Research Report 2011-04
Civil & Natural Resources Engineering

UNIVERSITY OF CANTERBURY
Te Whare Wānanga o Waitaha
CHRISTCHURCH NEW ZEALAND

Liquefaction Impacts on Pipe Networks

Misko Cubrinovski
Matthew Hughes
Brendon Bradley
Ian McCahon
Yvonne McDonald
Howard Simpson
Rod Cameron
Mark Christison
Bruce Henderson
Rolando Orense
Thomas O'Rourke

ISSN 1172-9511

UC ENGINEERING

December 2011

LIQUEFACTION IMPACTS ON PIPE NETWORKS

Short Term Recovery Project No. 6

Natural Hazards Research Platform

Misko Cubrinovski[1], Matthew Hughes[1], Brendon Bradley[1]

Ian McCahon[2],

Yvonne McDonald[3], Howard Simpson[4], Rod Cameron[4],

Mark Christison[4], Bruce Henderson[4],

Rolando Orense[5] and Thomas O'Rourke[6]

[1] University of Canterbury, Christchurch
[2] Geotech Consulting, Christchurch
[3] Practical Consulting, Christchurch
[4] Christchurch City Council, Christchurch
[5] University of Auckland, Auckland
[6] Cornell University, Ithaca, USA

December 2011

University of Canterbury

Table of Contents

1 INTRODUCTION

In the aftermath of the 22 February 2011 earthquake, the Natural Hazards Research Platform (NHRP) initiated a series of Short Term Recovery Projects (STRP) aimed at facilitating and supporting the recovery of Christchurch from the earthquake impacts. This report presents the outcomes of STRP 6: Impacts of Liquefaction on Pipe Networks, which focused on the impacts of liquefaction on the potable water and wastewater systems of Christchurch. The project was a collaborative effort of NHRP researchers with expertise in liquefaction, CCC personnel managing and designing the systems and a geotechnical practitioner with experience/expertise in Christchurch soils and seismic geotechnics. The project team members were:

Misko Cubrinovski, Professor, University of Canterbury (Project Leader)
Ian McCahon, Director, Geotech Consulting (Geotechnical Engineer)
Matthew Hughes, Research Associate, University of Canterbury (GIS Specialist)
Brendon Bradley, Lecturer, University of Canterbury
Yvonne McDonald, Civil Engineering Consultant, Practical Consulting
Howard Simpson, Team Leader, Capital Delivery, Christchurch City Council
Rod Cameron, Infrastructure Rebuild Leader, Christchurch City Council
Mark Christison, Unit Manager, City Water and Waste, Christchurch City Council
Bruce Henderson, Team Leader Asset Management, Christchurch City Council
John Walter, Senior Environmental Engineer
Aylwin Sim, Asset Management Analyst, Christchurch City Council
Gordon Taylor, Contracts Engineer, Christchurch City Council
Rolando Orense, Senior Lecturer, University of Auckland

The project was organized around three objectives:

1) Providing immediate advice, solutions and recommendations to the CCC with regard to geotechnical, liquefaction and seismic issues relevant for the systems
2) Documenting and evaluating the performance of the potable water and wastewater systems of Christchurch, and
3) Developing concepts and implementing procedures towards improved seismic resilience of the systems

Within this framework, a number of issues were addressed, as summarized in this report. Regular meetings of the project team were used for communication, exchange of ideas and progress reporting (minutes of 23 meetings available from Yvonne McDonald). The key findings are presented in eleven sections of this report including first a description of the intensity of seismic loads (Section 2) and liquefaction manifestation (Section 3) and their distribution through Christchurch. Sections 4 and 5 summarize the performance of the potable water and wastewater systems of Christchurch, while Section 6 summarizes the specific design and operational issues addressed including modifications of standards and practices resulting from this project work and associated activities. The discussion initiated on the performance objectives for the systems is summarized in Section 7. A Liquefaction Zoning Map was developed for Christchurch that classifies areas of the city in five different zones with respect to their liquefaction resistance (Section 8). The strength of each zone is defined relative to the reference Zone 1. For example, the average liquefaction resistance of Zone 3 is three times the lower bound resistance of Zone 1. Sections 9 and 10 present a summary of selected literature review and a simple tool provided to

the CCC for calculation of liquefaction-induced uplift of manholes. Finally Summary and Recommendations are presented in Section 11. A large number of appendices provide more details on each section, i.e. on lateral spreading (A), GIS analysis of the potable water system (B), liquefaction resistance index calculations (C), development of liquefaction zoning map of Christchurch (D), performance objective discussion document (E), lessons learned from the 1995 Kobe earthquake (F) and manhole uplift calculation tool (G).

A concise version of this project is also provided in the form of an executive summary, including key recommendations and conclusions.

2 SEISMIC DEMAND IMPOSED BY THE 2010-2011 EARTHQUAKES

The 4 September 2010 Darfield earthquake was caused by a rupture of a system of faults located to the west of Christchurch in the Canterbury Plains. The principal fault rupture (Greendale Fault) reached Rolleston or approximately 12 km from the west edge of the city and 18 km from its CBD. The 7.1 moment magnitude earthquake (M_w = 7.1) produced moderate to strong ground shaking within Christchurch with ground motions approaching the 475-design level in some period ranges.

The 22 February 2011 Christchurch earthquake was caused by a local fault just beneath the Port Hills to the south of Christchurch. The fault was practically within the city boundaries and approximately 5km to the south-east of the CBD. The M_w = 6.2 earthquake produced strong to very strong ground shaking within Christchurch with ground motions well above the 475-design level in the south, south-east and east suburbs of Christchurch as well as within the CBD. A number of factors such as the proximity of the fault to the city, rupture and wave propagation characteristics, and basin and site effects contributed to the very high ground motions.

The observations and effects of these earthquakes must be kept in the context of these very high and damaging ground motions produced by these events, particularly the 22 February 2011 earthquake.

As seismic waves propagate through the softer alluvial soils, from the basement rock towards the ground surface, alluvial soils significantly modify the characteristics of ground shaking. They amplify the shaking and seismic forces for some structures, while for others they reduce, or de-amplify, the shaking. The composition of alluvial soils, their stratification, thickness and stiffness (resistance to deformation) define the particular features of the subsequent modification of the ground motion. In addition, as seismic waves pass through the soils, they deform the soils producing both transient deformations (temporary displacements) and permanent movements and deformations (residual horizontal and vertical displacements, ground distortion, undulation of ground surface, ground cracks and fissures). In cases when the ground deformation is excessive and seriously affecting the performance of land or structures, the soils are considered to have 'failed'. Soil liquefaction is one form of such failure since it usually results in excessive ground deformation and displacement that severely affect the built environment. Lateral spreading is a particular phenomenon associated with liquefaction resulting in very large and damaging permanent ground displacements.

The seismic loads acting on structures can be generally classified into two groups: inertial loads (caused by inertial forces due to shaking or seismic accelerations) and kinematic loads (due to ground movement). The former are critical for buildings and structures above the ground level, while the later are very important for structures buried in the ground, such as foundations or subsurface pipelines. In the case of liquefaction and lateral spreading, the earthquake-induced ground movements are very large, and hence, the consequent kinematic loads on buried structures can be significant and often beyond the available capacity to sustain such loads.

To examine the seismic demand (inertial and kinematic loads) imposed by the two earthquakes, the distribution of the recorded peak ground accelerations (PGA), peak

ground velocities (PGV) and cyclic stress ratios (CSR) throughout Christchurch was first estimated (through direct GIS interpolation). Note that PGV and CSR are calculated using recorded acceleration time histories and PGAs respectively.

2.1 Peak Ground Accelerations

Records from 18 strong motion stations (SMS) were used in the presented analyses, 13 of which are within the city boundaries. The geometric mean PGAs (i.e. square-root of the product of the PGA recorded in two orthogonal horizontal directions) recorded during the two earthquakes are comparatively shown in Figure 1 indicating that generally much higher accelerations were recorded throughout Christchurch during the 22 February earthquake. At the stations to the west of Christchurch (Lincoln, Templeton, Rolleston) and in Kaiapoi, higher accelerations were recorded during the Darfield earthquake. The records at Papanui and Styx Mill show similar PGAs from both earthquakes.

The distribution of the geometric mean PGAs throughout the city (obtained by ordinary krigging interpolation) is shown in Figure 2 (O'Rourke and Milashuk, 2011) for the 2010 Darfield, 2011 Christchurch and 13 June 2011 earthquakes, respectively. These plots clearly show the dominant effects of fault location on the recorded PGAs, but also the important influences of other factors as reflected in the scatter and variation of accelerations recorded at stations located within small distances from each other. It is known that high frequency content of ground shaking is highly variable even on small length scales due to wave scattering.

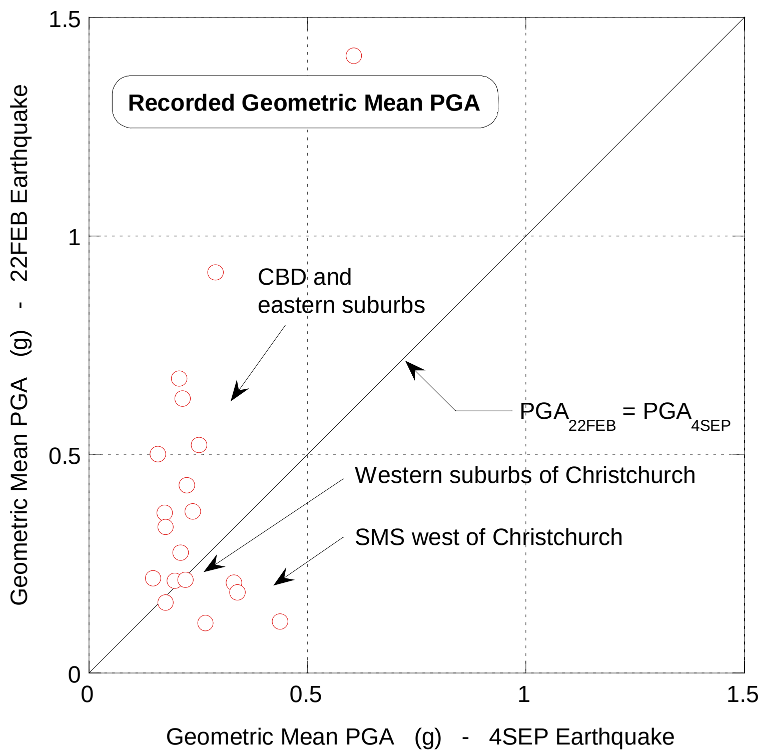
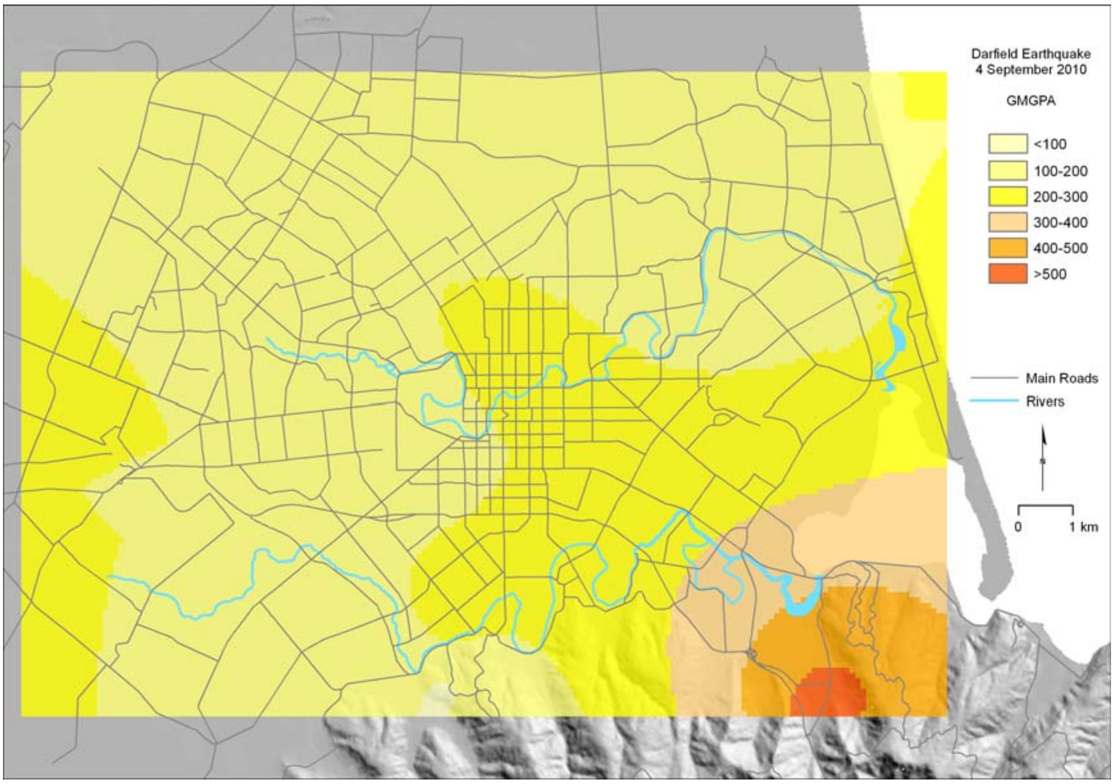
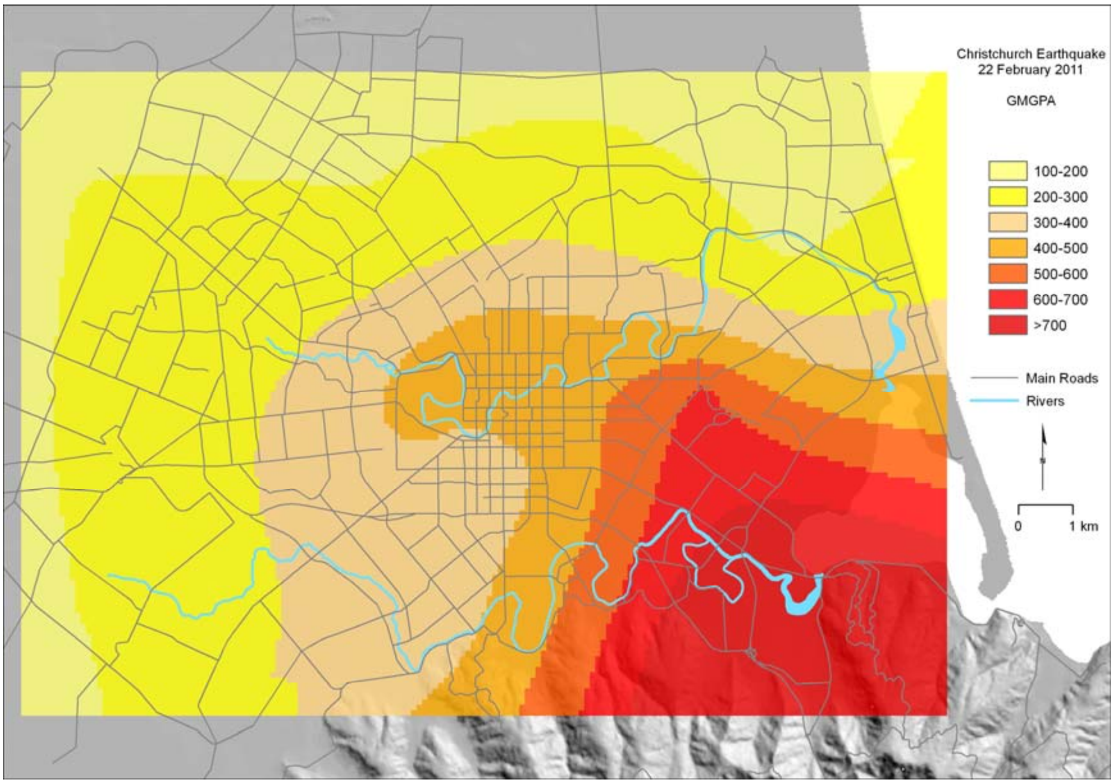


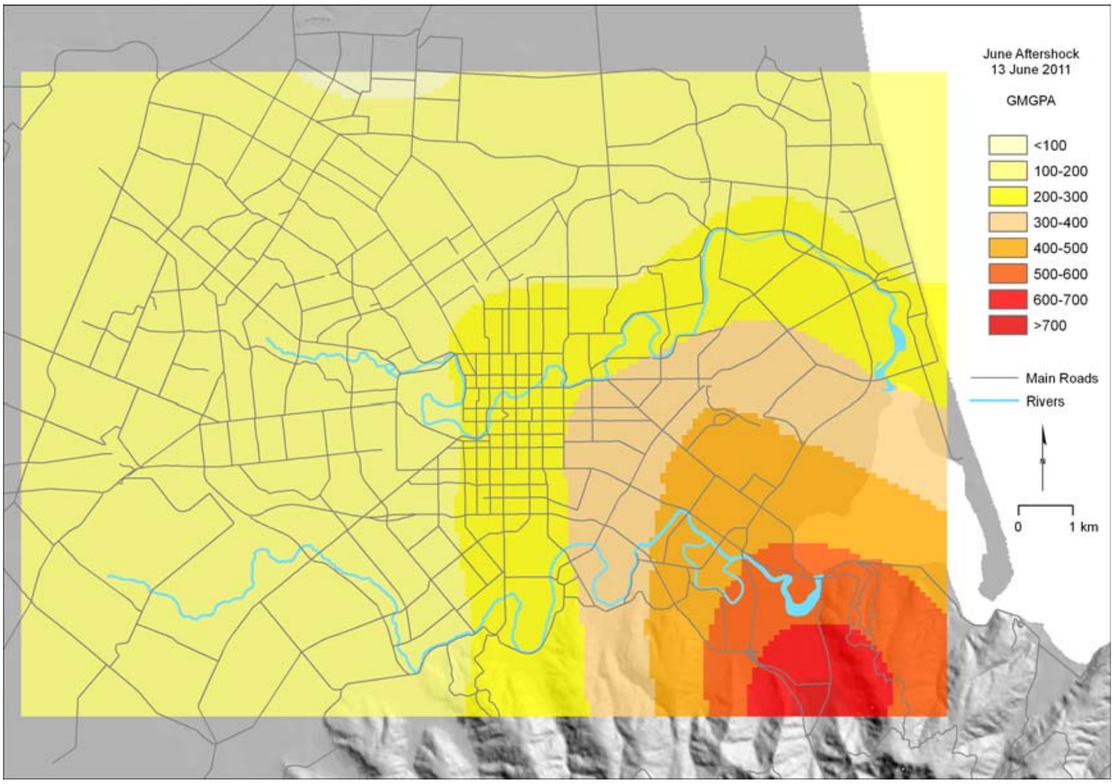
Figure 1. Comparison of geometric mean PGAs recorded during the 2010 Darfield earthquake (horizontal axis) and 2011 Christchurch earthquake (vertical axis); if two horizontal acceleration records were obtained in the NS and EW directions respectively, then the geometric mean PGA was calculated as $Geom.Mean\,PGA = \sqrt{PGA_{NS} \cdot PGA_{EW}}$



