

**Assessment of liquefaction and related ground  
failure hazards in Palmerston North, New Zealand**

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## EXECUTIVE SUMMARY

GNS Science has been engaged by Palmerston North City Council to assess liquefaction susceptibility for the city area and to suggest measures to help investigate and mitigate any areas of vulnerability identified. Specific reference was made to assessment of the PNCC wastewater treatment plant, Anders Racecourse and Kelvin Grove residential development areas. Arrangements for this work were made following the liquefaction which occurred during the 2010 Darfield Earthquake, and the report content was expected to reflect experiences gained from that earthquake. In the intervening period, the February 2011 Christchurch Earthquake occurred (see Appendix 2 for descriptions of the Darfield and Canterbury earthquakes). It resulted in increased liquefaction and damage in Christchurch. A revised delivery date has been arranged so that insights resulting from both these earthquakes could be more readily incorporated in this report.

Data for the report was collected from PNCC and other organisations. Our results and interpretations are assessed and presented in the report. Efforts have been made to separate detailed comment on the many contributing facets of the report from the principle conclusions. More detailed information is found in the 9 Appendices. The main report body discusses the liquefaction process, historical information, known active faulting and their characteristics, estimated return periods for strong ground shaking, geology, soils, subsurface investigations, the liquefaction susceptibility in Palmerston North and suggests mitigation measures based partly on experience gained from the two Christchurch earthquakes.

Following a general account of the phenomenon of liquefaction and its definitions, some historical international and New Zealand examples are outlined. This is followed by a brief explanation of methodology for assessing liquefaction susceptibility and opportunity. Strong ground shaking resulting from local moderate to very large regional earthquakes provides the opportunity for the process of liquefaction to occur in susceptible soils. The report presents information on the Modified Mercalli (MM Intensity) scale (fully described in Appendix 9), a descriptive method for quantifying strong ground shaking based on the observations of affected people. The report provides lists the main historical earthquakes that have affected Palmerston North, it provides a list of known potential seismogenic sources in the region and their characteristics, and it provides calculated values for the recurrence interval of strong ground shaking.

A summary of relevant information on the tectonic setting of the region, and the physiography, soils and geology of Palmerston North City precedes a section outlining the derivation of the “liquefaction map” (Figure 8b). We discuss the conventional method for developing a liquefaction map and describe how our map was generated using updated geological, soils, lidar and drillhole database data. Our map can be compared with that produced by Dellow *et al.* (1994); highlighting the differences between them.

The availability of a number of important digital datasets, including lidar topographic coverage, the Horizons Manawatu drillhole database, soils grading curves and geotechnical data provide an opportunity for improved input into our liquefaction assessment. We have developed a GIS database of subsurface information which enables the user to carry out 3D modelling, and a method for doing this is presented in Section 5.3 of the report. The Lidar data provide an accurate elevation model for the Palmerston North lowlands. The Horizons drillhole database provides significant sub-surface lithological information, as well as a basis for modelling static groundwater elevation. Downhole lithological information is supplemented

by geotechnical data. Modelling the static groundwater level and the upper surface of the top gravel deposits in conjunction with the lidar topographic model provide the tools to define thickness of fine-grained materials overlying gravel, and allow modelling of their saturated thickness across the urban area. Models derived from these data are presented in Figures 11, 12 and 13.

These models are not considered to be definitive statements regarding liquefaction susceptibility. This is because the liquefaction susceptibility of the fine-grained coverbeds cannot be adequately assessed from the drillhole logs. Additional data is required to improve the active database and the model it can generate. However, the modelling we have carried out defines areas of thick coverbeds that may be susceptible to liquefaction, and areas where there is unlikely to be liquefaction potential. This information can be used to indicate where careful geotechnical investigations are required.

The specific conclusion and recommendations of our report are as follows:

1. The 1994 (Dellow et al.) liquefaction assessment by GNS used mainly soil and geological information, supplemented with a small amount of available “geotechnical” criteria to carry out a regional liquefaction assessment, which included Palmerston North City. Holocene (last 10,000 years) geology areas were the main focus while the supplementary data included a few logged drillholes.
2. The present liquefaction assessment has concentrated on Palmerston North City. The data used has included the latest geological QMaps (Begg & Johnston 2000, & Lee & Begg 2002; Townsend *et al.* 2008), the soil map of Cowie & Kimpton (1976), lidar models and the drillhole data base of Horizons Manawatu Regional Council, supplemented with additional drillhole and geotechnical data located by GNS. The drillhole data base of some 1,600 records, includes lithology, groundwater level and relatively minor geotechnical data. We recommend that future site investigations in Palmerston North include more accurate drillhole information and required geotechnical data which can be added to improve the database and its resolution.
3. Internationally cities are starting to use drillholes and other subsurface data from tunnels, storage spaces, building basements, etc., to compile 3D geology maps of the ground beneath their city. This 3D geology is in an active database, in GIS format, which allows for the ready addition of new data and the retrieval of entered data for assessing new projects and developments. We have compiled all the available subsurface data into a GIS 3D database and have focussed our attention onto the top 30 m which is relevant for our liquefaction assessment. This database will be handed over to PNCC as a working database to which new and improved data can be added. This has the objective of refining the liquefaction modelling we have been able to formulate. We expect that the database will become increasingly useful to PNCC. As new data is added its uses will expand.
4. Our map differs significantly from the 1994 GNS assessment. We consider that the area of the city with the highest liquefaction susceptibility is the recent flood plain of the Manawatu River (Figure 8b) with its old infilled channels which show up beautifully on the lidar maps (Figures 5, 8c & 10). This flood plain has a variable thickness of loose, fine sand and silt flood deposits which range in thickness from zero to 6 m or more. However, this material is not always saturated as permeable gravels underly the fine sands. For this reason the liquefaction susceptibility is rated from 1 (very high) to 3 (moderate), although areas of gravel would have low to nil liquefaction susceptibility.

Where there is sloping ground combined with saturated highly susceptible soils, lateral spreading could occur in this zone.

5. The soils on which much of the City is constructed we rate as having “moderate” liquefaction susceptibility (Figure 8b) Zone 2 (high susceptibility) to 3 (moderate). This rating is based on the thickness of “soft” ground and high water table noted in the drillhole records. It is supported by a few soil grading curves obtained from Soil Bureau archives (Appendix 6). There is very little geotechnical data available to support this liquefaction susceptibility ranking. The soil mapping data (Cowie & Kimpton, 1976) and the associated grading curves indicate that the soft soils may be cohesive, in which case their liquefaction susceptibility will be reduced, possibly to a cyclic strain softening behaviour. This assessment will be improved and refined as more geotechnical data is added to the database.
6. The remainder of the City area (Figure 8b) is assessed as having low or negligible liquefaction susceptibility.
7. Liquefaction potential is assessed in terms of both the susceptibility of the soils and the opportunity for strong shaking to occur. Our study confirms that there are many local earthquake generating faults (seismogenic sources) within 40 km of Palmerston North which could generate strong ground shaking sufficient to cause liquefaction. Liquefaction begins to occur at MM7 intensity shaking and the amount and severity of liquefaction increases with increasing intensity. Using the GNS probabilistic seismic hazard model, we estimate that MM7 intensity shaking could be expected approximately every 30 years, MM8 every 130 years, MM9 every 700 years, and MM10 every 14,000 years. Based on the earthquakes that have now occurred in Christchurch, we consider that it is wise for Palmerston North City to assess what might happen at each of these levels of strong shaking.
8. Geotechnical data from the Totara Road waste water treatment plant shows that a variable thickness, ranging from 0 to 6 m depth of loose, uniformly graded, fine sand and silt are present at the site. When saturated these soils are very highly susceptible to liquefaction. However, the few measurements of static groundwater level are low possibly indicating that these susceptible materials are not saturated. In this case the site may not liquefy. As well the susceptible ground in newly constructed areas has been dug out and replaced with compacted, non-liquefiable material. These conclusions require confirmation. We recommend that a consultant experienced in the performance of the Christchurch area waste water treatment plants during the Christchurch earthquakes is engaged to carry out an assessment of the PNCC plant. This will provide PNCC with a rational basis for determining whether or not earthquake and liquefaction mitigation measures are required at their plant (see Section 7).
9. Experience from the Christchurch earthquakes shows that the seismic performance of a structure is closely related to its foundations. In the Christchurch area a huge amount of damage to houses was due to the failure of foundations, which were unsuitable for the known ground conditions (highly liquefiable soils). Where suitable foundations were used in highly liquefiable areas, such as driven tanalised timber piles capped by a well reinforced concrete slab, the house structure was invariably undamaged. Thus we do not exclude any areas from possible development. However, prior to undertaking any new developments in the City, suitable geotechnical investigations of the ground are required. Once the ground properties have been determined, suitable foundations which are needed to support the proposed structure during strong shaking in the prevailing ground conditions, can be rationally designed. This is a strong recommendation for the

Anders Road, Racecourse and Kelvin Grove growth areas, plus any new and infill housing and for new commercial buildings anywhere in the city (see Section 7). Using this process, the potential for damage to new houses and buildings can be mitigated with additional modest costs generally falling on developers. Experience from Christchurch shows that there is also a relatively urgent need for a robust assessment of older, earthquake risk buildings and their foundations, throughout the City.

10. We recommend that all new drillhole data, together with geotechnical data, such as CPT, seismic CPT, SPT, ground water levels, grading curves acquired from new site investigations, are entered into the subsurface 3D database. The database requires more geotechnical data to improve it and to refine the models obtained from it. Adding good data to the database will ensure it is a very useful and valuable asset for the City.
11. We recommend that PNCC approach all organisations that may have subsurface data on file. It is likely that engineering consultants will have a lot of useful subsurface data. They may be encouraged to share this data if they can obtain access to the complete database. Other organisations with subsurface data may include the NZ Transport Authority, the University and local building owners.
12. It is our hope that we have provided PNCC with some useful tools that are needed for the rational development of the City in light of the potential hazard from liquefaction, lateral spreading and other earthquake damage.

## 1.0 INTRODUCTION

### 1.1 Background

The area in which Palmerston North City currently lies was purchased by the New Zealand Government from the tangata whenua, the Rangitaane people, in 1864. Before this time the area was covered in dense native forest. From 1866 to 1926 Palmerston North area was largely a farming district with a small, sparse population. From 1926 onwards the City has developed rapidly and its area has increased to its current extent and population (Matheson, 2003).

On the 6th of September 2010, the Palmerston North City Council (PNCC) Planning and Policy Committee choose to defer the adoption of the "Residential Growth Strategy" until a comprehensive geotechnical report on the identified growth areas was presented to Council. This response was triggered by the liquefaction caused in Christchurch area during the Darfield Earthquake. The council wanted to avoid residential development on areas at greater risk of liquefaction, incorporate the experience gained from analysis in Christchurch and gain an appreciation of the liquefaction risk factors.

Following this deferment, the following was recommended during a full council meeting on the 29th of September:

***"That as part of the preparation of the Proposed District Plan Changes and associated section 32 RMA 1991 analysis being prepared to give effect to the Residential Growth Strategy, the Chief Executive be instructed to commission an integrated natural hazards analysis of the Residential Growth Strategy that includes a review of the risks posed by liquefaction and the options and costs to mitigate those risks."***

In 1994, GNS Science (GNS) completed two client reports (Berryman 1994 and Dellow *et al.* 1994) for Manawatu-Wanganui Regional Council on the assessment of liquefaction-induced ground failure susceptibility in the Manawatu-Wanganui region. The Dellow *et al.* (1994) report provided a regional liquefaction susceptibility map which included the Palmerston North City area. In September 2010, Catherine Stapp of PNCC approached GNS to find out if the map was available in electronic form so it could be loaded into Palmerston North City Council's GIS system. Of particular interest was the PNC waste water treatment plant's susceptibility to liquefaction. Catherine was also aware of other PNCC council departments seeking liquefaction susceptibility information based on the council motion. On further discussion, it was decided that a proposal from GNS be put forward to PNCC outlining a study for the assessment of liquefaction and related ground failure hazard in Palmerston North. This report results from that proposal

#### 1.1.1 Project Objectives

The following objectives were agreed to between PNCC and GNS:

1. To assemble, review, summarise and assess available subsurface information and historical liquefaction data from the Palmerston North area, and assess the applicability of relevant geological and engineering data.
2. To carry out a geotechnical assessment using existing information of the typical ground material to determine the scale of liquefaction and ground damage that might occur in

Palmerston North during strong (MM Intensity 7 to 10) earthquake shaking.

3. If it is found that significant liquefaction and associated ground damage could occur in Palmerston North, then measures that would help to investigate and mitigate the damage will be identified, together with comments on their applicability and likely costs.
4. To assess the liquefaction potential of the PNC wastewater treatment plant, Anders Road, Racecourse and Kelvin Grove residential growth locations and determine specific mitigation options and costs for each location.
5. In consultation with PNCC staff from both City Networks (Asset Management) and City Future (Strategic Planning), prepare a draft report, liquefaction susceptibility map and GIS layers that address these objectives by a negotiated date.

In light of the cost and damage caused by geotechnical phenomenon, such as liquefaction and lateral spreading, during the magnitude 7.1 Darfield Earthquake near Christchurch on the 4 September 2010, the PNCC commissioned GNS to assess hazards posed to Palmerston North City by liquefaction and the options and costs to mitigate those hazards. We note that the subsequent M6.3 aftershock on 22 February 2011 (now called the Christchurch Earthquake) caused deaths and a great deal more casualties, building damage and liquefaction in Christchurch than the larger magnitude, but more distant, Darfield Earthquake. A preliminary analysis of the reasons for this and their possible impacts on Palmerston North are discussed in this report. If required, a more complete analysis of the Christchurch Earthquake and its relevance to PNCC will be issued later as an Addendum to this report. The objective is for this hazard assessment to inform the Council's Residential Growth Strategy and associated proposed District Plan Changes and Section 32 RMA 1991 analysis.

Efforts have been made by GNS to ensure that this project has used all available and relevant information to assess the hazard of liquefaction within the Palmerston North City Council boundary. Areas of specific interest include the two potential residential growth areas; Kelvin Grove and Anders Road/Racecourse. Also of interest are the Totara Road Wastewater Treatment Plant and the Turitea Water Treatment Plant assets. Where liquefaction hazard is found to be significant, mitigation options are presented. This report includes a liquefaction hazard map and corresponding GIS layers.

Following collection of data from PNCC, it became clear that there are insufficient data available to provide a comprehensive assessment of liquefaction potential for the Turitea water treatment plant and that is excluded from this report by mutual agreement.

The report authors have presented the findings of this report to PNCC staff and have taken part in subsequent discussions with Staff. Questions raised by PNCC Staff are listed and answered in Appendix 8.

### **1.1.2 Project Design**

GNS proposed that the project be structured into two stages:

#### **Stage 1 Data gathering**

Gather geotechnical data from PNCC and other relevant organisations.

**Stage 2** Analysis and reporting  
Compile report, map and GIS layers.

It was proposed that the following topics would be covered by the report:

1. Introduction – scope, methodology
2. The liquefaction process
3. Geology of Palmerston North
4. Historical earthquakes and recorded liquefaction effects in Palmerston North
5. Known faulting and estimated recurrence intervals of shaking of sufficient strength to cause liquefaction.
6. Subsurface investigations
7. Potential for liquefaction and ground damage in Palmerston North, with particular reference to PNCC wastewater treatment plant, Anders Road, Racecourse and Kelvin Grove residential growth locations
8. Mitigation options

### **1.1.3 Timelines**

Data gathering for Stage 1 began as soon as the contract was agreed. The proposed and actual start date for Stage 2 was January 10th 2011. Preliminary findings from a current Christchurch liquefaction study on potential Greenfield development areas was regarded as being of direct relevance and benefit to the PNCC project. However, Palmerston North geotechnical information, such as subsurface investigations using (Seismic) Cone Penetrometer Testing (S)CPT, became available to GNS in late March 2011.

The initially proposed February reporting time was amended by mutual agreement so that immediate findings relevant to liquefaction from the Christchurch earthquakes could be incorporated in the report. This is particularly important in the light of decisions that may be made regarding possible mitigation or even abandonment of a few of the heavily liquefied and badly damaged areas of Christchurch. The final reporting time agreed by GNS and PNCC is 17 June 2011.

### **1.1.4 Previous GNS Reporting of Liquefaction in the region**

In 1994, GNS Science completed two client reports (Dellow *et al.* 1994/333902.4D and Berryman 1994/333 902.4B) for Manawatu-Wanganui Regional Council on:

1. Assessment of liquefaction-induced ground failure susceptibility in the Manawatu-Whanganui region and
2. Characterisation of near-surface geological conditions in urban areas of the Manawatu-Whanganui region.

The 1994 reports produced a deterministic regional liquefaction susceptibility map based on soils and geology with limited geotechnical assessment which included the Palmerston North City area.

In 1997 Wanganui City commissioned GNS to carry out a detailed study of liquefaction and related ground failure hazards in the Wanganui City Area (Beetham *et al.* 1997). This assessment, which included 17 sub-surface Cone Penetrometer Test (CPT) probes and other additional work, significantly refined the earlier 1994, more general study which was based on geology alone.

## **2.0 THE LIQUEFACTION PROCESS**

### **2.1 Introduction**

Strong shaking during earthquakes often results in the phenomenon known as liquefaction, as has been dramatically re-emphasised and illustrated in both the 2010 M7.1 Darfield and 2011 M6.3 Christchurch earthquakes. In this process, strong shaking may cause certain soils (mainly saturated, cohesionless, uniform fine sands and coarse silts) to consolidate, increasing pore water pressure and decreasing shear strength to a point where the soil is transformed to a liquid state. These changes can cause a loss of strength in near-surface materials, generally at depths of up to 20 m below ground level in areas where the water table is within about 5 m of the ground surface, often resulting in significant ground deformation.

Ground damage due to liquefaction processes occurs in the form of surficial ridges, hollows, fissures, lateral spreads and flow failures along with deposits of sand, silt and water in the form of “sand” volcanoes, vents and coalescing deposits of water-borne sands and silts. In places in Christchurch, these deposits formed a layer up to 1 m deep and flowed into and blocked drains. Liquefaction can allow dense, heavy objects, such as buildings, to sink, while lighter objects such as pipes, manholes and (empty) tanks may float upwards. All these expressions of ground damage, some of them spectacular and damaging, were observed and recorded in parts of Christchurch City, Waimakariri (Kaiapoi) and Selwyn Districts following the 2010 Darfield Earthquake (Allen *et al.* 2010). Even more severe liquefaction occurred within both previously liquefied areas and in new areas, following the 2011 Christchurch Earthquake (Figures 1 and 2).

Ground settlements, along with sand boils and water ejections in flat areas and lateral spreading, characterised by scarp-like ground cracking and settlements (again often with ejection of sand and water) on gently sloping ground adjacent to water courses, are the most common indication that liquefaction has occurred. Such ground deformation can be (extremely) damaging to buried underground services, and to poorly designed and constructed buildings. Complete liquefaction of loose, uniform, saturated fine sands and coarse silts, the most susceptible materials, can occur at distances of up to 150 km from large ( $\geq$  M 7.5) earthquakes, and can cause severe ground (and building) damage, as has been observed most recently in Liquefaction and lateral spreading accompanied both the M7.1 Darfield and M6.3 Christchurch earthquakes and has been associated with many historic earthquakes, both in New Zealand and overseas. Widespread liquefaction can occur at distances of up to 150 km from large ( $\geq$  M 7.5) earthquakes, and can cause severe ground (and building) damage.

## 2.2 Definition of liquefaction and related terms

Definitions of the liquefaction phenomenon and related effects caused by earthquakes have appeared in geotechnical publications, both in New Zealand and overseas. Liquefaction has been defined by Youd (1973) as "*the transformation of a granular material from a solid state into a liquefied state as a consequence of increased pore pressures*". Alternatively, Ziony (1985) defined liquefaction as "*the process by which water-saturated sediment temporarily loses strength, usually because of strong shaking, and behaves as a liquid*".

Widely accepted definitions of terms related to seismic liquefaction are those recommended by the American Society of Civil Engineers (ASCE Committee, 1978), and these definitions have been adopted for this study. The main terms that are relevant to this study are defined as follows:

### 2.2.1 Liquefaction

The act or process of transforming cohesionless soils from a solid state to a liquefied state as a consequence of increased pore pressure and reduced effective stress.

*Comments:*

- (1) Liquefaction is usually associated with and initiated by strong shaking during earthquakes, which causes certain soils (mainly cohesionless, uniformly-graded fine sands and coarse silts) to compact, increasing pore water pressure and decreasing shear strength. The term is strictly defined as a changing of state that is independent of the initiating disturbance that could be a static, vibratory, sea wave, or shock loading, or a change of ground water pressure. The definition is also independent of deformation or ground failure movements that might follow the transformation to a liquid state. The liquefaction process always produces a transient loss of shear resistance, but not a longer-term loss of shear strength.
- (2) Liquefaction is most likely to occur in saturated, relatively uniform, cohesionless, fine sands, silty sands, or coarse silts of low relative density (loose), generally at depths of up to 15 to 20 m below ground level, in areas where the water table is within 5 m of the ground surface. Such materials have relatively low permeability and only slowly dissipate (drain) increased pore-water pressures. Although liquefaction effects are observed only in loose soils, dense sands and silts may show initial liquefaction (strain softening) effects, but these are rapidly inhibited by the dilatancy characteristics of such soils.

### 2.2.2 Cyclic strain softening

Cyclic strain softening is a process defined in relation to liquefaction as a stress-strain behaviour under cyclic loading conditions in which the ratio of strain to differential shear stresses increases with each cycle. In saturated cohesionless soils, cyclic strain softening is caused by increased pore-water pressure.

Most saturated soils subjected to rapid undrained cyclic shearing (as is the case during an earthquake) will experience a change in their pore water pressure. Very permeable soils such as coarse sands and gravels are unlikely to experience much change in pore water pressure because pore water pressure changes can rapidly dissipate in such materials.

Dense sands may experience a decrease in pore water pressure due to dilation during shearing. However, more potentially damaging effects occur when seismic excitation generates an increase in pore water pressure with a consequential decrease in soil strength. Such soils are most susceptible to ground damage by strong earthquake shaking (for further discussion, see Appendix 1).

*Comments:*

- (1) Continued cyclic loading (as experienced during strong earthquake shaking) usually leads to increasing axial strains and increasing pore-water pressures, but does not necessarily lead to loss of ultimate shear strength if the material is dilative (such as dense sands and silts).
- (2) Cyclic strain softening occurs in cohesionless loose soils as a part of the liquefaction process. However, in cohesive soils (mud and clayey soils) cyclic strain softening effects (increased pore-pressures and decreased shear strength) can occur, resulting in some ground deformation or damage (collapse and settlement due to decreased bearing capacity), but complete liquefaction does not occur.

### 2.2.3 Ground failure

A term related to the field behaviour of soil and rock masses, and defined as a permanent differential ground movement capable of damaging or seriously endangering a structure. Related terms include:

- (a) **Lateral spread** - distributed lateral extensional movements in a fractured soil or rock mass, in which extension of the ground results from liquefaction or plastic flow of the materials. Lateral spreading commonly develops along the banks of rivers and streams, and man-made water courses (canals). Sand and water ejections are often associated with lateral spread fissures.
- (b) **Flow failure** - Flow failures (slides) are a form of slope movement involving the transport of earth materials in a fluid-like manner over relatively long distances, at least tens of metres.
- (c) **Sand boil** - An ejection of sand and water from cracks or fissures, and caused by piping from a zone of excess pore pressure within a soil mass. Sand boils commonly form during or immediately after earthquakes as pressures are relieved from liquefied zones, or zones of excess pore pressures in subsurface saturated cohesionless soils. *Sand boils are the most common and unambiguous indicator that liquefaction has occurred.*

*Comments:*

From a soil mechanics perspective, the basic cause of liquefaction in saturated cohesionless soils during earthquakes is the build-up of excess pore pressure due to the application of cyclic shear stresses induced by the earthquake ground motions. These stresses are generally considered to be due primarily to upward propagation of shear waves in a soil deposit, although other wave motions may also be influential. If the liquefaction effect is sufficiently severe and extensive, loss of ground strength may result in damage to any structures located in the affected area. Bearing capacity failure will cause buildings or

superficial structures to settle and tilt, and buried structures, such as underground pipes and tanks, may float upwards. Liquefaction of a confined sub-surface layer can cause large vertical and lateral displacements of the ground surface, to possibly only minor effects such as sand boils and water ejections, on otherwise unaffected alluvial (gravel) surfaces. If the area is on a gentle slope, or close to a free face such as an incised river channel or open drain, then lateral spreading failures can occur.

It has long been recognised that the intensity of ground shaking during earthquakes and associated damage to buildings are influenced by local site conditions, particularly soft sediment depth, soil and rock types, and depth to the water table. However, it has only been in the last 40 years that strong motion instrumental records have demonstrated the major effects of variations in local soil conditions on the characteristics of strong ground motions. Although liquefaction is a most damaging consequence of large and moderate earthquakes, other damage effects on "soft soil" sites are also important and are commented on in the report.

### **2.3 Liquefaction Background**

It is well documented in geotechnical literature that ground failures during earthquakes usually are of two main types:

- (a) Landsliding on steep slopes, and
- (b) Failures in flat, low-lying areas.

Both types are described by many authors. For example, landsliding on steep slopes has been described by Keefer (1984), Wilson and Keefer (1985), and Hansen and Franks (1991); the latter by Seed (1968), Youd and Perkins (1978), Seed and Idriss (1982), and Youd (1991). Failures on gentle slopes or flat-lying areas are usually attributed to the process of soil "liquefaction". This process is independent of any ground failure movements that might follow the transformation to a liquefied state. The four types of ground failures and typical ground damage effects due to liquefaction are summarised in Table 1.

Table 1 Typical ground failures and damage caused by soil liquefaction (after Tinsley et al., 1985)

<b>LIQUEFACTION FAILURE MODE</b>	<b>TYPICAL GROUND DAMAGE AND EFFECTS</b>
<b>1. LATERAL SPREADS</b>	Small to large lateral displacements of surficial blocks of sediments, on gentle slopes (< 3°). Movements, commonly of several metres to tens of metres, are usually toward a free face, particularly an incised stream channel, canal, or open cut. Particularly damaging to pipelines, bridge, structures with shallow foundations, particularly on flood plains adjacent to river channels.
<b>2. FLOW FAILURES</b>	Flow failures, the most catastrophic mode of liquefaction failure, are usually developed on slopes greater than 3°, with movements ranging from tens of metres to several km, at very rapid velocities. Such flows can be very large, and are highly damaging to all structures located on them, or in their paths.
<b>3. GROUND OSCILLATION</b>	Occurs when liquefaction occurs at depth, on slopes that are too gentle for lateral displacement, or are confined. Produces visible ground oscillation waves, ground settlements, opening and closing of fissures, ejections of sand and water from cracks and fissures (sand "boils"). Overlying and subsurface (pipes, tanks, etc.) structures often damaged, usually relatively minor compared to other failure modes.
<b>4. LOSS OF BEARING STRENGTH</b>	Strength loss caused by liquefaction can cause ground collapse and settlements; structures may settle and topple, and buried structures (pipelines, septic tanks, etc.) may rise to the surface. Spreading and collapse of embankment fills often occurs due to liquefaction of foundation soils.

In the past sixty years effort has been made to understand and predict liquefaction susceptibility and ground failures and hazards caused by liquefaction, as well as earthquake-induced landslides. Hansen and Franks (1991) suggest that efforts to understand liquefaction are because large areas within densely populated cities, particularly "port" cities such as Kobe in Japan (Park *et al.* 1995) (and now Christchurch), are often underlain by potentially liquefiable sediments, and these areas are regarded as vulnerable (see Appendix 2 for some case studies). This is the case in many New Zealand towns and cities where ground failures due to liquefaction at MM intensities of 7 or greater are likely to be serious and costly because of the essential infrastructure facilities, services and structures that are at risk. Liquefaction associated with two earthquakes focussed international concern and research.

The 1964 M7.5 Niigata (Japan) earthquake generated widespread and spectacular building and ground damage as a consequence of soil liquefaction. Loss of bearing strength of soils caused buildings to settle and tip, including several 4-storey apartment blocks in the Kawagishicho complex that were otherwise structurally undamaged by shaking effects of the earthquake. Liquefaction-induced lateral-spreading displacements of up to 10 m tore apart buildings, sheared piles, severed pipelines, compressed or collapsed bridges, and caused general destruction to the affected areas. Overall, liquefaction induced ground-failures caused severe damage to tens of square kilometres of Niigata and its environs (Hamada *et al.* 1986).

In the same year the M8.3 Great Alaska earthquake triggered liquefaction that resulted in large flow failures that demolished port facilities in Valdez, Seward and Whittier and carried large parts of those towns into the sea. Earthquake shaking and flow slides in turn generated seiches in surrounding bays, some of which over-ran coastal areas, creating additional damage and caused many deaths. The earthquake also caused numerous lateral spread failures that severely damaged the Alaskan highway and railway systems, and damaged 266 bridges, many beyond repair. That destruction prevented use of much of the highway and rail systems for months following the earthquake, greatly adding to the disruption caused by the event.

