowance should be made for loss of soil strength during earthquake shaking (from increased pore-water pressure and other forms of cyclic softening), for inertial effects (so-called seismic bearing factors – e.g., Ghahramani and Berrill9), and friction acting along the base of the footing from lateral loading (inclined loading).

For shallow foundations, the two available load paths for transmitting inertial forces into the building are:

• sliding friction between the supporting soils and the underside of the building; and

• passive resistance of the soil against downstand beams and other vertical faces such as basement walls and lift pits.

Where no basement is present there is a complex interaction between sliding friction and passive resistance against downstand beams (McManus6). This may result in unexpected distribution of forces, differential settlements and tilting. Sliding of buildings on shallow foundations should be avoided.

Recommendations

We recommend that:

19. The Department of Building and Housing should lead the development of more detailed guidance for designers regarding acceptable foundation deformations for the ultimate limit state (ULS).

20. Shallow foundations should be designed to resist the maximum design base shear of the building, so as to prevent sliding. Strength- reduction factors should be used.

### 4.10.7 Ground improvement

The objective of ground improvement is to treat loose, weak soils to prevent liquefaction and improve their strength and stiffness so shallow foundations may be safely used with satisfactory results. International experience has shown that buildings perform well where well-engineered, robust ground improvement has been carried out. The experience in Christchurch was more varied, despite the fact that the ground shaking was much more intense than the design ULS level. The performance of sites with ground improvement in Christchurch needs to be the subject of further detailed research to better understand the reasons for the variation in performance.

A very wide range of ground improvement techniques is available and these are subject to ongoing innovation. Techniques include in situ densification of loose and susceptible soils, improved drainage to reduce pore-water pressures during shaking, partial or total replacement of soils, in situ mixing of cementitious materials, and reinforcement to strengthen and stiffen soils. The many techniques and the parameters associated with each technique achieve a wide range of outcomes, both in terms of level of improvement and subsequent performance during shaking. The greater the improvement, the greater the cost, so there needs to be a degree of sophistication in specifying and monitoring these processes.

Many of the ground-improvement techniques are subject to proprietary technology and are heavily dependent on the knowledge, training and skills of the firms that developed the techniques and their site personnel carrying out the work. Many new techniques have been imported into New Zealand as a result of the Canterbury earthquakes and there are risks associated with the transfer of complex skills from well-established overseas operations to local operators. There are also risks associated with transfer of techniques developed overseas to the local geological situation. Track records established overseas may not be able to be immediately relied upon locally. On the other hand, the local presence of many top international firms provides a unique opportunity to inform and improve local practice and is a valuable resource for the rebuild of Christchurch.

Where ground improvement is being relied on to prevent soil liquefaction and to permit use of shallow foundations to support a building, the ground improvement effectively forms part of the foundation system of the building and should be considered as such. The considerable uncertainty in predicting the performance of the ground after treatment should be taken into account during design by applying appropriate factors (similar to the strength-reduction factors used in pile and footing design) to ensure a reliable outcome.

The objective when designing ground-improvement works should be to provide a level of confidence and robust performance, not simply to achieve some narrowly specified soil parameter. There needs to be good case-study evidence of the performance of each technique during earthquakes, especially where they are to be used as part of the foundation system for a multi-storeyed building.

The performance of a building foundation, including any ground improvement, depends on the design. Quality assurance during installation and performance testing of the finished work should enhance the understanding of satisfactory foundation behaviour.

The design basis of many ground-improvement techniques is dispersed through the literature, with little uniformity of approach. There is a need to collate and distil this information to promote a more consistent and robust approach to design of ground improvement works.

Recommendations

We recommend that:

21. The performance of ground improvement in Christchurch should be the subject of further research to better understand the reasons for observed variability in performance.

22. Ground improvement, where used, should be considered as part of the foundation system of a building and reliability factors included in the design procedures.

23. Ground-improvement techniques used as part of the foundation system for a multi-storey building should have a proven performance in earthquake case studies.

24. The Department of Building and Housing should consider the desirability of preparing national guidelines specifying design procedures for ground improvement, to provide more uniformity in approach and outcomes.

### 4.10.8 Deep foundation design

Deep piles can provide a good foundation for buildings at sites with poor foundation conditions near the ground surface, by transferring loads to deeper soil strata that are usually stronger, denser and older. They can also resist vertical uplift loads where required.

However, there are limitations and drawbacks with deep piles. They are vulnerable to relative lateral movements of the various soil strata during shaking (kinematic effects), loss of support and down-drag from liquefying intermediate soil strata, buckling within thick layers of liquefied soil, and damage from relative movements between the building and the ground surface. In loose, wet sands they can be difficult and expensive to install. Good performance is not assured without very careful engineering.