(a) 4 September 2010 (Darfield) earthquake



(b) 22 February 2011 (Christchurch) earthquake

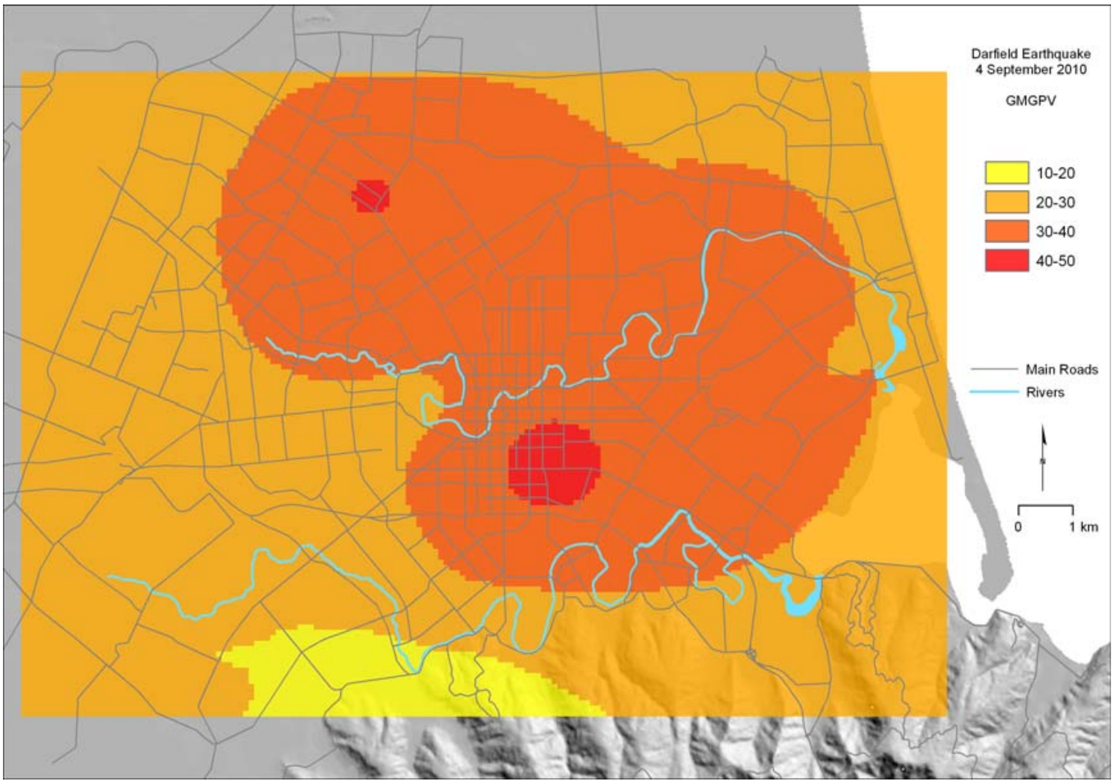


(c) 13 June 2011 earthquake

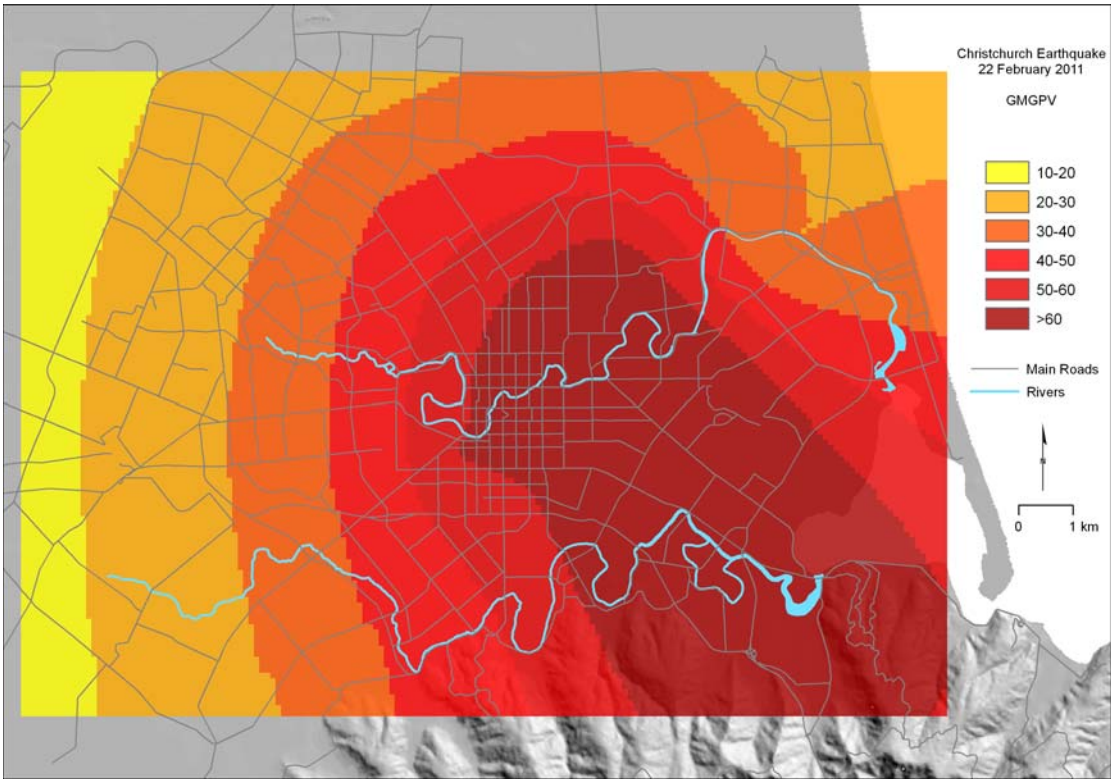
Figure 2. Distribution of geometric mean PGAs obtained by ordinary krigging interpolation of recorded accelerations (O'Rourke and Milashuk, 2011)

2.2 Peak Ground Velocities

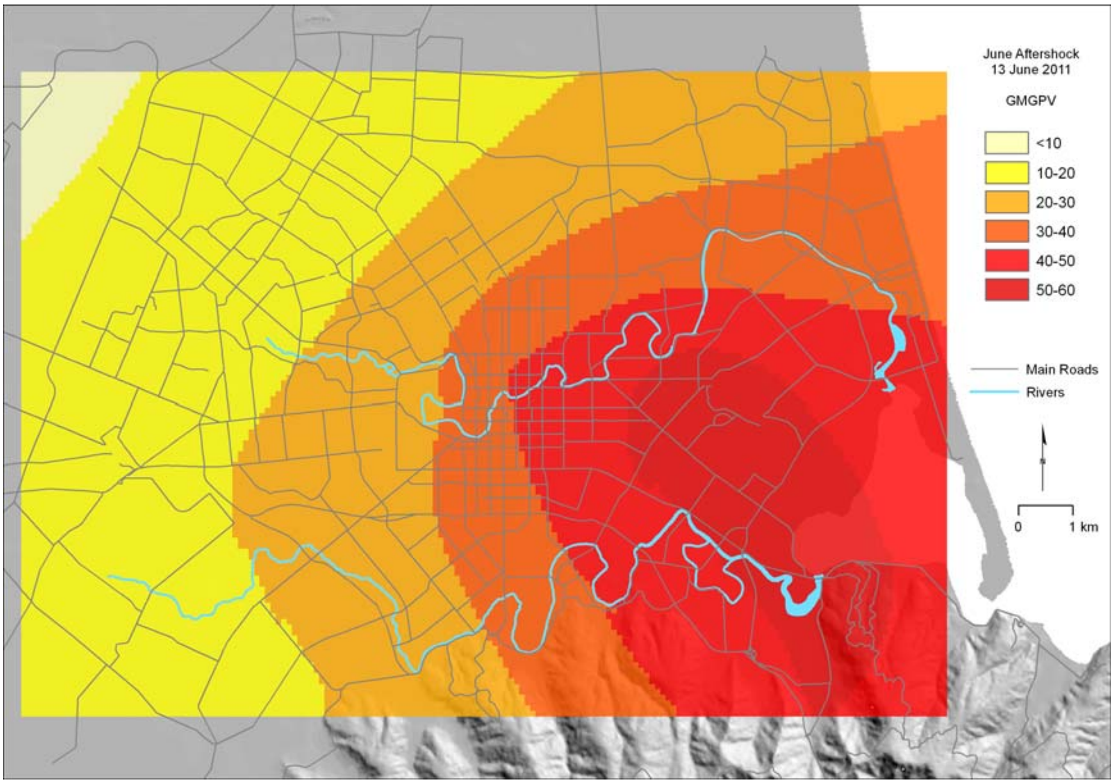
The peak ground velocity (PGV) is a better measure for the ground motion intensity and its effects on buried structures which are subjected to substantial ground movement and deformation than PGA (Bradley *et al.* 2009). This is why design codes often employ PGV rather than PGA when correlating ground motion intensity and damage to pipe networks (e.g. ALA 2004). PGV is also much less sensitive to the aforementioned high frequency content variability in the ground motion, and hence it has a smaller spatial variability as compared to the PGA. Figure 3 summarizes the PGV calculated from recorded acceleration time histories and interpolated by krigging for the three major events (O'Rourke and Milashuk, 2011).



(a) 4 September 2010 (Darfield) earthquake



(b) 22 February 2011 (Christchurch) earthquake



(c) 13 June 2011 earthquake

Figure 3. Distribution of geometric mean PGVs obtained by ordinary krigging interpolation of recorded accelerations (O'Rourke and Milashuk, 2011)

2.3 Cyclic Stress Ratios

In the simplified procedure for liquefaction evaluation (Seed and Idriss, 1971;Youd *et al.* 2001), the seismic demand (intensity of ground shaking) specific to liquefaction evaluation is defined by combining two key parameters of the ground motion, i.e. its amplitude and duration. The peak ground acceleration (PGA) is used as a measure for the amplitude of ground shaking while the earthquake moment magnitude (M_w) is used as a proxy for the duration of shaking (i.e. the number of significant stress cycles). By using this approach, it is possible to calculate the equivalent cyclic stress ratio, $CSR_{7.5}$, at any given site and depth inf the PGA at the site and the magnitude of the causative earthquake were known. $CSR_{7.5}$ in essence represents an equivalent amplitude of the shear stresses induced in the soil by the earthquake if they were to be expressed with 15 uniform stress cycles. The general form of the expression for $CSR_{7.5}$ is shown below

$$Normalized\ Intensity\ of\ ground\ \ shaking = CSR_{7.5} = \frac{CSR}{MSF} = \frac{f(PGA)}{f(M_w)} \tag{1}$$

This approach allows for comparison of the intensity (severity) of ground shaking imposed by different earthquakes at a given site, or over a given area (for an adopted reference depth in the deposit). Using this approach, the $CSR_{7.5}$ values induced by the 4 September 2010 and 22 February 2011 earthquakes were calculated for each of the 18 strong motion station (SMS) sites considered at the depth of the water table, as

shown comparatively in Figure 4. This plot shows that in the eastern and south-eastern suburbs, and the CBD, the ground shaking intensity specific to liquefaction triggering was higher or much higher during the 22 February earthquake. In the western and north-western parts of Christchurch (e.g. Riccarton, Papanui and Styx Mill stations), the Darfield earthquake produced slightly more severe ground shaking (i.e. combined effects of amplitude and duration of shaking). At the HPSC station, PGAs and hence CSRs appear to be somewhat 'anomalous', probably due to dominant effects of very severe liquefaction affecting the strong motion instrument at the site. When evaluating the observed manifestation of liquefaction or non-occurrence of liquefaction throughout different parts of Christchurch, it is important to have in mind these cyclic stress ratios because they demonstrate that different parts of the city were subjected to substantially different severity of ground shaking. For example, the severity of shaking at North New Brighton and Pages Road Pump stations was nearly three or four times the level of shaking experienced at the Papanui High School and Styx Mill Stations respectively. In general, the 4 September earthquake produced the maximum cyclic stress ratios in areas to the west and north-west of the CBD, whereas the 22 February earthquake produced the maximum CSRs to the south, south-east, east and north-east of the CBD, and the CBD itself. As illustrated in Figure 5, there is a reasonably wide interface zone where both earthquakes produced CSRs of similar magnitude.

Detailed analysis of the CSRs was further conducted to develop a liquefaction resistance map of Christchurch as described in the Liquefaction Resistance Index (LRI) section and Appendices C and D.

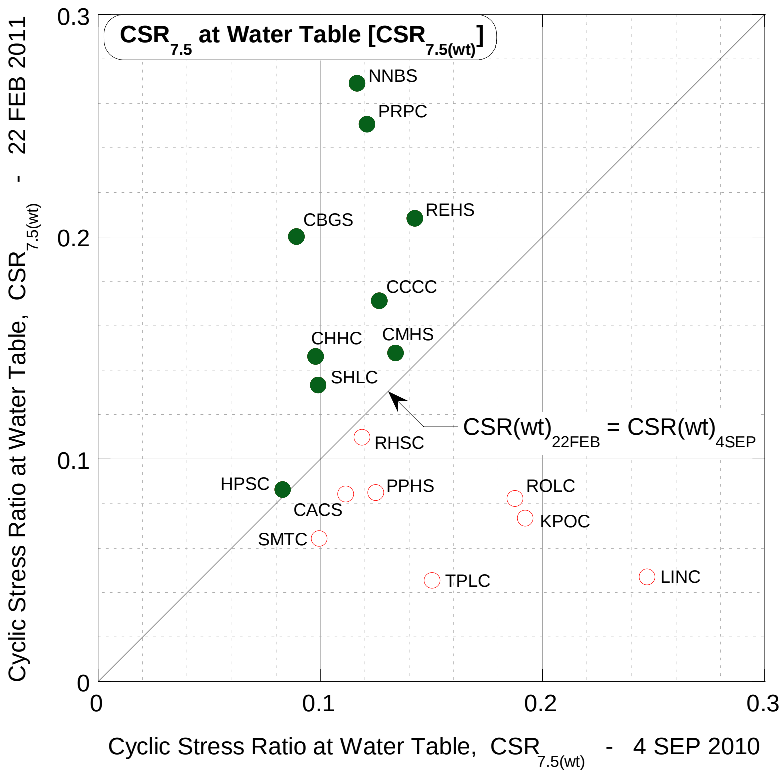


Figure 4. Comparison of equivalent cyclic stress ratios at water table depth, $CSR_{7.5(wt)}$, induced by the 2010 Darfield earthquake (horizontal axis) and 2011 Christchurch earthquake (vertical axis) computed using geometric mean horizontal PGAs recorded at SMS (SMS acronyms shown next tot each symbol)

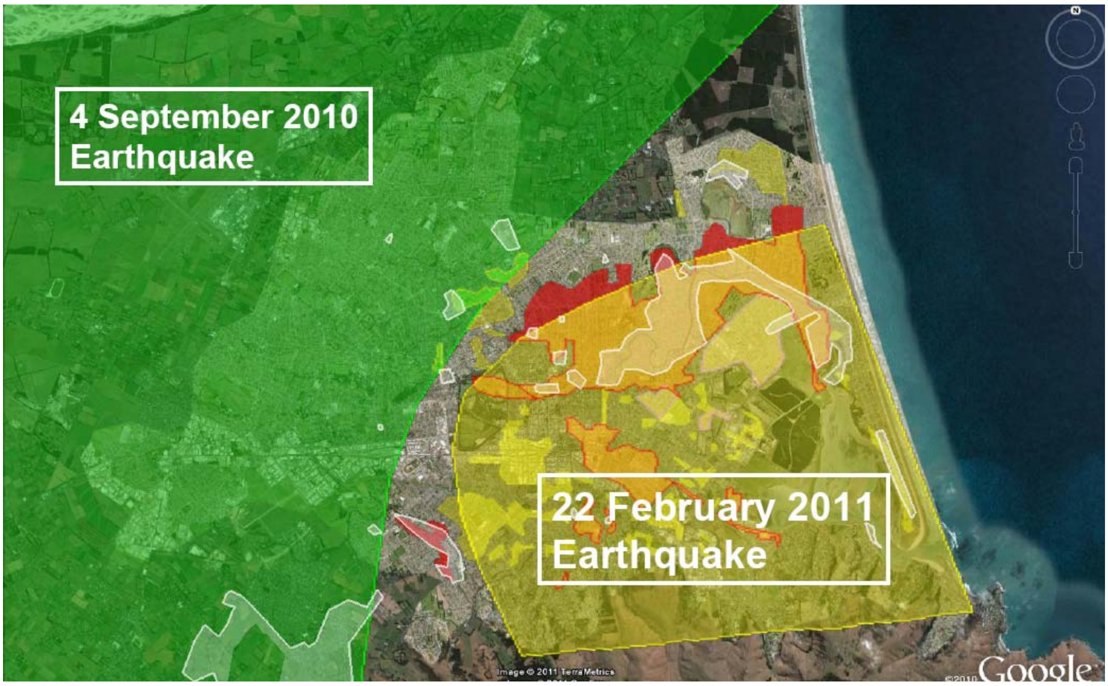


Figure 5. Dominant earthquake event producing the maximum cyclic stress ratios or shaking intensity pertinent to liquefaction triggering; in the green area the highest CSRs were produced by the Darfield earthquake (4 SEP 2010) while the Christchurch earthquake (22 FEB 2011) produced the highest CSRs in the area shown in yellow

2.4 Summary remarks

Table 1 summarizes representative geometric mean PGA and PGV values, and $CSR_{7.5(wt)}$ values at water table depth for the CBD, eastern and western suburbs of Christchurch. It indicates the following:

- The ground motions are characterized with high and damaging peak amplitudes, with horizontal PGA values of 0.18-0.674 g and horizontal PGV values of 27.6-72.8 cm/s.
- When combining the peak amplitude with the number of significant cycles, moderate to high cyclic stress ratios ($CSR_{7.5(wt)}$) were obtained for depths two metres below the water table. These stress ratios increase with depth as indicated in the footnote of Table 1.
- All intensity measures are consistent and indicate that the intensity of the ground motion in the CBD and eastern suburbs was on average 1.5 to 2.0 times that observed in the western suburbs during the 2011 Christchurch earthquake. Thus, any comparison of the performance of the water or wastewater networks must account for this difference in the seismic demand (different inertial and kinematic loads imposed by the earthquake in different parts of the city).
- The *PGA* is proxy for the inertial loads imposed on rigid structures above the ground, *PGV* is proxy for the kinematic loads on buried structures due to ground movement while the *CSR* is proxy for the intensity of the motion with respect to liquefaction triggering.

Table 1. Summary of representative geometric mean PGA and PGV values, and cyclic stress ratios at water table depth ($CSR_{7.5(wt)}$) for the CBD, eastern and western suburbs of Christchurch

Ground motion intensity measure	Western suburbs	CBD	Eastern suburbs
gmPGA (g)	0.176-0.275	0.366 - 0.522	0.216 - 0.674
gmPGV (cm/s)	27.6 - 36.7	46.3 - 65.4	35.1 – 72.8
$CSR_{7.5(wt)}$	0.100-0.125	0.146 - 0.209	0.251 - 0.269

*) Note that, for example, CSR values at depths of 2m below the water table can be simply calculated by multiplying $CSR_{7.5(wt)}$ with 1.51, 1.33, 1.22, 1.16 or 1.11 for water table depth of 1.0, 2.0, 3.0, 4.0 or 5.0m.

3 SOIL LIQUEFACTION IN THE 2010-2011 EARTHQUAKES

Following the 22 February earthquake, an intensive drive-through reconnaissance was conducted through parts of Christchurch to document the severity and extent of liquefaction throughout the city. The drive-through survey aimed at capturing surface evidence of liquefaction as quickly as possible and quantifying its severity in a consistent and systematic manner. The resulting liquefaction map (Cubrinovski and Taylor, 2011) is shown in Figure 6 where four areas of different liquefaction severity are indicated: (a) moderate to severe liquefaction (red zone, with very large areas covered by sand ejecta, mud and water, large distortion of ground and pavement surfaces, large fissures in the ground, and significant liquefaction-induced impacts on buildings and infrastructure), (b) low to moderate liquefaction (yellow zone, with generally similar features as for the severe liquefaction, but of lesser intensity and extent), (c) liquefaction predominantly on roads with some on properties (magenta zone, where heavy effects of liquefaction were seen predominantly on roads, with large sinkholes and 'vents' for pore pressure dissipation, and limited damage to properties/houses), and (d) traces of liquefaction (red circular symbols, with clear signs of liquefaction, but limited in extent and deemed not damaging for structures). The solid blue lines indicate roads where no signs of liquefaction were observed. The suburbs to the east of CBD along Avon River (Avonside, Dallington, Avondale, Burwood and Bexley) were most severely affected by liquefaction, which coincides with the area where about 5000 residential properties will be abandoned (New Zealand Government, 2011).