Similar severe effects have been reported during many other large earthquakes (see Appendix 2 for some case studies). Some well documented cases (by no means a complete list) of damage due to liquefaction effects during recent historical earthquakes are as follows:

- Destruction of sea walls and port facilities during the 1960 M9.5 Valdivia Earthquake in Chile (Duke and Leeds, 1963).
- Development of large landslides on gentle slopes, and partial failures and significant damage to earth dams during the 1971 M6.6 San Fernando Earthquake in California (Seed, 1972).
- Damage to buildings, roads, bridges, pipelines and farmland during the 1976 M7.8 Tangshan Earthquake in China (Yong *et al.* 1988).
- Widespread damage to roads, pavements, marina, harbour and airport facilities, buildings, utilities and services, with associated ground damage during the 1989 M6.9 Loma Prieta (California) Earthquake (Housner *et al.* , 1990; Seed, 1990).
- Considerable damage to buildings, bridges, roads and pipelines due to failure of liquefaction-affected soils adjacent to rivers and coastal areas, and on loose sandy fill during the 1990 M7.8 Luzon Earthquake in the Philippines (Hopkins *et al.* 1991).
- Liquefaction damage was recorded by NZ reconnaissance teams in Japan (1993 M7.3 Hokkaido-Nansei-Oki, Butcher *et al.* (1994); and the 1995 M7.3 Great Hanshin earthquakes.
- The 1995 Great Hanshin (Kobe) Earthquake caused extensive structural and liquefaction damage in a diverse, highly developed residential, city, industrial and port area which has been extensively studied (i.e. Park *et al.* 1995 and Brunnsden *et al.* 1996).
- The 2009 M7.5 Padang Earthquake resulted in liquefaction, landslides, building and infrastructure damage in the Padang area of North Sumatra (Bothara, *et al.* 2009).
- Records of liquefaction from large NZ earthquakes, as summarised by Hancox (1997) and by Allen *et al.* (2010).
- Liquefaction processes associated with the 2010 Darfield Earthquake caused an estimated NZ\$4 billion in damage to residential properties, infrastructure (mainly buried pipe networks, roads and bridges), and some commercial properties (Allen *et al.* 2010). Subsequently, the 2011 M6.3 Christchurch Earthquake exacerbated previous damage and resulted in extensive new liquefaction and damage. The Reserve Bank Governor, Alan Bollard announced that the damage due to Christchurch Earthquake was estimated to be NZ\$15 billion and of this sum, \$9 billion of the damage was to residential housing, \$3 billion to infrastructure and \$3 billion to other buildings. Of the housing and

infrastructure damage, we estimate that some 75% was due to liquefaction. Thus in total, we estimate that at least \$10 billion of damage was caused by liquefaction.

There are numerous publications and well documented records of damage, intensity and liquefaction effects in localised areas of soft soils, and numerous strong motion recordings from the Loma Prieta earthquake (e.g. as described by Housner *et al.* 1990). These are a valuable record of the event which is relevant to this liquefaction assessment. Similarly the Darfield and Christchurch earthquakes will provide a new body of strong-motion and other evidence relating to liquefaction.

Before the two recent highly damaging earthquakes in Canterbury, the most widespread historic liquefaction recorded in New Zealand since European settlement in c. 1840, was caused by the 1848 M7.8 Marlborough, 1855 M8.2 Wairarapa, 1931 M7.8 Napier and 1987 M6.5 Edgecumbe earthquakes. Liquefaction associated with these earthquakes occurred in coastal regions where plentiful saturated, well-sorted, fine-grained Holocene alluvial deposits are present (Fairless and Berrill, 1984; Hancox, 1997). The Edgecumbe and Christchurch earthquakes were smaller but shallow events close to low-lying areas underlain by saturated soils of alluvial and estuarine origin with high liquefaction potential. Other than these two earthquakes, liquefaction in New Zealand has been reported for many earthquakes of M6.9 or greater. However, perhaps in part due to its inland location, our searches of various archival sources reveal only one recorded instance of liquefaction in Palmerston North (see Section 2.4). The influence of soils on the behaviour of structures during earthquakes received very little attention from the geotechnical community before the catastrophic failures that occurred in Niigata and Alaska during 1964. The remarkable relationship between the intensity of structural damage and local soil conditions in Caracas, Venezuela during the 1967 Caracas earthquake stimulated great interest in the general field of earthquake soil dynamics. Fostered by public concern for the safety of nuclear power plants, the safety of dams, and the USA National Science Foundation programme on Earthquake Hazards Mitigation, liquefaction studies and research grew dramatically. They now represent a significant part of the work of many geotechnical engineering companies and research organisations. In New Zealand they have been the focus of renewed attention following the 2010 Darfield Earthquake.

## **2.4 Assessment of liquefaction Potential**

We have developed and used a number of existing spatial databases to compile a new map of liquefaction potential of the Palmerston North City area. Analysis of lidar-derived digital elevation models (DEMs) provide information on areas of deposition of young, potentially liquefiable materials. Soil and rock maps provide further information relevant to liquefaction. Drillhole logs and geotechnical investigations (CPT and SPT) help characterise subsurface materials. Available information on static groundwater elevations has also been compiled from topographic data and available drill-hole logs. Finally, qualitative experience from liquefaction in and around Christchurch resulting from both the 2010 Darfield and 2011 Christchurch Earthquakes has been applied in developing this report and maps.

The assessment of liquefaction potential involves two general steps (Table 2):

- (1) Evaluation of liquefaction **susceptibility**. This involves the identification of those areas or layers which have the characteristics of liquefiable materials and subsoil materials.
- (2) Evaluation of liquefaction **opportunity**. This involves determination of the occurrence of earthquake shaking strong enough to generate liquefaction in susceptible materials.

Liquefaction potential is site dependent. Certain soils and subsurface materials are more prone to liquefaction than others. Saturated, unconsolidated, relatively uniform fine sands or coarse silts at depths less than 20 m are most prone to liquefaction.

For particular sites or regions, information on liquefaction susceptibility and opportunity are assessed and considered together to determine the liquefaction potential, giving an indication of the relative likelihood that liquefaction will occur. Liquefaction susceptibility is a function of soil properties and water saturation, whereas opportunity is a function of the number of seismogenic sources in the region, and the periodicity of earthquake generation.

Table 2 Table illustrating the liquefaction hazard assessment process

<b>LIQUEFACTION SUSCEPTIBILITY</b>	<b>+</b>	<b>LIQUEFACTION OPPORTUNITY</b>	<b>=</b>	<b>LIQUEFACTION POTENTIAL</b>
a function of the soil's ability to resist liquefaction. This depends on: -soil type (grain size & composition) -relative density -water table depth -depth of soil layer	+	a function of the intensity of seismic shaking. This depends on: -magnitude and proximity of the earthquake -frequency of events $\geq$ MM VII -duration of shaking	=	<b>LIQUEFACTION POTENTIAL</b>
<b>NOTE: LIQUEFACTION HAZARD = LIQUEFACTION POTENTIAL + GROUND FAILURE POTENTIAL.</b>				

Prior to assessment of liquefaction hazard, we will provide some information on the geomorphology, soils, geology, historical earthquakes and seismogenic structures of the Manawatu region and Palmerston North City area.

### 3.0 SOILS AND GEOLOGY OF PALMERSTON NORTH

#### 3.1 Palmerston North Regional Tectonic Setting

Palmerston North City area lies about 200 km northwest of the Hikurangi Trough, the ocean-floor surface expression of the Australian-Pacific plate boundary (Fig. 3). The two plates move relative to one another on the surface of the earth, and are converging obliquely at a rate of about 40 mm/year (DeMets et al. 1994). Much of the strain associated with the collision occurs within the landmass of the North Island (Spörli 1980; Beanland 1995; Collot et al. 1996).

At the North Island the Pacific Plate dips beneath the Australian Plate, such that the fault interface (also known as the subduction interface) between the plates lies at a depth of c. 30 km beneath Palmerston North (Reyners et al. 2006). Strain associated with the plate boundary is transferred up through the over-riding Australian Plate to the ground surface in

the form of active faults and folds (Nicol et al. 2007). This deformation, both the broad regional strain and more local deformation associated with faults and folds, significantly influences the geological environment in the Palmerston North area. This geological influence is described below.

## **3.2 General Physiography**

The landforms of the Palmerston North City area, here substantially modified after Heerdegen & Shepherd (1982), comprise five geomorphic areas (Figure 4), the axial ranges; elevated southeast terraces, the Manawatu River floodplain, the elevated northwest terraces, and the low-lying plains.

The landforms of the entire city area are exclusively geologically young having emerged from the sea in less than the last million and a half years (including the axial range) and mostly within the last 125,000 years. The geomorphic characteristics of these areas are discussed in more detail in Appendices 3 and 4.

### **3.2.1 Historical Archived Maps**

The Palmerston North City Archives house valuable historic information on the city area. For this study an 1885 index plan drawn by city surveyors is useful for showing historic drainage and swamps over which city development now extends. Plans showing the development of the city over the years of European settlement are presented in Matheson (2003). We have shown the old areas of river and stream channels, swamps and cut-off oxbow ponds on Figure 8c.

## **3.3 General information on Lidar data**

Lidar (light detection and ranging) is a modern method of aerial mapping carried out by flying slightly overlapping lines across an area using a sophisticated laser distance scanner to very accurately measure the relative elevation of the ground and other surfaces. The data can be processed to provide an extremely accurate map of ground surface topography. Lidar data were collected over select areas of Palmerston North City Council and the Horizons Regional Council areas between November 2005 and February 2006. The airborne surveying was conducted by AAMHatch Pty Ltd., using an Optech ALTM3100 Lidar sensor at an operational frequency of 71 kHz. The horizontal accuracy of returned points was 0.55 m while vertical accuracy was 0.15 m. The lidar data were supplied to GNS Science in .las binary format. Following processing to remove points returned from materials above the ground surface, the three-dimensional point cloud data were interpolated to raster format to create a digital elevation model (DEM) of the ground surface at one metre spatial resolution. This very accurate elevation data were used in combination with a derivative hillshaded relief model of the landscape to accentuate geological and geomorphological features of relevance to liquefaction.

The accuracy and clarity of the Lidar-derived model (Figure 5; Appendix 4) has helped refine our understanding of the geomorphology of the PNCC area covered, and has played four key roles in our analysis of liquefaction susceptibility:

- Delineation of river channels and floodplains (Figure 5);
- Refinement of existing geology and soil map boundaries;

- Identification of a new fault line (Figure 6); and
- With drillhole logs from Horizons Manawatu, allowed spatial numerical modelling of subsurface materials, identifying areas likely to have thick soil materials which may be susceptible to liquefaction.

### 3.4 Soils of Palmerston North

Palmerston North, as depicted in Fig. 6 and Appendix 5, is founded on a variety of soil types. High quality soil maps cover some parts of Palmerston North City (Cowie, 1974; Cowie & Kimpton, 1976; Cowie, 1979; Figure 6). These soil types have characteristics that contribute to understanding liquefaction potential. Grading curves of soils within the PNCC area have been obtained from Soil Bureau archives and typical curves are presented in Appendix 6.

The soils of Palmerston North are described broadly within two groups:

- The soils of the river flats and
- Soils of the terrace land.

The predominant soil types of each group are described in some detail in Appendix 5.

Soils of the terrace land (especially the Milson, Marton and Tokomaru soils) are relatively old (thus are well graded with abundant interstitial clays and have almost certainly experienced repeated strong ground shaking in past earthquakes – see Section 4.3.2), and are elevated (and thus relatively well-drained). For these reasons, they are considered unlikely to liquefy (generally negligible liquefaction susceptibility). Soils of the river flats, especially the Rangitikei and Manawatu series, where they are thick, young and relatively poorly drained may be susceptible to liquefaction (moderate to very high liquefaction susceptibility). The soil map suggests large parts of the Palmerston North City area are unlikely to be prone to liquefaction, and identifies specific areas where liquefaction is a possibility.

From the grading curves shown in Appendix 6 we can make the following generalisations:

The Manawatu sandy loam has very highly liquefaction potential.

The Kairangi fine sandy loam has negligible to moderate liquefaction potential.

The TeArakura fine sand has moderate to high liquefaction potential.

The remaining soils in the PNCC area have negligible liquefaction potential. The location of all the soils in the PNCC area are clearly shown on Figure 6.

### 3.5 Geology of Palmerston North

The major rocks types occurring within the Palmerston North City area are discussed here in general terms (after Begg & Johnston 2000, Lee & Begg 2002, Townsend *et al.* 2008). Geological units exposed at the surface (Figure 8) are subdivided into:

- Basement rocks – exclusively Mesozoic greywacke rocks;
- Late Neogene and Early Pleistocene marine deposits;
- Early and middle Pleistocene non-marine and marine deposits;
- Last interglacial marine deposits;

- Early and middle last glacial alluvial deposits; and
- Holocene alluvial deposits.

Descriptions of these rocks may be found in Appendix 7.

In general terms the basement, Late Pliocene and early Pleistocene rocks are lithified or relatively well consolidated and will not liquefy under strong ground shaking. Because of their age, the early and middle Pleistocene non-marine and marine deposits, the last interglacial marine deposits, and the alluvial materials of the early and middle last glaciation are old enough to have been consolidated by natural processes (including soil development); their age also suggests they have experienced many episodes of strong earthquake shaking (Section 4.3.2). Their liquefaction susceptibility is regarded as negligible.

The most likely materials to liquefy are those low-lying materials of Holocene age, particularly loose sands and silts which are water saturated. On the basis of the geological map, large parts of the Palmerston North City area are unlikely to liquefy. Like the soil map, the geological map provides strong indications that large parts of PNC are unlikely to be susceptible but with restricted areas that may be susceptible to liquefaction. The geological map alone does not have sufficient detail to discriminate between Holocene materials susceptible to liquefaction from those that are not. Subsurface information on thickness of sand and silt in the sub-soil, and elevation of the static groundwater level is required before attempting to assess this.

The Manawatu region has a number of surficial active faults (Begg & Johnston 2000; Lee & Begg 2002). Many of those in the west are reverse faults that fail to reach the surface and are associated with actively growing folds, commonly asymmetric, with sigmoidal hinges and one limb dipping more steeply than the other (Te Punga 1957; Jackson *et al.* 1998). Ongoing deformation has long restricted the major drainage catchments to valleys between the fold crests (Begg *et al.* 2005). The crests of the anticlines are elevated and well-drained and therefore have reduced (negligible) susceptibility to liquefaction.

## **4.0 EARTHQUAKES**

### **4.1 Measuring Earthquake Strength**

The energy released by an earthquake, often quoted as its “size”, is described using Magnitude (M). Seismic waves which carry the energy released by the earthquake attenuate with distance from the source, so only part of the energy released is felt at the surface, and this energy dissipates with distance from the epicentre. The rate of attenuation depends on the earthquake’s magnitude, depth, the distance from the source, and the rock and soil materials beneath the surface.

The Modified Mercalli Intensity (MMI) scale provides a subjective scale, often described as “felt intensity”, for measuring an earthquake’s ground-shaking characteristics. For example, the 2010 M7.3 Darfield Earthquake was shallow, centred near Darfield, c. 30 km from Christchurch, and was accompanied by a ~30 km surface rupture extending in an east-west direction across the Canterbury plains from Greendale towards Christchurch (Van Dissen *et al.* 2011). This earthquake caused ~MM8 intensity in the CBD area of Christchurch. By comparison the smaller magnitude 2011 M 6.3 Christchurch Earthquake, also shallow, but

with its epicentre much closer to Christchurch City (c. 10 km), caused far more damage, building collapse, casualties and liquefaction in the CBD area. It was associated with ground shaking of MM9 to MM10 intensity in the CBD. Summary and full descriptions of the MMI scale are presented in Appendix 9.

## 4.2 Earthquake and liquefaction history of Palmerston North

### 4.2.1 Historic earthquakes felt in the Palmerston North area

Historic, strong earthquakes affecting the Manawatu area are summarised below.

- **M8.2, Wairarapa, January 23 1855.** The 1855 earthquake is the most severe earthquake to have occurred in New Zealand since systematic European colonisation began in 1840 (GeoNet, 2011). In the Palmerston North area the felt intensity was about MM8. Although Palmerston North City did not yet exist, numerous slump cracks, sand craters and ground water liberation were observed and reported in the greater Manawatu area where the MM Intensity was at least 8 and may have reached 9 (Downes, 1995).

Since European settlement in Palmerston North City area (post 1864), there have been three earthquakes which may have caused MM7 Intensity ground shaking. just strong enough to have generated liquefaction. However, there has been only one known report of liquefaction occurring in the city area (during the 1934 Pahiatua Earthquake).

- **M7.8, Hawke's Bay (Napier), February 3 1931.** The 1931 Hawke's Bay earthquake caused the largest loss of life and most extensive damage of any quake in New Zealand's recorded history (Geonet, 2011). The felt intensity of this earthquake in Palmerston North was between MM6-7, just below the intensity required for liquefaction. (Downes, 1995).
- **M7.6, Pahiatua, March 5 1934.** The 1934 Pahiatua earthquake shook the lower North Island on March 5 1934 and was felt as far away as Auckland and Dunedin (Geonet, 2011). The felt intensity in Palmerston North was MM7. Most of the ground and infrastructure damage reported was close to the epicentre in the town of Pahiatua and on the east coast (Downes, 1995). Reports of liquefaction-induced ground damage from the Manawatu-Whanganui Region include sand boils, ground cracking and subsidence at Foxton, and subsidence in Palmerston North and Eketahuna (Dellow et al. 1994).
- **M7.2, Masterton (Wairarapa) I, June 24 1942.** This earthquake severely rocked the lower North Island on June 24 1942, causing extensive damage to local buildings (Geonet, 2011). Ground shaking intensity in Palmerston North City reached MM7, however most damage was reported in Wairarapa and Wellington regions (Downes, 1995). Damage near Palmerston North was reported in Longburn where bridge approaches suffered subsidence/collapse (Downes, 2001).
- **M7.0, Masterton (Wairarapa) II, August 2 1942.** The shock that struck the Wairarapa Region on August 2 was nearly as severe as the June 24 earthquake five weeks earlier (Geonet, 2011). The intensity of this earthquake event in Palmerston North reached MM6-7. Again most damage was reported in Wairarapa and Wellington regions (Downes, 1995).

## 4.3 Liquefaction opportunity - known seismogenic structures in the PNCC region

Numerous active faults and potentially seismogenic structures are mapped within a 40 km radius of Palmerston North City (Begg & Johnston 2000, Lee & Begg 2002; Townsend *et al.*

2008; GNS Science Active faults database). Table 3 below lists their general characteristics, but each source is not discussed in detail. Each has the potential to generate felt intensities of MM7 or more in the Palmerston North City area.

In this report, we specifically include all known earthquake sources located within 40 km of the centre of Palmerston North City and a few at a greater distance. The 40 km radius was chosen to ensure that earthquakes that may be generated by the subduction interface at depth are included, and to limit the extent of seismogenic structures to those which are likely to produce the strongest shaking and are thus most important to Palmerston North City.

However, strong ground shaking that may cause liquefaction and some structural damage (i.e. at least MM7) in the Palmerston North City area could occur in association with major earthquakes centred >100 km away. This occurred in 1855 when the M8.2 Wairarapa Earthquake, whose epicentre was ~115 km from Palmerston North, caused MM8 to 9 shaking in the city area, the strongest in Palmerston North since European occupation. In addition, there are likely to be unknown seismogenic sources not listed in Table 3. These are covered largely by the probabilistic seismic hazard model (Stirling *et al.* 2002), as discussed below.

#### 4.3.1 Estimated fault recurrence intervals

The concept of recurrence interval for seismogenic structures is a worldwide research issue and the link between strain accumulation, single event slip values, fault rupture lengths, rupture plane areas and seismic moment (Gutenberg & Richter 1944; Schwartz & Coppersmith 1984; Wells & Coppersmith 1994; Wesnousky 1994; Anderson *et al.* 1996). This is important in the context of this report because, while we assume (on the basis of single event displacements, where measurable) that these faults rupture periodically with approximately subequal magnitude earthquakes, it is possible that some, or even significant amounts of accumulating strain may be released in a number of smaller earthquakes. If the latter is true, recurrence time between felt intensities large enough to generate liquefaction will be greater.

Table 3 Important known seismogenic structures of the Palmerston North City area and their characteristics.

**rv** = reverse, **ss** = strike-slip, **nn** = normal, **sn** = strike-slip with normal component; **sr** = strike-slip with reverse component.

FL = fault length, SED = single event displacement, RI = recurrence interval.

\* represents a fault length estimate made for this report on limited available data.

‘ indicates revised SED value in updated probabilistic seismic hazard model is significantly greater.

“ indicates revised SED value significantly lower.

^ indicates revised value in updated probabilistic seismic hazard (Stirling *et al.* 2010) model is significantly shorter.

~ indicates revised value is significantly longer.

Fault	Location /comment	Dist. from PNC (km)	Type	FL (km)	Likely EQ Mag	SED (m)	Slip rate (mm/yr)	RI (yrs)	Reference
Mascarin-Turakina Fault Zone	Offshore-coastal NW Manawatu	40	rv	92 - 102	M7.3 to 7.5	2.4'	0.06	c. 18000^	Lamarche <i>et al.</i> 2005; Nodder <i>et al.</i> 2007
Onepoto Fault	Offshore SWB	90	rv	78	M7.2 to 7.4	2.3'	0.4	568 to 3991	Lamarche <i>et al.</i> 2005; Nodder <i>et al.</i> 2007
Rangitikei Fault Zone	Offshore SWB	40	rv	27	M6.9 to 7.1	1.6'	0.7	1147 to 9578	Lamarche <i>et al.</i> 2005; Nodder <i>et al.</i> 2007
Otaheke Fault	Offshore SWB probably =	50	rv	38	M7.0 to 7.2	1.7'	1	1056 to 2471~	Lamarche <i>et al.</i> 2005; Nodder <i>et al.</i> 2007
Leedstown-Putorino Fault	Leedstown-Putorino Fault	30	rv		M6.7	2.5	0.3	8000	Stirling <i>et al.</i> 2002
Leedstown-Putorino Fault	Bulls-Marton entirely sub-surface, significance uncertain	28	nn	35*	M6.3	0.6	0.07	9164~	Stirling <i>et al.</i> 2002
Poroutawhao Fault	Halcombe-Glen Oroua	36	rv		M6.8	2.5	0.1	20000	Stirling <i>et al.</i> 2002
Mt Stewart-Halcombe structure	Halcombe-Glen Oroua	13	rv	25*	M6.8	2.5"	0.3	8000^	Jackson <i>et al.</i> 1998; Stirling <i>et al.</i> 2002
Himatangi structure	Carnarvon-Oroua Downs	21	rv	15*	M6.7	2.5"	0.3	8000^	Stirling <i>et al.</i> 2002
Feilding structure	Feilding – Beaconsfield	23	rv	12*	M6.9	2.5"	0.3	8000	Stirling <i>et al.</i> 2002
Levin structure	Levin – Koputaroa Takapari Rd-	30	rv	10*	M6.6	2.5"	0.3	6500^	Stirling <i>et al.</i> 2002
Pohangina structure	Palmerston North	?<5	rv	40*	M6.9	2.5	0.3	8000	Stirling <i>et al.</i> 2002
Ruahine (reverse) fault	possible = Pohangina structure	10	rv		M6.5			20000	Stirling <i>et al.</i> 2002
Probable new fault near Linton	Tokomaru-?Palmerston North	<8	?sn	>7					
Shannon structure	Shannon	26	rv	6*	M6.6	2.5"	0.1	20000^	Stirling <i>et al.</i> 2002
Northern Ohariu Fault	Otaki-Kahuterawa	12	ss	60*	M7.2	3.5	1.4	2500	Stirling <i>et al.</i> 2002
Moonshine-Otaki Fault	Akatarawa - Otaki Forks E Manawatu	38	sr?	75*	M7.2			125000	Stirling <i>et al.</i> 2002
Ruahine (South) Fault	Gorge-Te Hoe River	16	sr	60*	M7.2	3.5'	1.3	2800~	Stirling <i>et al.</i> 2002
Wellington NE - Mohaka Fault	S Wellington-Whakatane possibly =	14	ss	70*	M7.5	4.2"	7.1	592~	Stirling <i>et al.</i> 2002
Maunga	Woodville & Pahiatua faults	22	sn		M6.8			5000	Stirling <i>et al.</i> 2002
Pa Valley-Makuri Fault	Ihuraua - Mt Marchant	35	ss	38*	M7.4	6"	2.4	2500	Stirling <i>et al.</i> 2002
Mangaoranga Fault	Mangaoranga – Pleckville	38	sn		M6.1			5000	Stirling <i>et al.</i> 2002
Alfredton Fault	Ihuraua - Mt Marchant	39	ss	72*	M7.2	6"	3	4500^	Stirling <i>et al.</i> 2002

Fault	Location /comment	Dist. from PNC (km)	Type	FL (km)	Likely EQ Mag	SED (m)	Slip rate (mm/yr)	RI (yrs)	Reference
Subduction interface	Australia-Pacific plate boundary at depth	25 (vert.)	rv	very large	characteristics uncertain; may be capable of generating very large earthquakes;				recurrence interval uncertain

#### 4.3.2 Modelled Recurrence Interval of strong ground shaking

Using the current probabilistic seismic hazard model (W. Smith, modified after Stirling *et al.* 2002), recurrence intervals of strong ground shaking of various MM intensities can be calculated. This model (Table 4) predicts that MM7 shaking intensity, the threshold to trigger liquefaction, will occur in Palmerston North every few decades.

Table 4 Table illustrating recurrence intervals of various MMI intensities for Palmerston North City (calculated by N. Pondard using a model developed by W. Smith from the probabilistic seismic hazard model of Stirling *et al.* 2002).

Palmerston North (-40.35, 175.61)						
MMI	5	6	7	8	9	10
Return Period (yrs)	3	7	28	129	714	14,286

## 5.0 LIQUEFACTION SUSCEPTIBILITY

As noted in section 2.4, liquefaction hazard is a function of liquefaction susceptibility and opportunity. It is apparent from the historical earthquakes and the number of active faults located within 40 km of the city, that there are numerous potential seismic sources capable of producing ground shaking of MM7 or greater, sufficient to cause liquefaction in Palmerston North City, if susceptible soils are present. We have demonstrated that the seismic opportunity is high for Palmerston North.

But soil susceptibility is known to be variable, with widespread modest to low potential, but with a few areas where it may be high and very high. Soils close to former stream channels (the Mangaone, Kawau, Little Kawau, Terrace, Awatea and "Unnamed" Streams; see Matheson 2003), swampy depressions and lagoons (former channels and cut-off oxbows) close to the Manawatu River (the Awapuni, "Unnamed", Awatapu, Hokowhitu and Te Ngutu lagoons; Matheson 2003) are likely to contain soils susceptible to liquefaction. Street layout figures in Matheson (2003) show that streams have been diverted, modified and channelled since Palmerston North's founding in 1886. The city has gradually expanded into areas that have been modified, particularly the old Manawatu flood plain and channels and the cut-off oxbow lagoons, areas that may include places where liquefaction susceptibility is locally high to very high. The following work has been completed to identify areas that may include materials with liquefaction potential.

## 5.1 Conventional liquefaction map

Dellow *et al.* (1994) used deterministic criteria to compile an indicative liquefaction susceptibility map using the characteristics of different geological units and available geotechnical data. They classified geological units (on the basis of their known materials) into four zones, ranging from negligible liquefaction susceptibility (Zone 4) to those with high liquefaction susceptibility (Zone 1) (see Figure 8a).

The data used in generating our revised liquefaction ground damage potential map includes slightly modified versions of the 1:250,000 QMAP geological maps (Begg & Johnston 2000; Lee & Begg 2002), the soil map of Cowie & Kimpton (1976), supplemented by the drillhole database of Horizons Manawatu Regional Council and unpublished geotechnical data from the former Soil Bureau which is held at GNS, and Lidar topographic maps (Section 3.3).

We have adopted a 5 fold subdivision based on that presented by Environment Canterbury and BECA for the Christchurch area

(<http://ecan.govt.nz/publications/General/solid-facts-christchurch-liquefaction.pdf>).

The rationale for using a 5 fold scale is to encourage standardisation to a well defined, single national scale representing liquefaction ground damage potential. The ECan/Beca scale was developed prior to the recent Canterbury earthquakes and is better defined than most alternative scales published in New Zealand to date. The five classes range from Very High to Low and Negligible Liquefaction Ground Damage Potential, as described below:

1. **Very High Liquefaction Ground Damage Potential:** Area may be affected by lateral spreading and significant ground subsidence. Subsidence is likely to be greater than 300 mm.
2. **High Liquefaction Ground Damage Potential:** Area may be affected by significant ground subsidence. Subsidence is likely to be greater than 300 mm.
3. **Moderate Liquefaction Ground Damage Potential:** Areas may be affected by 100-300 mm of subsidence.
4. **Low Liquefaction Ground Damage Potential:** Areas may be affected by up to 100 mm of ground subsidence.
5. **Negligible Liquefaction Ground Damage Potential:** Areas where liquefaction is not expected to occur.

Christchurch is located close to the sea. Its eastern suburbs are partly underlain by unconsolidated Holocene marginal marine and estuarine deposits highly susceptible to liquefaction. By comparison Palmerston North is an inland city built largely upon Last Glacial and Holocene alluvial deposits materials more similar to those located to the west of Christchurch CBD. The 5-fold scale developed for Christchurch is therefore applicable to Palmerston North and we have preserved it as a qualitative scale for liquefaction ground damage potential. This 5-fold scale ranges from very high liquefaction ground damage potential (Zone 1) to negligible liquefaction ground damage potential (Zone 5) is used in Figure 8b.