Not all deep foundations performed satisfactorily in Christchurch. The reasons for this have not yet been identified but probably include failure to penetrate into suitable bearing strata, loss of support caused by liquefaction and cyclic softening, and load redistribution along the piles caused by liquefaction and cyclic softening.

The following requirements for deep pile foundations need to be carefully addressed by designers:

(a) There must be a clearly identified bearing stratum that will provide adequate support for the pile type and the building loads. Piles must be installed (driven, bored, screwed) to a target depth within the bearing stratum as determined by the site investigation and not simply driven to refusal or to a set.

(b) The bearing stratum must be sufficiently deep to be below any layers of liquefiable or weak soils, or be thick enough to bridge over any underlying liquefiable or weak soils.

(c) Caution is required where a bearing stratum is not continuous across the site. A conservative approach should be taken to ensure uniform support can be provided to the entire footprint of the building.

(d) Piles must be capable of reliably transferring the vertical loads (including uplift loads) from the building to the bearing stratum, and meet settlement requirements (even with liquefaction and cyclic softening of overlying soils), including the effects of loss of side resistance, load redistribution and down-drag.

(e) Piles must withstand relative lateral movements of intermediate soil strata (kinematic effects) including permanent lateral movement of the ground surface (lateral spread), without excessive damage that might compromise their ability to carry the building vertical loads reliably.

(f) Piles must be able to transfer the horizontal ground accelerations into the building without excessive damage that might compromise their ability to carry the building vertical loads reliably.

(g) Heavily loaded, slender piles penetrating through thick layers of liquefied soil may fail by buckling. The possibility of pile instability with liquefaction must be considered.

Deep foundations can provide very good foundations for buildings on difficult sites, but not all performed satisfactorily in the Canterbury earthquakes. A range of complex issues needs to be addressed for satisfactory performance during strong earthquakes.

Recommendation

We recommend that:

25. Detailed guidelines for deep foundation design should be prepared to assist engineers and to provide more uniformity in practice. The Department of Building and Housing should lead this process.

Many types of deep foundations are available and they are the subject of continual innovation. Each type has different advantages and disadvantages that make it more or less suitable for earthquake-resistant design. The most suitable types commonly used in New Zealand are discussed below.

### 4.10.9 Driven piles

Driven piles (treated timber, precast concrete, steel tubes, steel H-piles) have a significant advantage over other pile types for seismic design because the driving process pre-loads the base of the pile in the targeted bearing-stratum while simultaneously mobilising negative side-resistance along the shaft in the overlying soils. This effect may significantly reduce pile settlement if liquefaction or cyclic softening occurs in the overlying soils.

Driven piles have become less popular in recent years because of issues with noise and vibration during installation. Modern equipment and procedures help to minimise this and the temporary inconvenience during installation should be weighed against the significant performance advantages possible during earthquakes.

Where jetting, pre-drilling and vibrating hammers are used to help install piles through intermediate stiff strata and reduce noise and vibration, these procedures should not be used to penetrate into the targeted bearing stratum. The pile should be driven into the bearing stratum with a suitable hammer (gravity or hydraulic).

Recommendation

We recommend that:

26. Because driven piles have significant advantages over other pile types for reducing settlements in earthquake-resistant design, building consent authorities should allow driven piles to be used in urban settings where practical.

### 4.10.10 Bored piles

Bored piles can have advantages over driven piles, including better penetration of difficult intermediate layers to reach any desired target depth, the ability to observe and confirm the properties of the bearing-stratum during construction, and the fact that they can be made to a large diameter. In some situations, large-diameter heavily reinforced bored piles may be able to resist lateral spreading.

### 4.10.11 Continuous flight auger (CFA) piles

CFA piles are bored piles installed using a hollow-stemmed auger that eliminates the need for ground support in caving conditions. They have most of the same advantages and disadvantages as bored piles but are more limited in diameter and depth range and have more limited penetration of difficult intermediate layers.

The main disadvantage of bored piles for sites where soil liquefaction occurs is that they are susceptible to loss of side-resistance with liquefaction and attendant down-drag from overlying non-liquefied layers. Bored piles obtain most of their initial axial load capacity from side-resistance, which is a much stiffer load-transfer mechanism than end-bearing, and when completed most of the building weight will be carried by side-resistance. With liquefaction, much of the side-resistance may be lost, resulting in a significant transfer of load to the base of the pile.