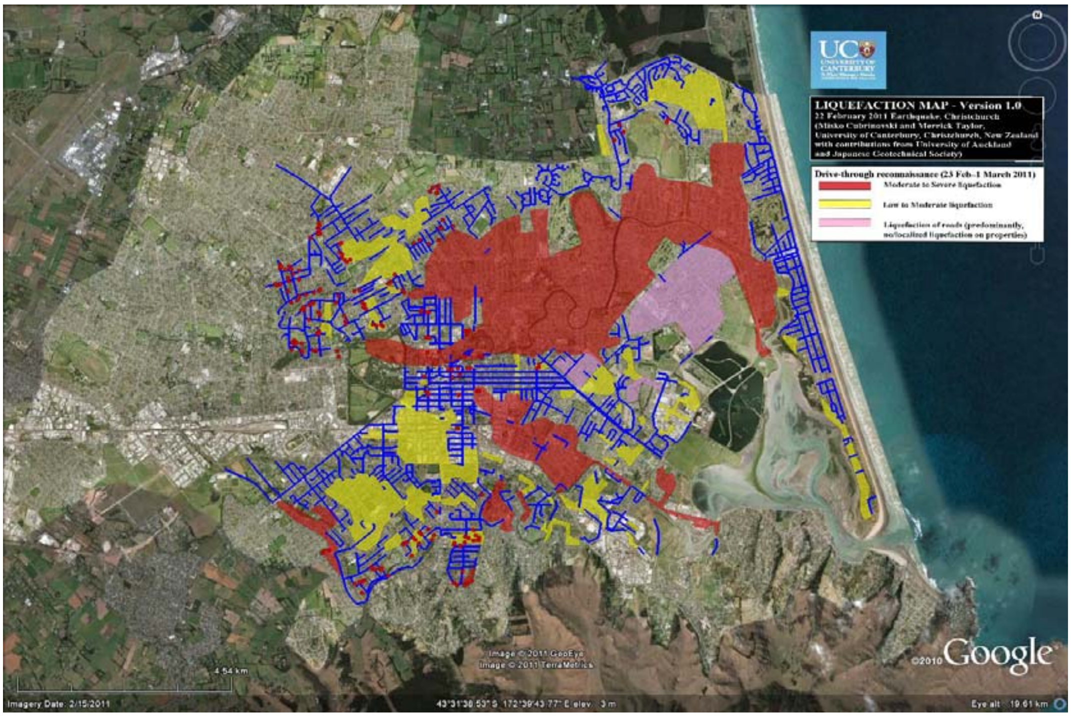


Figure 6. Liquefaction map of Christchurch from drive-through reconnaissance (Cubrinovski and Taylor, 2011); the map is incomplete (i.e. does not cover all of Christchurch) and shows only general overlay of areas (i.e. it cannot be used on property basis)

Ten days after the earthquake, after the urban search and rescue efforts had largely finished, a comprehensive ground survey was initiated within the CBD to document liquefaction effects in this area. Figure 7 shows the resulting liquefaction documentation map for the CBD together with the prevalent soil types in the top 7-8 m of the deposits (Cubrinovski and McCahon, 2011). The principal zone of liquefaction (shown in red) stretches west to east through the CBD, from Hagley Park to the west, along the Avon River to the northeast boundary of the CBD at the Fitzgerald Bridge. Many high-rise buildings on shallow foundations and deep foundations were affected by the liquefaction in different ways. Note that this zone consists mostly of sandy soils and largely coincides with the path of the Avon River and the network of old streams depicted in the 1850s survey maps (Archives New Zealand (2011)). Another zone of moderate to severe liquefaction was found in the south-east part of the CBD, though its effects were less significant in relative terms.

It is important to emphasize that in the maps shown in Figures 6 and 7 the manifestation (severity) of liquefaction even within one zone was not uniform, but rather varied substantially. In the red zone within the CBD, for example, the manifestation of liquefaction was primarily of moderate intensity with relatively extensive areas and volumes of sand/silt ejecta. There were also areas of low manifestation or only traces of liquefaction, but also pockets of severe liquefaction with very pronounced ground distortion, fissures, large settlements and substantial lateral ground movements. The zones of more pronounced liquefaction and ground distortion (black solid line and area) do appear somewhat to "line up" with the old stream channels, which sheds some light on the reasons for variability in liquefaction manifestation. One should not expect though that all liquefaction features and zones of pronounced ground weakness could be explained with reference to the stream channels dating back to 1850s, because the earlier history including deposition and re-working of surficial soils is also highly relevant for their liquefaction susceptibility.

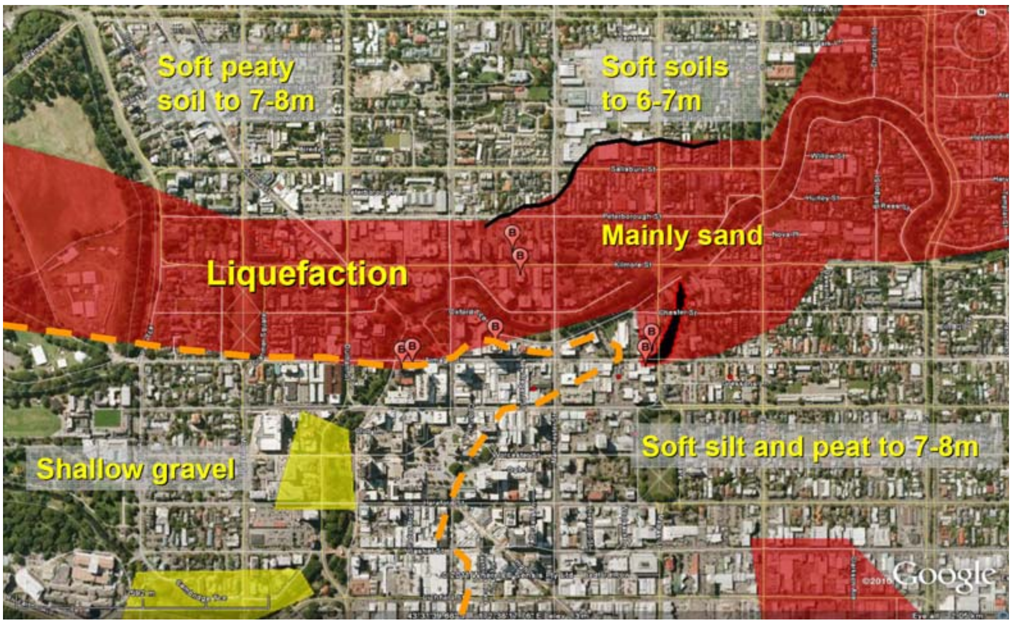


Figure 7. Liquefaction map indicating zones (in general terms, not on property basis) within the CBD affected by liquefaction in the 22 February earthquake (Cubrinovski and Taylor, 2011); predominant soils in the top part of the deposits are also indicated (Cubrinovski and McCahon, 2011)

In this sense, the liquefaction maps are generalized (both spatially and in terms of severity) and certainly are not applicable on a property basis. They do provide however a good basis for city-wide zoning based on actual observations of the ground performance during strong earthquakes and ground shaking. The map shown in Figure 6 is also not complete, because only the coloured areas were covered in the ground surveying, which was biased towards the areas affected by liquefaction.

Soil liquefaction repeatedly occurred in the same areas (i.e. at the same sites) during the multiple earthquakes producing strong ground shaking in Christchurch, and in particular during the 4 September 2010, 22 February 2011, and 13 June 2011 earthquakes. Figure 8 comparatively shows liquefied areas of Christchurch in these three events, as documented by field inspections.

The 22 February earthquake produced the most severe and widespread liquefaction within Christchurch which is consistent with the produced severity of ground shaking as described in terms of $CSR_{7.5}$ in the previous section. Again, consistent with the simplified procedure for liquefaction evaluation, in areas where the 4 September earthquake produced the highest CSRs (e.g. Kaiapoi and Halswell), the liquefaction manifestation was the most severe during the September 2010 earthquake.

The repeated occurrence of liquefaction at a given site during an earthquake is not surprising because liquefaction generally does not increase the liquefaction resistance of soils and hence does not prevent occurrence of soil liquefaction (re-liquefaction) at a site in subsequent earthquakes. The sequence of events in Christchurch has certainly

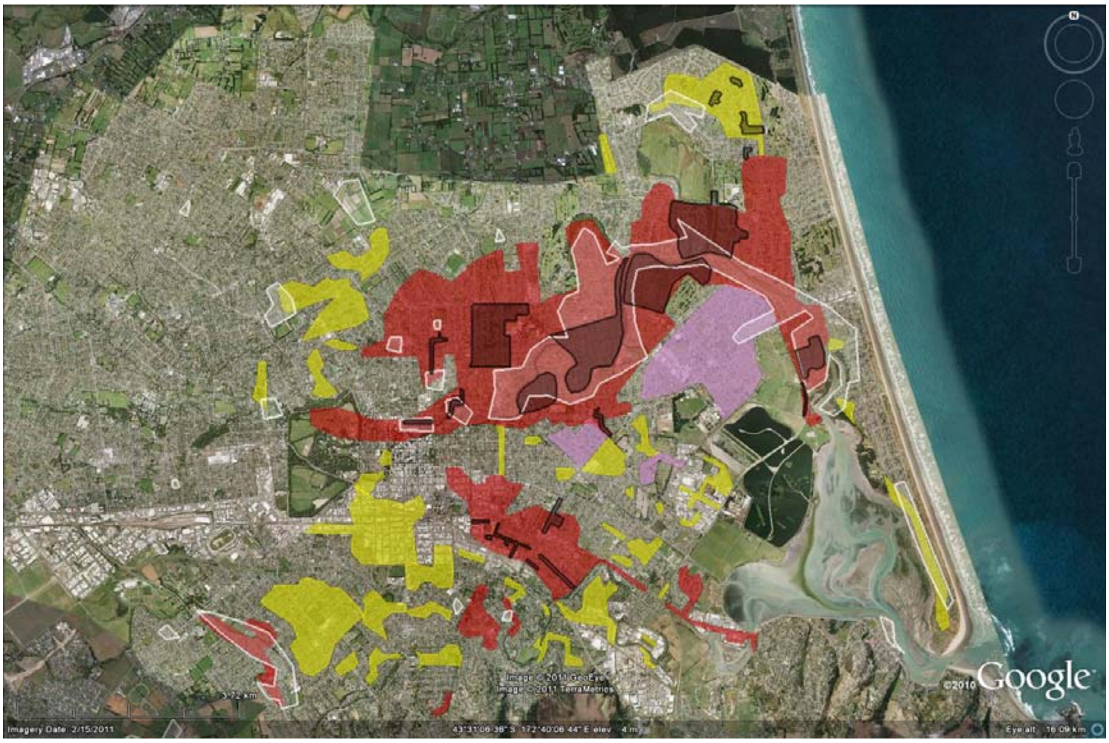


Figure 8. **Liquefaction maps documenting areas of observed liquefaction in the 4 September 2010 (white contours), 22 February 2011 (red, yellow, magenta areas; Cubrinovski and Taylor, 2011), and 13 June 2011 (black contours; Cubrinovski and Hughes, 2011) earthquakes; note that only parts of Christchurch were surveyed, and that the aim of the surveys was to capture general features and areas affected by liquefaction as observed from the roads, hence, the zoning is not applicable to specific properties**

proven this notion. The repeated liquefaction was often quite severe and some residents reported that the liquefaction severity increased in subsequent events. Again this type of behaviour is not surprising if one carefully considers the intensity and *volatility* of soil liquefaction process, and the characteristics of the re-solidified soil deposits post-liquefaction.

> *Soil liquefaction is a process in which over a very short period of time (several seconds or tens of seconds) during strong ground shaking, the soil transforms from its normal solid state into a heavy liquid mass. As a consequence of liquefaction, the soil essentially loses its strength and bearing capacity (i.e. the capacity to support gravity loads of heavy structures), thus causing sinking of heavy structures into the ground. Conversely, light and buoyant structures (that have smaller mass density than the liquefied soil mass) will be uplifted and float above the surface. Ground deformation associated with liquefaction takes various forms and is often excessive, non-uniform and involves large permanent vertical displacements (settlement) and lateral displacements commonly resulting in large cracks and fissures in the ground, substantial ground distortion and sand/silt/water ejecta covering the ground surface. The large pressures created in the groundwater during liquefaction are in excess of the equilibrium pressures, thus triggering flow of water towards the ground surface. Since these water pressures are very high, the water will carry a significant amount of soil on its way towards the ground surface and eject this on the ground surface. This process inevitably leads to loosening of some parts of the foundation soils and often results in creation of local 'collapse zones', sinkholes and 'vents' for pore pressure dissipation and flow of pore water* (Cubrinovski and McCahon, 2011).

The soil fabric of the re-solidified deposits post-liquefaction, is very 'weak', with low liquefaction resistance. Hence, even though in some cases parts of the deposit might have been slightly densified as a consequence of liquefaction, it would be misleading and very unconservative to assume that the liquefaction resistance of such soils has increased. One could assume that sites and areas that exhibited relatively severe liquefaction repeatedly indeed have low to very low liquefaction resistance and should be considered of high liquefaction potential in future earthquakes if no countermeasures against liquefaction are implemented. Using the recorded ground motions and liquefaction observations from the 2010-2011 Canterbury earthquake sequence, an approximate liquefaction resistance map for regional zoning is developed and presented in Section 8.

3.1 Lateral Spreading

Lateral spreading typically occurs in sloping ground or level ground close to waterways/open face (e.g. river banks, streams, in the backfills behind quay walls). Even a very gentle slope in the ground (of just few degrees) will create a bias in the cyclic loads acting on the soil mass during earthquakes which will drive the soil to move in the down-slope direction. If the underlying soils liquefy then the liquefied soil mass ('heavy liquid') will naturally move down-slope and will continue this movement until equilibrium is re-established (or resisting forces reach the level of driving forces). The process of spreading in backfills behind retaining walls is similar, with large ground shaking first displacing the retaining structure outwards (e.g. towards the waterway), which is then followed by lateral spreading in the backfills.

The temporal evolution of lateral spreading is closely related to the development of excess pore water pressures and soil liquefaction in the spreading deposit. While spreading due to the biased seismic loads might be initiated at the early stages of strong shaking during the pore pressure build-up, the magnitude of lateral spreading displacements will increase substantially once the soils liquefy, because soil liquefaction will dramatically reduce the stiffness and strength of soils and will reduce their shear resistance to levels below the amplitude of the driving shear stresses. The spreading may continue even after the strong shaking has diminished and is influenced by a number of factors such as the available soil resistance (soil properties and in-situ state), driving stresses (topography, and ground motion characteristics), dissipation of excess pore water pressures (dynamic permeability and water flow conditions) and magnitude of lateral displacements (change of overall soil volume during spreading). Clearly a complex interplay of the topography, soil characteristics and conditions, temporal and spatial development of pore pressures and strains in the ground during cyclic loading, residual strength and stiffness of liquefied soils, conditions for dissipation of excess pore water pressures, mode of deformation and characteristics of ground motion (earthquake loads) is affecting lateral spreading.

Following the 4 September 2010 and 22 February 2011 earthquakes, field measurements of lateral spreading were conducted by means of the ground surveying method at approximately 120 locations along Avon River in the affected area. Some details of the measured displacements, characteristics of lateral spreading and its impacts on buildings and infrastructure are given in the appended paper (Cubrinovski et al., 2012; Appendix A). In this section, a very brief discussion and excerpts from this document is given.

Figure 9 shows measured lateral spreading displacements along Avon River in Dallington due to the 4 September 2010 earthquake. They illustrate that at some locations the banks of the river moved laterally as much as two metres. The magnitude of spreading displacement typically decreases with the distance form the waterway and the spreading extended up to a distance of 100-250 m from the waterway.

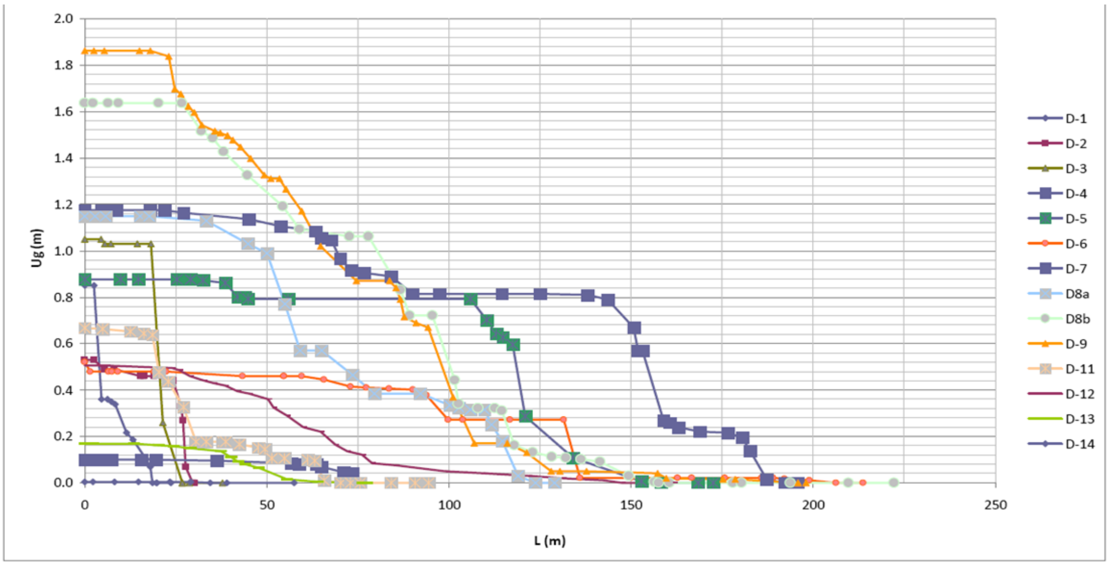


Figure 9. Permanent lateral ground displacements due to spreading along Avon River in Dallington caused by the 4 September 2010 earthquake (plotted as a function of the distance from the waterway)

After the 22 February 2011 earthquake, the ground surveying measurements were repeated at 25 locations to quantify the additional lateral ground movements due to spreading induced by the February event. Figure 10 comparatively shows the spreading displacements induced by the two earthquakes at nine locations along Avon River. Here the horizontal axis indicates the displacement induced by the 4 September 2010 earthquake while the vertical axis indicates the cumulative displacements due to both earthquakes. The contribution of all aftershocks preceding the 13 June earthquakes to the cumulative displacements shown in the vertical axis is considered negligible. The results indicate that the lateral spreading displacements along Avon River induced by the 22 February 2011 earthquake where of similar magnitude with those induced by the 4 September 2010 earthquake.

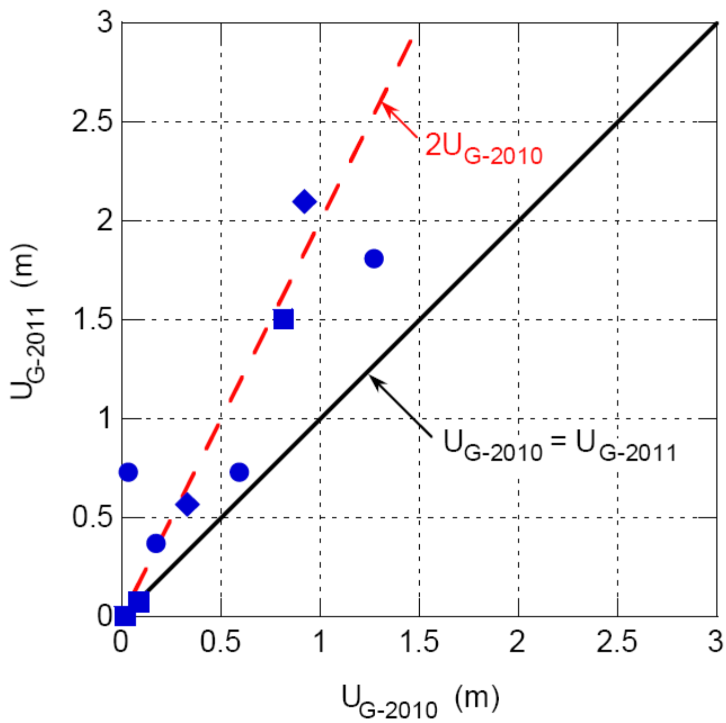


Figure 10. Comparison of permanent lateral ground displacements along Avon River (Avonside, Dallington, Bexley) caused by the 4 September 2010 earthquake (horizontal axis) and cumulative displacements due to the 4 September 2010 and 22 February 2011 earthquakes (vertical axis)

Lateral spreading involves large lateral ground movement at/near open faces (banks of waterways), but also substantial differential displacements in the direction of spreading resulting in large extensional deformation and ground fissuring. For example, the equivalent extensional strains in the zone of largest ground cracks were on the order of 5-10% while the 'average' extensional strains in the area affected by spreading were approximately 0.5-1.0%. The spreading was often accompanied by slumping of the soils near the banks (large settlement/subsidence), which was particularly noticeable at the approaches of bridges. Both lateral and vertical ground displacements induced by spreading where spatially non-uniform resulting in large localized deformation, stretching, tensile cracking and shearing of the ground. The non-uniformity of the ground deformation was further exacerbated by the spatial

variability in the severity of liquefaction as well as the influence of soil-structure interaction on the movement, deformations and flow of water during dissipation of excess pore water pressures. Finally, as implied in the description of the mechanism of lateral spreading, spreading-induced loads involve combined inertial (due to accelerations) and kinematic (due to ground movement) effects, the characteristics of which depend on the evolution of the lateral spreading process in relation to the particular acceleration time history (ground motion at the site) and the site/soil response. Clearly, engineering structures that were located within the spreading zone were subjected to very large and highly non-uniform (both spatially and temporally) ground deformation and seismic forces which were more often than not above the available capacity to sustain such movements/loads, hence resulting in substantial and widespread damage to buildings and infrastructure.