Because the alluvial depositional environment is less laterally persistent than marginal marine and estuarine environments, for Palmerston North the moderate to high liquefaction ground damage potential end of the scale is difficult to map within a single zone. For example an alluvial channel abandoned by the Manawatu River may be 5 m deep, 100 m

wide and hundreds of metres in length, and may be infilled by saturated fine grained material in an ox-bow lake situation, resulting in a high or very high liquefaction potential. This material may be surrounded by alluvial gravel that has limited liquefaction potential. Furthermore, in some cases, these local deposits may be indistinguishable in surface morphology. So accurate distinction between moderate, high and very high liquefaction potential here is difficult. For this practical reason, we characterise the higher end of the scale into broader groupings. The zones depicted on our map (Figure 8b) are effectively a four-fold classification but could be expanded to a five-fold classification, similar to that used in Christchurch, as more data becomes available:

- Zone 1-3 Moderate to Very High Liquefaction Ground Damage Potential
- Zone 2-3 Moderate to High Liquefaction Ground Damage Potential
- Zone 4 Low Liquefaction Ground Damage Potential
- Zone 5 Negligible Liquefaction Ground Damage Potential.

#### **5.1.1 Zone 5 – Negligible Liquefaction Ground Damage Potential**

Land underlain by elevated and well-drained deposits older than Holocene in age are considered to have a Negligible liquefaction susceptibility (equivalent to the Dellow *et al.*'s Zone 4; our Zone 5). The area underlain by these deposits includes a number of different geological units that include lithified basement rocks (Torlesse composite terrane), and consolidated late Pliocene marine deposits, early Quaternary alluvial gravels, middle and early late Quaternary marginal marine beds, and early last glacial alluvial deposits. These geological units cover much of the Palmerston North City area. Some of these geological materials are overlain by fine grained soil deposits (including tephra and loess), but because they are elevated and well-drained, pose little potential for liquefaction. Most of the geological units were classified within Zone 4 of Dellow *et al.* (1994), but they recognised a zone of weak marine deposits south of Bunnythorpe that they considered provisionally to have low liquefaction susceptibility (their Zone 3). The lidar DEM indicates that this zone is elevated and drillhole/geotechnical data show that cover materials are between 1 and 3.5 m thick. Depth to static groundwater for the few data points available is between 3.5 and 5.5 m. These data suggest that negligible liquefaction susceptibility is more appropriate.

#### **5.1.2 Zone 4 – Low Liquefaction Ground Damage Potential**

A second geological group, late last glacial alluvial deposits, are also elevated on terrace lands, and comprise coarse and compact gravel and sand, with a single, thin overlying loess and soil that is generally well drained. These materials are considered to have a low to negligible liquefaction potential, and are conservatively classified here as Zone 4 (Zone 3 of Dellow *et al.* 1994).

#### **5.1.3 Zones 2 – 3, Moderate to High Liquefaction Ground Damage Potential**

The northwest part of the map area is characterised by a slightly elevated early to middle Holocene river course remnants, leveed stream deposits or low-lying flood plains. The age of these materials ranges from a notional “early to middle Holocene” to modern. This area includes defined (see Appendix 5) soil types of the Kairanga silt loam and Te Arakura silt loam locally of significant thickness (Cowie 1974). On the basis of historical information (Matheson 2003) and lidar data, a part of western Palmerston North City is underlain by

swamp and stream overbank silt materials up to several metres thick. In many places, these soils are fine-grained and occupy low-lying land with relatively high static groundwater levels. These areas are therefore considered to have a moderate to high liquefaction susceptibility (roughly equivalent to Dellow *et al.*'s Zone 2 – Moderate) and we rank them as Zone 2-3.

#### **5.1.4 Zones 1 – 3, Moderate to Very High Liquefaction Ground Damage Potential**

The modern course of the Manawatu River (from immediately prior to settlement is underlain by materials comprising loose sand and gravel alluvium overlain in places by silt, some swamp deposits and locally soil that may be relatively thick. Soil conditions in this area are highly variable, with relict river channels, many partially infilled by fine grained overbank flood deposits. There is potential for relatively deep (at least up to several metres deep) highly liquefiable materials deposited locally in these former river channels and pools. Logs of a number of drillholes within this part of Palmerston North City record soil and silt as thick as 9 m and that the static groundwater level is shallow. Overall we class materials zoned as 1-3 having very high liquefaction susceptibility locally, although we recognise many gravel-filled former river channels may have low liquefaction susceptibility.

Generally, our map corresponds to that of Dellow *et al.* (1994). However, our map differs principally in that we have downgraded liquefaction susceptibility of Dellow *et al.*'s area of Zone 3 in the Bunnythorpe area on the basis of detailed (lidar) topographic data, and recognise the variability of deposits of the Manawatu Plains area from Flyers Line to Longburn on the basis of drillhole and limited geotechnical data. As well, we recognise the possibility of localised pockets of fine-grained materials within the Manawatu River floodplain that may have high liquefaction susceptibility, so have zoned that area Zone 1-3.

## **5.2 Liquefaction susceptibility incorporating subsurface information**

A comprehensive desk study has been conducted to assemble, review, summarise and assess available sub-surface drillhole and geotechnical information from the Palmerston North area. Drillhole data from over 1582 boreholes obtained from Horizons Regional Council and GNS archives, includes lithologies, soil penetration tests (SPT, CPT, Scala PT) and representative soil analysis held by the former Soil Bureau, DSIR. These data have been compiled in GIS format to provide a database of easily retrievable sub-surface information. Sub-surface data from the Totara Road Wastewater Treatment Plant were obtained from the June 2006 MWH engineering report commissioned by PNCC.

Our aim is to develop models of subsurface materials that will supplement and support the conventional liquefaction map. These models will provide an element of geotechnical quantification of liquefaction potential zones on a conventional map. As well the models directly incorporate information derived from drillhole and geotechnical information and more accurately characterises thickness and properties of the subsurface coverbeds (those finer grained, weak/soft materials overlying the uppermost gravels) and water table levels.

### **5.2.1 Horizons Manawatu drillhole database**

The Horizons Manawatu drillhole database includes static groundwater depth measurements, collar height RL's and downhole lithological information. Despite the fact that the database is heterogeneous, containing variable quality data supplied by many contributors, it is the most comprehensive information on subsurface materials available for the Palmerston North area.

### 5.2.2 Geotechnical information

There is a relatively small amount of geotechnical information available for the City area but there is a moderate to good distribution of CPT testing. The distribution of scala penetrometer data is poor and confined to the waste water treatment plant. SPT data are few in number with patchy distribution.

The information required to determine the liquefaction susceptibility of a soil is:

- Soil cohesiveness. CPT and STP can be used to determine this. Low SPTn values are a good, internationally used indicator of liquefaction susceptibility;
- Soil saturation. The drillhole database contains available information on static groundwater levels;
- Soil grading. Uniform fine sand and silts are the most susceptible. While cohesive soils (clayey and well graded) are not susceptible to liquefaction, they may be susceptible to cyclic strain softening, a lesser problem. SPT samples can be used to establish grading curves;
- Soils that meet the criteria of being saturated, uniform and fine grained are highly susceptible to liquefaction.

### 5.3 Steps in modelling liquefaction potential using digital datasets

In order to advance from the regional liquefaction assessments made earlier using geology and soils maps with limited geotechnical input (i.e. Dellow *et al.* 1994) we have been able to use a number of digital datasets that are helpful in refining liquefaction potential in the Palmerston North area. Some of these are only recently available and they include:

- A DEM derived from lidar data, and derivative shaded relief models;
- The Horizons Manawatu drillhole database, that records information on the location, collar heights (both with 1582 records), static groundwater elevation (237 records), and downhole from- and to- lithological information (331 drillholes);
- Published geological and soil maps; and
- Geotechnical data (87 CPT records, 7 scala records, 19 SPT records).

When combined, these data provide a significant amount of information useful in quantifying liquefaction potential. The lidar data provides a very accurate topographic model, and assists in interpreting the Holocene (last 10,000 years) depositional environments across the area (e.g. Figure 13 & 14).

Of the 1582 simplified digital well drillhole records ([Wells.shp](#)) within the broader Palmerston North area, 546 are located within the Palmerston North City boundary. Of these, 172 drillholes have associated logs. Drillholes outside the PNC boundary are included in modelling to provide a broader context for the study and help improve model precision in the northwest of the Palmerston North City area.

Because geological, soil and groundwater conditions changes radically southeast of the Manawatu River, and there is a relative paucity of drillhole and geotechnical data there, modelling southeast of the river is poorly constrained and unrealistic. A model barrier

([GroundwaterBoundary.shp](#)) was placed at the southeast side of the Manawatu River to isolate the influence of these data points and no weight should be attached to models southeast of the boundary line.

Information relevant to liquefaction contained within the drillhole log file ([Wells.shp](#)) include:

- Location (NZMG easting and northing)
- Elevation (m – metres above sea level – masl; relative level - RL)
- Depth to water (m below collar)
- Groundwater static level (m amsl or RL)
- Whether water is artesian or otherwise
- Downhole lithologies (from\_ and to\_ values, with lithological descriptors)

The drillhole database contains static groundwater elevations for many locations. Downhole lithological information defines in many places the uppermost gravel depth and elevation, and descriptions of overlying coverbeds. Geotechnical information (especially CPT and SPT logs) provide similar information on the depth of gravels, strength, and to some degree, the nature of overlying coverbeds.

A detailed step by step description of how this digital modelling was undertaken follows.

**Step 1: Assembling point data:** Relevant drillhole and geotechnical data was integrated (to [DrillholeGeotech\\_gravel.shp](#)). This point shapefile provides identification, location, depth to the uppermost gravel, and the RL of this surface for each drillhole/geotechnical location. Note that the file has been cleared of drillholes that recorded the uppermost gravel surface at depths of greater than 40 m, and of drillholes where the RL of the uppermost gravel surface is recorded as above the collar height recorded. The file contains this information for a total of 459 locations across an area that includes the Palmerston North City urban area (269 records within the Palmerston North urban area), and 153 records northwest of the urban area. Thirty seven records lie southeast of the model barrier to the south of the Manawatu River.

**Step 2: Modelling the depth of the uppermost gravel surface:** The point data above ([DrillholeGeotech\\_gravel.shp](#)) was used to model the depth to the uppermost gravel surface (attribute: Grvl\_depth). The resulting modelled surface at the top of the uppermost gravel is [grav\\_depth](#). The depth of the modelled surface represents the thickness of coverbeds resting on the uppermost gravel surface. Several different modelling methods were assessed, including IWD, kriging, splining, and splining with a barrier. The method adopted in this report as most appropriate is splining with a barrier. This method allows data in the area southeast of the river to be isolated from the rest of the model, improving the quality of the model elsewhere in the modelled area. It is important to note that this upper gravel surface is not a stratigraphic or chronostratigraphic horizon. Gravels modelled are of different ages and differing stratigraphic position, but the information is important for liquefaction, as it provides an indication of the thickness of the finer-grained materials (hereafter described as “coverbeds”) that rest upon the uppermost gravel.

The modelled thickness of coverbeds on the gravel surface was then contoured ([grav\\_depth\\_1m.shp](#)). Primary features of the model include the presence of areas of thick coverbeds on gravels in the area immediately southeast of the Oroua River and beneath the

hill country to the north of the city (Nannestad Rd to Setters Line to south of Grove Rd). An area within the city, along strike from the southwest end of the Pohangina Anticline also has a number of “spikes” of high covered thickness (here, the model is influenced heavily by drillholes 336251, 336253, 336257, 336111, 336141, 336005, 336232 and 336391), perhaps resulting from penetration into older middle Pleistocene marginal marine deposits in the subsurface extension of the Pohangina Anticline. This feature lies beneath most of the area designated as the Kelvin Grove development area. Areas underlain by older alluvial deposits, including the central part of the city, are underlain by coverbeds generally less than 2 m thick, although a notably thick “hole” at Terrace End (model influenced by drillhole 336111), presumably represents a site where no gravels overlie the older middle Pleistocene marine deposits present further up the Pohangina Anticline. Southeast of these older gravels, between the central city and the lower river flats (Horowhitu to Tiakitahuna), coverbeds are commonly significantly thicker, up to depths of 15 m. These elevated coverbed thickness extend across the southern end of the Anders Racecourse development area. Coverbed thicknesses beneath the late Holocene floodplain of the Manawatu River are generally thin, but there are some areas (e.g. Te Matai Rd-Koehlers Rd area) where they are of significant thickness (drillhole 336141 and O336/141).

**Step 3: Modelling the uppermost gravel surface RL:** The point data above ([DrillholeGeotech\\_gravel.shp](#); attribute: gravel\_RL) was used to model the RL of the uppermost gravel surface across the area ([grav\\_rl](#)). The modelled surface was contoured ([grav\\_rl\\_1m.shp](#)). Contours with negative values are a modelling anomaly caused by smoothing of the model by local variations in the surface; we assume that gravel is at, or close to the topo surface in these areas. Positive peaks in the north-eastern area are assumed to represent areas where late last glacial and Holocene gravels are absent, and the uppermost gravel modelled is within middle Pleistocene marginal marine materials. A number of “highs” in the Terrace End/Kelvin Grove area (model influenced heavily by drillholes 336251, 336253, 336257, 336111, 336141, 336005, 336232 and 336391) may represent the sub-surface extension of the Pohangina Anticline. In general, modelling confirms the expected general westward inclination in the elevation of the uppermost gravel surface. The surface is not a simple westward inclined one, but includes some features we interpret as buried channel features, including some near the Oroua River area, one in the Cloverlea Rd area and one near State Highway 56 in the Longburn area.

**Step 4: Modelling the static groundwater surface:** Using [gw\\_rl.shp](#) (attribute: Depth\_to\_W), a point shapefile developed from the Horizons Manawatu drillhole data collar location spreadsheet, the depth of the static groundwater was modelled ([gw\\_depth](#)). This shapefile includes drillhole logs that have Depth\_to\_W values of < 20 m and excludes all entries characterised as “artesian”. The modelled surface was contoured ([gw\\_depth\\_1m.shp](#)).

The resulting model reveals that the depth to static groundwater across most of the area lies between -1 and -5 m; in the central city area between the Square and the Manawatu River, it is modelled as relatively deep (c. -4 to -8 m), but that is heavily influenced by a single drillhole (336611). Slightly shallower static groundwater levels may exist in the Anders Racetrack development area and in a belt between Terrace End and Whakarongo. Static groundwater depth in the Kelvin Grove development area is deep, and may reflect the sub-surface extension of the Pohangina Anticline. Depressed static groundwater depths in the southeast city and Koehlers Rd-Te Matai Rd areas may reflect the presence of thick coverbeds of limited permeability in the area. Isolated areas of deeper groundwater depths

between Kairanga and the Oroua River and upper tributaries of Mangaone Stream may also reflect thicker coverbeds of limited permeability.

**Step 5: Modelling the RL of the static groundwater surface:** Using [gw\\_ri.shp](#) (attribute: [gwl\\_masl\\_](#)), the RL of static groundwater was modelled ([gw\\_ri](#)). The model was contoured ([gw\\_ri\\_1m.shp](#)). The resulting model predictably shows a westward inclination in the elevation of static groundwater. A steeper gradient in the model between Terrace End and the central city may reflect the sub-surface extension of the Pohangina Anticline. “Channels” in the modelled surface in the Cloverlea Rd area and SH 56 near Longburn probably reflect similar features in the model of the RL top of the uppermost gravel.

**Step 6: Checking the modelled thickness of coverbeds on gravels:** The model derived above (Step 2) for the RL of the uppermost gravel surface can be checked by subtracting the modelled RL to of the uppermost gravel from the lidar surface model ([cov\\_thick](#)). This model was contoured ([cov\\_thick\\_2m.shp](#)) and compared against the previously derived model (Step 2). Negative values that are found locally, mostly in the Manawatu River area are interpreted to represent areas where drillhole RLs conflict with lidar elevations, or where the modelling barrier has not compensated for very different elevations immediately across the boundary line. Extremely high values (180 to about 15 m) reflect relative absence of gravels in drillholes that penetrate middle Pleistocene marginal marine beds. The two models broadly conform, although the latter ([cov\\_thick](#)) is the model adopted for coverbed thickness, because the lidar surface model is a far more accurate one for surface morphology.

**Step 7: Modelling saturated coverbed thickness:** The final step in this modelling process is to model areas where coverbed sequences are saturated, and therefore potentially liquefiable. To identify such areas, the model for the RL of the uppermost gravel bed (from Step 3; [grav\\_ri](#)) was subtracted from the RL of the static groundwater model (from Step 5; [gw\\_ri](#)). The resulting model ([cov\\_sat](#)) and its contours ([cov\\_sat\\_1m.shp](#)) provide an indication of thickness of saturated coverbeds that may potentially be liquefiable during strong ground shaking.

In areas of this model with negative values, static groundwater is below the upper surface of the topmost gravel, so any coverbeds present are not saturated. Areas with values between 33 and about 15 m are found in areas underlain by elevated and permeable middle Pleistocene marginal marine beds. Areas with saturated coverbeds between 1 and 10 m thick have attributes that warrant further investigation. It is important to note that drillhole data are not adequate to define whether these saturated coverbeds consist of lithologies that are capable of liquefaction.

Significant areas of potentially liquefiable materials are found in a number of places in the Oroua River valley and in the Longburn area, including an area near Cloverlea Rd. A further belt between Longburn and West End passes through the Anders Racecourse development zone. A number of local areas with thick potentially liquefiable coverbeds are found between Terrace End and Raukawa Rd, and the significance of these is uncertain. One of these, at Terrace End, is based on a single drillhole record that appears to have uncharacteristically deep coverbeds, and this may be due to the presence at shallow depths of materials comparable with the subsurface extension of the Pohangina Anticline. Another, beneath Te Matai Rd, close to its junction with SH3, is based on a deep elevation of the uppermost gravel surface.

### List of file names for modelled points, surfaces and the boundary:

NB Shapefiles are delivered in both NZMG and NZTM coordinate systems. NZMG and TM are suffixed to the following shapefile names to discriminate them.

[GroundwaterBoundary.shp](#)

#### Drillhole point data

[Wells.shp](#)

[DrillholeGeotech\\_gravel.shp](#)

[gw\\_rl.shp](#)

#### Groundwater depth

[gw\\_depth\\_1m.shp](#)

[gw\\_depth](#)

#### Groundwater RL

[gw\\_rl\\_1m.shp](#)

[gw\\_rl](#)

#### Depth to gravel

[grav\\_depth\\_1m.shp](#)

[grav\\_depth](#)

#### Gravel\_Surface\_RL

[grav\\_rl\\_1m.shp](#)

[grav\\_rl](#)

#### Coverbed thickness

[cov\\_thick\\_2m.shp](#)

[cov\\_thick](#)

#### Depth of saturated coverbeds

[cov\\_sat\\_1m.shp](#)

[cov\\_sat](#)

In summary, significant areas of soil materials potentially susceptible to liquefaction are found in a number of places in the Oroua River valley and in the Longburn area, including an area near Cloverlea Rd. A further belt between Longburn and West End passes through the Anders Racecourse development zone. A number of local areas with thick potentially liquefiable coverbeds are found between Terrace End and Raukawa Rd, and the significance of these is uncertain. One of these, at Terrace End, is based on a single drillhole record that appears to have uncharacteristically deep coverbeds, and this may be due to the presence at shallow depths of materials comparable with the subsurface extension of the Pohangina Anticline. Another, beneath Te Matai Rd, close to its junction with SH3, is based on a deep elevation of the uppermost gravel surface. More data from future subsurface investigations would help improve these models.

### 5.4.2 Conclusions from digital liquefaction modelling

Inspection of data available from the Horizons Manawatu drillhole database, from available geotechnical data and from lidar topographic data suggest that there are some areas that may be susceptible to liquefaction to the northwest of the existing Palmerston North urban area. There are also indications that potentially liquefiable materials may underlie the southwest part of the developed urban area, including parts of Longburn, Cloverlea, and in places within the Anders Racecourse development area.

In general, these data are consistent with the conventional materials-based liquefaction susceptibility mapping. The method developed above is capable of characterising subsurface variability of coverbeds and saturated coverbeds. However, variability in the model may be controlled by a single record, so at this stage the models are regarded as a supplementary guideline for liquefaction potential when using the conventional map. Smaller areas identifiable using the conventional materials-based mapping (such as soils information) will not be identifiable from this digital modelling technique because most of these smaller areas are not sampled by the drillhole database or the geotechnical database (see limitations of digital modelling, below).

In this modelling, the geotechnical nature of the coverbeds on gravel has not been fully identified, as they are derived in part from the heterogeneous drillhole logs. This model records the presence and thickness of saturated coverbeds above the underlying gravel which is not susceptible; if weak and loose, they may be susceptible to liquefaction. Materials logged by drillers as “clay” may actually represent clay, or silt or mud or even fine sand. While clay and mud are unlikely to liquefy, they could be susceptible to cyclic strain softening (see Appendix 1 and Section 2.2.2). Uniformly graded silts and fine sand are susceptible to liquefaction. For these reasons, our 3D models have limitations because of a lack of geotechnical data. However, they do raise caution regarding the (further) development of areas where thick, saturated coverbeds are modelled. A suitable response and strong recommendation to this note of caution would be to require careful geotechnical investigation of all new development sites in these suspect areas.

### **5.4.3 Limitations of our subsurface interpretation**

There are a number of limitations associated with the subsurface interpretations that are mentioned below.

- The lidar DEM coverage is limited to about half the PNC area of interest;
- Drillhole coverage is limited and does not extend across the full area;
- The detailed soil map covers only part of the area;
- There is very limited geotechnical data;
- Drillhole logs are variable with inconsistent material/soil descriptions;
- There are inconsistently documented static groundwater levels;
- Simplistic representation of the top gravel surface – this is not a stratigraphic or a chronostratigraphic surface. Rather, it is an amalgamation of a large number of different gravel horizons of differing ages which we interpret as underlying the finer grained “soil” coverbeds.

#### **5.4.3.1 Lidar DEM (digital elevation model)**

The lidar DEM provides a high resolution and accurate surface model on which to base other data and our Holocene geology interpretations (see Figure 5, also, Figures 14 & 15, Appendix 4). The DEM constrains and improves the accuracy of drillhole collar heights, and therefore downhole depths. It also provides accurate information that feeds into interpretation of the origin of materials underlying the surface.

#### **5.4.3.2 Drillhole coverage**

There are variable derivations of drillhole collar heights within the drillhole database. Some elevations have been estimated by drillers, some following surveying, some from GPS surveying, and some may be acquired from the lidar DEM. The database may be improved through standardising by deriving spot elevations for drillhole locations from the DEM. There are few or no drillhole data for areas upstream from Raukawa Rd, or for much of the Tararuas, so we are not able to carry out modelling in the Ashhurst area and SE of the Manawatu River.

#### **5.4.3.3 Soil layer**

The Cowie (1974; Cowie & Kimpton 1976) soil map is a very useful check on geological data and on near-surface materials. Because it is at a larger scale, it has more detail and potentially more information on liquefaction potential, particularly on the present flood plain of the Manawatu River. It is reasonable to integrate the liquefiable areas derived from the geological map units with areas of deep soft soils identified from the soil map and supported by drillhole data. However, the Cowie & Kimpton map does not cover the Ashhurst area, and similar applications there are not possible.

#### **5.4.3.4 Geotechnical data**

The amount and distribution of geotechnical data available for analysis in this project are useful, though limited in coverage and extent. There are good geotechnical data available for the waste-water treatment site which confirms that the Manawatu River over-bank flood deposits are loose, uniform fine sand and silt up to 5m thick at this site. Where saturated these materials will be highly susceptible to liquefaction and lateral spreading. This information has been used to help classify all the recent Manawatu River plain (Figure 8b) as liquefaction zone 1 – 3, with highest liquefaction susceptibility. Because of a lack of geotechnical data elsewhere, depth to gravel was used in our 3D modelling to supplement the drillhole data. CPT data, through measuring soil strength with depth (point resistance plots), provided an independent check on the depth to gravel in the city area.

### **5.4.4 General statement of limitations**

The figures produced from the subsurface models (Figures 11 – 13, particularly Figure 13) provide an indication of where soil conditions favouring liquefaction may exist in the Palmerston North area. However, because of our interpretation and extrapolation of data, such indications have inherent inhomogeneities and errors. The entire area southeast of the Manawatu River has not been incorporated because the underlying materials here are last interglacial or older marginal marine and older alluvial deposits.

A second qualification is that the modelled surface on the topmost gravel is not really a single surface. The topmost gravel is not a single homogeneous stratigraphic unit. Some of the gravels at this surface are Holocene gravels, while some may be as old as 200,000 years. Drillhole static groundwater levels are measured all year round, without regard to seasonal fluctuations. A further limitation on the accuracy of the models is variable quality in loggers' observations, interpretations and reporting. In addition, elevations of drillhole collar heights, derived by drillers/loggers without the benefit of lidar maps, and recorded depths in drillhole logs are subject to significant error.

We urge PNCC to be cautious in using the numerical results of this modelling. Rather the maps should be used to indicate the possibility that unconsolidated, saturated sand and silt may be present, and may represent a possible liquefaction potential. Where this is the case, soils should be tested on a site by site basis where the opportunity arises with new developments. We recommend that all new subsurface data is added to the database to refine the precision and numerical usefulness of the models from it.

## **6.0 POTENTIAL FOR LIQUEFACTION AND GROUND DAMAGE IN PALMERSTON NORTH**

### **6.1 PNC Totara Rd. wastewater treatment plant**

The area of the wastewater treatment plant lies close to the Manawatu River floodplain. The area is of low elevation (c. 20 to 25 m asl) and is of low relief. There are a number of relict, partially buried river meander channels within the general area, suggesting potential exists for locally thick flood deposit/soil sequences. A relatively large number of geotechnical records (8 CPT; 8 scala; 13 SPT) and 14 drillhole records are available for the Totara Rd Wastewater Treatment Plant area (MWH 2006). The deepest CPT record in the area, close to Pioneer Highway, reaches almost 10 m depth, and in general the CPT records indicate that weak sediments at the site vary in thickness from 1.8 to 5.2 m, with an average of c. 3.3 m. Nine test pits excavated for geotechnical investigations at the wastewater site revealed a range in thickness of weak sediments of 3.7 to 5.1 m, with an average thickness of c. 4.3 m. These subsurface records have been entered into the database.

Borehole logs 2, 3, 4 & 5 (of BH's 1 to 6) reported in MWH (2006) show that in places, the top 2 to 5 m of the site is covered by brownish, loose, fine sand and silt. This material is interpreted as deposits from sediment laden, over-bank flood waters when the river was in high flood. The SPT values of <5 and sometimes as low as 1 or 2, show that these deposits are very loose and when saturated they will be highly susceptible to liquefaction and lateral spreading. However, in the site drillholes, the groundwater table is lower than the deposits and they are therefore not saturated and are unlikely to liquefy at these locations. The geotechnical report (MWH, 2006) recognises the possibility of liquefaction and settlement of these materials, but does not consider the possibility of lateral spreading. Because of the damage and disruption that might possibly occur in strong earthquake shaking, we recommend that a re-assessment is made of these materials on the site and their capacity for causing damage due to liquefaction and lateral spreading.

The Dellow *et al.* (1994) liquefaction map classed the Wastewater treatment plant area within their Zone 2, indicating moderate liquefaction susceptibility. Our updated map classes the site within our zones 1-3 (moderate to high). Our groundwater elevation model places the static groundwater level at c. 3 to 6 m below the surface, and our modelled thickness of "coverbeds" suggests a variability in the thickness of potentially susceptible materials, with a belt of thick, saturated "coverbeds" extending east-northeast across the site.

### **6.2 Anders Road, Racecourse residential growth option**

The Anders Road, Racecourse residential growth area is low-lying and traversed by at least two old courses of the Mangaone Stream (Figure 8c). The slightly elevated overbank deposits are clearly shown on the lidar hillshade. Cowie's (1979) soils map does not cover

the whole area, but at least the eastern two thirds of the area is underlain by lowland soil types of the Manawatu, Kairanga and Te Arakura series (3, 4 and 8 – Appendix 5). The normal range of thickness of these soils is 1 to 3.5 m, and they vary in their texture. Based on these soil descriptions (Appendix 5 & 6) these soils are silty and fine sandy loams which appear to be well graded. Thus in our assessment, based on the soils descriptions alone, these soils are likely to have low liquefaction susceptibility. However, geologically, they are categorised as Holocene alluvium (Begg & Johnston 2000). There are few geotechnical data available, with only 4 CPT logs available (P57, P70, P80, P86), these clustered in two small areas. These logs record “soft” soils of thicknesses ranging from 3.6 to 6.5 m. Three drillhole logs fall within the area (ids 335031, 335505, 335511), recording depths to the uppermost gravel ranging from 2 to 11.4 m. Dellow *et al.*'s (1994) liquefaction susceptibility map registers the area within zones 1 (high liquefaction susceptibility) and 2 (moderate susceptibility). Based on the scarce available geotechnical data, for our revised map we characterise the area as having moderate to high (zone 2-3) liquefaction susceptibility. In light of the soils maps information, this assessment may be conservative and with additional data it may be possible to lower the susceptibility assessment.