However, mobilisation of the end-bearing mechanism requires significant settlement to take place, typically 5–10 per cent of pile diameter. Greater settlement may occur where the base of the pile has been excessively disturbed during construction or poorly cleaned out.

Special construction techniques may be used either to pre-load the base of the pile or to reduce side-resistance through the upper liquefiable strata, but these all add cost. It is possible to pre-load the base of bored piles using pressure-grouting techniques or special devices. Excavating bored piles using bentonite slurry is known to reduce side-resistance and permanent sleeves may be installed. CFA piles pre-load the pile base to a limited extent by injecting concrete under pressure during installation.

Installation of deep bored piles in Christchurch (and in some other urban centres in New Zealand) may be complicated by the presence of artesian ground water pressures within the target gravel bearing strata.

### 4.10.12 Screw piles

Screw piles consist typically of one or more steel plate helices welded to a steel tube. The pile is screwed into the ground and the tube filled with concrete. Torque measurements are used to identify penetration into the target-bearing stratum. These piles have the advantage for seismic loading that almost all the load is transferred to end-bearing on the steel helices embedded in the target bearing stratum, with minimal side-resistance along the shaft.

For all pile types, the risk of “punch through” into underlying layers of liquefiable soil needs to be considered. In the case of a very weak soil layer underlying a strong bearing layer, the weak layer may influence the bearing capacity of the pile for a thickness of least five diameters above the interface, and possibly more if high excess pore water pressures penetrate upwards into the bearing layer.

### 4.10.13 Uplift capacity

Deep piles, especially bored piles, have often been used to resist large overturning actions generated by certain structural forms. Provided the site soils are not at risk of liquefaction or cyclic softening during earthquakes, the side-resistance mechanism will provide a stiff response in both compression and uplift. Cyclic degradation of the side-resistance mechanism may occur where the pile carries only light gravity loads, and should be considered.

Where liquefaction and/or cyclic softening of soils is likely to occur, deep piles may have limited capacity to resist uplift loads. Only the side-resistance from soil strata below the deepest liquefiable layer should be relied on to resist either uplift or compression during and after shaking.

In some locations (notably Wellington) belled piles have frequently been used to improve the uplift resistance of bored piles. The upper surface of the bell is considered to act as an upside-down footing and treated as such for the calculation of capacity. However, the mobilisation of end-bearing in soil, upwards or downwards, may require significant movement of the pile (5–10 per cent of diameter in each direction) and is likely to result in a very soft load-displacement response, especially if gapping develops. The resulting structural response may be more like foundation rocking, which can be quite different to that intended by the designer. Foundation movements, both upwards and downwards, are likely to govern design and need to be considered.

An upside-down punching shear failure is also possible where weak or liquefied soil overlies the founding stratum. Penetration of about five diameters into the founding stratum is necessary to develop maximum uplift capacity. The “punch through” failure mechanism should be considered for lesser embedment depths.

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

Belled piles should only be used in firm, cohesive soils or weak rock in dry-hole conditions where the bell can be excavated without risk of collapse and carefully cleaned out and confirmed before concreting. Drilling belled piles in granular soils under fluid should not be permitted where the integrity of the bell cannot be assured.

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

As for belled piles, the uplift capacity will be reduced by the presence of weak or liquefied overlying strata unless the helix is embedded at least five diameters into the bearing stratum. An “upside-down punch through” mechanism should be considered where embedment into the bearing stratum is less than five diameters.

### 4.10.14 Kinematic effects

All deep pile foundations are vulnerable to kinematic effects where different soil strata undergo differential lateral movements during and after shaking. The most damaging effects arise where the non-liquefied surface crust undergoes significant permanent lateral deformation relative to the underlying bearing stratum. With significant soil liquefaction, permanent lateral movements of the surface crust are widespread and vary from extreme (severe lateral spreading near watercourses) to subtle but potentially damaging movements caused by minor surface gradients. In Christchurch, permanent lateral deformations of the ground surface, of up to 300mm, were widespread even in areas far from watercourses and without obvious surface manifestation of lateral-spreading damage. All deep pile foundations in areas where there is a risk of significant liquefaction should be designed to accommodate such movements, even when they are far from any watercourse.

Even where soil liquefaction is not considered an issue, kinematic effects can still arise through deformations of other weak soils, especially adjacent to steep slopes such as waterfronts and bridge abutments.

The main protection against kinematic effects for deep piles is flexibility and/or ductility. In most cases it will be impractical to make deep piles strong enough to prevent the kinematic movements because the mass of the moving soils is enormous and the non-liquefied soils will rapidly develop full passive pressure to act against the face of each pile. In most cases it will be acceptable for the piles and the supported building to move with the surface crust, provided the piles are sufficiently flexible and ductile to continue to safely carry the vertical loads from the building.