Starting from the Colombo St Bridge, practically all bridges downstream Avon River were severely impacted by lateral spreading. Rotational movements of abutments, damage to foundation piles, subsidence of approaches to bridges and in some cases structural damage were the most typical spreading-induced damage to bridges. Typical damage pattern for bridges is illustrated in Appendix A.

Buried structures were subjected to very large and variable kinematic loads within relatively short distances. For example, a pipeline segment of about 50 m located within the spreading zone could experience large differential lateral ground movement on the order of 0.5 – 1.0 m and similar level of differential settlement of the ground. In addition, parts of the pipeline and adjacent manholes may have experienced uplift due to liquefaction-induced buoyant pressures. The loads on the pipeline and particular modes of deformation depended on the particular layout of the pipeline with respect to the spreading direction and ground movements (parallel, perpendicular or at angle to the spreading).

A wide range and variation of maximum spreading displacements was measured even within a given area illustrating the complex influence and interplay of various factors affecting lateral spreading. The spreading was often very pronounced and of large magnitude in point bar deposits, but much smaller at the cut-banks on the opposite side of the river. Old river channels, streams and gullies, and artificial infilling of wetland areas during the European settlement have also contributed to the variability and complexity in the manifestation of lateral spreading. Along Avon River, the meandering loops (present and past) and topography were affecting the spreading in a very complex way. This is apparent in the pre-earthquake ground elevation map shown in Figure 11 (Landcare Research 2011) where elevation above sea level is indicated. Clearly, the ground is sloping towards the Avon River, though the slopes are very gentle and generally less than 1.5 degrees (2.6 %), and hence the bias in the cyclic load due to the sloping ground is relatively small.

In addition to the ‘localized spreading’ along waterways, there are indications from aerial observations (aerial photogrammetry and LiDAR) that global patterns of spreading driven by topographic and geometry conditions may have occurred. Such spreading movements may have involved quite large areas but they do not induce large localized strains and deformations at shallow depths, and hence are much less damaging to the water and wastewater systems than the differential spreading movements along waterways described above. Further investigations of the global patterns of spreading are currently under way (Tonkin and Taylor, 2011).

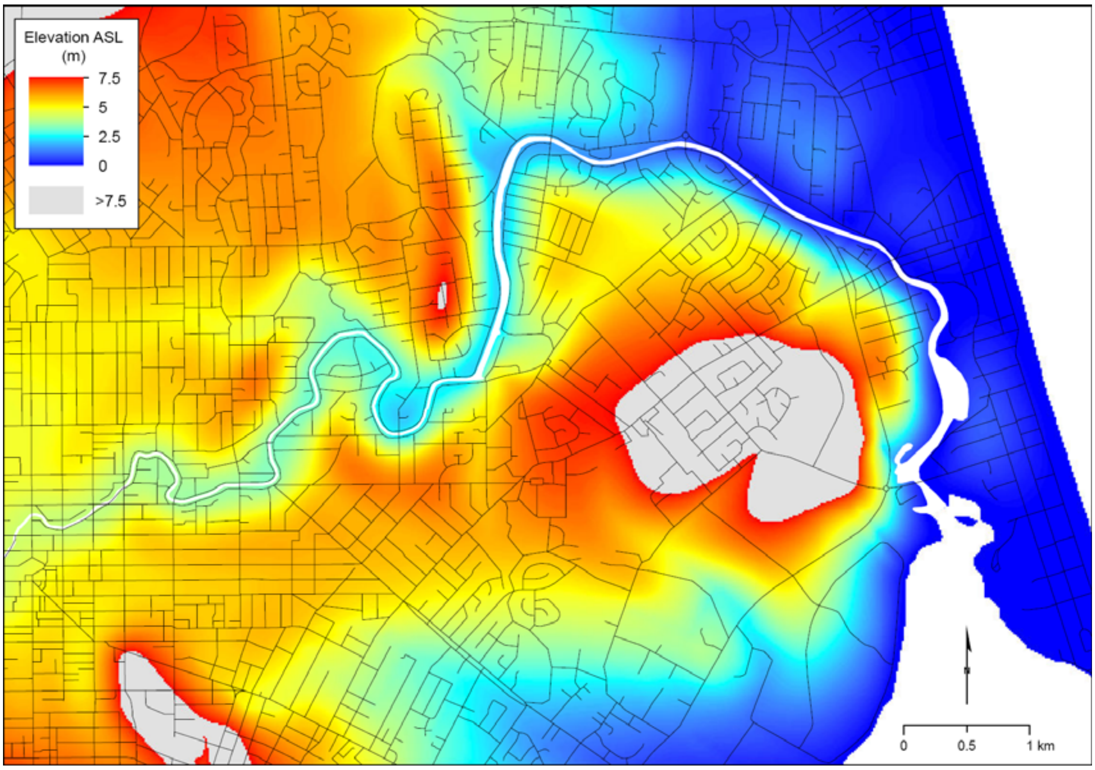


Figure 11. **Pre-earthquakes ground elevation above sea level along Avon River, from the CBD to the estuary (derived from the Digital Elevation Model of Christchurch, Landcare Research, 2011)**

3.2 Summary remarks

- Widespread and severe liquefaction occurred in the suburbs of Christchurch and its CBD. Such extensiveness and severity of liquefaction in native soils is exceptional by international standards.
- The repeated and very severe liquefaction particularly along the Avon River and in some other localized areas, clearly indicate that such soils have very low liquefaction resistance. There are several contributing factors to such low resistance:
  - i) By composition and their plasticity, (non-plastic sands, silty sands, sandy silts, and silt-sand-gravel mixtures) the soils are highly susceptible to liquefaction.
  - ii) Their in situ state (conditions) including full saturation (high water table), medium or loose to very loose density and fluvial deposits fabric (granular structure of river deposits) all point out to a high liquefaction potential.
  - iii) The soils are relatively young and apparently free of any serious aging effects, which again suggests a low liquefaction resistance.
  - iv) The groundwater regime of Christchurch is exceptional with significant groundwater flow through aquifers and many wells and natural springs in the area. The artesian pressure and upward water flow reduce the effective stress in the subsurface soils and reduce (eliminate) the possibility for soils to get stiffer and stronger due to

aging effects. Liquefaction resistance is known to increase with the age of soils due to changes in their micro-structure and cementation (aging effect). We speculate that such aging effects on soils could not develop in the Christchurch groundwater environment.

v) Finally, we have to emphasize again that the severity of ground shaking together with the aforementioned factors also played key role in the severity and extensiveness of the induced liquefaction. Preliminary analyses suggest that liquefaction impacts of an M_w=8.0 Alpine Fault earthquake will be much less severe than those of the 22 February 2011 earthquake.

3) Ground surveying measurements at approximately 80 locations along Avon River indicate maximum (relative) magnitudes of permanent lateral ground displacements due to spreading of liquefied soils on the order of 1.0 – 2.0 m. The spreading typically extended inland up to a distance of 100 m to 250 m from the waterway.

4) Different spreading patterns and distribution of lateral displacements with distance from the waterway were observed in North Kaiapoi, South Kaiapoi and along Avon River in Christchurch. In addition to the more conventional 'exponential decay' distribution where the spreading displacements rapidly decrease with the distance from the waterway, a block-mode failure was observed in South Kaiapoi with the largest and very damaging ground fissures opening at a distance of approximately 125-250 m from the waterway. The spreading along the meandering loops of Avon River showed very complex pattern and was affected by the interplay of soil conditions, topography, river geometry and local depositional environment.

5) The spreading induced very large and non-uniform ground deformation/displacements in the affected zone severely impacting infrastructure in the area. Road bridges suffered consistent spreading-induced damage and deformation mechanism including rotation of the abutments associated with deck pinning and damage at the top of the piles. Slumping of the approaches was also typical damage feature at locations of large lateral spreads. Loss of grade in gravity pipes, breakage of brittle pipes, failure of joints and connections were typical failures in potable water and wastewater pipe networks of Christchurch.

6) When evaluating lateral spreading one should carefully consider ground elevation (direction of sloping), river geometry (meandering, loops, cutbanks, point bar deposits), presence of weakened zones (old river channels, fills, etc.) and geotechnical conditions, next develop lateral spreading zoning and probable range of spreading displacements and their distribution, and then anticipate loads and deformation of the pipeline having in mind its particular layout relative to the direction of lateral spreading.

4 PERFORMANCE OF THE POTABLE WATER SYSTEM

4.1 Main characteristics of the potable water system

The Christchurch water supply system is an integrated citywide network that sources high quality groundwater from confined aquifers, and pumps the water into a distribution pipe network throughout the city consisting of 1600 km of watermains and 2000 km of submains (CCC 2010a).

The water is supplied from approximately 150 wells at over 50 sites, 8 main storage reservoirs, 37 service reservoirs and 26 secondary pumping stations. The system is divided into distinct pressure zones and uses bulk storage reservoirs to assist in meeting peak demands and providing for emergencies. The wells and pumping stations are evenly distributed throughout the city, providing efficient delivery of water at a relatively uniform pressure within each zone. Secondary pumping stations and reservoirs are used in areas of undulating terrain (e.g. Port Hills). The system is centrally controlled from the main wastewater treatment plant.

Watermains and submains are located almost exclusively within legal roads, at shallow depths. The preferred location for principal watermains is in the carriageway, about 2.0-2.5 m from the kerb. Submains are typically installed beneath footpaths approximately 150mm from boundaries. Submains are served from crossovers which are usually located at fire hydrants. All crossovers are 50mm in diameter regardless of the submain size, with the preferred connection into either a tapped hydrant riser or into the main at a hydrant tee. The system is designed so that turning off a maximum of five valves can isolate any area in the network that serves no more than 50 properties. A typical layout of watermains and submains is shown in Figure 12.

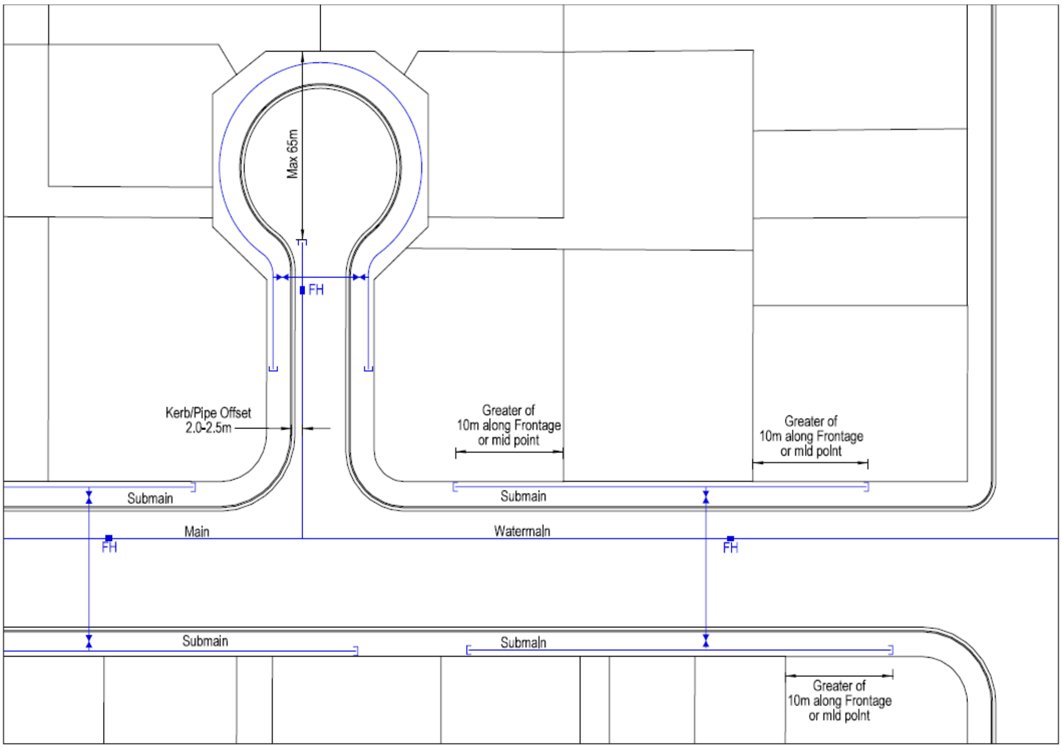
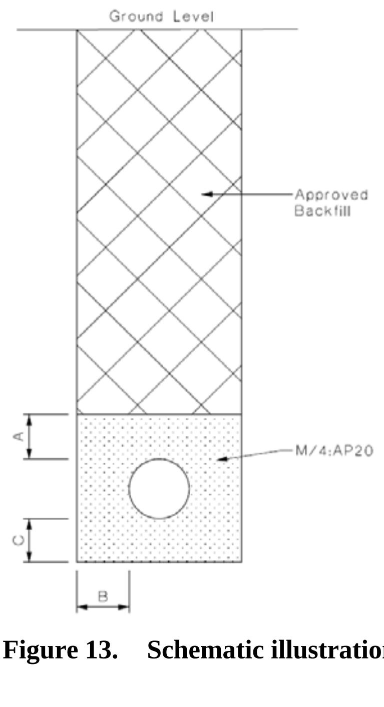


Figure 12. Typical layout of watermains and submains in the carriageways and footpaths of Christchurch streets (CCC, 2010a)

Standard diameters of watermains are 100 mm to 600 mm, while submains have diameters of 50 mm and 63 mm. Watermains are laid in trenches 200-300 mm wider than the pipe diameter, at relatively shallow depths. The cover thickness depends on the pipe size, location and material, but is usually about 800mm (at least 750mm, but no more than 1.5m for the standard watermains diameters). Typical thickness of cover for submains is 300-500 mm. The trenches are backfilled with native soils and are compacted to 95%, 90% and 70% of the material's maximum dry density (NZS 4402.4.1.1) for trafficked, pedestrian and landscape areas, respectively. Note that the backfill excludes haunching and bedding, for which as indicated in Figure 13 AP20 material is used (sandy gravel with at least 55% gravel size particles and 8-15% fines).

A GIS layout of the watermains network is shown in Figure 14 in which three different colours are used to distinguish between different pipe materials: polyvinyl chloride (PVC) pipes (green), polyethylene (PE) pipes (magenta) and other material pipes (grey). As indicated with the pie chart in the inset of the figure, out of the 1511km pipe length (covered in the analysis excluding the hills), 797km, or 52.7%, of the watermains are asbestos cement (AC) pipes; 398km, or 26.4%, are PVC pipes; 27km, or 1.8%, are steel pipes; only 15km, or nearly 1.0%, are PE pipes (which as illustrated subsequently are the most robust regarding liquefaction-induced ground deformations); and 273km, or 18.1%, are pipes made of other materials. The stated percentages and distribution of materials comprising the watermains system reflects various phases in the historical development of the system and selection of pipe materials. In recent years, three pipe materials have been used for watermains: ductile iron, PVC and PE, with a number of criteria being used in the selection of the pipe material (CCC, 2010a).

The submains network predominantly consists of PE pipes with a pipe length of 1318km, or 84.6%, out of the total length of 1557km, with PVC pipes and Galvanized Iron (GI) having 52.3km and 161.9km, or 3.3% and 10.4%, of the total length, respectively (Figure 15).



Nominal Pipe Diameter Φ	A	B	C
< 150	100	100	100
≥ 150 <300	150	150	100

Figure 13. Schematic illustration of backfill and pipe-laying details (units in mm)

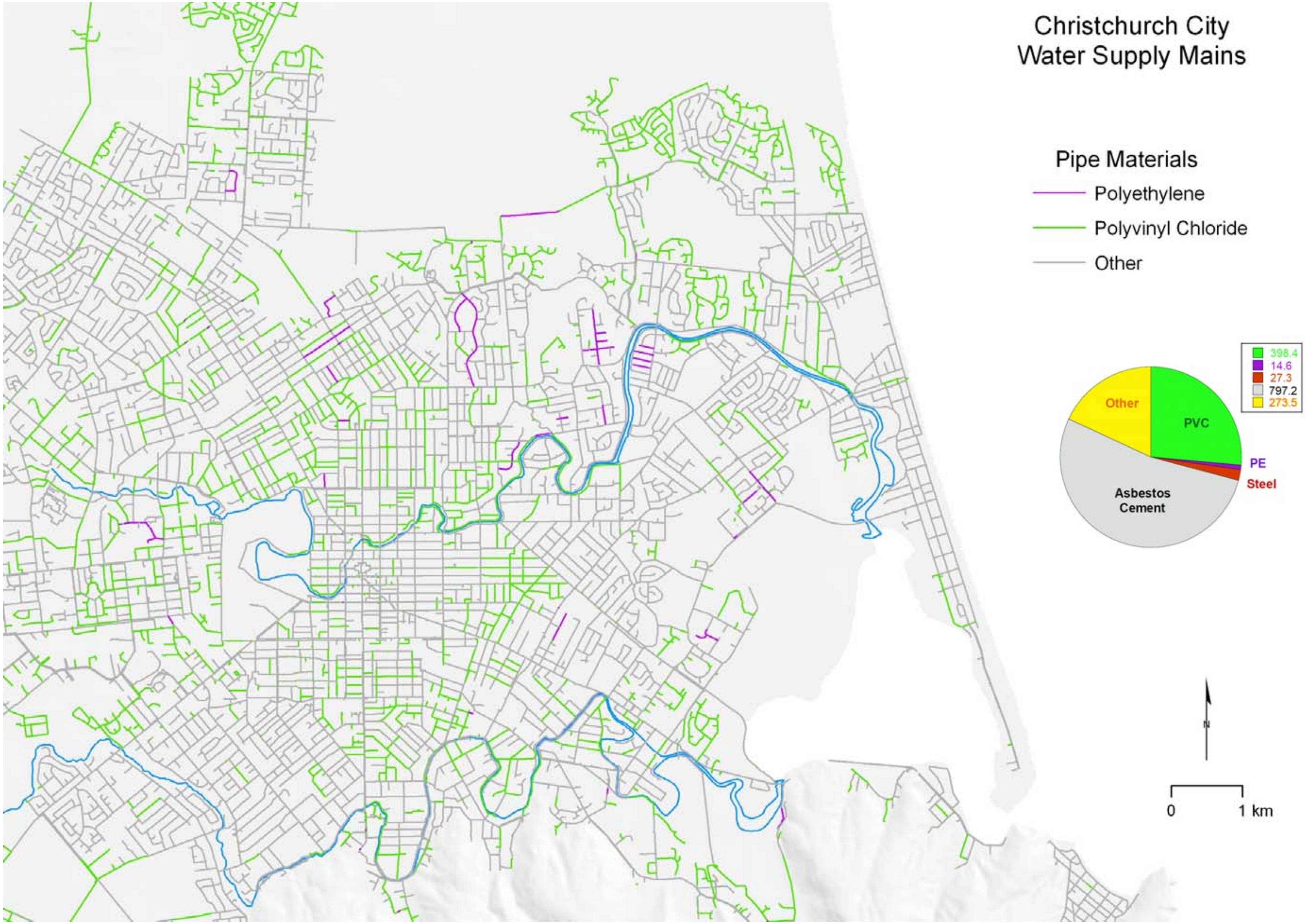


Figure 14. Christchurch watermains network indicating different pipe materials (location and length of prevalent or relevant materials from a resilience perspective)