The modelled depth of “coverbeds” (Figures 12 & 13) suggests there may be a belt of thick soils across the southern one third of the area, up to 11 m thick. Static groundwater depths across the area are modelled at < 3 m across the eastern half. The modelled thickness of saturated “coverbeds” across the area varies from 0 to 7 m. Existing data suggest that there may be few liquefaction problems in the north-central part of the area (Westberg Rd/Whitehorse Dr area), but there may well be areas possibly susceptible to liquefaction near Pioneer Highway and Tremaine Ave. These characteristics provide a strong argument for exercising caution in development of this area. We recommend that comprehensive site investigations with good geotechnical advice precede development. A major lesson from the Canterbury earthquakes damage is that provided houses and buildings are constructed on foundations that take account of local site subsurface conditions and continue to provide support when the foundation soils yield, deform inelastically, and/or liquefy, then the houses and buildings can survive undamaged. Measures can also be taken to minimise damage to infrastructure, such as buried pipelines.

### **6.3 Kelvin Grove residential growth option**

The Kelvin Grove residential growth area is almost entirely contained within Dellow *et al.*'s (1994) zone of negligible liquefaction susceptibility. Similarly, in our revised map, it is almost all within zones of negligible and low liquefaction susceptibility. The modelled thickness of coverbeds in this area is probably heavily disguised by inability to distinguish between the underlying middle Pleistocene marginal marine deposits and Holocene “coverbeds” associated with liquefaction susceptibility. There is limited data available within the area for constraining the depth to groundwater. However, most of the area is relatively elevated, and thus unlikely to have shallow static groundwater elevations. Because of the lack of data, the modelled thickness of saturated “coverbeds” (Figure 13b) in the area is unreliable. The southern quarter of the area lies on the Holocene floodplain of the Manawatu River, with a number of relict channel forms, at least partially infilled. This area features on the saturated coverbeds model as having thick fine-grained materials resting on the uppermost gravel.

The soil (11 and 12, Milson loam and Marton silt loam which are described as being clayey/cohesive) and geological units (Q5b and Q3a1) underlying at least the northern three quarters of this area, and the area's relatively high elevation and good drainage means there

is little to no potential for liquefaction. Given the Holocene materials in the southern quarter, the presence of relict river channels, the presence of thick fine-grained materials in the top 20 m of drillhole 336581, we recommend caution in development of this area. Again we recommend good subsurface investigations and geotechnical advice prior to development with building foundations suitable for the local site conditions.

## **7.0 MITIGATION OPTIONS & COST**

### **7.1 PNCC Totara Rd Wastewater Treatment Plant**

Our knowledge of the Totara Road wastewater treatment plant is derived from a geotechnical consultants report to PNCC (MWH 2006). It is clear from this report that the plant site, at the south-western margin of the City close to the Manawatu River is in places covered with 2 to 6 m of loose (SPTN values < 5), uniform fine sand and silt, which are interpreted to be sediments deposited from past floods in the river. When such loose sediments are saturated they are very highly susceptible to liquefaction and lateral spreading. However, in a few places where static groundwater level has been measured, the water level is near or below the base of the susceptible deposit, so these deposits are not saturated and are unlikely to liquefy. However, this assumption is based on little data and needs to be confirmed for the site.

MWH (2006) assessed the liquefaction settlement of these susceptible deposits but did not consider lateral spreading, which Christchurch experience shows is potentially more damaging. We understand from the MWH (2006) report that in new parts of the wastewater treatment site the susceptible materials were excavated out and replaced with better compacted material. In this case assessment of entire site is required to determine if any of the key features of the plant could be affected by liquefaction and lateral spreading. If any of the plant is affected, works should be undertaken to mitigate this possibility.

In Christchurch we understand that part of the waste water treatment plant at Bromley was constructed on stone columns. After the Darfield Earthquake, damage to the plant appears to have been minimal, but it may have been greater after the Christchurch Earthquake. We recommend that the performance of the waste water treatment plant in Christchurch and Kaiapoi (both in liquefied areas), and other places nearby are studied and compared to the situation at Palmerston North. We recommend that a consultant who is familiar with the water treatment plants in the Christchurch area, and the damage that has occurred to them from both earthquakes, is engaged to assess the Palmerston North plant for liquefaction, lateral spreading and other possible earthquake damage. An initial assessment is estimated to involve 2 to 3 weeks work by a high calibre consultant, with an estimated cost of \$20 to \$30K. Once this assessment has been undertaken, much better decisions could be made about whether or not liquefaction is a problem at the waste water treatment site which requires mitigation, and if it is, then what sort of mitigation would be appropriate.

### **7.2 Anders Road, Racecourse and Kelvin Grove residential growth areas**

A take-home lesson from the two Christchurch earthquakes is that “new” houses were often built with inadequate foundations for the prevailing, known, ground conditions. For example, areas of eastern Christchurch (i.e. adjacent to the Kaiapoi River and parts of Bexley) which were mapped as highly susceptible to liquefaction, have in recent years been subdivided with extensive new housing constructed, many of which are higher value, half to one million

dollar properties. In most cases the new houses have been constructed on unreinforced or lightly reinforced concrete foundation slabs, which are quite inappropriate for the known ground conditions. During the earthquakes there has been strong liquefaction in these areas with associated differential ground settlements and lateral spreading. In this situation the unreinforced concrete slabs have cracked, tilted and pulled apart, often allowing the liquefaction products, fine sand and silt with water, free access to the interior of the house. Although the houses have not collapsed, they are no longer habitable and cannot readily be repaired, so they are a complete write-off.

The (relatively few) houses that were built on good foundations (well reinforced foundation slab on compacted hard fill grade, well reinforced slab on driven piles, well reinforced slab with ground beams) survived with little or no damage although their services, particularly those buried in the ground, such as water and waste water, but sometimes including power and phone lines, were often cut. There are, however, techniques for minimising damage to services. These include:

- Keeping power and phone lines above ground, or with very shallow burial;
- Use of flexible (polybutinol type) pipes for water and waste water. These pipes should have a minimum of joints and should be buried as shallow as possible (which minimises long-term repair and maintenance costs anyway);
- Pile manholes and pump stations with screw or driven piles to prevent them from floating up during liquefaction and thus shearing off pipe connections;
- Use flexible service connections from the ground to houses.

In our view, (high) liquefaction susceptibility alone need not rule out an area from subdivision and development. What is required in areas of weak, or for that matter, in any type of ground, is an appropriate site investigation which characterises the ground at that site. For better ground a scala penetrometer test may be sufficient. In areas of deeper, weak ground a CPT and/or SPT testing together with soil sample(s) for testing, such as particle size tests, would be suitable. Once the properties of the ground have been determined, an appropriate foundation for a structure on the site can be designed.

In no cases should any buildings be constructed on an unreinforced concrete slab. This has apparently been allowed under NZS 3604, nonspecific design, but is likely to change following the lessons of the Christchurch earthquakes.

We estimate that a “standard” sized house built in an area with high liquefaction susceptibility would cost an additional \$15,000 to \$20,000 to build with a well reinforced concrete slab on a grid of driven tanalised post piles. For a house costing \$200,000 to build, this is 10% of the house cost. For a house costing \$400,000 it is 5%. Many owners of badly damaged, written-off houses in Christchurch would now consider this to be an excellent investment if it had saved their house – (which it did in at least one case).

For any new developments in Palmerston North we strongly recommend that the above site investigation and foundation design procedures are carried out in every case. We note that in some places there may be additional “hazard” considerations prior to development. In a liquefaction area ground settlement may be sufficient to cause an unacceptable increase in the potential for flooding. In Kaiapoi this was the case where both land settlement and stop bank settlements due to liquefaction and lateral spreading caused an increase in flooding hazard and a requirement for expensive top-up remedial works for the stop banks.

## 8.0 CONCLUSIONS AND RECOMMENDATIONS

1. The 1994 (Dellow et al.) liquefaction assessment by GNS used mainly soil and geological information, supplemented with a small amount of available “geotechnical” criteria to carry out a regional liquefaction assessment, which included Palmerston North City. Holocene (last 10,000 years) geology areas were the main focus while the supplementary data included a few logged drillholes.
2. The present liquefaction assessment has concentrated on Palmerston North City. The data used has included the latest geological QMaps (Begg & Johnston 2000, & Lee & Begg 2002; Townsend *et al.* 2008), the soil map of Cowie & Kimpton (1976), lidar models and the drillhole data base of Horizons Manawatu Regional Council, supplemented with additional drillhole and geotechnical data located by GNS. The drillhole data base of some 1,600 records, includes lithology, groundwater level and relatively minor geotechnical data. We recommend that future site investigations in Palmerston North include more accurate drillhole information and required geotechnical data which can be added to improve the database and its resolution.
3. Internationally cities are starting to use drillholes and other subsurface data from tunnels, storage spaces, building basements, etc., to compile 3D geology maps of the ground beneath their city. This 3D geology is in an active database, in GIS format, which allows for the ready addition of new data and the retrieval of entered data for assessing new projects and developments. We have compiled all the available subsurface data into a GIS 3D database and have focussed our attention onto the top 30 m which is relevant for our liquefaction assessment. This database will be handed over to PNCC as a working database to which new and improved data can be added. This has the objective of refining the liquefaction modelling we have been able to formulate. We expect that the database will become increasingly useful to PNCC. As new data is added its uses will expand.
4. Our map differs significantly from the 1994 GNS assessment. We consider that the area of the city with the highest liquefaction susceptibility is the recent flood plain of the Manawatu River (Figure 8b) with its old infilled channels which show up beautifully on the lidar maps (Figures 5, 8c & 10). This flood plain has a variable thickness of loose, fine sand and silt flood deposits which range in thickness from zero to 6 m or more. However, this material is not always saturated as permeable gravels underly the fine sands. For this reason the liquefaction susceptibility is rated from 1 (very high) to 3 (moderate), although areas of gravel would have low to nil liquefaction susceptibility. Where there is sloping ground combined with saturated highly susceptible soils, lateral spreading could occur in this zone.
5. The soils on which much of the City is constructed we rate as having “moderate” liquefaction susceptibility (Figure 8b) Zone 2 (high susceptibility) to 3 (moderate). This

rating is based on the thickness of “soft” ground and high water table noted in the drillhole records. It is supported by a few soil grading curves obtained from Soil Bureau archives (Appendix 6). There is very little geotechnical data available to support this liquefaction susceptibility ranking. The soil mapping data (Cowie & Kimpton, 1976) and the associated grading curves indicate that the soft soils may be cohesive, in which case their liquefaction susceptibility will be reduced, possibly to a cyclic strain softening behaviour. This assessment will be improved and refined as more geotechnical data is added to the database.

6. The remainder of the City area (Figure 8b) is assessed as having low or negligible liquefaction susceptibility.
7. Liquefaction potential is assessed in terms of both the susceptibility of the soils and the opportunity for strong shaking to occur. Our study confirms that there are many local earthquake generating faults (seismogenic sources) within 40 km of Palmerston North which could generate strong ground shaking sufficient to cause liquefaction. Liquefaction begins to occur at MM7 intensity shaking and the amount and severity of liquefaction increases with increasing intensity. Using the GNS probabilistic seismic hazard model, we estimate that MM7 intensity shaking could be expected approximately every 30 years, MM8 every 130 years, MM9 every 700 years, and MM10 every 14,000 years. Based on the earthquakes that have now occurred in Christchurch, we consider that it is wise for Palmerston North City to assess what might happen at each of these levels of strong shaking.
8. Geotechnical data from the Totara Road waste water treatment plant shows that a variable thickness, ranging from 0 to 6 m depth of loose, uniformly graded, fine sand and silt are present at the site. When saturated these soils are very highly susceptible to liquefaction. However, the few measurements of static groundwater level are low possibly indicating that these susceptible materials are not saturated. In this case the site may not liquefy. As well the susceptible ground in newly constructed areas has been dug out and replaced with compacted, non liquefiable material. These conclusions require confirmation. We recommend that a consultant experienced in the performance of the Christchurch area waste water treatment plants during the Christchurch earthquakes is engaged to carry out an assessment of the PNCC plant. This will provide PNCC with a rational basis for determining whether or not earthquake and liquefaction mitigation measures are required at their plant (see Section 7).
9. Experience from the Christchurch earthquakes shows that the seismic performance of a structure is closely related to its foundations. In the Christchurch area a huge amount of damage to houses was due to the failure of foundations, which were unsuitable for the known ground conditions (highly liquefiable soils). Where suitable foundations were used in highly liquefiable areas, such as driven tanalised timber piles capped by a well reinforced concrete slab, the house structure was invariably undamaged. Thus we do not exclude any areas from possible development. However, prior to undertaking any new developments in the City, suitable geotechnical investigations of the ground are required. Once the ground properties have been determined, suitable foundations which are needed to support the proposed structure during strong shaking in the prevailing ground conditions, can be rationally designed. This is a strong recommendation for the Anders Road, Racecourse and Kelvin Grove growth areas, plus any new and infill housing and for new commercial buildings anywhere in the city (see Section 7). Using this process, the potential for damage to new houses and buildings can be mitigated with additional modest costs generally falling on developers. Experience from

Christchurch shows that there is also a relatively urgent need for a robust assessment of older, earthquake risk buildings and their foundations, throughout the City.

10. We recommend that all new drillhole data, together with geotechnical data, such as CPT, seismic CPT, SPT, ground water levels, grading curves acquired from new site investigations, are entered into the subsurface 3D database. The database requires more geotechnical data to improve it and to refine the models obtained from it. Adding good data to the database will ensure it is a very useful and valuable asset for the City.
11. We recommend that PNCC approach all organisations that may have subsurface data on file. It is likely that engineering consultants will have a lot of useful subsurface data. They may be encouraged to share this data if they can obtain access to the complete database. Other organisations with subsurface data may include the NZ Transport Authority, the University and local building owners.
12. It is our hope that we have provided PNCC with some useful tools that are needed for the rational development of the City in light of the potential hazard from liquefaction, lateral spreading and other earthquake damage.

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## FIGURE CAPTIONS



Figure 1 Photos from Christchurch showing evidence of liquefaction in the form of sand boils along “fissures”. a) The Avon River right bank at Anzac Bridge. b) Mona Vale gardens in Fendalton. c) This swimming pool was full of water during the Darfield Earthquake and stayed in the ground when the site liquefied. However, it was empty and “floated” when the site liquefied in the Christchurch Earthquake. d) This pumping station caisson floated and moved laterally during both earthquakes.



Figure 2 These photos provide examples of lateral spreading and settlement adjacent to the Avon River after the Christchurch Earthquake: a) Fitzherbert Terrace near the junction with Avonside Drive; b) Bridge St. bridge near New Brighton showing abutment settlement and rotation; c) the Avondale Rd. bridge with lateral spreading cracks and d) abutment rotation and settlement.

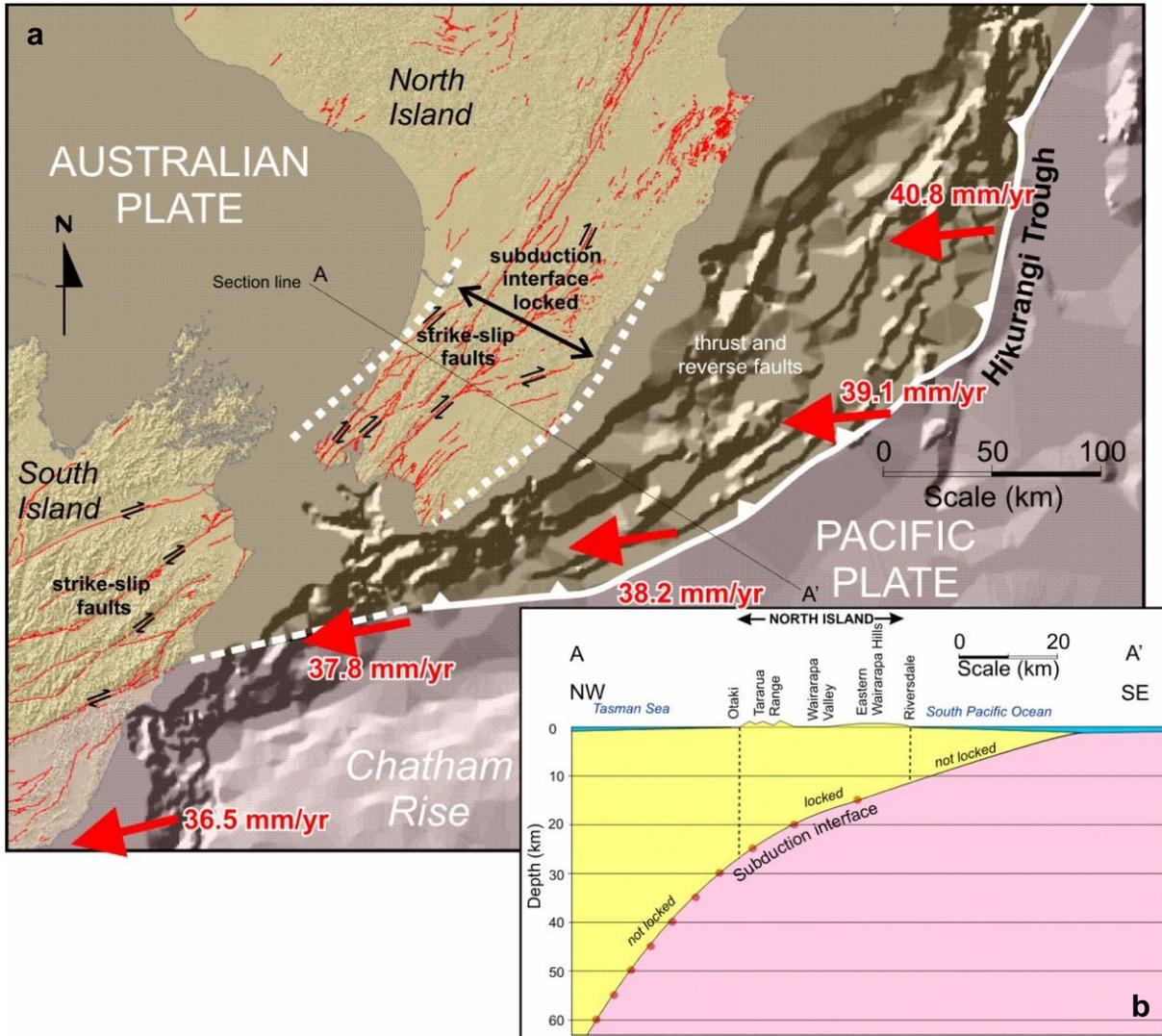


Figure 3 a) Palmerston North City lies close to the boundary between the Pacific (here coloured pink) and Australian (yellow) tectonic plates. The plates are moving across the surface of the globe at differing rates and the relative difference is represented here by convergence rates at specific places along the boundary (red arrows and numbers). b) The Manawatu region sits behind the outer edge of the Australian Plate at the western margin of a zone where the west-dipping plate interface is locked. Locking of the plate interface, and the relative convergence rates and vectors contribute to the upward bulge in the Australian Plate that is the landmass of the southern North Island. The offshore DEM is derived from bathymetric contours.

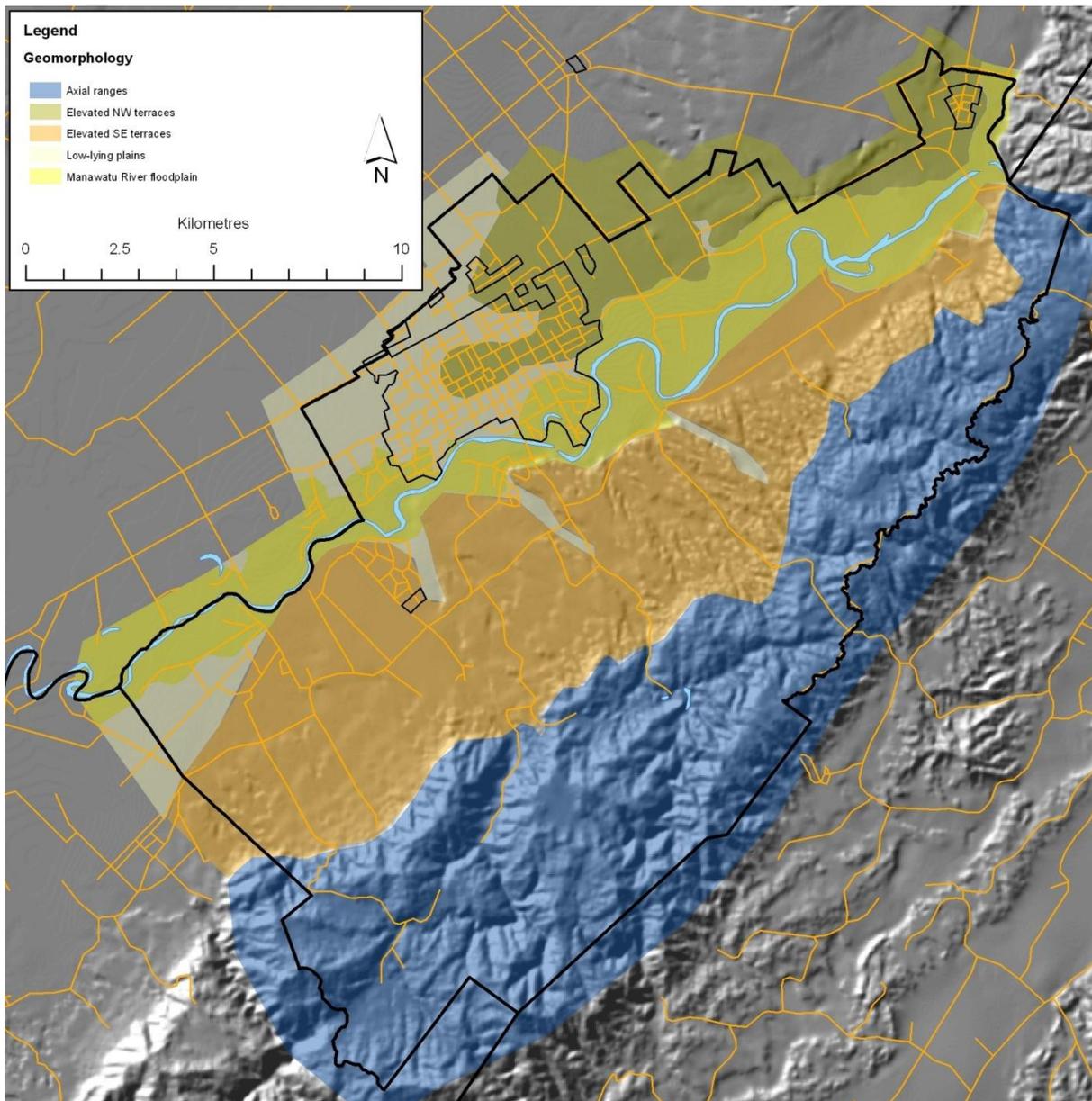


Figure 4 Generalised geomorphology of Palmerston North City. The authority boundary is shown as a heavy black line, and the built-up urban areas are also outlined in black. Further discussion of the physiography can be found in Appendices 3 and 4.

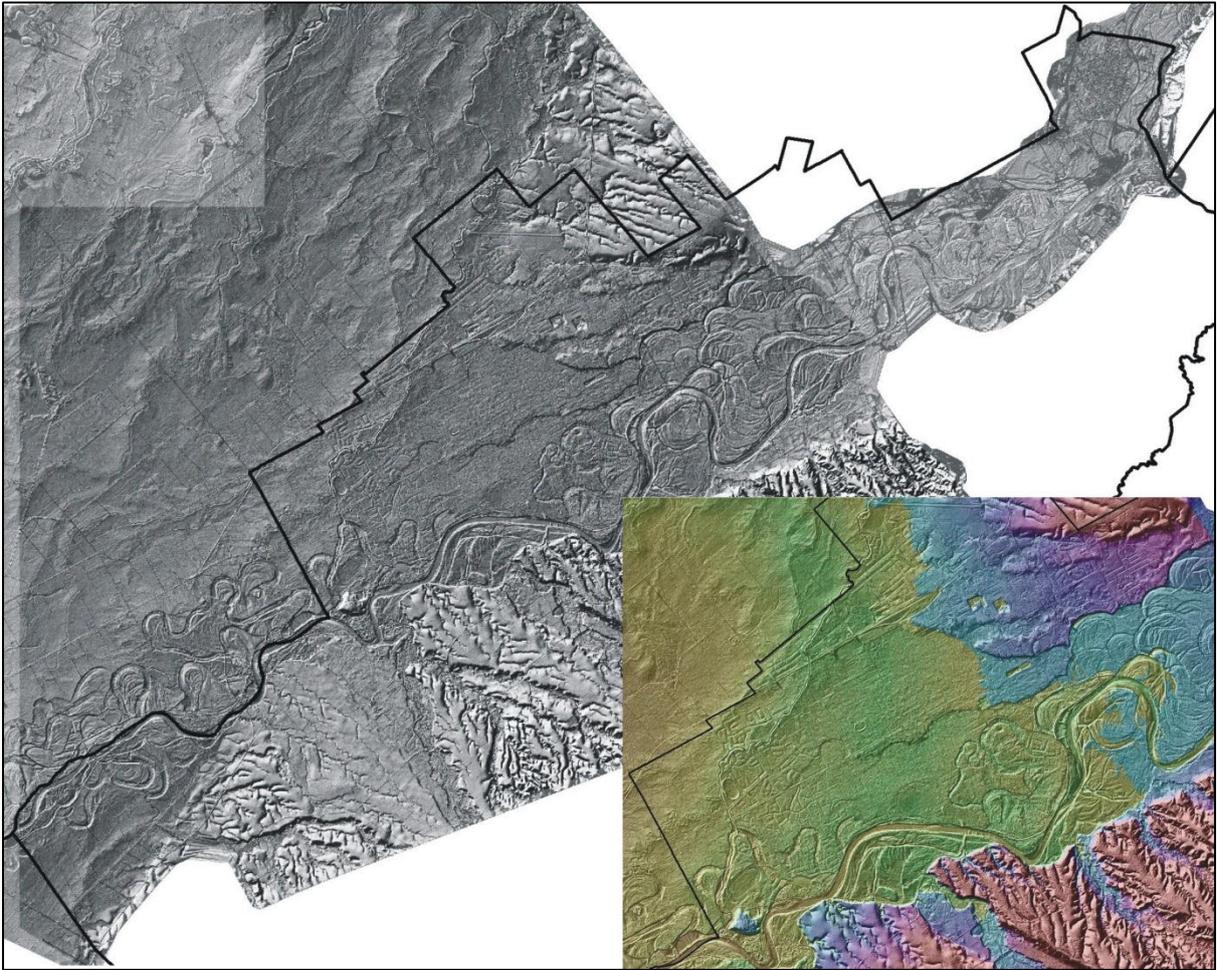


Figure 5 A hillshade derived from Lidar Digital Elevation Models (DEMs) of the Palmerston North City area. The heavy black line marks the Palmerston North City boundary. A smaller area of the City itself is imaged in the inset; here the hillshade is underlain by a colour-ramped DEM, where colour bands depict areas of similar elevation.

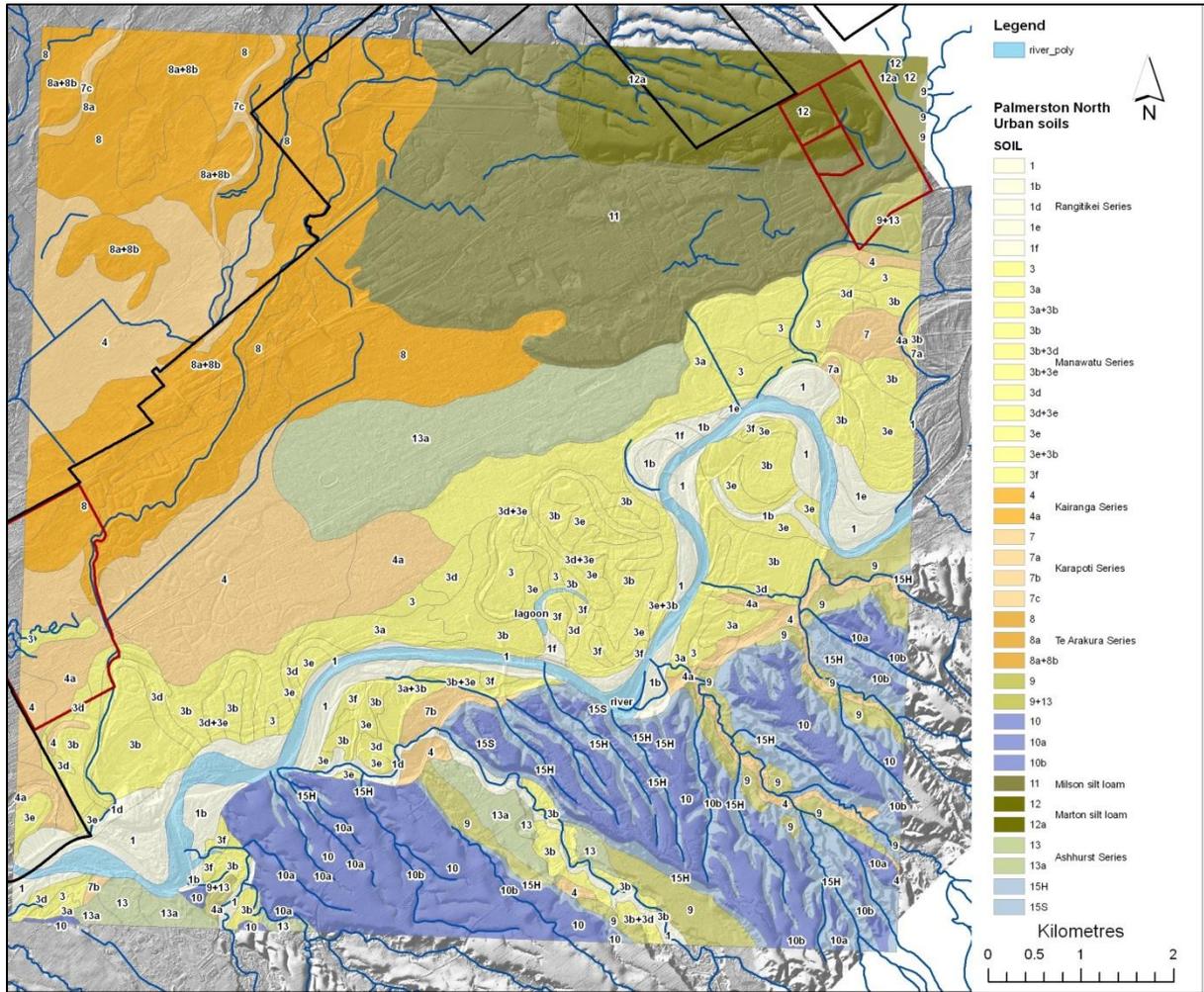


Figure 6 The soils of Palmerston North (after Cowie 1974) are subdivided into two, soils of the river flats (yellow and orange), and those of the terraced land (blue and green). Soils on the terraced land are usually thicker and more structured than those on the river flats.