Recommendation

We recommend that:

27. Where there is a risk of significant liquefaction, deep piles should be designed to accommodate an appropriate level of lateral movement of the surface crust even when they are far from any watercourse.

### 4.10.15 Lateral loading

Where deep pile foundations are used, friction acting between the ground floor or basement slab should not be relied on to transmit base shear into the building, because of likely settlement of the ground surface relative to the building. Passive resistance of soil acting against vertical surfaces such as downstand beams and liftshafts may continue to provide a load path, provided ground settlement is not excessive. However, the passive resistance mechanism is often soft and in most cases it is likely that the main load path for lateral loads entering the building will be through the piles.

For some sites with weak or liquefied soils and insubstantial surface crust, it may not be possible to develop the calculated ULS design base shear for the building before the soil around the piles will yield. This may have the benefit of reducing the building accelerations and reducing the kinematic deformations of the piles and should be accepted. However, neither benefit should be counted on, either to reduce building element design forces or to reduce pile kinematic loads, because it is difficult to accurately predict the maximum passive resistance of the soil.

Care must be taken to ensure that a pile does not suffer a brittle structural failure that might compromise the axial capacity of the pile. The pile should be detailed with adequate ductility to withstand the maximum lateral load from building inertia when it is also being subjected to maximum kinematic deformation and carrying the necessary axial loads.

Recommendations

We recommend that:

28. Base friction should not be included as a mechanism for lateral load transfer between the ground and the building when it is supported on deep piles.

29. If reliance is to be placed on passive resistance of downstand beams and other vertical building faces, a realistic appraisal of the relative stiffness of the load-displacement response of the passive resistance compared to the pile resistance should be made.

30. For buildings on deep piles, it is not essential that the calculated lateral capacity of the foundations should exceed the design base shear at the ULS, provided that the piles have sufficient flexibility and ductility to accommodate the resulting yield displacement and kinematic displacements.

31. There are major problems in the use of inclined piles where significant ground lateral movements may occur. Where the use of inclined piles is considered, the kinematic effects that may generate very large axial loads that could overload the pile and damage other parts of the structure connected to the pile should be considered.

### 4.10.16 Buckling and P-delta effects

Evidence from overseas indicates that heavily loaded, slender piles may buckle in thick layers of liquefied soil (Bhattacharya et al.10), although there are no known examples from Christchurch. P-delta effects have also been discussed as a possible problem for deep piles, but in most cases the pile head will be well restrained by the surface crust.

### 4.10.17 Cyclic effects

Even where soil liquefaction does not occur, cyclic axial loading of deep piles may cause degradation of the side-resistance mechanism, resulting in load transfer to the pile base and attendant settling and loss of uplift resistance. The most vulnerable piles are those carrying relatively light gravity loads or where cyclic load reversal (from compression to uplift) occurs.

## References

1. Cubrinovski, M. and McCahon, (August 2011). *Foundations on Deep Alluvial Soils*. Christchurch, New Zealand: Canterbury Earthquakes Royal Commission.
2. Letter from Jonathan D. Bray, J.D. Bray Consultants, LLC to Canterbury Earthquakes Royal Commission, 18 October 2011.
3. McManus, K.J. (2011). *Foundation Design Reliability Issues*. Christchurch, New Zealand: Canterbury Earthquakes Royal Commission.
4. Tonkin and Taylor Limited (2011). *Christchurch Central City Geological Interpretative Report*. Christchurch, New Zealand: Christchurch City Council.
5. NZS 1170.5:2004. *Structural Design Actions Part 5, Earthquake Actions*. Standards New Zealand.
6. McManus, K.J. (2003). Earthquake Resistant Foundation Design*, Proc. N.Z. Geotech. Soc. Symp., Tauranga, N.Z*., (Invited theme paper).
7. AS/NZS 1170.0:2002. Structural Design Actions, Part 0, *General Principles*, Standards Australia/Standards New Zealand.
8. AS/NZS 2159:2009. *Piling – Design and Installation*. Standards Australia.
9. Ghahramani, A. and Berrill, J.B. (1995). Seismic Bearing Capacity Factors by Zero Extension Line Method, Proc. *Pacific Conference on Earthquake Engineering*, November 1995, 147-156.
10. Bhattacharya, S., Madabhushi, S.P.G. and Bolton, M.D. (2004). An alternative mechanism of pile failure in liquefiable deposits during earthquakes, *Geotechnique 54*, No. 3, 203-213.