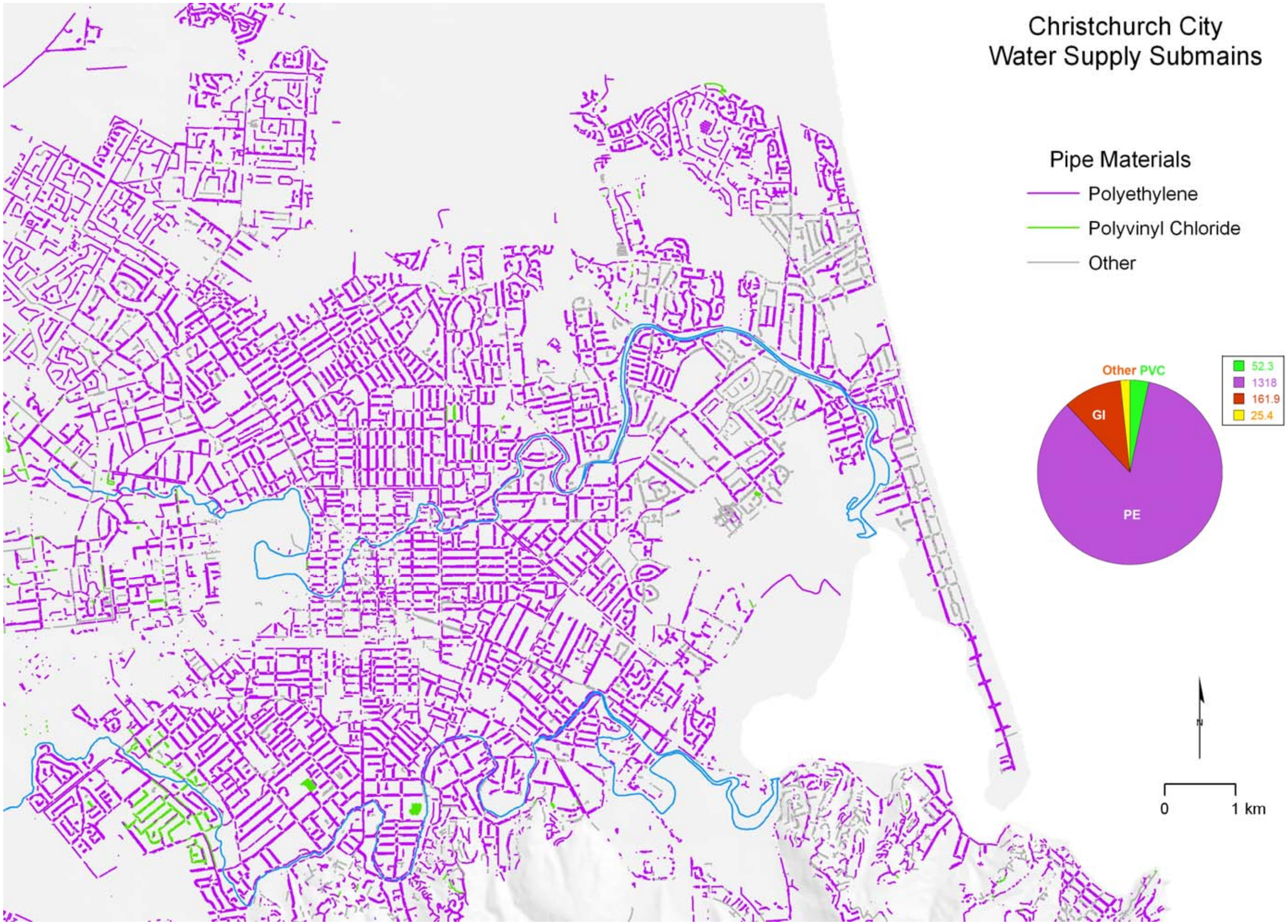


Figure 15. Christchurch submains network (PE pipes cover approximately 85% of pipe length)

The water supply network is designed for an asset life of 100 years, which is also the minimum required design life of the pipes and fittings. The Infrastructure Design Standard (CCC, 2010a) stipulates that

> *"all structures must be designed with adequate flexibility and special provisions to minimise risk of damage during earthquakes" and to "provide flexible joints and isolation valves at all junctions between rigid structures (e.g. reservoirs, pump stations, bridges, buildings, manholes) and natural or made ground".*

There are no provisions specific to soil liquefaction in the aforementioned Infrastructure Design Standard.

4.2 Performance of the potable water system (PWS) in the 2010-2011 earthquakes

To rigorously evaluate the performance of the potable water system, at least three levels of analyses are envisioned for the pipe network within the GIS framework. Ideally, each analysis level should be applied to individual major seismic events, such as the 4 September 2010, 22 February 2011 and 13 June 2011 earthquakes. The three levels of analyses are:

1. Level 1 Analysis (L1A), which provides assessment of the performance of different pipe materials and associated design/construction details. In its simplest form (presented in this report), this analysis allows a comparison of the repair/break rates for different pipe materials. A more rigorous approach would allow us to discriminate between different types of failures/breaks, and hence, to identify the particular design, construction, and material details that contributed the most to the network damage. It is critically important to collect further data and conduct this more rigorous analysis in order to develop an enhanced understanding of the performance of the potable water system in the 2010-2011 earthquakes.
2. Level Two Analysis (L2A), is equivalent to LA1 except that it focuses on correlating the network damage with the observed liquefaction severity (land damage). Again, several levels of rigour could be used for LA2. In addition to discriminating between different failure types as mentioned for LA1, the severity of liquefaction or land damage (ground deformation) can be expressed either in simple qualitative terms (presented in this report) or in terms of quantitative measures of ground movement such as PGV or transient/permanent ground displacements (lateral displacements and settlements).
3. Level Three Analysis (L3A), which uses geological and geotechnical data (i.e. SPT, CPT and Vs from field data, and laboratory soil data from physical/deformational tests) to estimate liquefaction potential and consequent ground deformation, and then correlate the predicted land damage with the actually observed pipe network damage. This approach allows to develop predictive models for seismic performance of potable water networks that could be applied to areas with similar ground conditions within Christchurch that were not subjected to very severe ground shaking or to areas outside of Christchurch.

Summary of the applied analysis methodology and results of LA1 and LA2 are presented below whereas more details of the analyses and results are given in Appendix B.

Data and Methodology

The Council provided the water supply Geographic Information System (GIS) layer through their Web Features Service. Attribute information included pipe segment unique identifiers, year of pipe installation, pipe lengths, diameters, depths and materials. Five pipe materials were separately considered in the analyses: polyvinyl chloride (PVC), polyethylene (PE), steel (S), galvanised iron (GI) and asbestos cement (AC). All other materials were lumped in the category "other materials". L1 and L2 Analyses were conducted on watermains and submains networks, using repair data for the three major earthquakes. This summary provides the analyses results only for the 22 February 2011 earthquake, which as previously mentioned inflicted the largest network damage. The analyses were carried out in their simplest form (due to lack of data and time), and hence, types of failures were not differentiated and liquefaction effects were not rigorously quantified.

One of the key issues encountered in the repairs data was that often several repairs were lumped into a single record (repair point, CCC, 2011), thus making it impossible to identify the exact number of repairs. In addition, the description of repairs was either not sufficient or not presented in a readily available format for analysis. In this context, the presented analyses and results should be considered as preliminary and as a work-in-progress. However, we do not anticipate significant changes in the figures/results presented herein, and hence they should be treated as representative for early decision-making. The data used in the analyses presented herein was for the period "February to June", and was assumed to include the repairs carried out in the period 23 February 2011 to 12 June 2011.

Using GIS tools, the total length of pipes, lengths of different pipe materials, lengths of pipes in areas of different liquefaction severity and number of repairs per unit length were calculated and then used in further statistical analysis. The L2 Analysis was conducted only for the area covered in the drive-through ground surveying shown in Figure 6, in order to compare rigorously the performance of the network across areas with similar seismic demand but with different liquefaction severity including areas of no liquefaction.

Level 1 Analysis Results for Watermains

Figure 16 shows the location of repairs/faults on the watermains network following the 22 February 2011 earthquake. In the background GIS layer the pipe materials are indicated with different colours. In the inset of the figure, the performance of different pipe materials is summarized with a bar chart (for areas in the plains, excluding the hills). It shows that 5.1% of the total length of watermains was damaged, or 77.5 km out of 1511 km considered in the analysis. Steel pipes suffered the largest damage (8.9%), followed by AC pipes and other material pipes (6.1% and 6.8%, respectively), whereas much better performing were the PVC (1.8%) and PE (0.5%) pipes. The results of the analyses are summarized in Table 2. It is noted that the sample lengths of PE and S pipes are considered insufficient for a robust statistical analysis and hence the respective results should be treated with caution.

Level 2 Analysis Results for Watermains

Figure 17 shows the same repairs/faults data for the watermains network and 22 February earthquake with the liquefaction map (Figure 6) shown in the background. Using this setup in the GIS framework, it was possible to calculate and correlate the pipe damage with the observed severity of liquefaction. The pie chart shown in the

inset of Figure 17 indicates that 34 km of the damaged pipes or 58% of the damaged length in the area covered by ground surveying (Figure 6) was in areas of moderate to severe liquefaction, 20.2% were in areas of low to moderate liquefaction, 2.5 % in areas where traces of liquefaction were observed and 19.3% in areas where no signs of liquefaction were observed. Hence, there is a clear link between liquefaction severity and damage to the pipe network. To further scrutinize the correlation between the damage to pipes and liquefaction severity, the results are summarized separately for different materials in Table 2 and with a series of bar charts shown in Figure 18. These results indicate the following:

- For all pipe materials except PE pipes, there is a clear increase in the affected length (percentage of damage) with increasing liquefaction severity
- For S, AC and other materials pipes, the percentage of damaged pipes in areas of severe liquefaction was very high, between 15% and 22%.
- PVC pipes suffered two to four times less damage than S, AC and other material pipes
- There is an indication that PE pipes performed well though the sample is too small for any definitive conclusions. For the same reason, the "anomalous" result obtained should be ignored until data and details/reasons for the failure/repair of the short pipe segment of PE pipe in the no liquefaction area are available/clarified.
- The level of pipe damage in no liquefaction and not inspected areas are similar indicating that ground displacements/performance were similar in these areas (with general absence of liquefaction manifestation).

Table 2. Results of Level 1 and Level 2 Analyses for the Christchurch watermains network, using repairs/faults data for the 22 February 2011 Earthquake

Pipe material	Total length (km)	Damaged length						
		Level 2 Analysis					Level 1 Analysis	
		Severe Liquefaction, in km & (%)	Low-Mod. Liquefaction, in km & (%)	Traces, in km & (%)	No Liquefaction, in km & (%)	Not inspected, in km & (%)	Total length, km	Damaged length (%)
PVC	398.4	3.8 (7.9)	1.1 (3.7)	0 (0)	0.5 (0.73)	1.8 (0.72)	7.2	1.8
PE	14.6	0 (0)	0 (0)	0 (0)	0.07 (2.7)	0 (0)	0.07	0.5
S	27.3	1.0 (20.7)	0.5 (17.6)	0.04 (5.3)	0.3 (6.5)	0.6 (4.1)	2.4	8.9
AC	797.2	20.2 (22.1)	7.9 (10.6)	1.0 (9.9)	8.0 (5.8)	12.0 (2.5)	49.1	6.1
Other	273.5	9.0 (15.4)	2.3 (7.8)	0.4 (11.4)	2.5 (3.3)	4.5 (4.2)	18.7	6.8

*) Figures in brackets indicate percentage of damaged pipes within the particular class

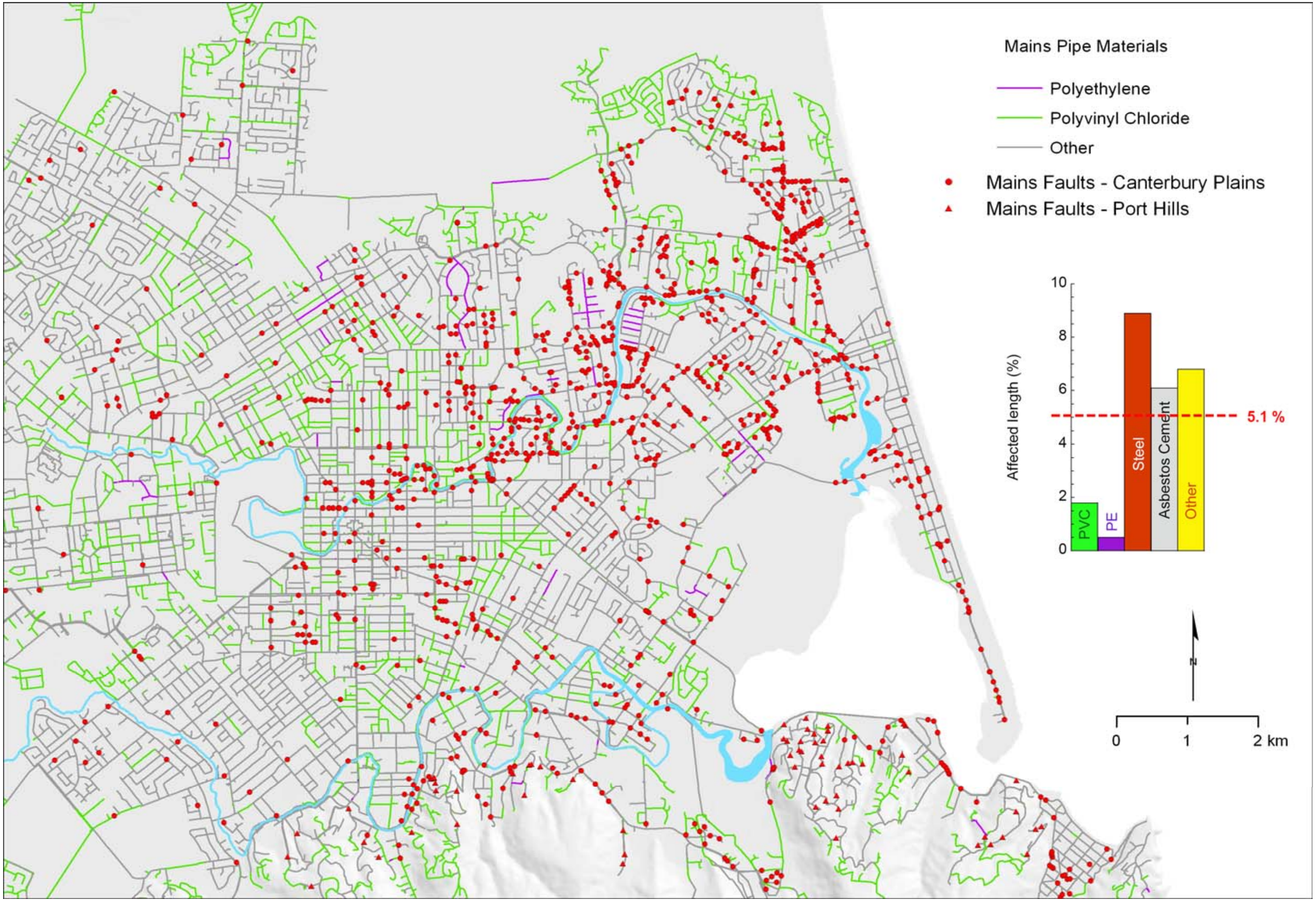


Figure 16. Locations of repairs/faults (indicated by red symbols) on the Christchurch watermains network following the 22 February 2011 earthquake

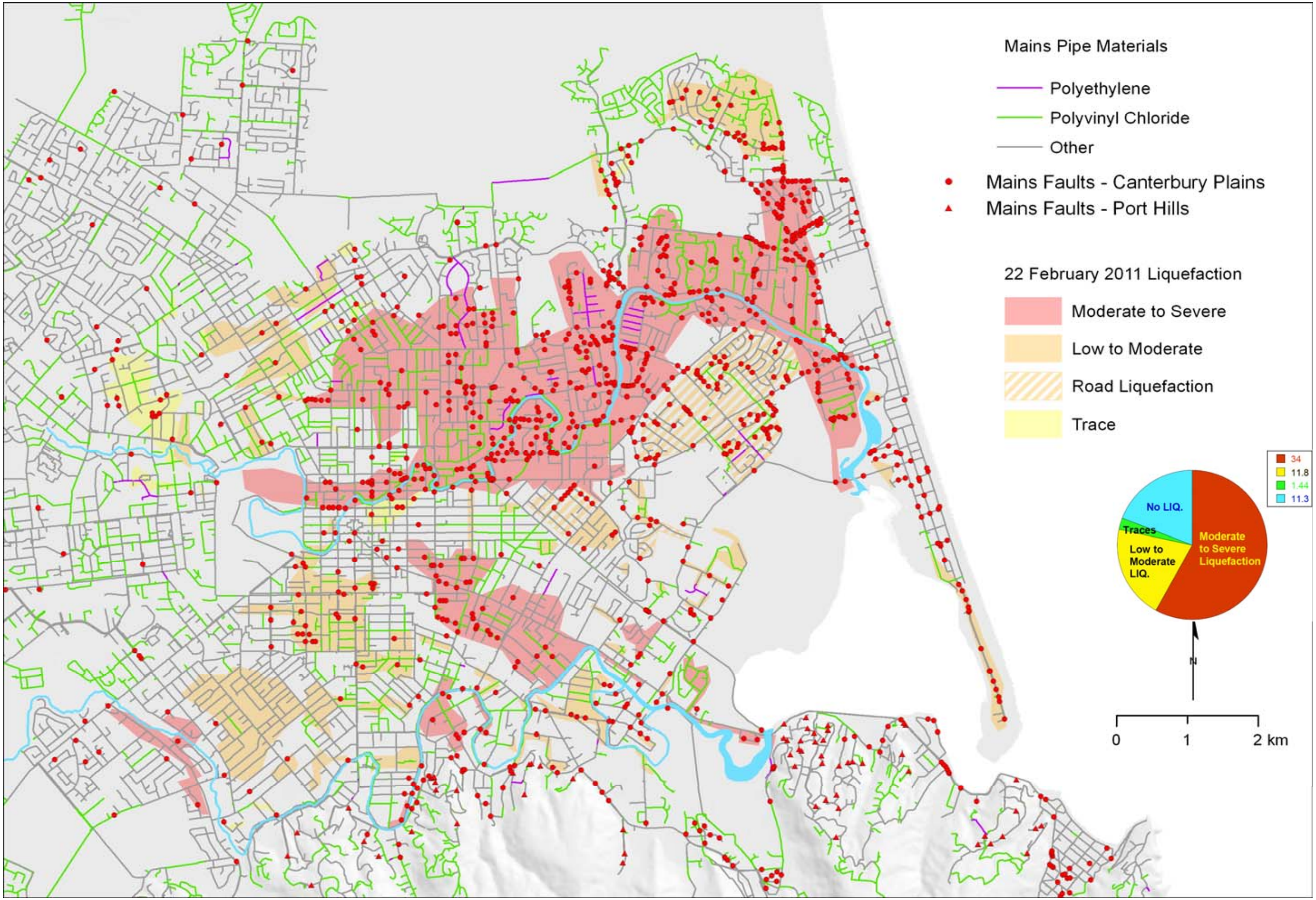


Figure 17. Locations of repairs/faults (red symbols) on the Christchurch watermains network and areas of liquefaction following the 22 February 2011 earthquake