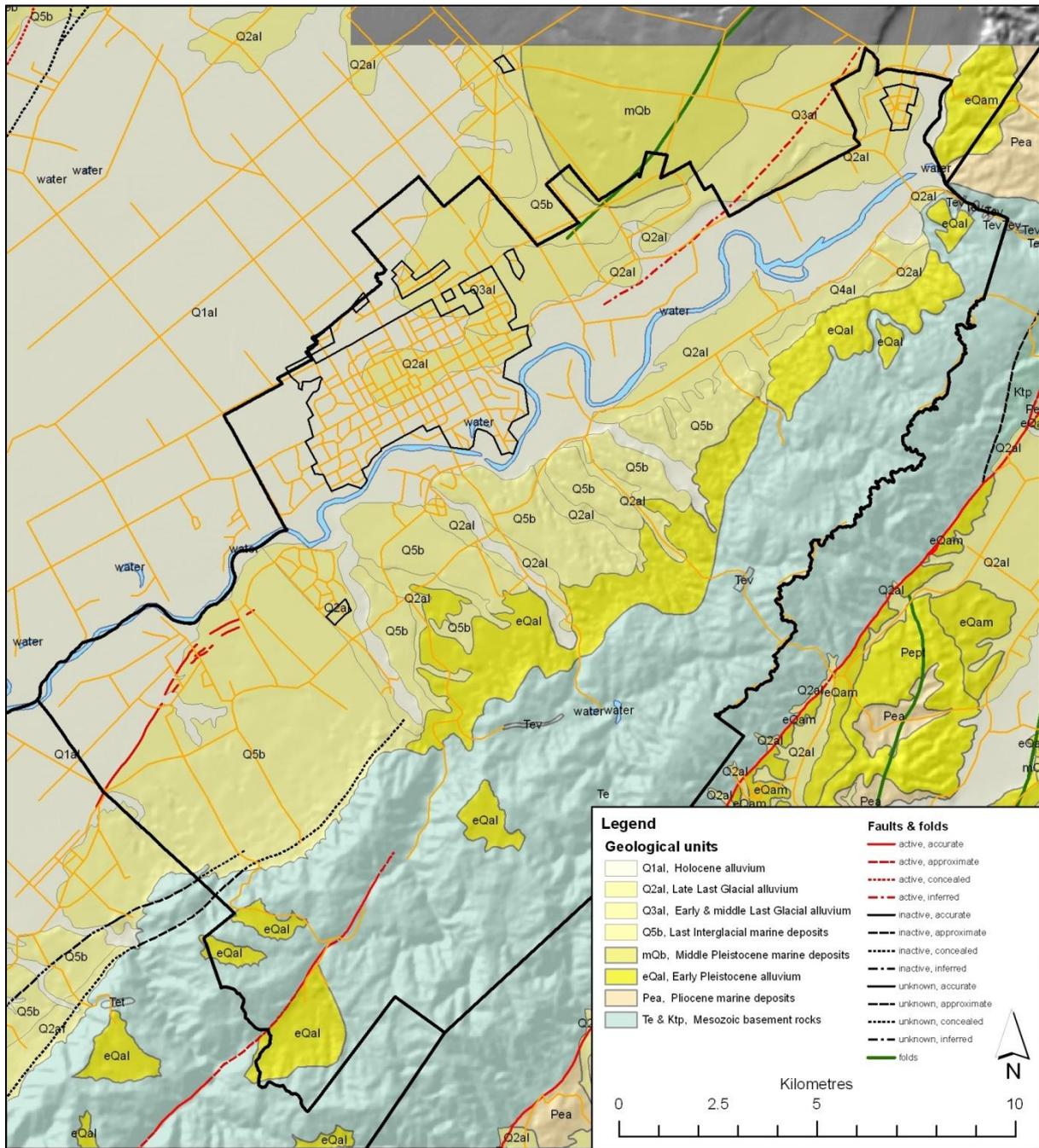


Figure 7 A simplified geological map of the Palmerston North City area.

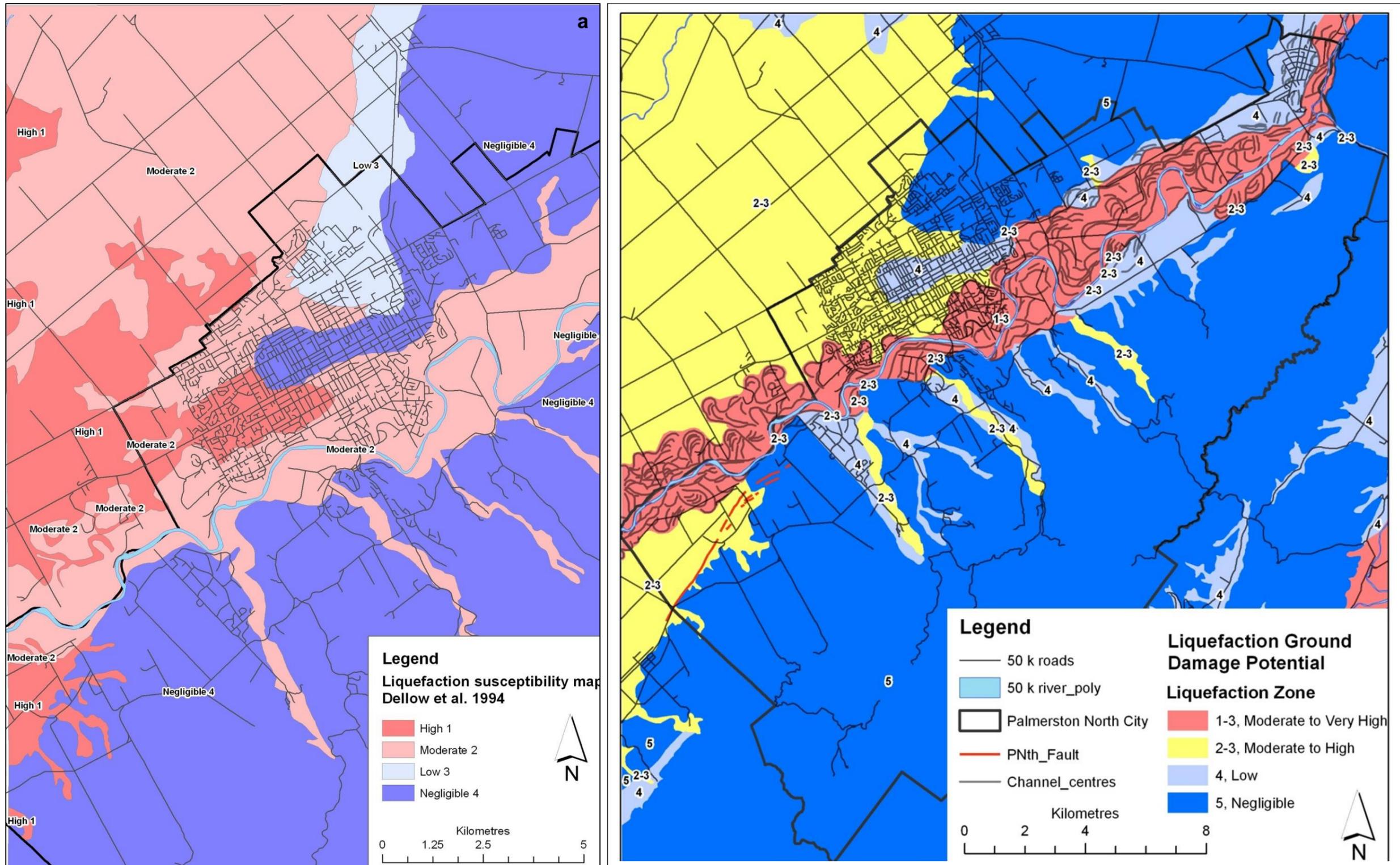


Figure 8 a) Comparison between a) the liquefaction susceptibility map of Dellow et al. (1994), and b) a liquefaction susceptibility map generated using the same methodology, but independently, for this job using higher quality physiographic and more recent geological information. In both figures, dark blue represents areas where there is negligible liquefaction susceptibility; light blue, low liquefaction susceptibility. In the left map, pink represents areas of moderate, and red, high liquefaction susceptibility. In the right hand map, pink represents areas zoned 2-3 (possible to moderate liquefaction susceptibility and red, areas zoned 1-3 (possible to high liquefaction susceptibility). In the new map, river channel centres and relict river channel centres are added as black lines, mostly across the late Holocene floodplain of the Manawatu River.

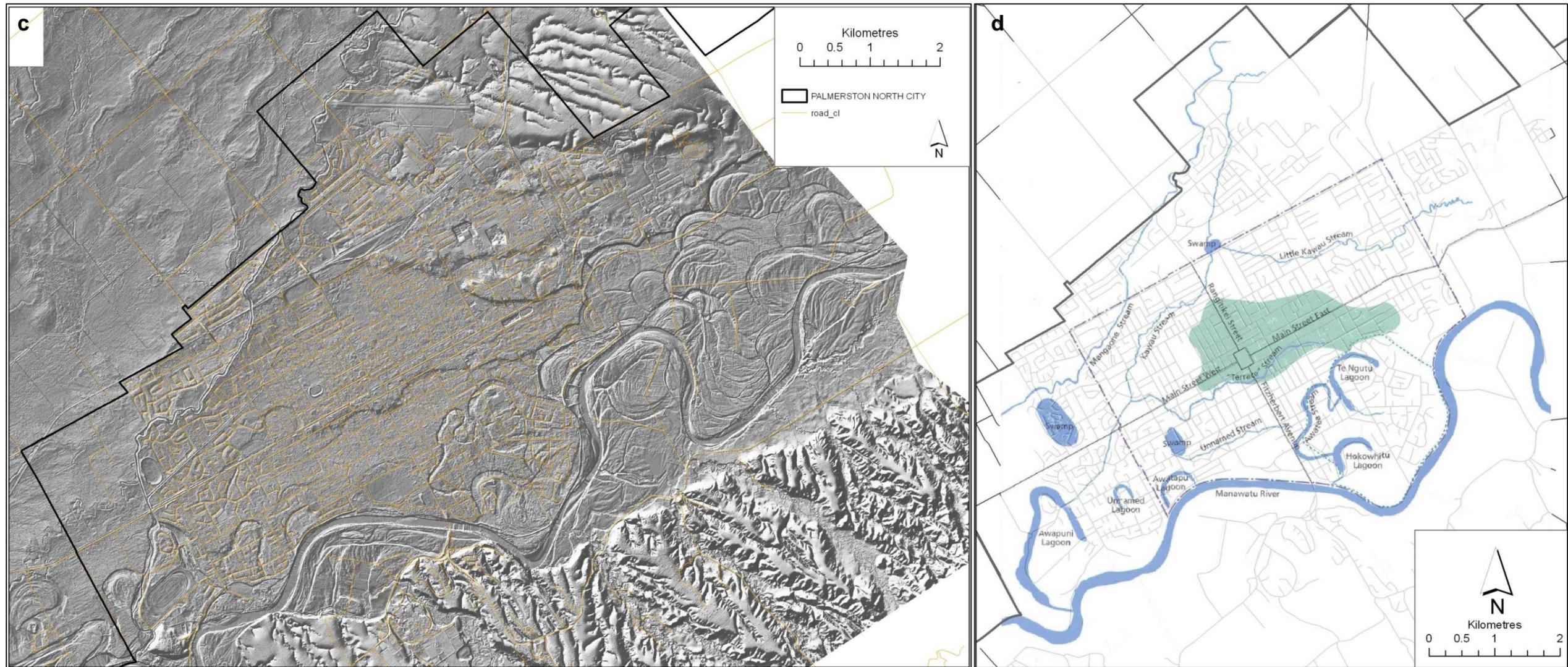


Figure 8 c) The Lidar DEM with shading (top) compared at the same scale to an early map of Palmerston North showing the city area as it was in 1896. Areas of old stream and river channels, old lagoons and swamps seen on both images may contain weak sediments, including peats, that are susceptible to settlement and earthquake induced liquefaction and lateral spreading effects. Special care is needed with site investigations and foundation design in these areas; d) Matheson (2003) compiled a map from early surveyors records of the location of swamps and lakes in Palmerston North City area. The green area represents the original clearing that Palmerston North was built on.

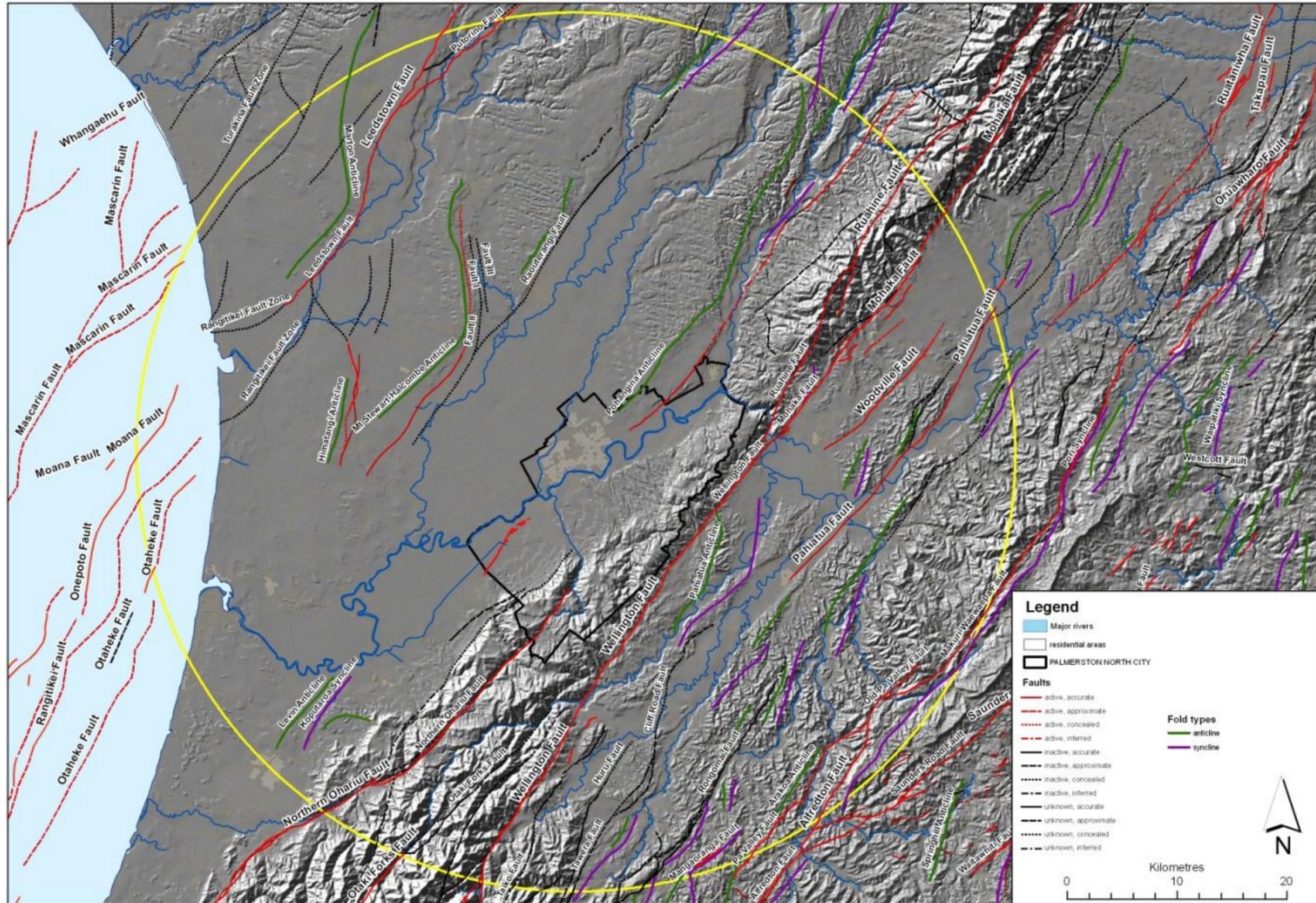


Figure 9 Surface and immediately subsurface seismicogenic faults and folds of the Palmerston North area. The yellow circle has a radius of 40 km from Palmerston North City Centre. The most significant source is the Wellington-Mohaka Fault which lies within 15 km of the City Centre. Other important faults include the subduction interface, c. 25 km beneath the city, the Ruahine and Northern Ohariu faults, and a newly discovered feature (arrowed), probably an active fault in the southwest of the City area. Anticlinal folds are commonly associated with underlying active reverse faults west of the Wellington Fault.

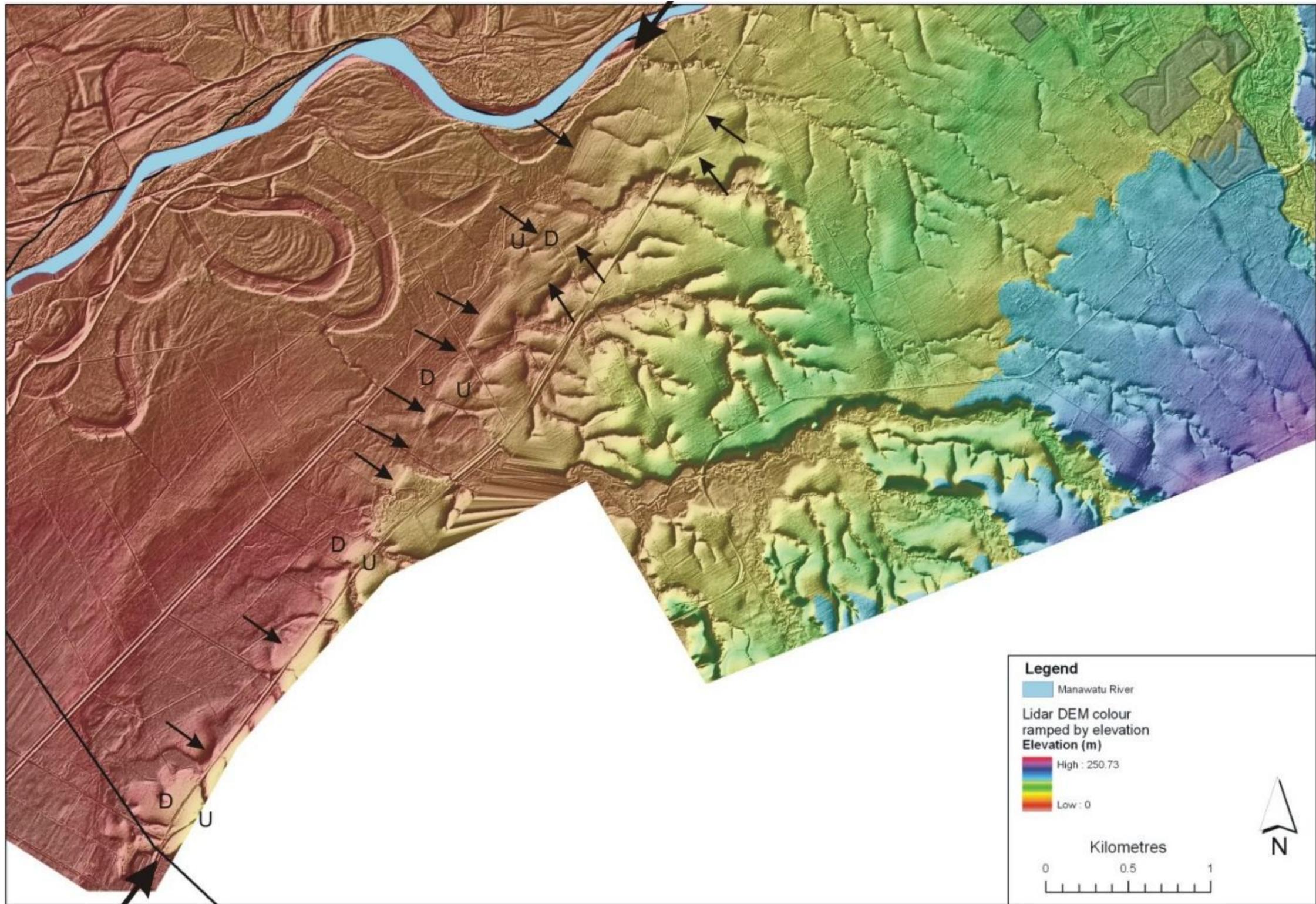


Figure 10 A geomorphic feature in the extreme SW of the Palmerston North City area shows many of the hallmarks of an active fault, as illustrated in this high resolution lidar DEM and hillshade. The sharp, linear feature (arrowed) is consistently elevated on the SE side and appears to displace Holocene fan deposits. Close to the NE end of the preserved lineament, it splays eastwards in a number of traces, and a small graben has developed, with the northwestern bounding feature downthrown to the SE. If the feature is a fault, it strikes towards Palmerston North City, its northeastern visible extent becoming obscured by active alluvial processes of the Manawatu River floodplain. In this map, Linton is marked on the northeastern edge as a grey shaded area. U = upthrown side, D = downthrown side.

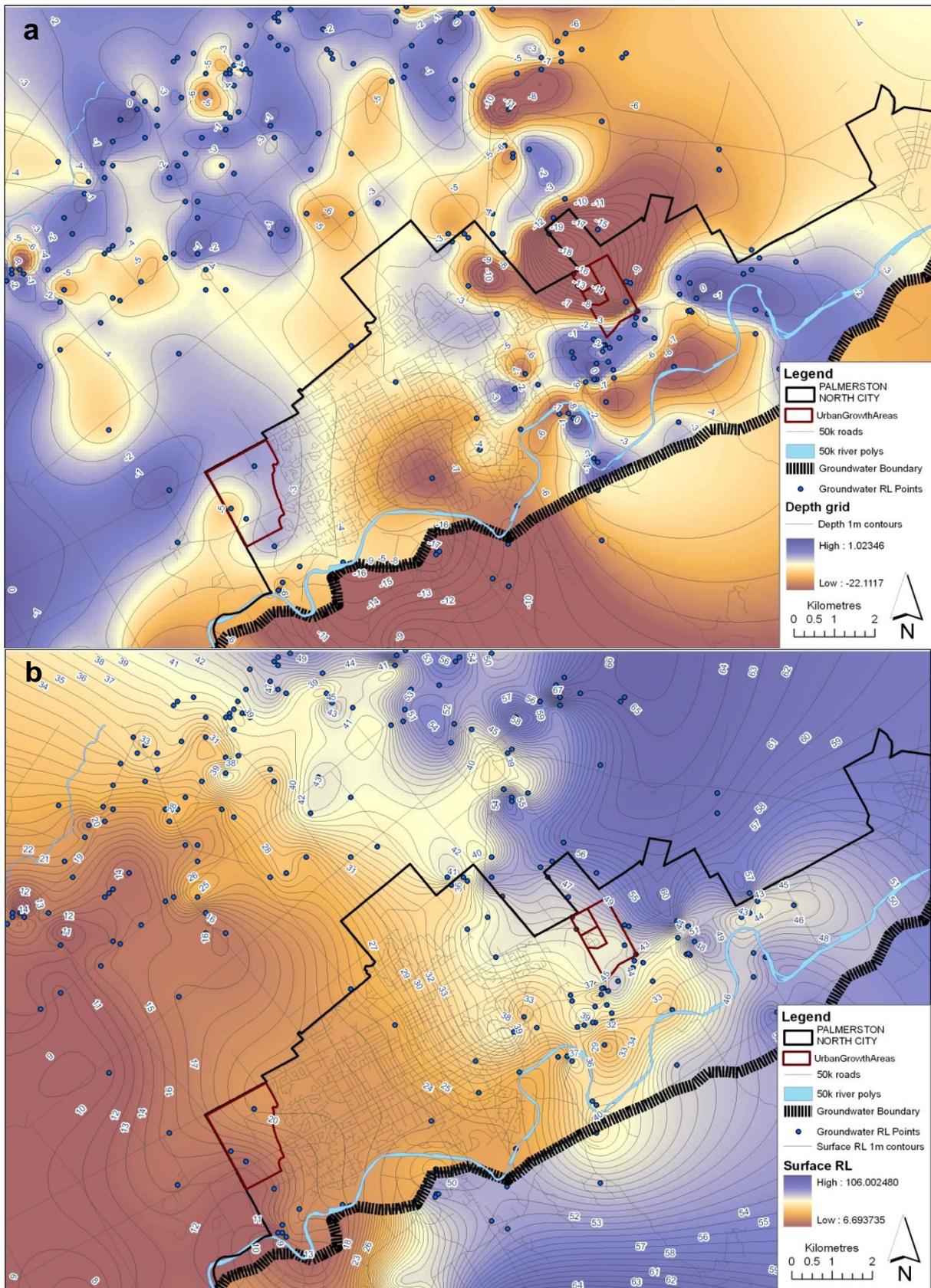


Figure 11 a) The modelled depth to groundwater in the Palmerston North City area showing point data available (depth and RL of groundwater surface). Modelled 1 m contours on the surface are labelled (depth in m to groundwater). b) The RL of the modelled groundwater surface, with data points in blue and labelled 1 m contours across the area.

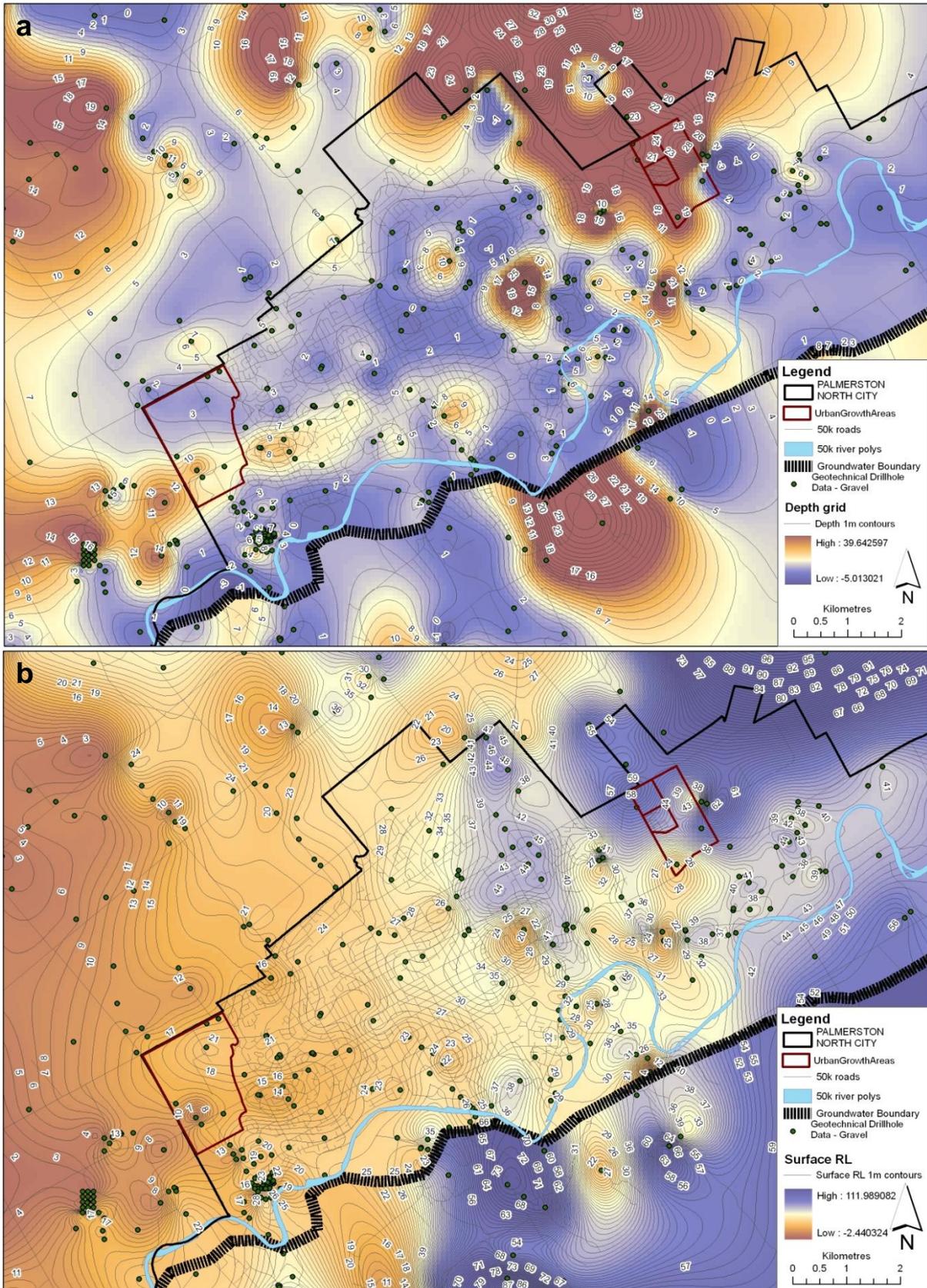


Figure 12 a) The modelled depth to the uppermost gravel unit in the Palmerston North City area. Data points (comb\_geotech\_drillhole\_depth\_to\_grav\_rl.shp) used to generate the model are shown as yellow points. 1 m contours indicate depth to the modelled surface. b) The modelled RL of the uppermost gravel surface with 1 m contours.

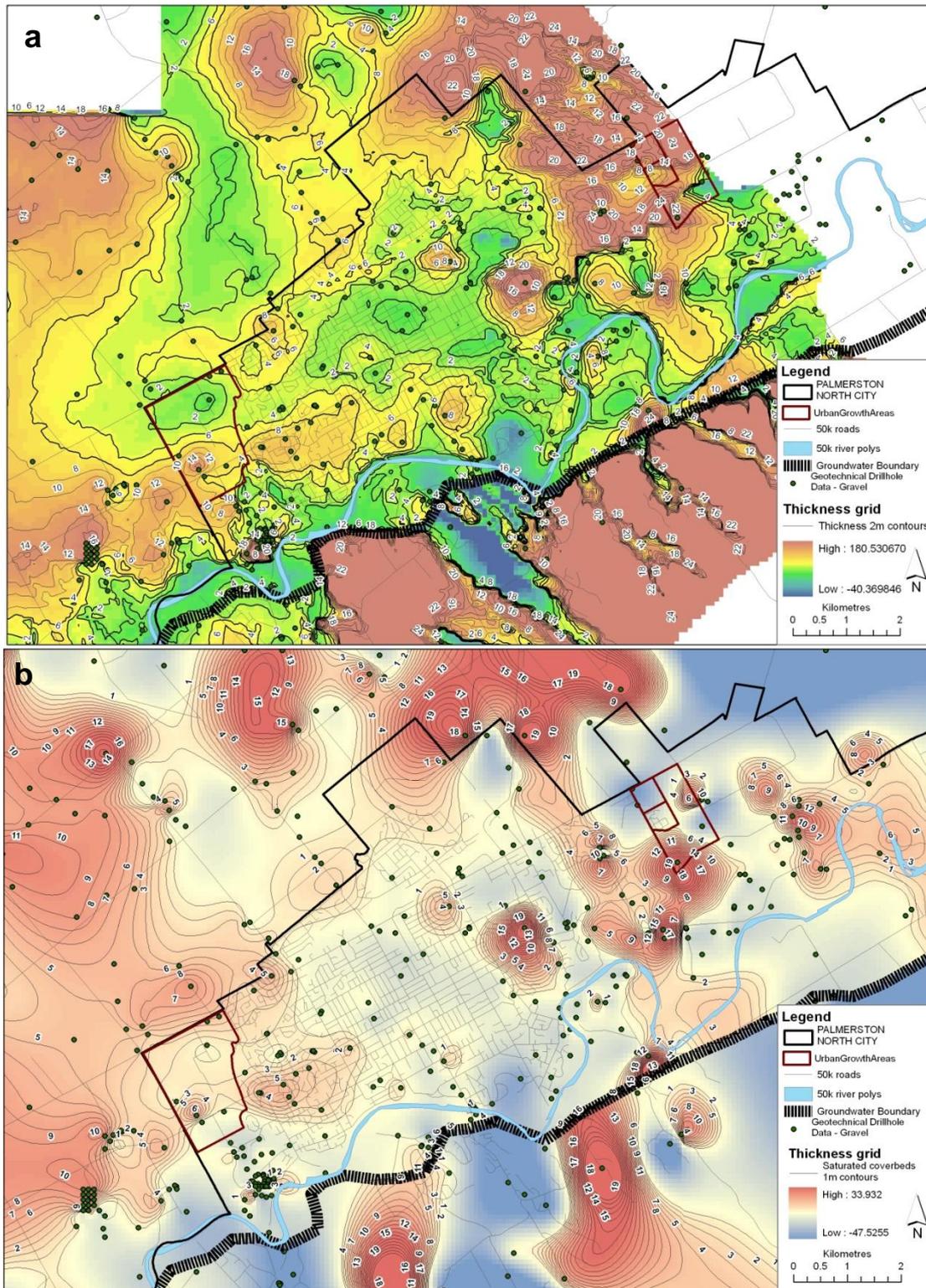


Figure 13 a) The modelled thickness of cover deposits upon the topmost gravel in the Palmerston North City area. One m contours are shown where positive values less than 20 m are indicated. High positive values are assumed to represent areas where gravel is deep and presumably materials are older marine Quaternary deposits. b) The modelled thickness of saturated cover materials (on gravel) that may have high liquefaction susceptibility. Two metre contours are present in areas that have thick cover sequences that may have high liquefaction susceptibility. Yellow areas are underlain by thin saturated cover materials.

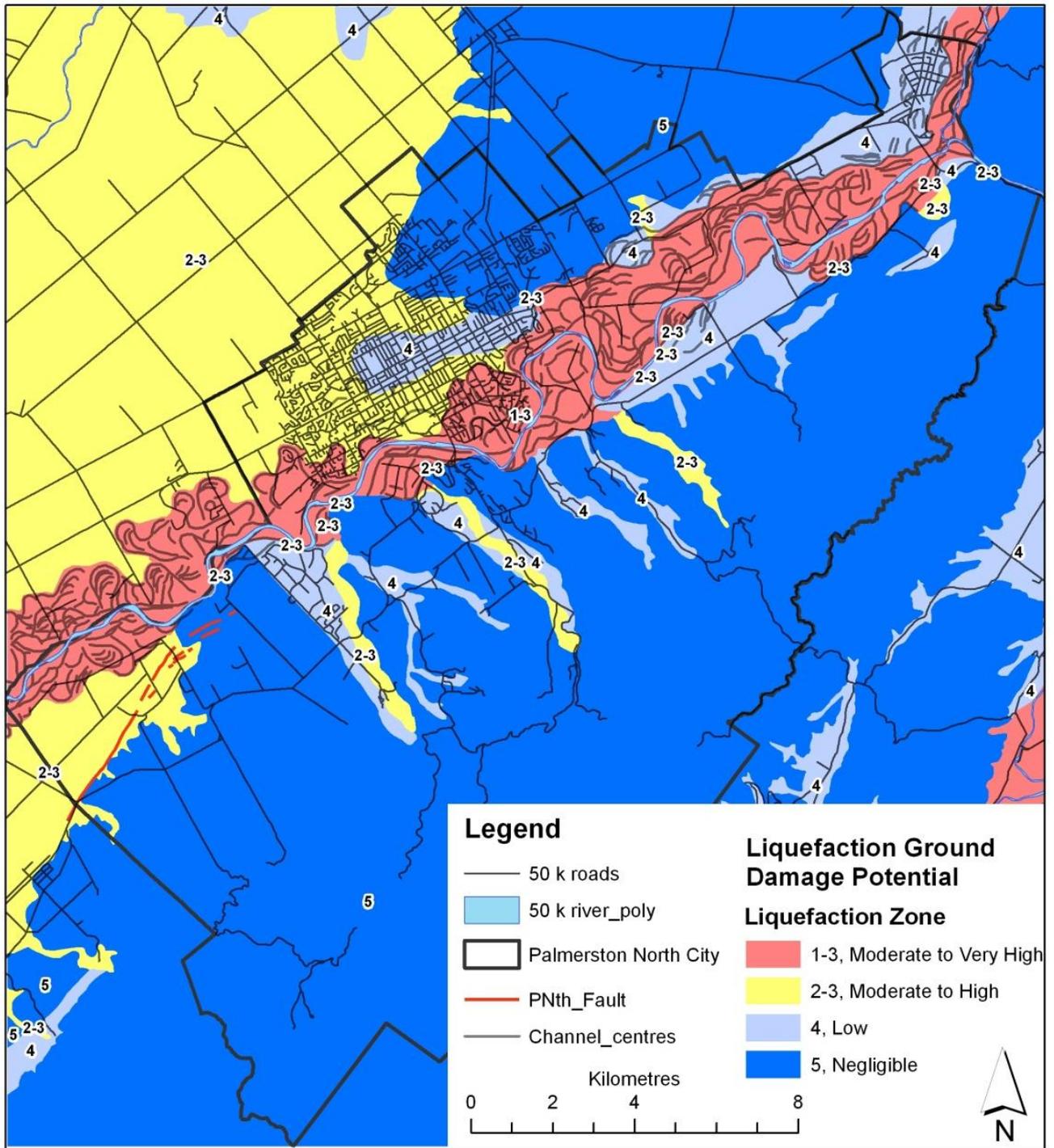


Figure 14 Enlarged version of the liquefaction ground damage potential map developed in this report.

- 1. = Very High Liquefaction Ground Damage Potential
- 2. = High Liquefaction Ground Damage Potential
- 3. = Moderate Liquefaction Ground Damage Potential
- 4. = Low Liquefaction Ground Damage Potential
- 5. = Negligible Liquefaction Ground Damage Potential

## APPENDICES

### APPENDIX 1 CYCLIC STRAIN SOFTENING

Laboratory studies show that both cohesive and non-cohesive soils can experience an increase in pore water pressure during cyclic loading. During the application of cyclic loading to a soil, slip occurs at the grain-grain contacts, causing a volume change. For a saturated, undrained soil the tendency for volume change results in the transfer of some of the intergranular stresses to the pore water. If the soil is contractive, a pore water pressure increase results, and if the soil is dilative, a decrease in pore water pressure occurs. This increase in pore water pressure is thought to occur at the major peak seismic stresses, or during the unloading part of the stress cycle. However, for sands at least, there appears to be a critical shear stress level below which no pore water pressure increase occurs.

Some researchers (e.g. Tsatsanifos, 1982) consider that the (lower) shear stresses induced by the initial part of a seismic motion record may contribute more to the development of pore water pressure within a layer than the higher stresses in the main part of the record. This is due to a decrease in effective strength resulting from the pore water pressure rise in the early part of the motion, which means *a reduced ability for the soil to transmit the higher shear stresses*. Consequently these later higher shear stresses may have a reduced effect upon the pore water pressure development in a soft soil, but will lead to greater displacements and non-linear behaviour in the weakened soil. A number of researchers have found that the duration of large shear stresses during seismic motion is a major factor governing pore water pressure development.

In undrained cyclic triaxial tests on a fully remoulded silty clay, Ogawa *et al.* (1977) found that even for a large number of stress cycles, the pore water pressure in the sample would not quite reach the full confining pressure to give complete liquefaction. Moreover it was found that even after cessation of the cyclic loading, the pore water pressure continued to rise for some minutes to almost reach the confining pressure. This result raises the possibility of post-earthquake shear strength failures. In such a case, actual failure may be induced by an aftershock. During cyclic triaxial testing of clay, as the pore water pressure increases, the strain amplitude also steadily increases until eventually a yield point is reached, after which the *axial strain increases* and the deviatoric stress decreases rapidly as the number of cycles increases.

This type of plastic, non-linear soil behaviour can lead to large permanent displacements at the surface of a soil mass during a strong earthquake, such as experienced in the epicentral area of the 1987 Edgecumbe earthquake, which may be damaging to buried services (pipelines, etc.) paths, roads, rail lines and poorly designed structures (see Section 2.6.3)

In summary, therefore, it can be seen that the development of excess pore water pressures leading to the liquefaction of a soil mass during earthquakes depends on:-

- Soil grain size, permeability, and thickness
- Soil layer up to 20 m below ground surface
- Depth to water table of about  $\leq 5$  m (possibly 10 m) below ground surface
- Duration and intensity of strong shaking (at least MM 7, or usually MM 8 for liquefaction)

- Hydraulic boundary conditions and confining pressures
- Level of stress and amplitudes of strain in soil mass

The situation where the pore water pressure generated during an earthquake becomes equal to the overburden pressure (i.e. a situation of zero effective stress exists in the soil) has been called "true" or "complete" liquefaction, and available evidence indicates that "true" liquefaction occurs commonly in non-cohesive, uniformly graded fine sands and coarse silts. [Note that the ASCE Committee (1978) discourages the use of such qualifiers in relation to liquefaction as they can create confusion.]

Liquefaction generally occurs at strong levels of ground shaking. On the Modified Mercalli intensity scale (Appendix 9), liquefaction effects become evident at MM Intensity 7, at distances of 100-150 km from the epicentre for earthquakes of magnitude 7 to 7.5, and up to 400 km from the epicentres of great (M 8-9) earthquakes (Ambraseys, 1988).

The condition where cyclic stresses induce increased pore water pressures, reduced soil strength, and plastic or non-linear behaviour, leading to permanent ground deformations (**cyclic strain softening** or **cyclic mobility**), can occur in both saturated cohesive and non-cohesive soils. Field evidence indicates that cyclic strain softening occurs most commonly in "soft" soil sites (where the resulting ground damage is most apparent), and at higher intensity shaking (MM 8 or greater) generally close to the epicentral areas of large-magnitude earthquakes.

It is clear that "true" liquefaction can lead to significantly greater ground damage and is potentially more destructive than ground damage associated with cyclic strain softening. However, unless there is detailed and specific sub-surface information available on the nature of the soils at a particular site, it often cannot be readily determined which has caused the resulting ground damage. Hence in many cases they are not differentiated and both are often included under the general banner of "liquefaction".

## APPENDIX 2 CASE STUDIES OF LIQUEFACTION ASSOCIATED WITH HISTORIC EARTHQUAKES

Some earthquakes that have produced significant damage due to liquefaction, and which have been well studied, serve to illustrate a variety of different points relevant to this study. Five events are briefly discussed below.

### The 1964 Niigata Earthquake

Although the epicentre of the Niigata earthquake ( $M \approx 7.5$ ) was located some distance (about 56 km) from Niigata, and the maximum ground accelerations recorded in the city were only about 0.16 g, the earthquake induced extensive liquefaction in the low-lying areas of the city. Water began to flow out of the ground from boils and cracks during and immediately after the earthquake. This liquefaction caused widespread damage. Many buildings settled by more than 1 m in the liquefied soil, often accompanied by severe tilting. Thousands of buildings collapsed or suffered major damage as a result of these effects.

The city is built upon a deep deposit of uniformly graded medium sand. Following the earthquake an extensive survey of the distribution of the damaged structures was made. It was found that buildings in the coastal dune area (Zone A) suffered practically no damage. The major damage and liquefaction were concentrated in the lowland areas, where two distinct zones (B and C) could be clearly recognised. In Zone B the damage was relatively light, but in Zone C the damage and liquefaction effects were most extensive. Since all zones contained similar types of structures, the differences in the extent of damage could be attributed to differences in the subsoil and foundation behaviour.

Studies were subsequently carried out to determine the differences in soil conditions in the three zones. These showed that the difference in behaviour in Zone A from that in Zones B and C could readily be attributed to two major differences in soil characteristics. Although all zones were underlain by sandy soils to a depth of approximately 30 m, in Zone A the sands were considerably denser than those in Zones B and C, and the water table was at a much greater depth beneath the ground surface. However, in Zones B and C the general topography and shallow depth to the water table was essentially the same. Hence the difference in extent of damage was considered to be related to the characteristics of the underlying sands. Accordingly considerable effort was made to determine any significant differences in the general soil conditions in those zones.

As the soils involved are sands, subsurface investigations efforts were concentrated on the determination of their relative density by means of standard penetration test (SPT) probing. Although there is considerable scatter in the SPT results from any one zone, a clear trend emerges showing that the sands below 4.5 m depth are slightly denser in Zone B than Zone C. Down to 4.5 m depth both zones are essentially the same and below about 14 m depth the sands in both zones are relatively dense and are unlikely to be involved in liquefaction. It is therefore concluded that the relatively small differences in penetration resistance (of about 5 to 10 blows per 300 mm) reflect the differences in sand density that were responsible for the major difference in liquefaction and foundation behaviour in the two zones.

At Niigata, extensive studies were also made of the influence of foundation type on settlement and tilting. Raft foundations were compared with piles, and piles of various length were studied (Seed and Idriss, 1982). The publishing of all these results has made available

much information from the extremely damaging earthquake which is invaluable for others planning and designing to mitigate earthquake hazards.

### **The 1964 Great Alaskan Earthquake**

The M<sub>w</sub>8.3 Alaska earthquake of 29 April 1964 produced major and widespread liquefaction effects, including several massive landslides (spreads and flows) in the cities of Anchorage, Seward, and Valdez, and around the shores of Kenai Lake (Seed, 1968). The landslide at Valdez led to a decision to relocate the entire town on stronger foundation materials several kilometres away. Damage in Seward and Anchorage was catastrophic.

The massive landslides occurred on gently sloping ground when extensive cohesionless sandy silt soil layers were sufficiently weakened by earthquake-induced cyclic strain softening and/or liquefaction, resulting in massive translational flow failures.

On relatively flat ground away from the towns, soil liquefaction caused extensive damage to a wide variety of bridge foundations and approaches at locations 80 to 130 km from the epicentral area. Damage included horizontal movement of abutment foundations towards stream channels, spreading and settlement of approach and embankment fills, horizontal displacements and tilting of piers, and severe differential settlements of abutments and piers.

The greatest concentrations of severe damage occurred in places characterised by thick deposits of saturated, cohesionless soils. Ample evidence exists of liquefaction of such materials during the earthquake, which is likely to have played a major role in the development of foundation displacements and observed damage to bridges and other structures. Typical foundations in these areas consisted of piles driven through saturated sands and silts of low to medium relative density (SPT N values of about 20 to 25). Of about 60 tested samples from the heavily damaged areas, two-thirds were uniform fine sands. By contrast, bridges founded on gravels or gravelly sands, regardless of their penetration resistance (SPT) values, generally showed little or no displacements, indicating that such materials have higher resistance to liquefaction during large earthquakes.

### **The 1987 Edgecumbe earthquake**

On 2 March 1987 a M 6.3 shallow (~8 km) earthquake occurred underneath the Rangitaiki Plains close to the township of Edgecumbe in the Bay of Plenty. The earthquake caused widespread damage throughout the Bay of Plenty area, with much of the damage due to permanent inelastic ground displacements caused by liquefaction of the saturated alluvial deposits of the Rangitaiki Plains. During the earthquake, eyewitness accounts in the epicentral area around Edgecumbe describe the ground as rolling, with surface waves up to 0.75 m in height, so that people were unable to stand. The duration of strong shaking (>0.05 g) for the earthquake was 8-9 seconds, recorded at Matahina Dam 11 km from the epicentre.

The area most significantly affected by the earthquake was a low-lying, swampy flood-plain adjacent to the Tarawera, Rangitaiki and Whakatane rivers. The flood plain has been formed during the Holocene (last 10 000 years) by the deposition of large amounts of volcanic-derived pumice alluvium (gravels, sands and silts), which have buried pre-existing river channels, and incorporated varying amounts of peaty material. Ground damage caused by the earthquake included ground surface fault ruptures up to 2.5 m in height and 7 km long, widespread but mainly minor failures of surficial materials on steep slopes, incipient cracking

on ridges, slopes and cuttings, and widespread liquefaction effects and localised ground settlements across the plains (Franks *et al.* 1989).

Widespread subsurface liquefaction was indicated by the ejection of sand, silt and water from linear cracks and holes in the ground forming sand boils, and more significant ground damage was indicated by well developed fissures, sand ejections, and lateral spreading failures along the banks of river and stream channels, particularly along the banks of the Whakatane and Rangitaiki rivers.

Most of the soils underlying the Rangitaiki Plains are saturated, with the water table ranging from close to the ground surface in the coastal strip, to about 3 m below ground surface further inland (at Te Teko). The area of sand boils was restricted mainly to the immediate epicentral area of the Rangitaiki Plains. Within this area probing indicates that the loose saturated alluvial materials with low penetration resistance are some 10 to 20 m thick, and overlie dense sands.

The main ground damage that resulted from the liquefaction processes was that caused by lateral spreading. Such damage included open tension cracks, sand ejections, and settlements of bridge approach fills and flood-control and roading embankments close to the main rivers. There was also widespread (minor) slumping of stop banks.

Very strong shaking and the resulting ground damage in the epicentral area caused widespread damage to buildings, roads, railways and underground services, especially in and around Edgecumbe. Such damage included severely bent and distorted railway lines, compressional ripples and tensional cracking in the asphalt surfacing of roads, compressional ruptures of concrete paving and kerbs, and severe compressional damage to underground reticulation services. It is considered that the soft, loosely compacted sediments around Edgecumbe have experienced inelastic deformations during the main earthquake, producing zones of permanent ground deformation and damage (Franks *et al.* 1989). It was apparent that modern, well-constructed dwellings built on good foundations such as a monolithic, well reinforced concrete slab, suffered virtually no structural damage in the event, and were not influenced by the ground damage.

Localised elongate ground depressions or areas of subsidence, up to 0.3 m in depth and up to 100 m wide, were common across the plains. These depressions appear to coincide with near-surface peaty soils, and are considered to be the result of differential settlements associated with earthquake induced compaction of the peat materials.

### **The 1994 Kobe Earthquake**

The extensive, damaging liquefaction and lateral spreading caused by the Kobe Earthquake (also alternatively referred to as the Hyogo-ken Nanbu and the Great Hanshin earthquake) are referred to in New Zealand reconnaissance team reports by Park *et al.* 1995 and Brunson *et al.* 1996. Kobe is a coastal port city with extensive wharf and reclaimed land areas which were badly damaged by liquefaction and lateral spreading.

### **The 2010 Darfield (Canterbury) Earthquake**

A recent damaging earthquake in New Zealand was the September 4, 2010, moment

magnitude ( $M_w$ ) 7.1 earthquake. It occurred on a buried fault beneath the Canterbury Plains near Darfield, some 30km West of Christchurch. The earthquake rupture has a ~30km long complex surface fault trace which is now called the Greendale Fault. It was an unexpected and possibly rare (long recurrence time) event at this location. Extensive damage was caused to lifelines and residential houses due to widespread liquefaction and lateral spreading in areas close to rivers, streams and wetlands throughout Christchurch and Kaiapoi. Unreinforced masonry buildings also suffered extensive damage throughout the region. Despite the in places severe damage, fortunately no one was killed and only two serious injuries were reported. From the engineering point of view, one can argue that the most significant aspects of the 2010 Darfield Earthquake were geotechnical in nature, with liquefaction and lateral spreading being the main causes of the inflicted damage (Allen, *et al.* 2010).

Immediately following the Darfield Earthquake an intensive geotechnical reconnaissance was conducted to capture evidence and perishable data. Surveys were carried out by foot, car and helicopter to record general and detailed observations and to make field tests, including dynamic cone penetration tests, Swedish Weight Soundings and Spectral Analysis of Surface Waves (Allen, *et al.* 2010). Following the immediate geotechnical reconnaissance, intensive work led by Dr. Misko Cubrinovski of Canterbury University has continued on studies and analyses of liquefaction and related effects. Where possible and relevant, the findings of the Darfield Earthquake liquefaction studies have been incorporated into this report work.

### **The 2011 Christchurch Earthquake**

On 22 February 2011 at 12.52pm, a  $M_w$  6.2 earthquake occurred under the northern margin of the Port Hills with its epicentre under the Heathcote Valley. This smaller magnitude earthquake caused far stronger shaking than the Darfield Earthquake 6 months earlier, with associated increased building damage and collapse, a large number of casualties and deaths, and increased liquefaction and landslides (rockfalls) on the Port Hills.

A full assessment of this earthquake is still underway. Areas that showed no surface signs of liquefaction in the Darfield Earthquake (MM7 to 8 intensity in the city) were extensively liquefied in the later earthquake, which caused MM 9 to 10 shaking in the city.

Once fully assessed, the two Christchurch earthquakes are bound to lead to changes in building codes and standards which will improve building and construction requirements throughout New Zealand.

## **APPENDIX 3    PHYSIOGRAPHY OF THE PALMERSTON NORTH AREA**

JG Begg

This appendix provides a geomorphic context for the brief notes made within the text, and the spatial distribution of the units discussed are illustrated in Figure 4.

### **Axial ranges**

The northern Tararua Range dominates the eastern Palmerston North landscape and rises to an elevation of 767 m at Arawaru at the southeast end of the City area. The axis of the range trends approximately northeast-southwest, and the structural grain (and the alignment of streams and gullies) within the eroded part of this area is similar. An increasingly intact remnant of an ancient (c. 2 Ma) erosion surface is preserved as elevation decreases toward the Manawatu Gorge from the high part of the Tararua Range in the south. It takes the form of a gently folded surface (the fold axis again aligned northeast-southwest) of gentle relief, with relatively steep north-western and south-eastern flanks.

### **Elevated southeast terraces**

The land between the axial ranges and the Manawatu River is characterised by its slope from the ranges towards the Manawatu River, by deeply eroded streams with planar-crested ridges between.

### **Manawatu River floodplain**

The Manawatu River floodplain has a reducing gradient from the mouth of the Manawatu Gorge to the southwest edge of the City boundary. The low-lying floodplain is characterised by dense active and abandoned channelling, little (or only partially) infilled by subsequent sedimentation (e.g. including the historical Te Ngutu, Hokowhitu, Awatapu and Awapuni lagoons).

### **Elevated northwest terraces**

An area in the northwest of the City consists of elevated terrace land at the southern extension of the Braemar-Hiwinui ridge.

### **Low-lying plains**

The southwestern part of the City includes the eastern margin of the Manawatu Plains.

## **APPENDIX 4 SUPPLEMENTARY PHYSIOGRAPHIC INFORMATION FROM LIDAR ANALYSIS**

S Levick, JG Begg

The lidar information reviewed in this study provides an understanding of the landscape one or two orders of magnitude more precise than available using conventional methods. The value of the data is summarised briefly in the section within the main text body, and this appendix is designed to supplement that summary and substantiate our interpretations.

### **Elevated southeast terraces**

Surface remnants of the terraced land on the southeastern side of the Manawatu River dip gently northwest at slopes ranging from 0.5° to 4°. Between terrace surface remnants, larger streams are deeply incised (c. 30 m) and commonly include a flat-floored aggradation surface that dips at a similar slope to the older ridge-crests on each side (e.g. Tiritea Stream). This aggradation surface is itself incised (c. 20 m) by Holocene alluvial degradation. Smaller streams are V-shaped with convex-walls. The northwest margin of this terraced area is truncated and cliffed by alluvial processes of the Manawatu River.

### **Channelled floodplain of the Manawatu River**

The active floodplain of the Manawatu River is characterised by relict sinuous alluvial channel forms of the late Holocene Manawatu River (Figure 15). The depth of well-preserved channels on the floodplain ranges from < 2 m to c. 5 m.

### **Elevated northwest terraces**

Some parts of Palmerston North City are built upon older, elevated terraces that no longer preserve channel features at the surface; a series of these terraces rise to the hill crest behind Kelvin Grove.

### **Low-lying Manawatu Plains**

The Lidar DEM and derivative hillshades show that much of the area previously described as the Manawatu Plains (Begg & Johnston 2000) consists of late Holocene levee-ed stream systems and their associated deposits (Figure 16). The perching elevation of such levee-ed streams above the surrounding plains ranges from c. 2m to c. 5 m.

### **Active fault trace**

A sharp landscape lineament, c. 7 km long and striking c. 035°, is revealed by the lidar data in the extreme southwest corner of Palmerston North City (see Figure 10). The feature apparently extends southwest of the limits of lidar coverage and bears many characteristics of an active fault. The last interglacial surface is apparently downthrown to the west, and there is possibly a subtle displacement of the late Holocene surface material. The feature splays into several branches in its northern third, some with apparent downthrow to the NW, and some to the NE. A small graben feature is developed near its northeastern extent. In this report, it is listed in this report as a potential seismic source.

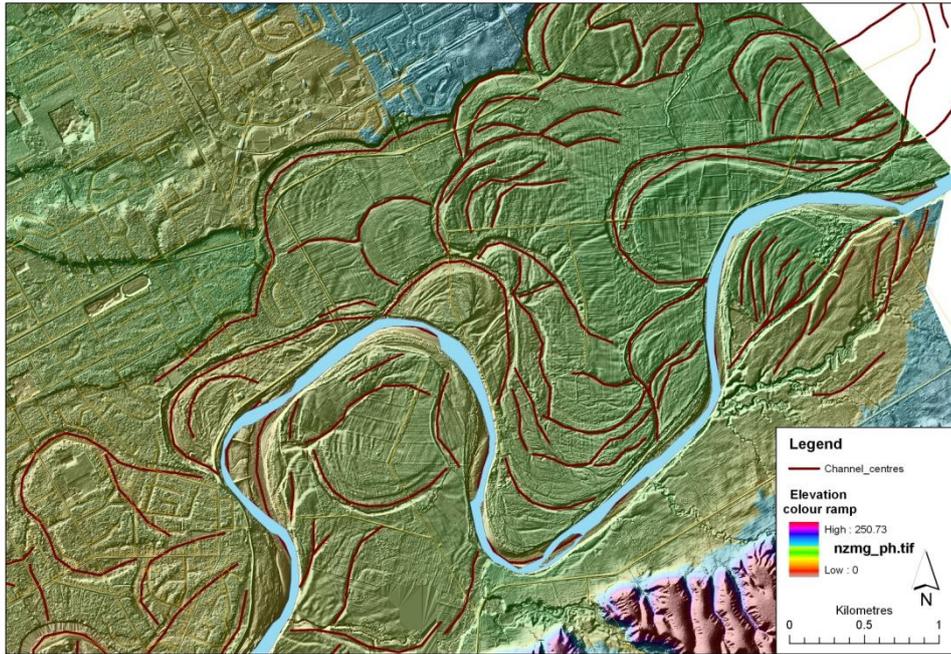


Figure 15 Palmerston North City lies adjacent to the late Holocene floodplain of the Manawatu River. The Lidar DEM allows a far more accurate definition of the active late Holocene floodplain which is characterised by densely spaced, partially infilled alluvial channel features.