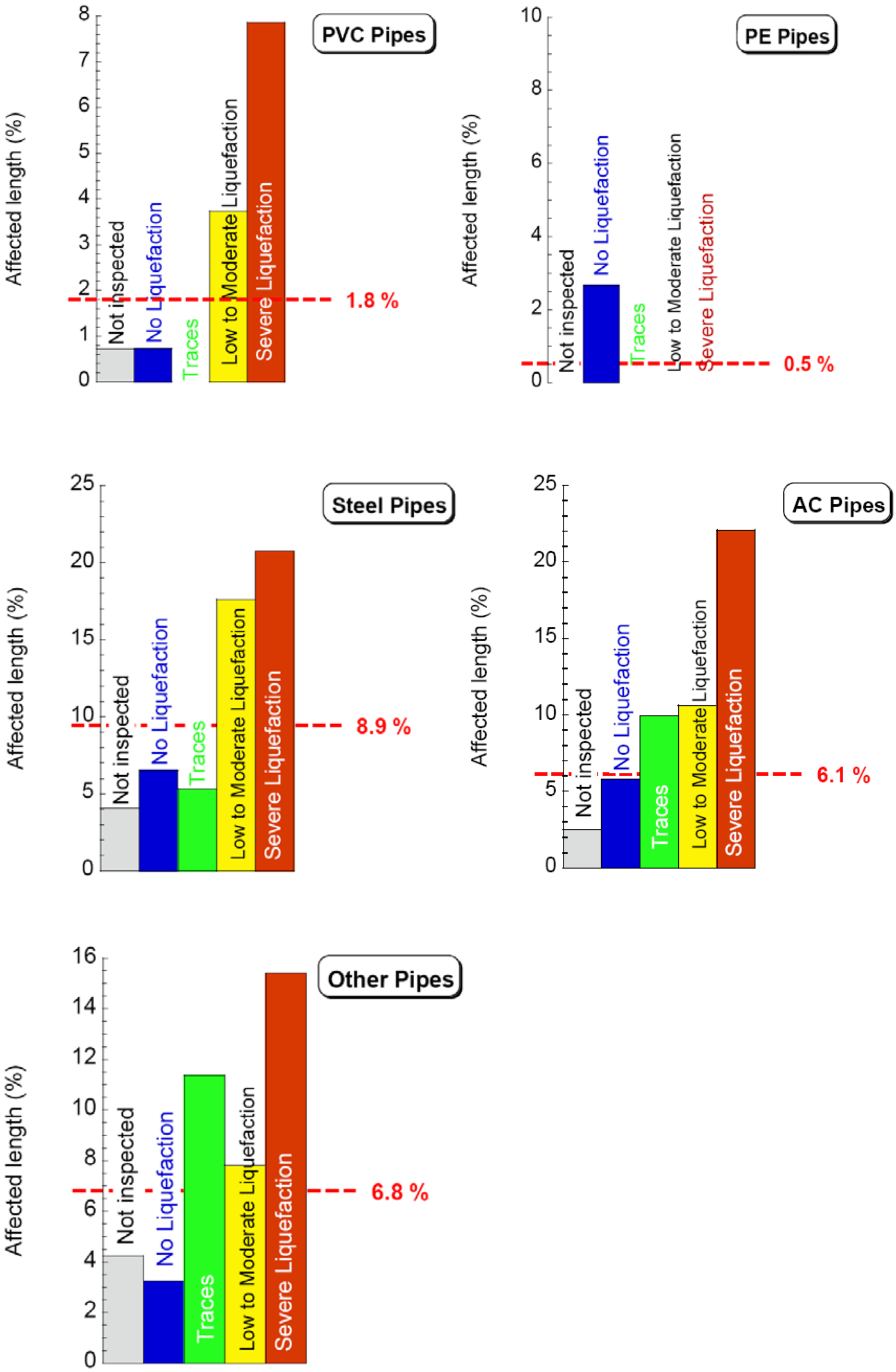


Figure 18. **Percentage of length damaged for water mains due to the 22 February earthquake, for different pipe materials and liquefaction severity (dashed red lines represent the average affected length across all ground damage levels).**

Analysis Results for Submains

An equivalent analysis was conducted for submains, the results of which are summarized in Table 3. and Figures 19, 20 and 21. The key results from these analyses can be summarized as follows:

- For PE pipes, the percentage of damaged length ranged between 1.4% (not inspected areas) and 5.2% (Areas of severe liquefaction). Again, there was a clear increase in damage with liquefaction severity.
- PE pipes suffered, on average, five to six times less damage than GI pipes.
- GI pipes performed poorly with 17% damaged length in areas of low to moderate liquefaction and 26% in areas of severe liquefaction.
- For PVC pipes, the percentage of damaged length ranged between 2% and 3% (for non-liquefied and severe liquefaction areas respectively). However, the PVC pipes sample size was insufficient and hence the PVC submains results should be treated with caution.
- Comparing the damage to watermains and submains, it appears that for each pipe material the damage to the submains was larger than the damage to the mains. The total damaged length of submains was smaller, however, because over 80% of the submains were comprised of the well performing PE pipes.
- Even though in this simplest form of the analysis the damage is always associated to a certain pipe material, the nominally defined "failures" include (and probably are dominated at least for the PE pipes) by failures of particular components (joints, connections, fire hydrant details, crossovers, laterals) rather than material failures. *It is critically important therefore to discriminate between different types of failure and carry out a more rigorous second stage of analysis, which will help to identify key weaknesses and also "good design/construction details/characteristics" of the Christchurch potable water system that was subjected to the series of severe earthquakes.*

Table 3. **Results of Level 1 and Level 2 Analyses for the Christchurch submains network, using repairs/faults data for the 22 February 2011 Earthquake**

Pipe material	Total length (km)	Damaged length						
		Severe Liquefaction, in km & (%)	Low-Mod. Liquefaction, in km & (%)	Traces, in km & (%)	No Liquefaction, in km & (%)	Not inspected, in km & (%)	Total length, km	Damaged length (%)
PVC	52.3	0.1 (2.9)	0.04 (2.6)	0 (0)	0.05 (1.6)	1.0 (2.2)	1.2	2.3
PE	1318	11.3 (5.2)	5.2 (4.5)	0.4 (2.5)	6.8 (2.6)	9.4 (1.4)	33.1	2.5
GI	162	9.3 (26.1)	4.8 (17.0)	0.05 (5.5)	4.5 (11.7)	5.4 (9.2)	24	14.8
Other	25.4	0.23 (16.4)	0.09 (4.7)	0 (0)	0.02 (2.8)	0.45 (1.2)	0.79	3.1

*) Figures in brackets indicate percentage of damaged pipes within the particular class

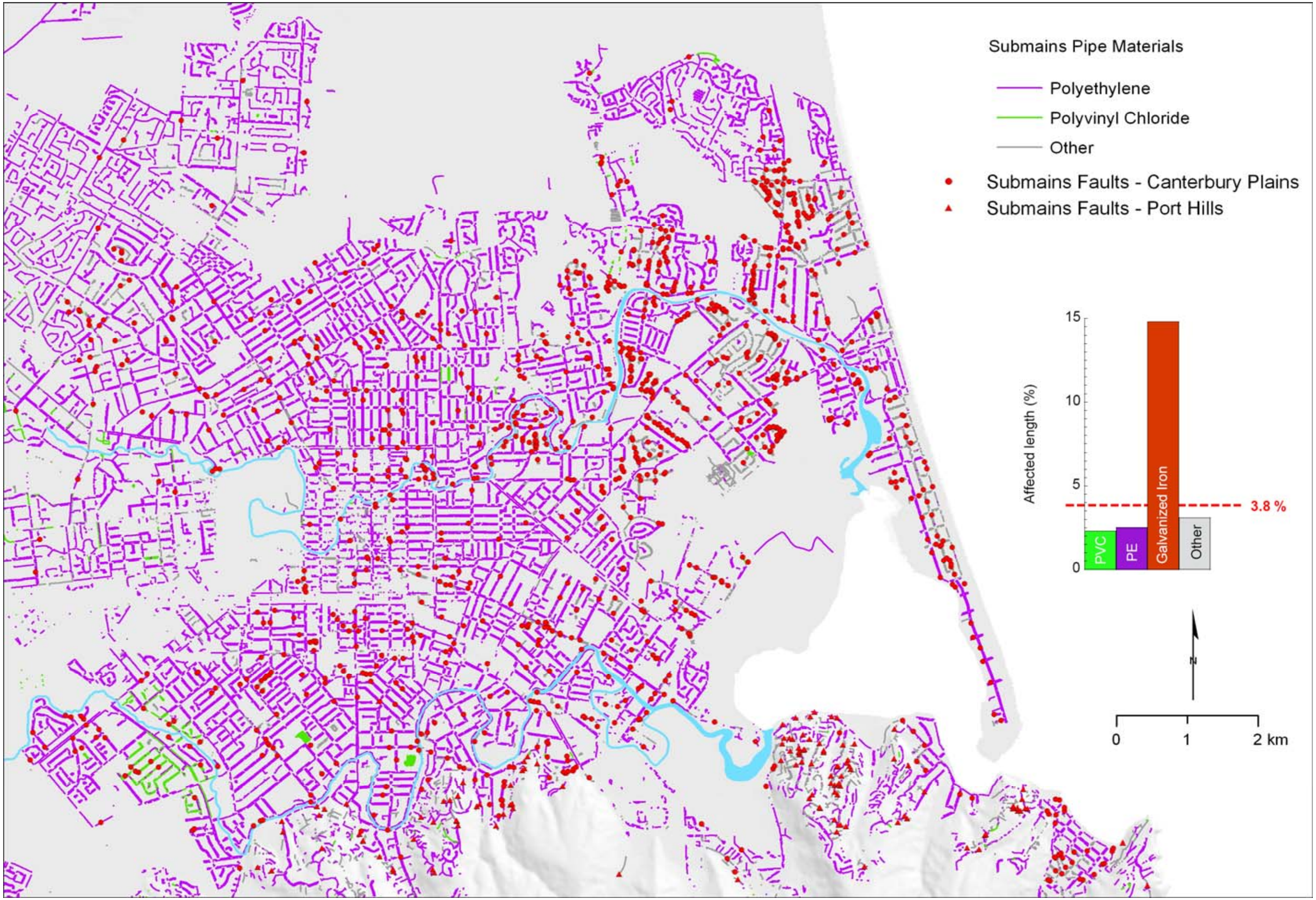


Figure 19. Locations of repairs/faults (red symbols) on the Christchurch submains network following the 22 February 2011 earthquake

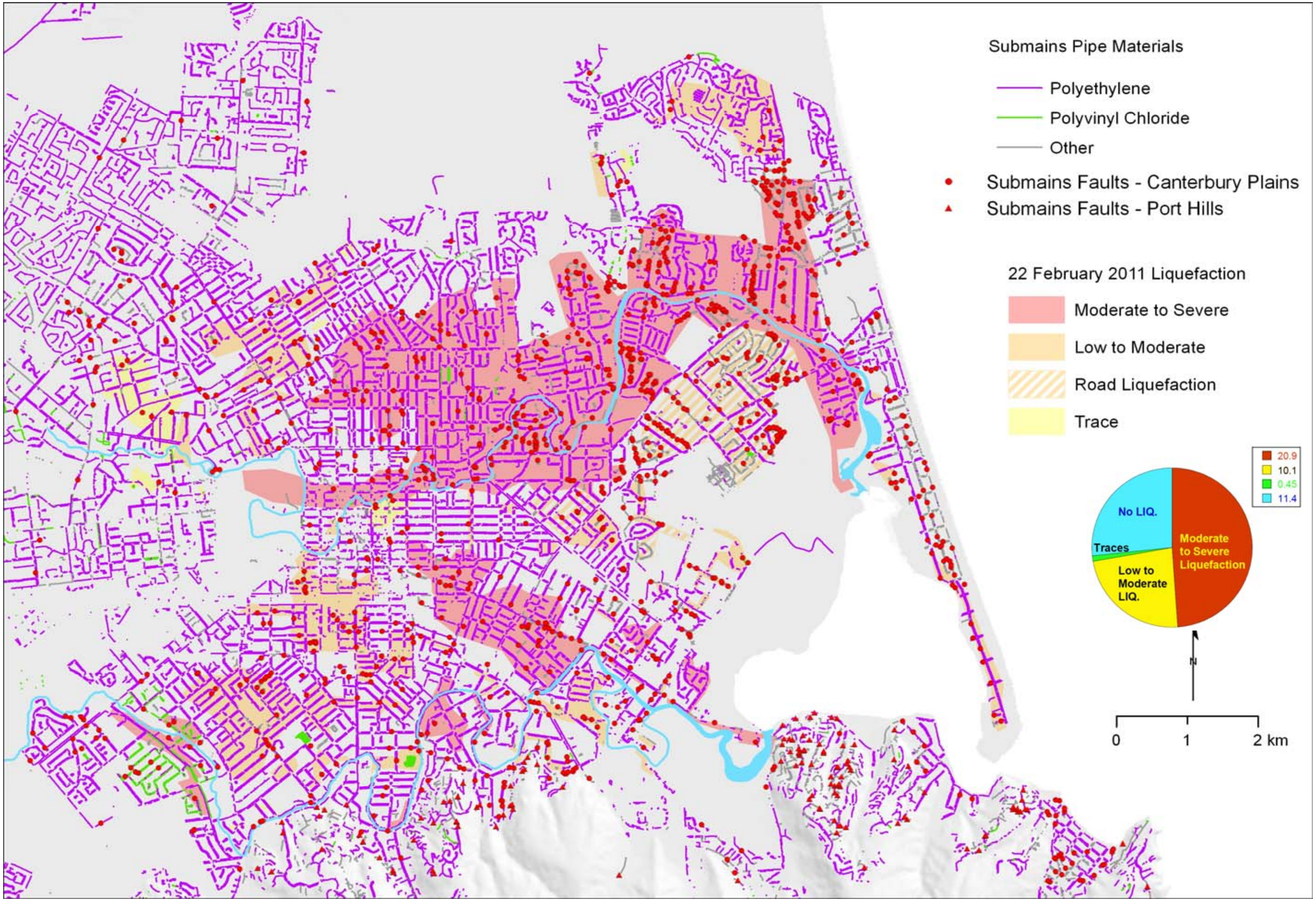


Figure 20. Locations of repairs/faults (red symbols) on the Christchurch submains network and areas of liquefaction following the 22 February 2011 earthquake

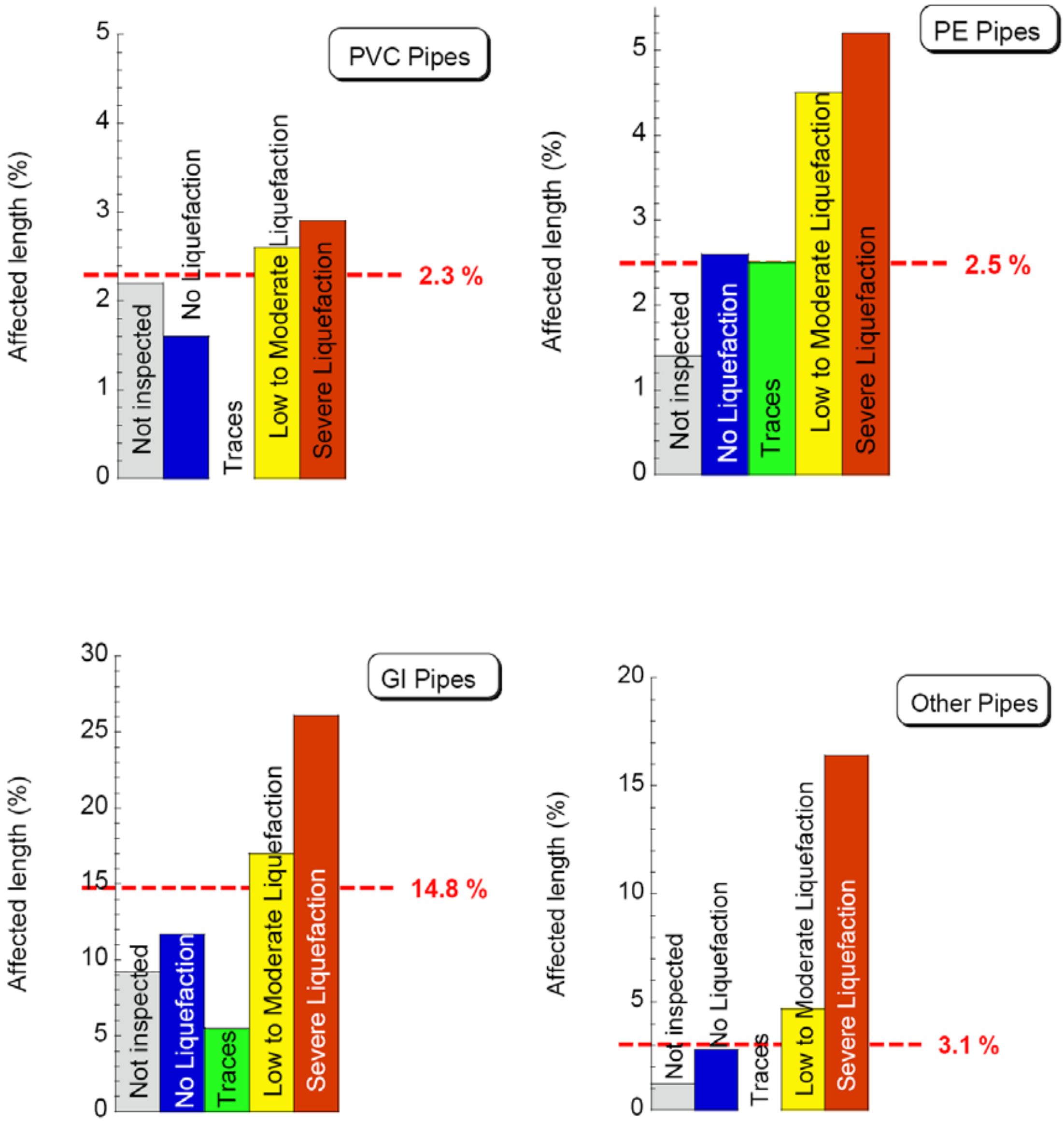


Figure 21. Percentage of length damaged for submains due to the 22 February earthquake, for different pipe materials and liquefaction severity (dashed red lines represent the average affected length across all ground damage levels).

5 PERFORMANCE OF THE WASTEWATER SYSTEM

5.1 Main characteristics of the wastewater system

The terrain of Christchurch (excluding the hills to the south) is very flat with an average slope of approximately 0.1 – 0.2% from the west part of the city towards the coastline to the east. For this reason, the Christchurch wastewater system differs from most of other cities in New Zealand: it makes extensive use of flatter grades than normal sewer grades and also a large number of pumping stations. The flat grades often result in velocities significantly lower than the widely used self-flashing velocity of 0.7 m/s. The system consists of a citywide network of about 1800 km of pipes that collect and transport the wastewater to the Bromley treatment plant.

Before the earthquake, on average, the system was considered in a good condition despite its age. It is interesting though that infiltration (entry of subsurface water) and inflow (entry of surface water) together accounted for approximately one third of the annual wastewater flow. This fact should be considered in the context of the very intense surface water flow and groundwater regime of Christchurch.

Wastewater pipes are laid in the centre of the road with minimum vertical cover of 1.2 m. Gravity pipes are laid in straight lines and at a constant gradient between access points (manholes and inspection chambers). The minimum size of private sewers is 100 mm. Majority of the pipes are at depths between 2.0 m and 3.5 m.

The maximum spacing between manholes is from 100 m (for D = 150 – 225 mm pipes) to 180 m (for D ≥ 1600 mm pipes). Two types of manholes have been used: cast in situ square shape manholes (an older version of the manholes), while the new version of the manholes is a lighter precast circular in shape manhole. The design factor of safety against floating is set at FS ≥ 1.2 assuming an empty manhole, fully saturated soil and no contribution of skin friction.

The trenches for the pipes are 300 mm wider than the external diameter of the pipe. Haunching and surrounding soils are compacted to 95% maximum dry density using AP40 and AP20 materials. Approved geotextile is used to protect the haunching and pipe foundation soils from infiltration of surrounding soils and backfills.

The existing pipe materials and their distribution throughout the city reflect various phases and preferred choice of materials in the historical development of the city and the system. Current specifications (CCC, 2010a; CCC, 2010b) indicate that PVC, PE and ceramic pipes are suitable for use in gravity sewers while pressure pipelines are normally constructed from PVC-U, PE or concrete pipes.

The wastewater system has a design asset life of at least 100 years.

5.2 Performance of the wastewater system (WWS) in the 2010-2011 earthquakes

In both earthquakes (4 September 2010 and 22 February 2011), the wastewater system was hit particularly hard resulting in numerous failures and limited or no-service provided to large areas. As described in Section 6, typical damage to the network included loss of grade in gravity pipes, breakage of pipes/joints and

infiltration of liquefied silt into pipes (often accompanied by depression of carriageways, undulation of road surface and relative movement of manholes), and failure of joints and connections (particularly numerous failures of laterals).