Figure 16 Detailed colour ramped DEM of the western part of Palmerston North City illustrating the presence and extent of levee-ed stream systems on this part of the Manawatu Plains. The main stream, Mangaone Stream, is contained behind man-made stop-banks. The trotting track is at the lower centre of the image.

## APPENDIX 5 SOILS OF PALMERSTON NORTH

P. Barker

This appendix supplements the brief comments on soil types made in the main body of the text. A soil map of the Palmerston North City area is included in the illustrations to the main text (Figure 7).

### 1. Soils of the river flats

#### Rangitikei Series (1 -1f)

The Rangitikei soils are restricted to the low flat areas bordering the river. They are generally shallow in depth (< 1.5 m) and overlie weakly weathered and poorly sorted sandy gravels with lenses and bands of sands. There may also be occasional layers of silt. The soils are well drained but are subject to flooding.

#### Manawatu Series (3 – 3f)

The Manawatu soils border the Manawatu River on higher parts of the river flats than the Rangitikei soils and constitute the largest series within the city. The soils are well to moderately well drained and have been subjected to infrequent flooding. The soils are 1 – 3 m thick, and are underlain by weakly weathered and poorly sorted gravels.

Manawatu silt loam occurs in small areas of the river flats in the southern part of the city. Below about 0.4 m the sediments become sandier and gravels occur at a depth of about 3 m.

Manawatu mottled silt loam is the most extensive of the Series within the city. It occurs on lower lying ground than the Manawatu silt loam. Gravels are encountered at about 3 m.

Manawatu fine sandy loam is formed on slightly sandier textured alluvium. Gravels occur at about 3 m.

Manawatu sandy loam is formed on coarser textured alluvium and is confined within the city to the borders of present and old river channels. Gravels occur at about 1.8 m.

#### Karapoti Series (7 -7c)

Karapoti Series occur in only small areas of the higher parts of the river flats. They are about 1 m thick and overlie sands between 2 and 4 m thick. Beneath this there are weakly weathered sandy gravels.

#### Kairanga Series (4 - 4a)

These soils occur on the low-lying river flats in the north and south-western parts of the city. They are heavier in texture than the Manawatu soils and tend to be less well drained. There are two main types in the series based on differences in texture.

Kairanga silt loam occurs on the poorly drained heavier textured low-lying river flats in the south-western parts of the city. These soils are silt loam topsoil overlying clay to clay loam subsoils and are between 1.5 and 2.5 m thick. Below this, the most common material is a plastic silty clay loam.

There is also a small area bounded by Maxwells Line & Ronberg, Cardiff and College Streets where there is peaty layer about 0.3 and 0.6 m thick and 0.6 – 1.5 m deep.

Kairanga fine sandy loam. This soil is of limited extent in the city area. It differs from the Kairanga silt loam in that it is distinctly sandier in texture.

Within the city boundaries the gravels are often deeper than 6 m.

#### Te Arakura Series (8 – 8b)

Soils in this series occur in the west and north-western part of the city.

Te Arakura silt loam occurs on the lower lying areas in the north-western part of the city. Sand, which overlies gravels, occurs between 1.2 and 3.5 m.

Te Arakura fine sandy loam occurs in the north-western part of the city on levees of the Mangaone Stream. The surface is higher than the Te Arakura silt loam, and the boundary between the two soils is gradual. Below about 1.5 m depth there are bands of sands and clay loams which overlie gravels at about 3.5 m.

## **2. Soils of the Terrace Land**

### **Ashhurst series (13-13a)**

Ashhurst silt loam occurs in a small area near Linton military camp. Gravels with a sandy clay loam matrix are generally found at about 0.75 m depth.

Ashhurst silt loam, stony phase occurs on the terrace which runs through the centre of the city from the showgrounds through to Terrace End. In the south-east there is a distinct terrace edge about 2.5 m high which runs just east of Church Street and separates this soil from the Manawatu and Kairanga soils of the river flats. In the north-west the soil merges into Te Arakura soils without any distinct terrace edge. The sediments on this terrace are generally shallow with a thin surface of fine textured material overlying weakly cemented sandy gravels.

### **Milson silt loam (11)**

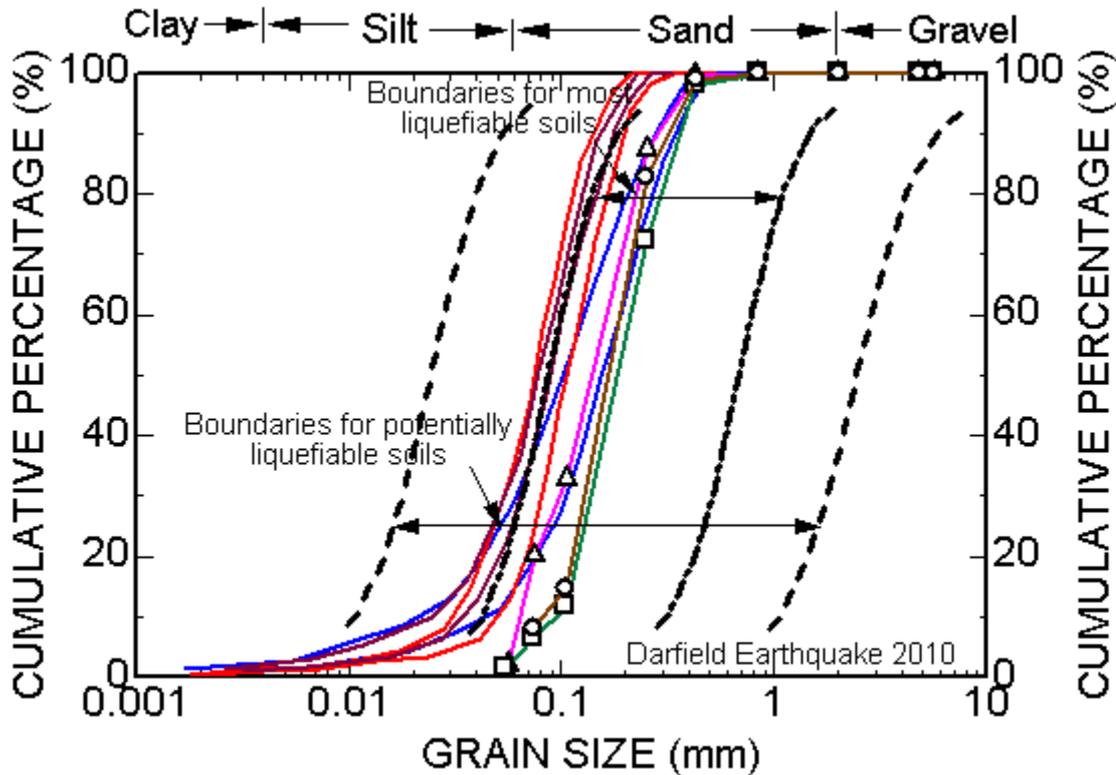
Milson silt loam occurs on the terraces (Milson Terrace) in the north-eastern part of the city. On the south-eastern and southern boundaries it is separated from the Ashhurst and Te Arakura soils by a well-defined terrace scarp; at Roberts Line it is 9 m high. To the west the scarp becomes lower and as it swings across to the airport it becomes only about 1 m high. Moderately cemented gravels, with some sand, are encountered at about 2.5 m depth.

### **Marton silt loam (12)**

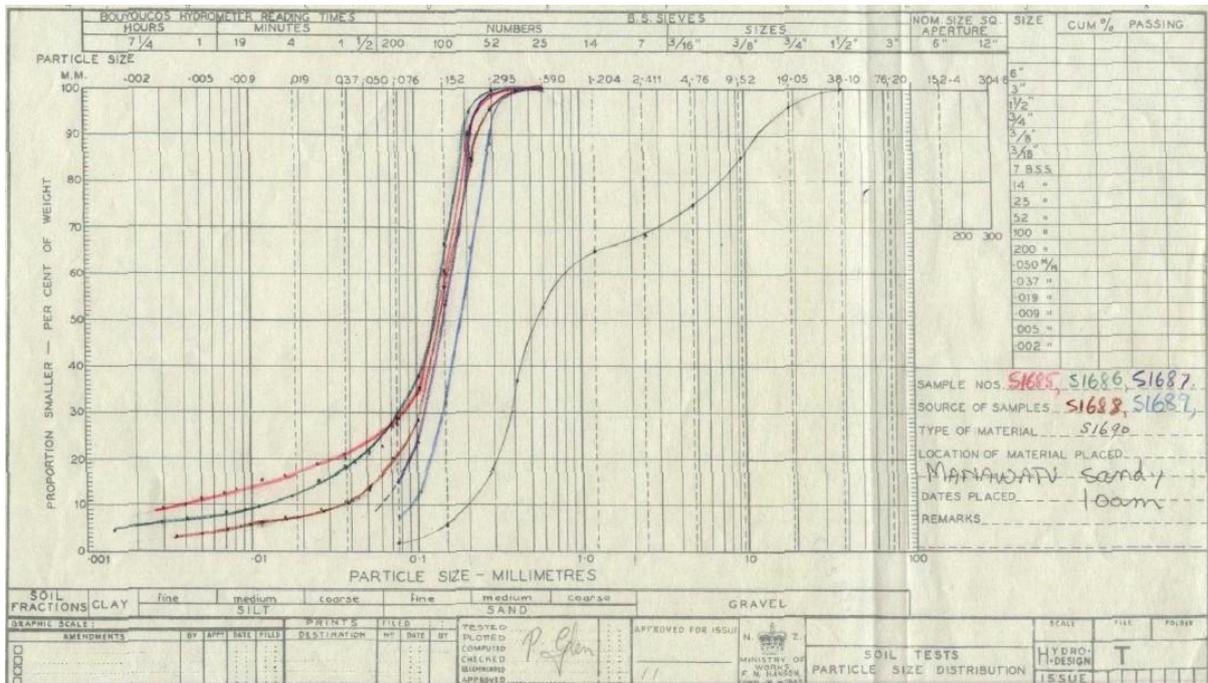
Marton silt loam occurs to the north of the city on a higher terrace to that covered by Milson soils. The terrace is dissected by moderately deep valleys resulting in a gently rolling topography with extensive flattish terrace tops.

Beneath the shallow silt loam topsoil the sediments become more clay rich to about 1.2 m depth. From 1.2 to about 4.2 m there is a layer of clay with lenses of concretions. This overlies a thin layer of banded sands above gravel.

**APPENDIX 6 TYPICAL GRADING CURVES FOR SOILS IN THE PNCC AREA**



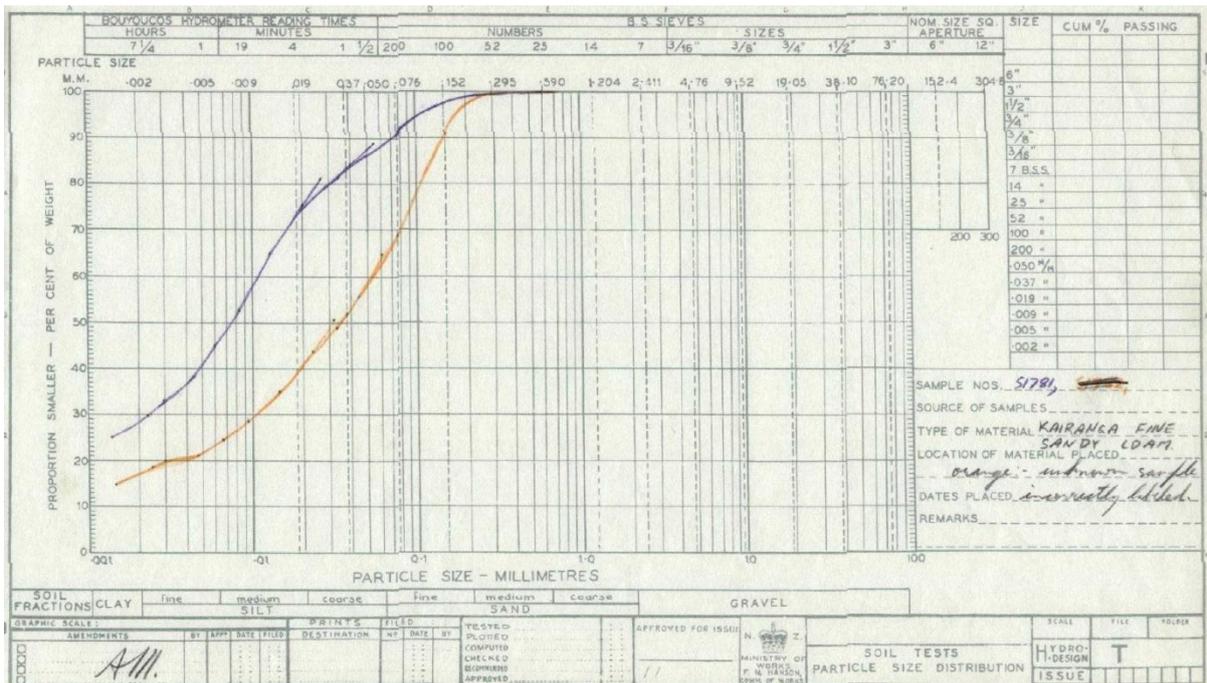
1. An example showing grading curves for liquefied soils from Christchurch. Samples taken after the Darfield Earthquake. These grading curves represent very highly liquefiable soils.



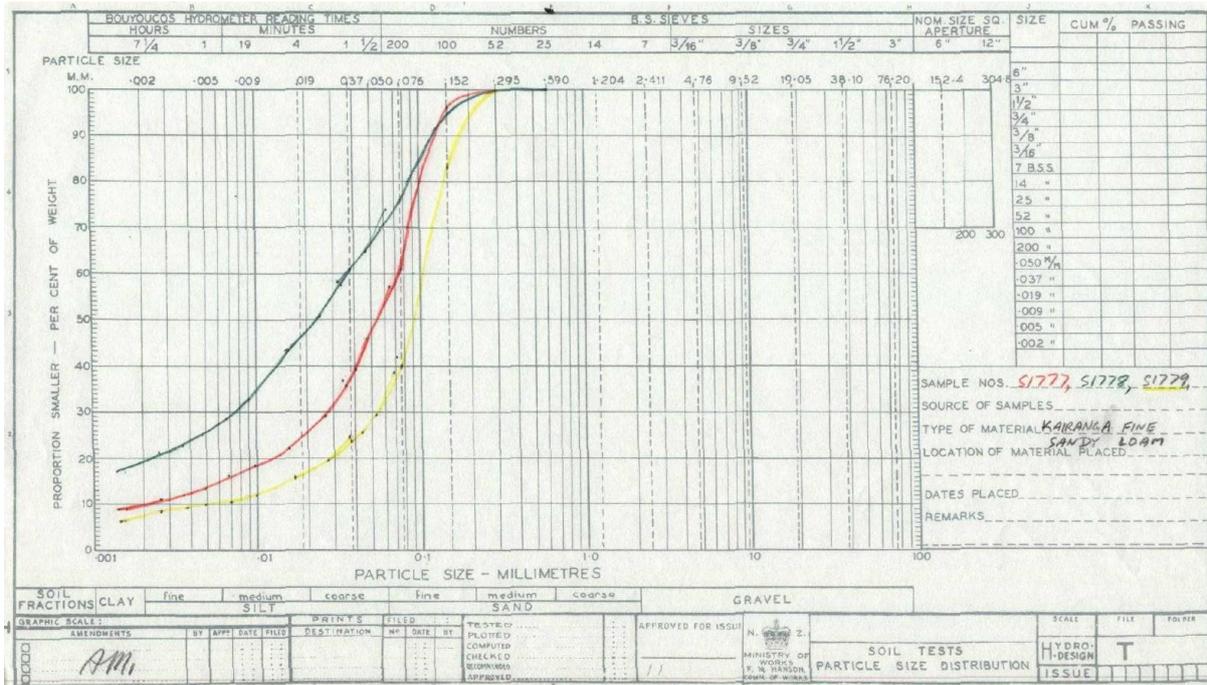
2. Manawatu Sandy Loam. Most samples have steep, uniform grading curves indicating very high liquefaction susceptibility. Such soils will liquefy when saturated.



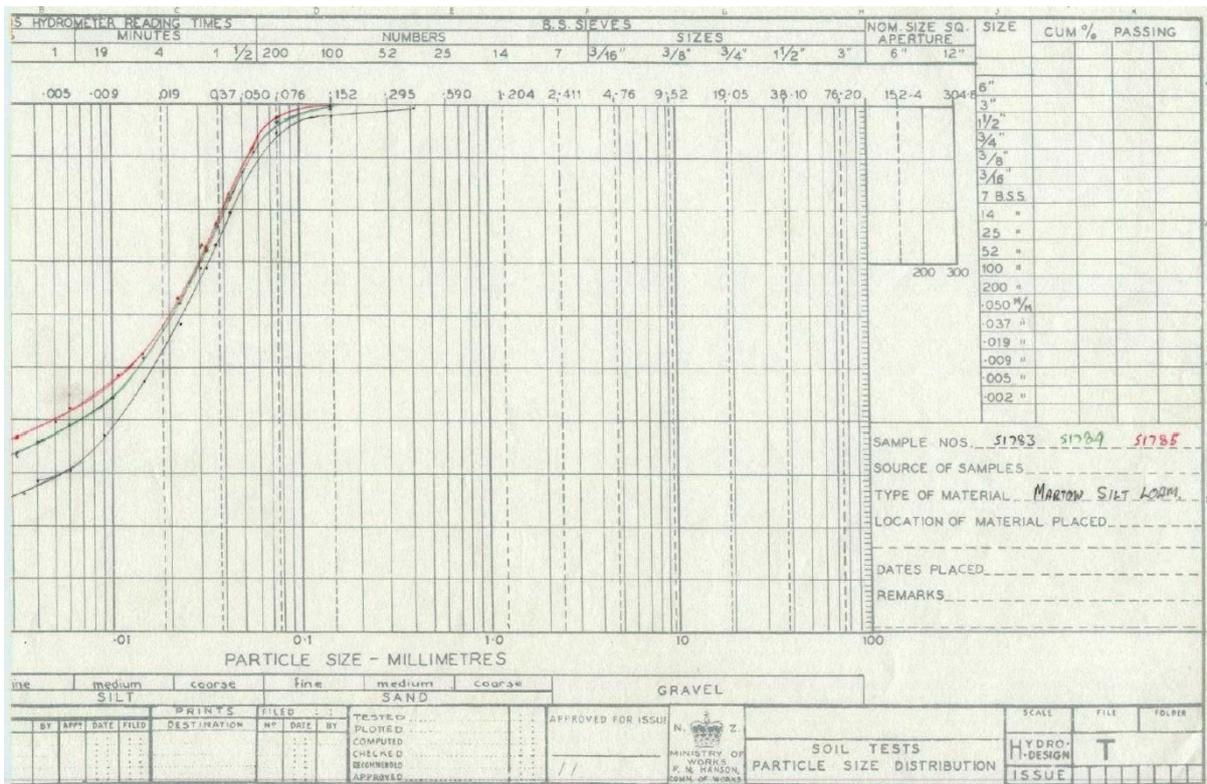
**3. Ashurst stony silt loam.** These grading curves indicate well graded soils with negligible liquefaction susceptibility.



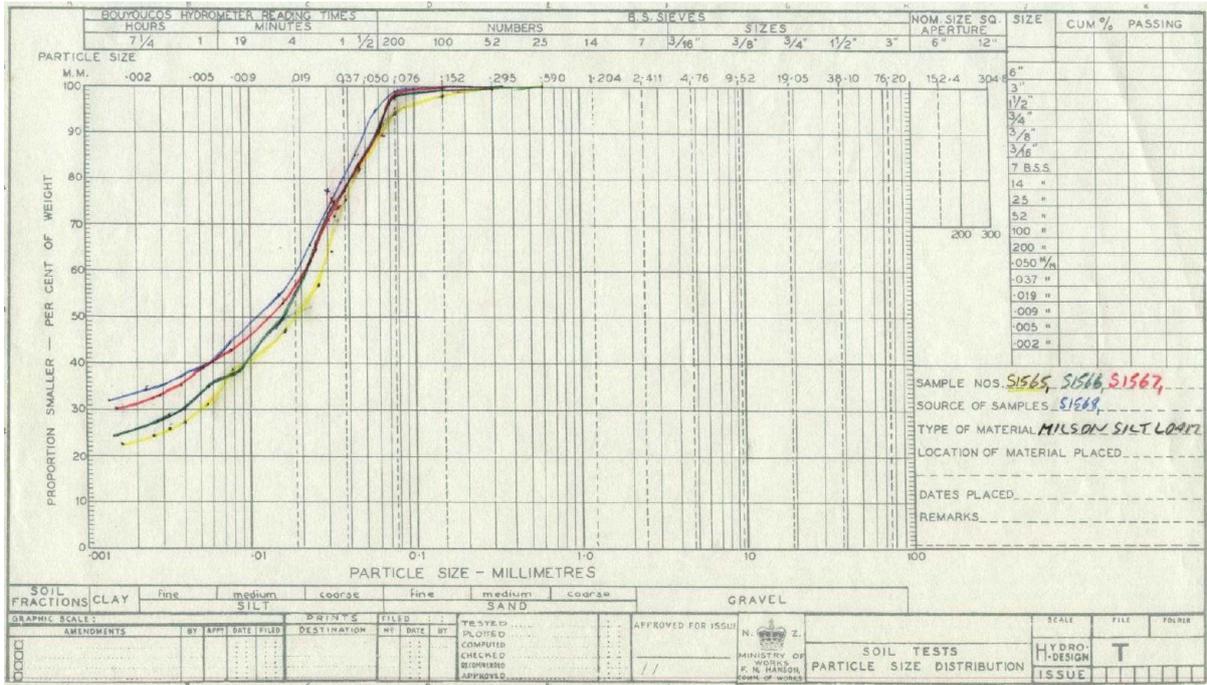
**4. Kairanga fine sandy loam.** These grading curves have ~20% fines which will tend to fill pore spaces. They are generally outside the range of susceptible soils.



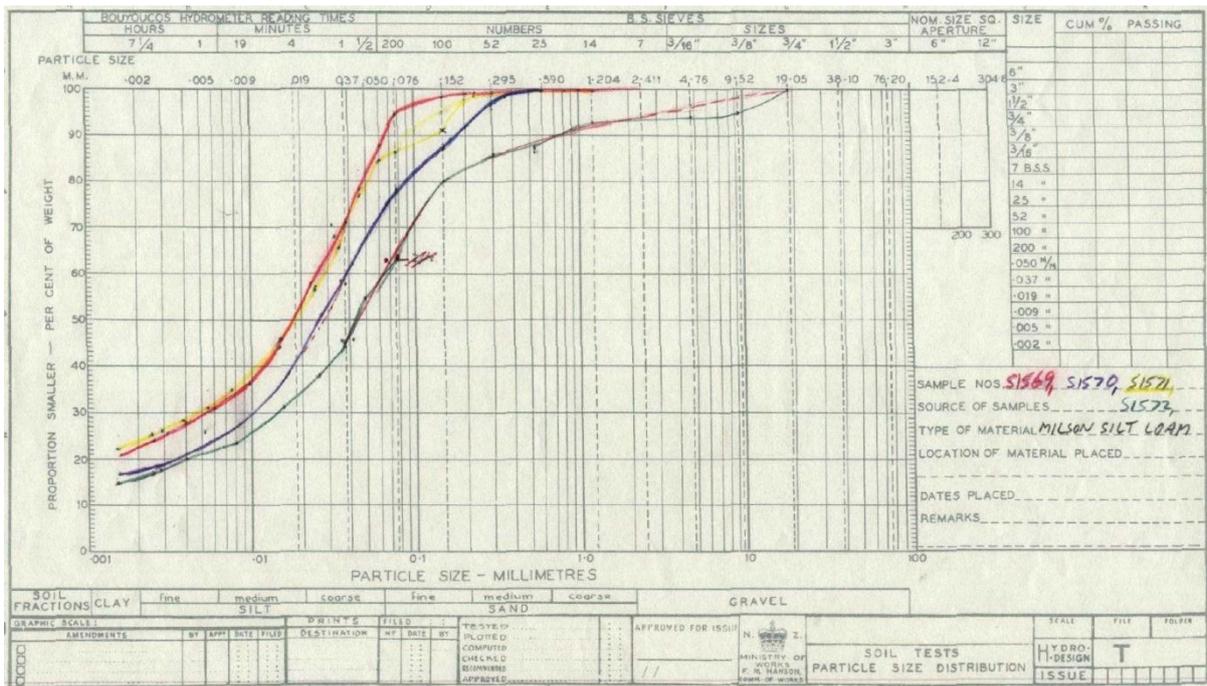
**5. Kairanga fine sandy loam.** The red and yellow curves are likely to be susceptible to liquefaction. The Kairanga soils have negligible to moderate liquefaction potential.



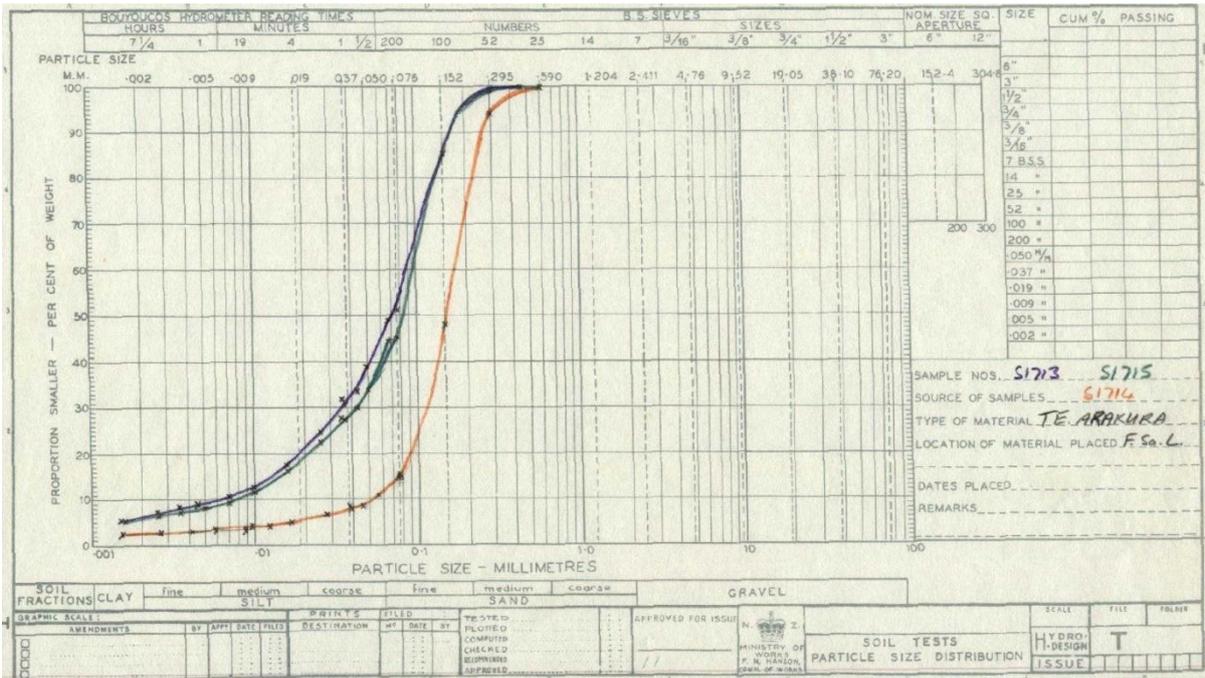
**6. Marton silt loam.** These curves are outside the range of liquefiable soils (see 1 above) and have a high (~30%) fines content. They have negligible liquefaction potential.