Detailed information on the damage to the wastewater system was still not available at the completion of this project predominantly for two reasons. A very large portion of the network was severely damaged and more importantly, the inspection of the damage and repairs of the wastewater network (unlike the water supply system) is much more difficult because the network is installed at larger depths (often exceeding 2.5 m depth) and therefore there was a need for large number of repairs at depth, which in turn requires dewatering and trench support. It is estimated that a full recovery of the system will take about 2 to 3 years to complete, or even longer.

The status of the damage inspections and data collection on the wastewater system can be summarised as follows:

- CCTV inspections have been completed for parts of the damaged network after the Darfield earthquake. This data was still not readily available in analysis format at the completion of this project.
- CCTV inspections have been also conducted for parts of the network after the 22 February earthquake. This data was in early stages of compilation during this project.
- Information on manholes including their movement relative to the adjacent road surface has been also compiled, but again was not available during the project.

It is critically important to systematically collect, interpret and analyse these data in order to find out what worked well and what didn't work in the existing wastewater system that was hit by the earthquakes, and also to identify key modes of failure/damage. In addition to the abovementioned data/information, the project team recommended and initiated:

- Detailed case studies in selected areas in order to collect data and information on the performance of the network targeting specific sections/details of the system.
- Development of systematic field inspection procedures/form for documenting relevant damage data during repairs over the period of reinstatement of the system.

Service status maps for the wastewater network are shown in Figures 22 and 23 (showing the status of the system on 16 March 2011 and 27 April 2011 respectively) to illustrate the extent of the damage and its distribution due to the 22 February 2011 earthquake. Red lines indicate parts of the network without service, orange lines indicate limited service and green lines show full service of the network. The maps indicate the speed of recovery of the service into partial or full service though one should note that this includes also temporary solutions and restoration of service rather than full reinstatement of the network.

Further studies and analyses of the wastewater network are required and strongly recommended (at all three levels of analyses and both in its simplified form and also by discriminating different failure types and damage contributions).

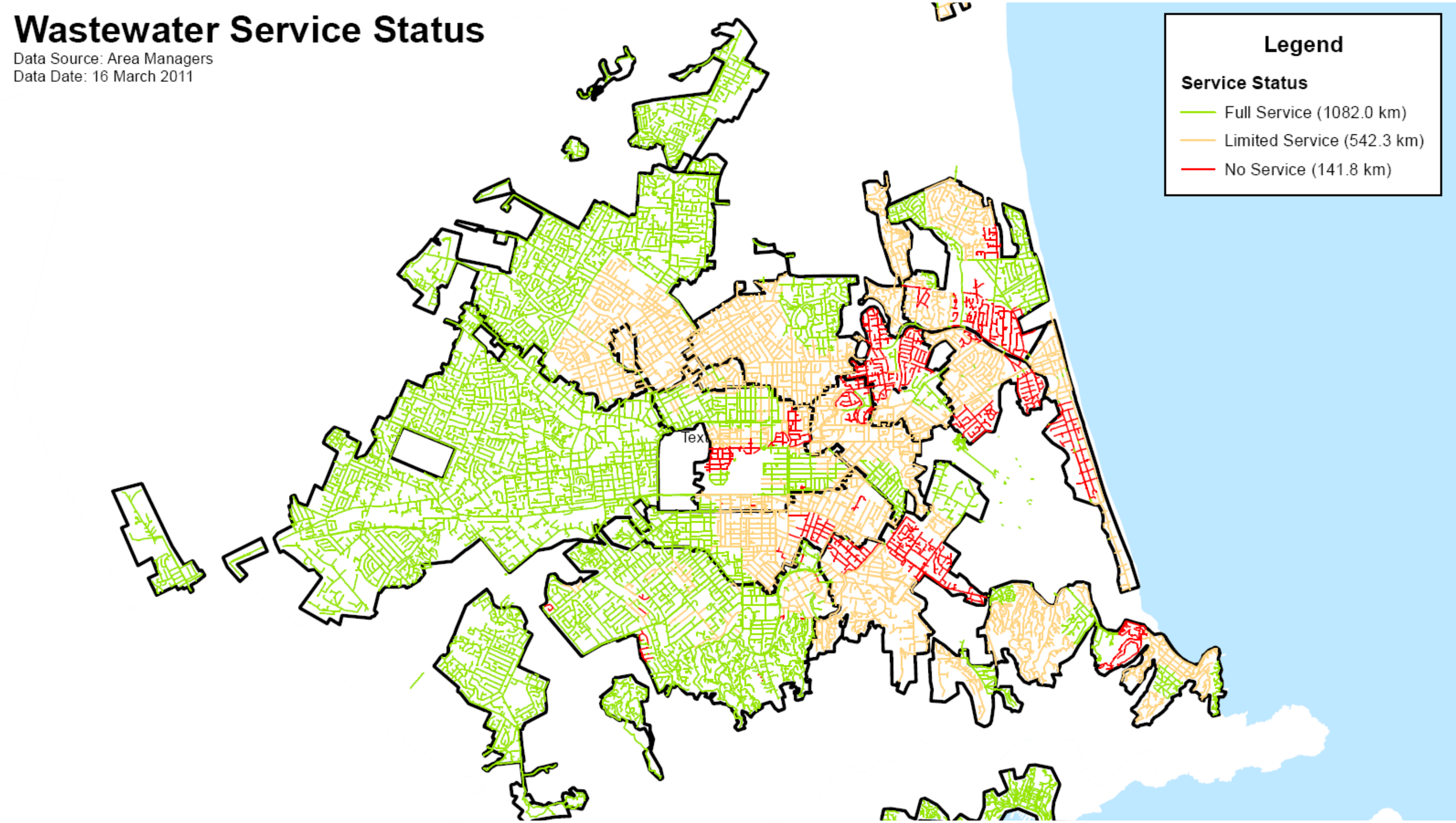


Figure 22. Wastewater network status as of 16 March 2011 illustrating distribution and severity of damage inflicted by the 22 February 2011 earthquake

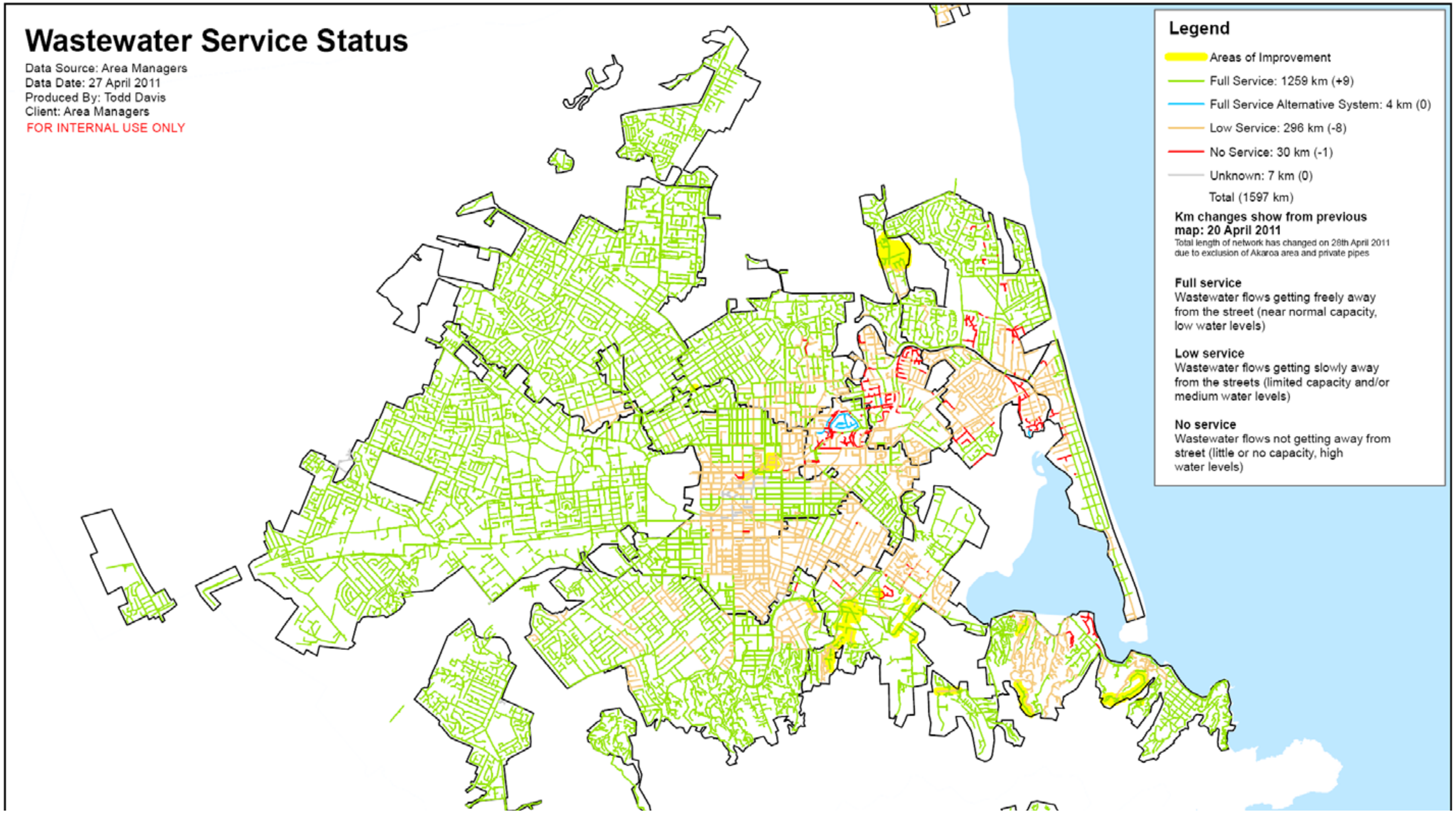


Figure 23. Wastewater network status as of 27 April 2011 indicating the speed of reinstatement of services in conjunction with the status map shown in Figure 22

6 CCC ISSUES AND CONSIDERATIONS

6.1 Material, Design and Construction Specifications

The series of earthquakes experienced since 4 September 2010 has highlighted deficiencies within the specification of design and construction of the piped reticulation network for wastewater, storm water and potable water within Christchurch City.

In order to inform and improve the design and construction of replacement infrastructure and to ensure future assets perform to acceptable levels of service, current Christchurch City Council material, design and construction specifications were reviewed.

The standard CCC details were reviewed with respect to concept, geofabric selection and grading of the backfill and foundation material. The standard details were accepted as suitable for ongoing use in liquefiable parts of the city with only minor changes.

Matters considered included a review of current best practice, an examination of alternative design and construction options, and possible opportunities to utilise these to increase the resilience of the built infrastructure. Overseas experience was also canvassed and the proposals from such experiences were examined in light of Christchurch conditions for their applicability.

6.2 Seismic Effects

The 2010-2011 Christchurch earthquakes affected the pipe networks in various ways and impacted their ability to provide adequate service to large areas of Christchurch. This was demonstrated in the following ways, amongst others:

- Gravity reticulation experienced reduced capacities.
- Potable water networks and pressure sewers experienced leakage and loss of pressure.
- Wastewater reticulation had increased flows.
- Pump stations were incapacitated.
- All networks experienced damage leading to blockage or leakage at the interface with structures.

A significant effect on the gravity pipe network has been the movement of pipe sections relative to each other, the surrounding ground and/or structures. This movement is evidenced by changes in grade, varying grades along a pipe length, or joint dislocation either within the pipe length, or at connections to structures. Effects included:

- The loss of grade, which has reduced capacity particularly in gravity lines.
- Inconsistencies in the invert level, which has encouraged deposition and reduced capacity.
- Partial or complete blockage of main lines or laterals.
- Joint damage or movement, allowing silt and groundwater infiltration or discharge of wastewater to groundwater.
- Depressions in carriageways caused by infiltration of subgrade materials into the gravity system (or removal of liquefied subsoils through groundwater flow towards the surface).

Pressure networks have also experienced

- Leakage and loss of pressure due to pipe and joint damage.
- Blockage and damage to pumps and treatment facilities from ground movement including settlement, uplift and lateral displacements; and from mobilised silt in the wastewater stream.

Some portions of Christchurch's gravity reticulation network are installed at depths exceeding 2.5m. Damage to these lines has meant that there have been:

- Large numbers of repairs at depth.
- Delays when repairing, and high reinstatement costs, due to the required dewatering and trench support.

6.3 Decisions

The alternatives for repairing the pipe networks considered were assessed for their ability to counter seismic effects and their anticipated performance over the lifecycle of the infrastructure. The ability of alternative repairs to reduce the recovery time after a subsequent seismic event and the relative costs and benefits under these scenarios were considered, along with the applicability of these alternatives to the various component criticalities or liquefaction damage potential zones. These zones are shown on the Liquefaction Resistance Index (LRI) map produced through this project and discussed elsewhere.

Where solutions are applicable to all zones within Christchurch City and to all infrastructure types, they have generally been published through amendments to Council Approved Material Specifications, the Infrastructure Design Standard (IDS, CCC, 2010a) or the Construction Standard Specifications (CSS, CCC, 2010b). These solutions are formatted as below and the location of this amendment is indicated to the right of the solution:

✓ Recommended solutions are ticked and are applicable in all of Christchurch City. Where solutions are not used, record this non-conformance in the design report. ✘ Crossed items have been considered and dismissed at this time for the reasons stated.	Location of solution in Council documentation

The tabulated solutions will be applied as best practice for the relevant Liquefaction Resistance Index zones as shown. These will only be adopted after consideration of site information, network requirements and the condition of the infrastructure requiring repair. Some of these solutions may be prohibitively expensive if applied city-wide. Criticality of the infrastructure is also a key consideration when assessing the more expensive options.

Suggested best practice solution, to be assessed against other options and considered in light of site investigations, network requirements and reticulation condition	Zones for which solution is acceptable, as defined on LRI map			
	orange	yellow	green	blue

6.4 Construction Alternatives

Construction must comply with the CSS. The designer designs the project to the requirements of the IDS, referencing individual standard details from the CSS where appropriate. Materials to be installed within Christchurch City must comply with the Council's requirements, set out in the Approved Materials list and webpage and must be approved, as detailed in CSS: Part 1 clause 4.0. The designer selects from the approved materials, considering the requirements of the IDS for the application.

1. Pipe haunching or surround and backfill details

The Council has, and continues to use, imported gravel backfill for pipelines under roads to reduce the potential for trench settlement. This backfill also increases resistance to liquefaction damage by forming a zone of non-liquefiable soil above the pipe, providing a zone of much more permeable ground to relieve excess pore pressures immediately under and around the pipe and higher strength material along the trench.

Alternative methods and materials for pipe haunching, surround and backfill investigated included: wrapping of the haunching material in geotextiles, providing reinforcing along the trench base through special foundations, different backfill or haunching materials and their compaction requirements, amending the specifications for geotextiles currently detailed, reinforcing the backfill immediately adjacent to manholes to prevent differential movement.

Recommended solutions included:

✓ Detail a "soft ground" or "raft" foundation wrapped in geotextile with strength class C, installed to TNZ F/2 to improve seismic strength	CSS: Part 3 SD 344
✓ Continue using M/4: AP40 or AP20 as pipe haunching or surround as they provide optimum seismic permeability and strength versus long term stability against fines migration.	No amendment
✓ Continue using M/4: AP40 or AP65 as trench backfill as they provide optimum seismic permeability and strength versus long term stability against fines migration.	No amendment
✓ Improve the pipe's resilience by tightening bedding or haunching compaction requirements	CSS: Part 3 clause 8.5.1 CSS: Part 4 clause 11.4.2

Best practice solutions included:

	orange	yellow	green	blue
Wrap pipe haunching or surround for flexible gravity pipes in geotextile, strength class C, installed to TNZ F/2 to improve seismic strength	Yes	Yes	No	No

Alternatives looked at and subsequently discarded for the reasons shown are:

✖ Provide geogrid reinforcement to backfill immediately adjacent to manholes to encourage the manhole and the pipe to move as one – It is felt that the mesh won't activate sufficiently early to prevent this movement.

Research is continuing into the performance of various haunching options through site investigations and into the relative benefits of completely encasing the trench backfill in a geotextile.

2. Polyethylene pipe construction

The currently preferred material for pressure networks that require improved seismic resistance is polyethylene. The larger volume of polyethylene being installed highlighted some deficiencies in the CSS relating to installation, welding and testing. Amendments subsequently made to these documents and the related processes included: improving quality records, welder competence and welding methods, clarifying weld testing and pressure testing.

Recommended solutions included:

✓ Provide improved quality records through the Contract Quality Plan, including methodologies and weld records, to support weld and welder competence	CSS: Part 3 clause 7.3, 7.4 CSS: Part 4 clause 10.2, 10.3
✓ Ensure polyethylene welders are competent, through requiring current industry qualifications and proof of experience relevant to pipe diameter being welded. Provide process for assessment of welder competence and consequences of weld failure	CSS: Part 3 clause 7.3, 7.4 CSS: Part 4 clause 10.2, 10.3
✓ Improve weld construction through amendments to construction specifications including equipment and processes	CSS: Part 3 clause 7.3, 7.4, 14.2.6 CSS: Part 4 clause 10.2, 10.3
✓ Provide commentaries and graphical plots of electrofusion peel decohesion test results to confirm weld competence and allow tracking of material or welder related performance issues	CSS: Part 3 clause 14.4 CSS: Part 4 clause 17.2
✓ Update polyethylene pressure test requirements to ensure testing is relevant to pipe size and use and results are clear	CSS: Part 3 clause 14.3 CSS: Part 4 clause 17.3
✓ Update ovality test requirements to current best practice	CSS: Part 3 clause 14.4.6

3. Manhole construction

The performance of manholes under seismic loading has not been consistent. Research is continuing into how manholes have performed through the events experienced in the various liquefaction areas, relative to the adjacent ground.

Alternatives looked at and subsequently discarded for the reasons shown are:

✖ Provide drainage into and around manholes to decrease seismically generated porewater pressures – evidence from the past events does not consistently illustrate that manholes

float, rather than the land settling relative to them in some situations. There are potential problems with this drainage facility diverting groundwater flows into the wastewater network. This may increase overall flow rates and compromise existing groundwater flow patterns.

✘ Provide additional weight to manholes to prevent flotation – evidence is not consistent that manholes always float.

4. Material Selection

Material alternatives and changes to current specifications investigated included: increasing the pipe stiffness, specifying ductile materials, improving the material specification for connections between pipes and for fittings.

Recommended solutions included:

✓ For gravity applications, use PVC-U pipe, SN16 for 100 and 150 diameter, SN8 for 225 and above, to improve the pipe's resistance to becoming oval or buckling under seismic loading	Website Approved Materials List V 11
✓ For wastewater pressure applications, use polyethylene pipe as it has experienced no known failures under seismic loading	IDS clause 6.8
✓ Increase the minimum PN for polyethylene pipe in wastewater pressure applications to PN10 to improve resistance to seismic loading	Website Approved Materials List V 11

Best practice solutions included:

	red	orange	green	blue
Use polyethylene pipe for potable water applications as the pipe body has experienced no known failures under seismic loading	Yes	Yes	No	No

Site research and testing is continuing into actual material and connection or fitting performance under seismic loading and into the performance of reinforced concrete pipes.

5. Joint details

Investigated joint alternatives included: wrapping pipe joints in geotextiles, installing PVC-U long socket connectors at manholes, increasing the socket length on PVC-U pipes.

Recommended solutions included:

✓ Wrap PVC-U gravity pipe joints, including on laterals, in geotextile with strength class C to prevent ingress of silt where joints open up under seismic loading	IDS clause 5.13.3 and 6.12 CSS: Part 3 clause 8.5.6
✓ Install long socket connectors to manholes on PVC-U gravity reticulation to increase the potential to accommodate longitudinal joint	IDS clause 5.10.6 and 6.6.3 CSS: Part 3 clause 8.10.5

movement ✓ Improve socket lengths and so joint movement capacity on PVC-U pipes by specifying minimum socket lengths and marking two witness marks (one as a reference mark) through CCC PVC-U material approval	CCC Webpage - Approved materials, Witness mark memo

Research is continuing into actual material performance, how their connections reacted under seismic loading, and the performance of polyethylene mechanical couplers under simulated seismic loading.