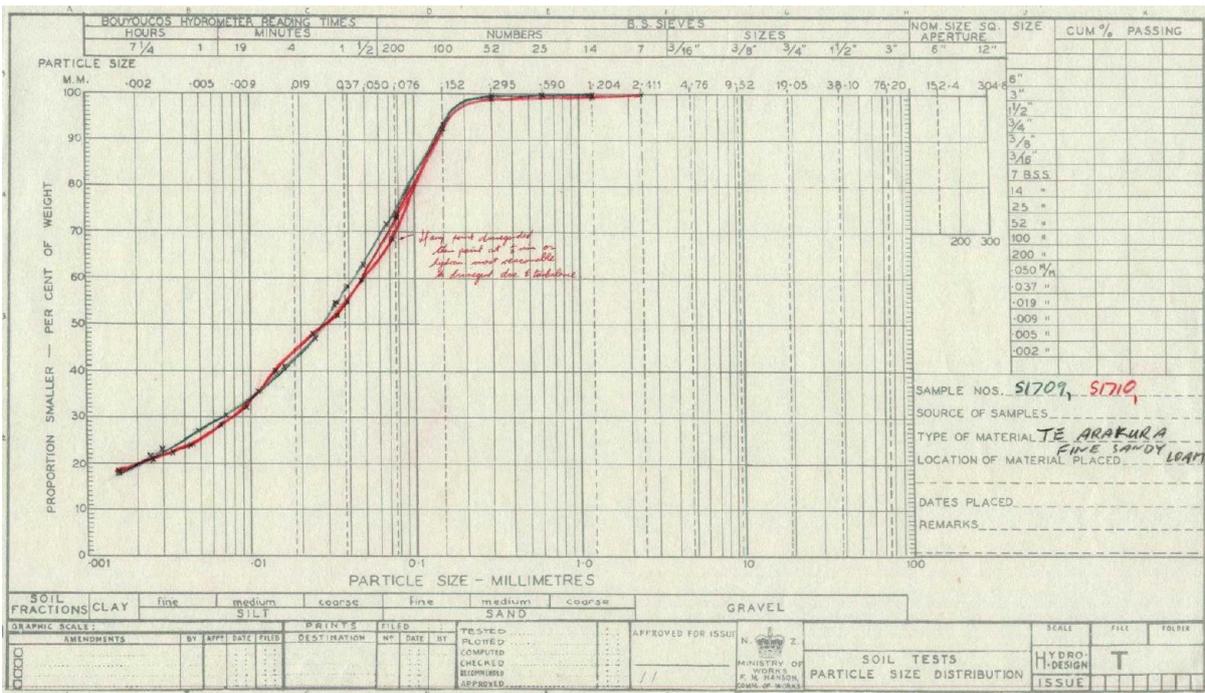
7.



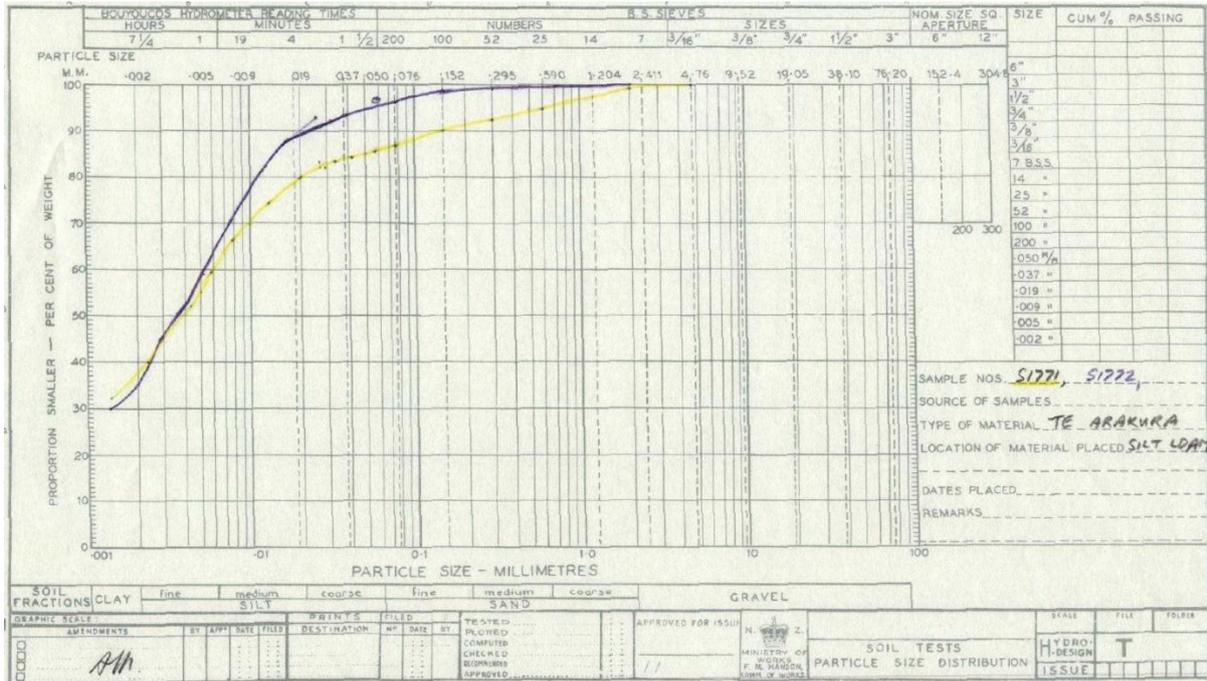
8. Curves 7 & 8 are Milson silt loam. They are outside the range of liquefiable soils and have negligible liquefaction potential.



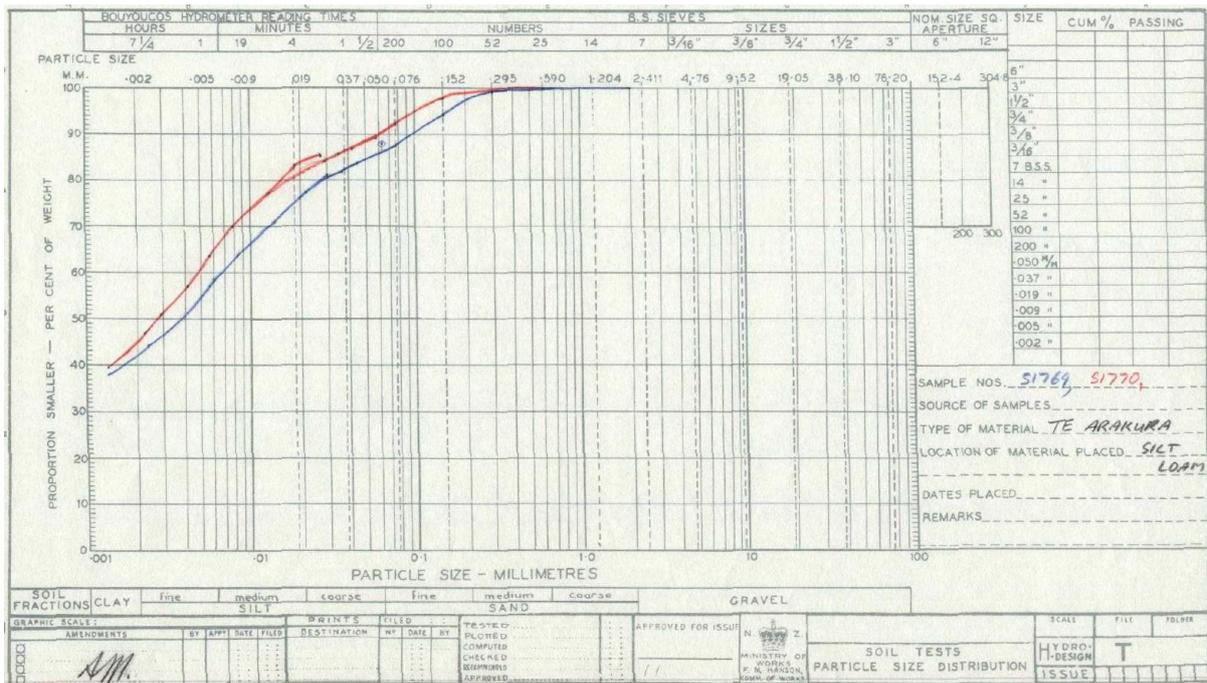
9



10. Curves 9 & 10 are TeArakura fine sand. Curves 9 show liquefaction susceptibility whereas 10 do not. We conservatively regard the TeArakura fine sand to be moderately to highly susceptible to liquefaction.



11.



12. Curves 11 & 12 are TeArakura silt loam. These soils have a high fines content and are outside the range of liquefiable soils. They have negligible liquefaction susceptibility.

## **APPENDIX 7 GEOLOGY OF THE PALMERSTON NORTH AREA**

JG Begg

The surface rocks and deposits mapped are described below in order of reducing age and elevation. Younger deposits on the valley floor are underlain by older units, the age of which increase with depth.

### **Basement rocks (“greywacke”) (age 260 – 150 Ma)**

This group comprises the old, hard greywacke rocks that form the Tararua and Ruahine ranges. They undoubtedly lie beneath the thick younger deposits of the Manawatu Plains in the Palmerston North area. The predominant rock type is hard sandstone, and interbedded mudstone. They are commonly thoroughly deformed, sometimes to the extent that they comprise mixtures of blocks floating within a sheared mudstone matrix. These rocks are hard and not prone to liquefaction.

### **Late Pliocene and Early Pleistocene marine deposits (age 3.6 – 0.7 Ma)**

These deposits consist mostly of siltstone, mudstone, sandstone, pumiceous sand, and minor limestone and conglomerate and are of continental shelf origin. They are compact and not prone to liquefaction.

### **Early to middle Quaternary deposits (age 700 - 128 ka)**

The Palmerston North area has been subject to repeated inundation by the sea during high sea level stands of the late Pleistocene. The area has been subaerial during the intervening cold climatic cycles for at least the last 400 ka. Materials are moderately consolidated to loose and comprise loose sand and silt (warm climatic phases) and sand, silt, peat, gravel, loess and tephra (cool climatic phases). These materials are mantled by loess deposits of increasing thickness with increasing age.

Marine benches, left stranded above sea level by ongoing regional uplift have been folded and faulted through the intervening period which has restricted river courses to areas between anticlinal crests, and alluvial gravels lap against older elevated deposits around the anticline axes.

### **Last Interglacial and early to middle last glacial deposits (age 128 to 24 ka)**

This unit consists of marine sand, gravel and silt (deposits of the last interglacial high sea level stand) overlain by alluvial gravel, sand, silt and peat (alluvial and swamp deposits of the early to middle last glacial period). Materials range in consolidation from moderately consolidated to loose. They are often overlain by loess and tephra coverbeds that vary (and increase) with the age of the deposit; these usually immediately underlie a flat or gently dipping surface.

### **Late last glacial alluvium (age 24 to 12 ka)**

This unit is listed separately to the previous one because it is less consolidated. It consists of little consolidated to loose alluvial gravel and sand, with minor silt and peat. In places, the

surface of these materials is covered by thin loess deposits, but in many places, alluvial gravel clasts lie immediately beneath topsoil. In some places, relict channels are preserved on this surface.

### **Holocene alluvium, silt and peat**

Poorly consolidated Holocene (Q1) alluvial gravel, sand, silt and peat deposits are present in the Manawatu River floodplain, as mantling deposits around levee-ed streams and beneath the Manawatu Plains.



## APPENDIX 8 GNS RESPONSE TO QUESTIONS FROM PNCC

June 2011

Palmerston North City Council  
Private Bag 11034  
The Square  
Palmerston North

Attention: Daniel Batley, David Murphy and Catherine Stapp

Dear Daniel, David and Catherine

### PNCC liquefaction assessment

*Thank you for your warm and positive reception when we visited to deliver our report to you. For us it has been an interesting and thought provoking task that we have enjoyed being involved with.*

*I respond below to the questions posed in you memorandum dated 31 May 2011*

**Thank you for your draft report and findings for the potential risk of liquefaction and related ground failure within Palmerston North City. The report provides some useful conclusions and recommendations regarding future development within Palmerston North. Thanks to you and the GNS team for coming up to Palmerston North to present the report. The Council officers involved in the presentation found it very useful.**

**We have now had the chance to assess the report and bring together feedback from within Council to formulate a response. In an endeavour to finalise the report and make progress on the implementation of Residential Growth Strategy work, there are some points of clarification sought. The questions are as follows:**

*Our report has now also been reviewed internally by two of GNS staff. Their main comment is that it is too long.*

*As discussed with Daniel we have located soil grading curves done by Soil Bureau and now archived at Landcare Research. We hope to have copies of these within a few days and will include them in a new Appendix. These curves will help us resolve the liquefaction susceptibility of the soils mapped by Soil Bureau and will thus help us refine our map. In my view it is essential to include these in our report.*

*I understand also from comments made by Daniel that Catherine may have located some soils data at PNCC? If this is the case we could include that as well.*

*I do not expect that these refinements and additions to our report will alter the main conclusions and recommendations we have given you. So in the mean-time you could use the draft report.*

- 1. Some parts of the Kelvin Grove and Anders Road residential growth areas, in addition to some existing residential areas within the City, are susceptible to flooding and require earthworks to take place to raise ground levels. The existing residential area on the lower terrace bounded by Roberts Line, James Line and Napier Road requires a 2m increase in the ground level (including foundations) to mitigate potential flooding issues. What impact would such extensive earthworks have on the risk and occurrence of liquefaction?**

*The Christchurch suburb of Bexley consisted of 1.5 to 2m of fill over estuary wetland. The estuary wetland contains ~10m of fine sand and silt sediments which are highly susceptible to liquefaction and it was not consolidated prior to filling. This area performed very poorly during the earthquake with extensive liquefaction and lateral spreading causing differential settlements of house concrete slab foundations, which are generally only lightly reinforced. About half a metre thickness of sand was expelled in each of the Darfield and Christchurch earthquakes, leading to huge clean-up efforts and settlement of the ground surface by about a metre. There is so much liquefaction damage to houses that it may not be viable to rebuild there (in Bexley).*

*A geotechnical assessment is required of your proposed areas to determine:*

- 1. If the materials are susceptible to liquefaction. To be susceptible they need to be uniform and fine grained and they need to be saturated (these criteria were both met at Bexley).*
- 2. If they are susceptible it may be viable to economically treat the area at the time of filling by using drainage, compaction, stone columns, etc., or some combination of these. Treatment at the time of filling would allow development of an earthquake mitigated subdivision with more standard (but adequately designed) house foundations, buried services (as shallow as possible) that will not float to the surface in strong earthquake shaking and the subdivision would not have risk of liquefaction and lateral spreading.*

- 2. Bearing in mind that there may be a number of unknown active faults within the City that may cause ground failure that could in turn result in liquefaction; one question that has been raised by Council engineers relates to potential risk of liquefaction versus the potential occurrence of an event that would cause liquefaction. As the fiscal cost of mitigating the impact that liquefaction can have on a dwelling, building or infrastructure is substantial, if an event is not forecasted to occur for a long period of time then what is the risk of doing nothing at this stage? How do they weigh up against each other?**

*GNS recommends that all new buildings and developments are designed following a geotechnical investigation which assesses their site for liquefaction and the strength of the ground materials to be able to support the proposed structure. The structure then needs to have well designed and engineered foundations which are compatible with the ground materials found at the site.*

*This is not new – it is standard good engineering practice which should be followed everywhere.*

*As Palmerston North already has a high probability of strong earthquake shaking, the location of new and presently unknown faults (seismic sources) will not increase this probability much. Thus designing new buildings with a good standard of engineering input is already a requirement. Older buildings should be assessed for strengthening then retrofitted if required. The assessment and retrofitting should include assessment of the ground strength and the capacity of the building foundations to support the building if an earthquake occurs. Ground investigation and treatment if required is part of the old building assessment and retrofitting process. Experience suggests that older and larger buildings where many people are located should be the first to be assessed and retrofitted. Houses are shown to not be prone to collapse, even with extreme liquefaction and ground damage, but they have a very high repair and social cost if they are not well engineered for earthquakes. The location of a fault under the town would increase the surface fault rupture hazard at the location of the fault on the ground surface, but I note that the Darfield Fault rupture did not cause houses to collapse even when it passed through them, and this possibility can only be addressed when and if such a fault is located.*

- 3. Clarification is sought on the accuracy of the findings of the report, particularly in relation to the Kelvin Grove growth area. The preferred location for residential growth at Kelvin Grove is on the lower terrace, it is noted that there was/is minimal data (bore hole) for this area. Are site specific investigations at time of development sufficient for this location or should additional bore hole data be sourced prior to rezoning and development?**

*In my view it would be worth having a modest level of subsurface geotechnical information in your preferred development areas before rezoning as this would allow for better development of ideas and planning such as for schools, commercial areas, parks and housing to utilise/avoid natural features and for assessment of development costs. Later during development more extensive data could be obtained to meet the specific planning and design requirements of the subdivision.*

- 4. Another area of interest is the potential additional cost for developing the growth areas with regard to mitigating the impacts of liquefaction (engineered foundations). First, in terms of the \$20,000.00 that is referred to in the report, what floor area is this based upon? Second, how do the costs increase as the floor area increases and is this cost just for the foundations alone?**

*The \$20,000 referred to in the report is a rough estimate for the additional cost of a driven grid of tamped piles, the tops of which are cast into the locally thickened reinforced concrete foundation slab. We saw a house in Avonside Drive, Christchurch with this type of foundation. The house was apparently undamaged in an area of severe lateral spreading, and I expect the owners of this new house were delighted to be one of the few undamaged houses in the area. Their investment in their house foundations had paid off handsomely. Many, many people with expensive new homes had their*

*dreams shattered because of poor foundations to their houses. Most would have been happy to pay \$15 - \$20K extra for their house if it had prevented damage. The estimate is for an "average" larger, upmarket house.*

- 5. How strong are the recommendations regarding the requirements for above ground servicing for power and phone lines to residential dwellings? This is an important consideration in moving forward with the residential growth areas as having lines above ground has potential impacts on the urban design outcomes for new residential areas.**

*The possibility of above ground servicing for power and phone lines in a residential area is mentioned as a possibility for consideration by PNCC.*

*Power and phone poles were tilted and will need replacing in older, heavily liquefied areas of Christchurch.*

*A lesson from Christchurch is that deep services are very expensive to dig up and repair/replace. Buried services should therefore be as shallow as practical – even in areas where ground damage is not expected, as this will help reduce long-term maintenance and replacement costs too. Flexible and ductile underground pipes are less prone to fracture and damage in the event of ground movement.*

**Can you please provide comment on the above matters and confirm the final report by Friday 17 June 2011.**

**It would also be appreciated if you can invoice the Council for the report prior to the end of the Council financial year (30 June 2011)**

*GNS plans to deliver our amended final report next week and will invoice PNCC before the end of June.*

**Can you please liaise with Tiwene Roberts regarding the handover of the 3D model.**

*The 3D model developed by GNS for PNCC is currently being checked by GNS. It will be handed over to PNCC (Tiwene Roberts) in good working order. If required GNS would be pleased to assist PNCC with the maintenance of the database.*

*If you have any questions about what is contained above then please contact Dick Beetham.*

*Yours sincerely  
Dick Beetham*



*Engineering Geologist & CPEng (geotechnical)*

## **APPENDIX 9 MODIFIED MERCALLI INTENSITY SCALE SUMMARY DESCRIPTIONS**

### **MM1: Imperceptible**

Barely sensed only by a very few people.

### **MM2: Scarcely felt**

Felt only by a few people at rest in houses or on upper floors.

### **MM3: Weak**

Felt indoors as a light vibration. Hanging objects may swing slightly.

### **MM4: Largely observed**

Generally noticed indoors, but not outside, as a moderate vibration or jolt. Light sleepers may be awakened. Walls may creak, and glassware, crockery, doors or windows rattle.

### **MM5: Strong**

Generally felt outside and by almost everyone indoors. Most sleepers are awakened and a few people alarmed. Small objects are shifted or overturned, and pictures knock against the wall. Some glassware and crockery may break, and loosely secured doors may swing open and shut.

### **MM6: Slightly damaging**

Felt by all. People and animals are alarmed, and many run outside. Walking steadily is difficult. Furniture and appliances may move on smooth surfaces, and objects fall from walls and shelves. Glassware and crockery break. Slight non-structural damage to buildings may occur.

### **MM7: Damaging**

General alarm. People experience difficulty standing. Furniture and appliances are shifted. Substantial damage to fragile or unsecured objects. A few weak buildings are damaged.

### **MM8: Heavily damaging**

Alarm may approach panic. A few buildings are damaged and some weak buildings are destroyed.

### **MM9: Destructive**

Some buildings are damaged and many weak buildings are destroyed.

### **MM10: Very destructive**

Many buildings are damaged and most weak buildings are destroyed.

### **MM11: Devastating**

Most buildings are damaged and many buildings are destroyed.

### **MM12: Completely devastating**

All buildings are damaged and most buildings are destroyed.

### MODIFIED MERCALLI INTENSITY SCALE

The following version of the MM scale is as used in New Zealand (Dowrick, 1996).

Dowrick D J (1996) The modified Mercalli earthquake intensity scale-Revisions arising from recent studies of New Zealand earthquakes. *Bull NZ National Soc for Earthquake Eng* **29**: 92-106.

#### MODIFIED MERCALLI INTENSITY SCALE - NZ 1996

Items marked \* in the scale are defined in the note following.

#### MM1

##### *People*

Not felt except by a very few people under exceptionally favourable circumstances.

#### MM2

##### *People*

Felt by persons at rest, on upper floors or favourably placed.

#### MM3

##### *People*

Felt indoors; hanging objects may swing, vibration similar to passing of light trucks, duration may be estimated, may not be recognised as an earthquake.

#### MM4

##### *People*

Generally noticed indoors but not outside. Light sleepers may be awakened. Vibration may be likened to the passing of heavy traffic, or to the jolt of a heavy object falling or striking the building.

##### *Fittings*

Doors and windows rattle. Glassware and crockery rattle. Liquids in open vessels may be slightly disturbed. Standing motorcars may rock.

##### *Structures*

Walls and frame of buildings, and partitions and suspended ceilings in commercial buildings, may be heard to creak.

#### MM5

##### *People*

Generally felt outside, and by almost everyone indoors. Most sleepers awakened. A few people alarmed.

##### *Fittings*

Small unstable objects are displaced or upset. Some glassware and crockery may be broken. Hanging pictures knock against the wall. Open doors may swing. Cupboard doors secured by magnetic catches may open. Pendulum clocks stop, start, or change rate (H\*).

##### *Structures*

Some windows Type I\* cracked. A few earthenware toilet fixtures cracked (H).

#### MM6

##### *People*

Felt by all.

People and animals alarmed.

Many run outside.\*

Difficulty experienced in walking steadily.

##### *Fittings*

Objects fall from shelves.

Pictures fall from walls (H\*).

Some furniture moved on smooth floors, some unsecured free-standing fireplaces moved.

Glassware and crockery broken.

Very unstable furniture overturned.

Small church and school bells ring (H).

Appliances move on bench or table tops.

Filing cabinets or "easy glide" drawers may open (or shut).

*Structures*

Slight damage to Buildings Type I\*.

Some stucco or cement plaster falls.

Windows Type I\* broken.

Damage to a few weak domestic chimneys, some may fall.

*Environment*

Trees and bushes shake, or are heard to rustle.

Loose material may be dislodged from sloping ground, e.g. existing slides, talus slopes, shingle slides.

**MM7***People*

General alarm.

Difficulty experienced in standing.

Noticed by motorcar drivers who may stop.

*Fittings*

Large bells ring.

Furniture moves on smooth floors, may move on carpeted floors.

Substantial damage to fragile\* contents of buildings.

*Structures*

Unreinforced stone and brick walls cracked.

Buildings Type I cracked some with minor masonry falls.

A few instances of damage to Buildings Type II.

Unbraced parapets, unbraced brick gables, and architectural ornaments fall.

Roofing tiles, especially ridge tiles may be dislodged.

Many unreinforced domestic chimneys damaged, often falling from roof-line.

Water tanks Type I\* burst.

A few instances of damage to brick veneers and plaster or cement-based linings. Unrestrained water cylinders (Water Tanks Type II\*) may move and leak.

Some windows Type II\* cracked. Suspended ceilings damaged.

**MM8***People*

Alarm may approach panic.

Steering of motorcars greatly affected.

*Structures*

Building Type I, heavily damaged, some collapse\*.

Buildings Type II damaged, some with partial collapse\*.

Buildings Type III damaged in some cases.

A few instances of damage to Structures Type IV.

Monuments and pre-1976 elevated tanks and factory stacks twisted or brought down.

Some pre-1965 infill masonry panels damaged.

A few post-1980 brick veneers damaged.

Decayed timber piles of houses damaged.

Houses not secured to foundations may move.

Most unreinforced domestic chimneys damaged, some below roof-line, many brought down.

*Environment*

Cracks appear on steep slopes and in wet ground.

Small to moderate slides in roadside cuttings and unsupported excavations.

Small water and sand ejections and localised lateral spreading adjacent to streams, canals, lakes, etc.

**MM9***Structures*

Many Buildings Type I destroyed\*.

Buildings Type II heavily damaged, some collapse\*.

<p>Buildings Type III damaged, some with partial collapse*.</p> <p>Structures Type IV damaged in some cases, some with flexible frames seriously damaged.</p> <p>Damage or permanent distortion to some Structures Type V.</p> <p>Houses not secured to foundations shifted off.</p> <p>Brick veneers fall and expose frames.</p> <p><i>Environment</i></p> <p>Cracking of ground conspicuous.</p> <p>Landsliding general on steep slopes.</p> <p>Liquefaction effects intensified and more widespread, with large lateral spreading and flow sliding adjacent to streams, canals, lakes, etc.</p> <p><b>MM10</b></p> <p><i>Structures</i></p> <p>Most Buildings Type I destroyed*.</p> <p>Many Buildings Type II destroyed*.</p> <p>Buildings Type III ∇ heavily damaged, some collapse*.</p> <p>Structures Type IV ∇ damaged, some with partial collapse*.</p> <p>Structures Type V ∇ moderately damaged, but few partial collapses.</p> <p>A few instances of damage to Structures Type VI.</p> <p>Some well-built* timber buildings moderately damaged (excluding damage from falling chimneys).</p> <p><b>MM11</b></p> <p><i>Environment</i></p> <p>Landsliding very widespread in susceptible terrain, with very large rock masses displaced on steep slopes.</p> <p>Landslide dams may be formed.</p> <p>Liquefaction effects widespread and severe.</p> <p><i>Structures</i></p> <p>Most Buildings Type II ∇ destroyed*.</p> <p>Many Buildings Type III ∇ destroyed*.</p> <p>Structures Type IV ∇ heavily damaged, some collapse*.</p> <p>Structures Type V ∇ damaged, some with partial collapse.</p> <p>Structures Type VI suffer minor damage, a few moderately damaged.</p> <p><b>MM12</b></p> <p><i>Structures</i></p> <p>Most Buildings Type III ∇ destroyed*.</p> <p>Structures Type IV ∇ heavily damaged, some collapse*.</p> <p>Structures Type V ∇ damaged, some with partial collapse.</p> <p>Structures Type VI suffer minor damage, a few moderately damaged.</p>
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**Note to 1996 NZ MM Scale**

Items marked \* in the scale are defined below.

**Construction Types:**

*Buildings Type I (Masonry D in the NZ 1965 mm scale)*

Buildings with low standard of workmanship, poor mortar, or constructed of weak materials like mud brick or rammed earth soft storey structures (e.g. shops) made of masonry weak reinforced concrete or composite materials (e.g. some walls timber, some brick) not well tied together. Masonry buildings otherwise conforming to buildings Types I - III, but also having heavy unreinforced masonry towers. (buildings constructed entirely of timber must be of extremely low quality to be Type I).

*Buildings Type II (Masonry C in the NZ 1966 MM scale)*

Buildings of ordinary workmanship, with mortar of average quality. No extreme weakness, such as inadequate bonding of the corners, but neither designed nor reinforced to resist lateral forces. Such buildings not having heavy unreinforced masonry towers.

*Buildings Type III (Masonry B in the NZ 1966 MM scale)*

Reinforced masonry or concrete buildings of good workmanship and with sound mortar, but not formally designed to resist earthquake forces.

*Structures Type IV (Masonry A in the NZ 1966 MM scale)*

Buildings and bridges designed and built to resist earthquakes to normal use standards, i.e. no special collapse or damage limiting measures taken (mid-1930's to c. 1970 for concrete and to c 1980 for other materials).

*Structures Type V*

Buildings and bridges, designed and built to normal use standards, i.e. no special damage limiting measures taken, other than code requirements, dating from since c. 1970 for concrete and c. 1980 for other materials.

*Structures Type VI*

Structures, dating from c. 1980, with well-defined foundation behaviour, which have been specially designed for minimal damage, e.g. seismically isolated emergency facilities, some structures with dangerous or high contents, or new generation low damage structures.

**Windows**

Type I - Large display windows, especially shop windows.

Type II - Ordinary sash or casement windows.

**Water Tanks**

Type I - External, stand mounted, corrugated iron tanks.

Type II - Domestic hot-water cylinders unrestrained except by supply and delivery pipes.

H - (Historical) More likely to be used for historical events.

**Other Comments**

“Some” or “a few” indicates that the threshold of a particular effect has just been reached at that intensity.

“Many run outside” (MM6) variable depending on mass behaviour, or conditioning by occurrence or absence of previous quakes, i.e. may occur at MM5 or not till MM7.

“Fragile Contents of Buildings”. Fragile contents include weak, brittle, unstable, unrestrained objects in any kind of building. “Well-built timber buildings” have: wall openings not too large; robust piles or reinforced concrete strip foundations;

Superstructure tied to foundation

∇ Buildings Type III - V at MM10 and greater intensities are more likely to exhibit the damage levels indicated for low-rise buildings on firm or stiff ground and for high-rise buildings on soft ground. By inference lesser damage to low-rise buildings on soft ground and high-rise buildings on firm or stiff ground may indicate the same intensity. These effects are due to attention of short period vibrations and amplification of longer period vibrations in soft soils.