6.5 Design Alternatives

The process of design is iterative in that various options are considered, assessed against each other and the hazards or risks that the infrastructure is expected to experience, and finally the optimum solution presented for construction. The series of earthquakes have highlighted the need to consider a range of effects, many of which cannot be precisely defined, against which the options are evaluated. Because of this and the ongoing work to define the expected performance of the infrastructure, some of the solutions are current 'best practice' rather that recommendations.

Designs must comply with the stated requirements of the IDS. Where the design varies from these requirements, a non-conformance report is generated and presented to council as a record of the above decision making process.

6. Providing for future events

Historically there has been little geotechnical investigation for pipeline construction, other than some bores at manhole locations to the manhole depth.

Improved resilience can be provided through increasing the capability of the network to withstand seismic events by allowing for future settlement, by providing a system that will not be as affected by liquefaction or land movement e.g. pressure systems, by adding redundancy into the network, by using more robust materials and by designing to reduce the recovery time involved in repairs or replacement.

Best practice solutions include the following:

	orange	yellow	green	blue
Allow in designing gravity line grades for liquefaction settlement as determined by the LRI zone and associated settlements table	Yes	Yes	Yes	No
Carry out detailed geotechnical investigations of sites to determine the liquefaction potential and therefore likely settlement or lateral spread and subsequent movement that major or critical infrastructure must resist.	Yes	Yes	No	No
Apply the guidelines from NTC 33 clauses 32-37, detailing what the geotechnical investigation for pump station sites should address.	Yes	Yes	No	No

Consider alternative depths or wastewater reticulation systems instead of large scale gravity networks serviced by substantial lift pump stations	Yes	Yes	No	No

7. Differential movement risk areas

Network analysis suggests that the water reticulation experienced greater damage rates at the hill/plain interface by comparison to similar reticulation in other areas. This area may require special consideration to ensure there is sufficient ductility in the reticulation and the reticulation performance is still being analysed.

8. Lateral spread risk areas

Liquefaction encourages lateral spread in those areas where the land is sloping or is not confined e.g. adjacent to rivers and slopes.
Measures to counter damage to reticulation in these areas include:

- ✓ Design for ease of repair e.g. fittings between the structure and the pipe should be placed above-ground for easy access.
- ✓ Improve pipe flexibility through using polyethylene in lateral spread areas and through designing adequate compensatory flexibility into the connections to structures etc

9. Sewer depths and grades

There are a number of ways to reduce the depth of gravity sewers in selected areas and as a larger scale solution. These include: detailing collector sewers, laying gravity lines to the flattest functioning grade, restricting sewer depths and lateral connection depths.

Recommended solutions included:

✓ Install collector sewers over existing deep (over 2.5m) sewers, where depth permits. This is to prevent future repairs on laterals and junctions at depth	IDS clause 6.5.8 CSS: Part 3 clause 8.10.6
✓ Apply depth restriction of 3.5m to gravity sewers to prevent possibility of repairs at depth	IDS clause 6.5.6
✓ Apply depth restriction of 2.5m for the connection of laterals to gravity sewers to prevent possibility of repairs at depth	IDS clause 6.10.1

Research is continuing into the minimum grades at which sewers can be laid in Christchurch City. Allowances for liquefaction settlement can then be more clearly defined.

10. Material selection

Seismic events load pipes in all directions. Ductility within and between the pipe segments and robust connections between pipes and fittings or structures are fundamental to maintaining a functioning network after an earthquake event.
Material choices to counter this include:

✓ Avoid using brittle pipe materials wherever possible	No amendment
✓ Detail long socket connectors to manholes on PVC-U gravity reticulation to provide increased longitudinal joint movement	IDS clause 5.10.6 and 6.6.3

11. Foundation treatments

Liquefaction substantially reduces the strength of the pipe foundation materials.
Foundation treatments designed to counter this include:

✓ Use "soft ground" or "raft" foundation options for pipes laid in areas where foundation bearing pressures are less than 50kPa	CSS : Part 3 SD 344/3

Research is continuing into the performance of various haunching options through site investigations and into the relative benefits of completely encasing the trench backfill in a geotextile.

12. Redundant infrastructure

Methods of treating the large volume of damaged infrastructure potentially remaining in the ground require consideration due to the cost of this treatment as a component of the rebuild works.
Recommended solutions include the following:

✓ Removal is preferred because these pipes form voids which can undermine the foundations of pavements and adjacent services and can disrupt groundwater flows.	Not yet incorporated into Council documentation
✓ Treatment is dependent on the proximity to all services, the pipe's position in the road cross-section and the size of the pipe. If grouting, ensure it is continuous along the pipe length. Low strength concrete (3MPa) is preferred to prevent future issues where the pipe may require removal.	Not yet incorporated into Council documentation
✓ Obsolete AC pipes should preferably be left in the ground due to contamination problems.	Not yet incorporated into Council documentation

7 PERFORMANCE OBJECTIVES

Performance-based design of lifelines requires, as a starting point, to set the performance objectives of the system. Since such objectives were loosely defined under the "provision of service", a discussion was initiated within the Council to address the following questions: "what are according to the Council, acceptable levels of service for the water and wastewater systems, for major events such as these earthquakes?"; "what is the acceptable percentage of the population to be without service, and for what period of time?". The ultimate goal was for the CCC to establish specific performance objectives for the Water Supply and Wastewater Systems which will provide appropriate design objectives and performance that is balanced between effort and cost (capital and operational), and aims at realistic (achievable) and acceptable performance levels. The initial document stimulating this discussion is given in Appendix E and the tables below summarize the provisional performance objectives (currently under discussion) derived independently by the CCC asset management team based on their technical and operational scrutiny of the performance of the systems in the 2010-2011 earthquakes and community reaction.

Domestic Service - Disaster Recovery- Design Level of Service		
	Water Supply	**Waste Water**
48 (72)hours	90% of Premises	85% of Premises
48 (72) hours	95% Critical facilities	95% Critical Facilities
4 (7) days	95% of premises	n/a
7 (14) days	99.5% of premises	90% of Premises
1 (2) month	n/a	99.5% of Premises

Quality -Disaster Recovery -Design Levels of Service		
	Water Supply	**Waste Water**
2 weeks	n/a	80% of effluent reaches treatment plants
1 month	90% of city receives water conforming to NZDWS	90%
3 (6)months	n/a	99%
3 months	n/a	Treatment (or lack of) not causing significant adverse environmental impacts
6 months	n/a	Full Consent Compliance

Business Continuity -Disaster Recovery -Design Level of Service		
	Water Supply	**Waste Water**
1 month	95% of Industry/ commercial activity able to resume normal business	90% of Industry/ commercial activity able to resume normal business
3 months		95%
6 (12) months		99%

8 LIQUEFACTION RESISTANCE MAP

8.1 LRI Concept

In the simplified procedure for liquefaction evaluation (Youd et al., 2001) a factor of safety against triggering of liquefaction (in a free field level ground deposit), *FS*, is calculated as:

$$FS = \frac{CRR_{7.5}}{CSR} MSF \tag{1}$$

where $CRR_{7.5}$ is the Cyclic Resistance Ratio or liquefaction resistance while the seismic load (demand) is defined by the Cyclic Stress Ratio (*CSR*) and Magnitude Scaling Factor (*MSF*). Here *CSR* accounts for the amplitude of the seismic load (using the peak ground acceleration as a measure for the amplitude) while *MSF* accounts for the duration of shaking (or number of significant load cycles) using the earthquake magnitude as a proxy for the shaking duration. If $FS \leq 1.0$, then the available liquefaction resistance is smaller than (equal to) the seismic load (demand) and hence liquefaction will be triggered (will occur) for the considered ground motion (earthquake). The simplified method is used as a predictive tool to evaluate the liquefaction potential at a given site for an assumed ground motion (*PGA*, M_w) and estimated liquefaction resistance $CRR_{7.5}$ using empirical relationships based on penetration resistance or shear wave velocity.

Using this approach, the inverse problem could be solved to back-calculate the liquefaction resistance $CRR_{7.5}$ based on records of ground motions and observed liquefaction manifestation due to an earthquake. In the inverse problem, *CSR* and *MSF* are calculated using actual records of peak ground accelerations (*PGA*) and the earthquake magnitude (M_w) respectively, while *FS* is estimated from the observed severity of liquefaction manifestation, and eventually the liquefaction resistance is back-calculated as:

$$CRR_{7.5} = \frac{CSR \cdot \overline{FS}}{MSF} = CSR_{7.5} \cdot \overline{FS} \tag{2}$$

Here $CSR_{7.5}$ is a function of *PGA* and M_w whereas $\overline{FS}$ is a function of the severity of liquefaction manifestation. This approach was adopted to calculate a so-called Liquefaction Resistance Index (*LRI*) map and develop liquefaction zoning for Christchurch based on *LRI*, as described below.

There are a couple of advantages of this approach. First, it allows us to quantify actual earthquake observations and summarize them in the form of a liquefaction zoning (hazard) map. Second, using this approach we could quickly develop preliminary liquefaction zoning for the needs of CCC and their immediate decision making before a more robust zoning/analyses based on high-quality geotechnical data could be completed. A brief summary of the development of the Liquefaction Resistance Map is presented below whereas details are given in the Appendices C and D.

8.2 $CSR_{7.5(wt)}$ values from the Darfield and Christchurch earthquakes

As shown in the appendix, $CSR_{7.5}$ is a function of the peak ground acceleration, considered depth in the deposit and water table depth, i.e. $CSR_{7.5} = f[PGA, z, wt(z)]$. When the water table is at shallow depths, then the effects of z and *wt(z)* diminish and the cyclic stress ratio effectively reduces to a function of *PGA* alone, i.e. $CSR_{7.5(wt)} = f[PGA]$. Thus, using the geometric mean peak ground accelerations recorded at the strong motion stations within and in the vicinity of Christchurch during the Darfield and Christchurch earthquakes, $CSR_{7.5(wt)}$ were computed at the strong motion stations and then were interpolated across Christchurch using ordinary kriging interpolation, as shown in Figure 24.

As described in Section 4, for the Christchurch potable water system the pressurised pipe network is typically at shallow depths of about 0.8 m, while the wastewater pipes are predominantly at depths from 2.0-3.5 m. In addition, for most of the suburbs that experienced liquefaction in Christchurch, the water table was high, at about 1 m to 1.5 m from the ground surface. For these reasons, the liquefaction zoning for pipe networks was focused on the shallow depths of the deposits corresponding from the depth of the water table to 2 metres below the water table. However, it should be noted that the results would not change significantly (in relative terms) if slightly larger depths are considered (say 4 to 5 m below the water table).

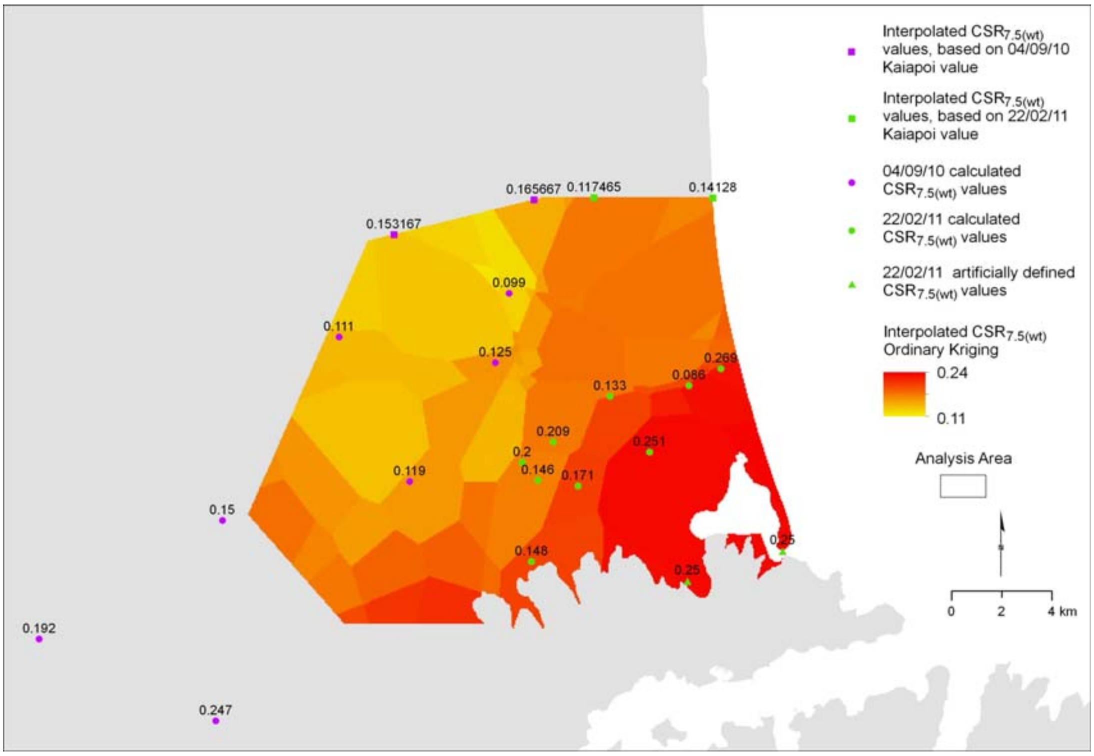


Figure 24. **$CSR_{7.5(wt)}$ values produced in Christchurch by the 4 September 2010 and 22 February 2011 earthquakes (ordinary krigging interpolation with a spherical variogram model; note that the CSR-values at the north boundary and the area at the HVSC station were constrained to eliminate spurious effects of either lack of data or extremely high accelerations due to localized effects**

8.3 Estimated FS values based on liquefaction observations from the Darfield and Christchurch earthquakes

A key issue in the calculation of LRI maps is the assumption/evaluation of the factor of safety $\overline{FS}$. We have to assume the factor of safety for both areas that did, and areas that did not liquefy during the earthquakes. Details about the reasoning behind the selection of particular $\overline{FS}$ values are given in Appendix C whereas here a brief summary of the results is presented.

For the liquefied areas, the factor of safety was defined based on the severity of manifested liquefaction in the field, as summarized in Table 4. Since triggering of liquefaction yields by definition FS = 1.0, traces of liquefaction, low to moderate liquefaction and moderate to severe liquefaction were given FS values of 0.9, 0.75 and 0.50 respectively. In other words, FS decreases with increased severity of liquefaction manifestation. An FS of 0.5 indicates that the available cyclic strength of the soil was half of the seismic load induced by the earthquake. For cases of extreme or very severe effects of liquefaction, an FS value of 0.25 was adopted.

In the non-liquefied areas, it was conservatively adopted that in areas where the water table was at 1m or 2m depth, that $\overline{FS}$ was slightly above the threshold triggering value or 1.1 and 1.25, respectively. Then $\overline{FS}$ was increased with the water table depth since it is well known that a thick crust decreases the likelihood of occurrence and surface manifestation of liquefaction. Thus, $\overline{FS}$ = 1.5, 1.75 and 2.0 was adopted for areas with depth to water table of 3.0, 4.0 and 5.0m. The adopted $\overline{FS}$ values across different severity of liquefaction and water table depths are summarized in Figure 25.

This approach was applied to establish an LRI map for Christchurch using the liquefaction map shown in Figure 6 and $CSR_{7.5(wt)}$ distribution (Figure 24) calculated based on the magnitude and recorded PGAs for the Darfield and Christchurch earthquakes. Figure 26 shows the $\overline{FS}$ values in different areas of Christchurch using liquefaction severity and depth of water table as quantifiers for the factor of safety. Figure 27 shows nominal $\overline{FS}$ values based on the water table depth in Christchurch. Note that in Figure 26 a large area shown in grey was not covered in the liquefaction inspections since there was no accurate information whether liquefaction occurred or not (or what was its severity of liquefaction) in this area. By multiplying the $\overline{FS}$ values shown in Figure 26 with the $CSR_{7.5(wt)}$ of Figure 24 the LRI value was calculated and summarized in Figure 28 in the form of a Liquefaction Resistance Index map of Christchurch. Here orange, yellow, green and blue indicate Zones 1, 2, 3 and 4, with Zone 1 being the reference zone. The red zone covers part of the abandoned areas and is below the established threshold LRI value of 0.065. Note that the zone numbers also indicate the relative liquefaction resistance. Thus, for example, Zone 3 has three times the liquefaction strength of the lower bound value of Zone 1.

To further facilitate the use of the LRI map in preliminary design evaluations, Table 5 summarizes the typical range of settlements associated with each zone. These are based on expert judgement and should be taken only as preliminary estimates with further updates to follow based on more robust interpretation and analysis.

For the grey zone we recommend to use a comparative analysis of the performance of the network for provisional classification. For example, if the wastewater system performance in a grey zone was similar to the performance of a green zone, then the

zone could be provisionally classified as 'green' or Zone 3. One should be rigorous when comparing different parts of the network (and apply this to network segments with similar design/material characteristics) and also reasonably conservative when adopting this approach of provisional zoning of the uninspected grey areas.

Table 4. Adopted correlation between $\overline{FS}$ and liquefaction severity

Average Factor of Safety, $\overline{FS}$	Liquefaction Severity	Typical Manifestation and Damage to Structures	Estimated Ground Settlement
0.90	Traces of liquefaction	Some evidence of liquefaction, but limited both in extent and impacts, and judged non-damaging for structures	< 50 mm
0.75	Low to moderate	Clear evidence of liquefaction, with scattered sand boils (sand ejecta) and ground distortion; low damage to residential buildings and buried pipe networks.	50 – 200 mm
0.50	Moderate to severe	Very large, continuous and thick sand ejecta, severe ground distortion (undulations, fissures) and substantial total and differential settlements; moderate to severe damage to residential buildings and buried pipe networks.	200 – 400 mm
0.25	Very severe (extreme)	Extreme manifestation of liquefaction with excessive ground distortion including very large total and differential settlements, vertical offsets and ground fissures, often accompanied with severe effects of lateral spreading; excessive (most often beyond repair) damage to residential buildings and buried pipe networks.	> 400 mm

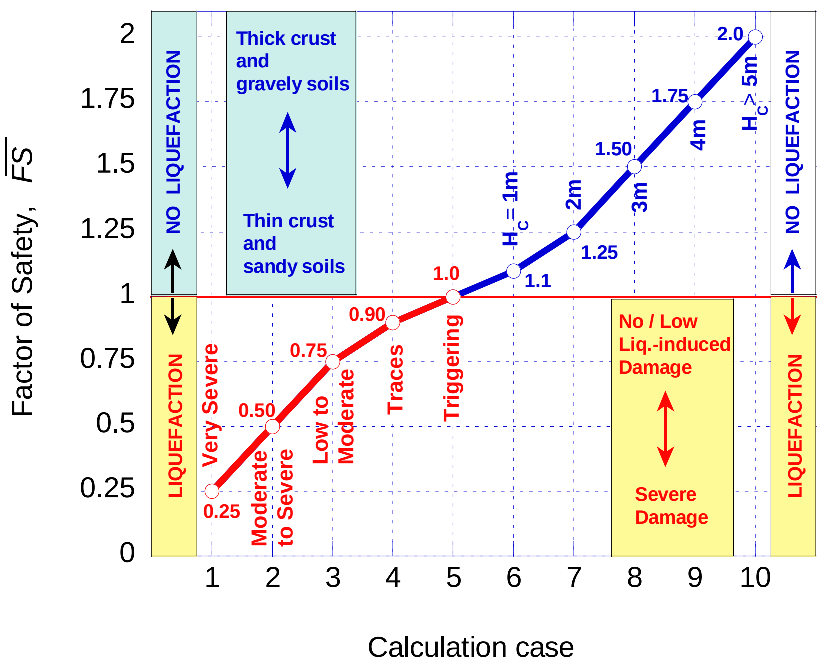


Figure 25. **Adopted factors of safety (against liquefaction) in areas of manifested liquefaction (*FS* < *1.0*) and areas of no liquefaction (*FS* > 1.0) with different water table depth**

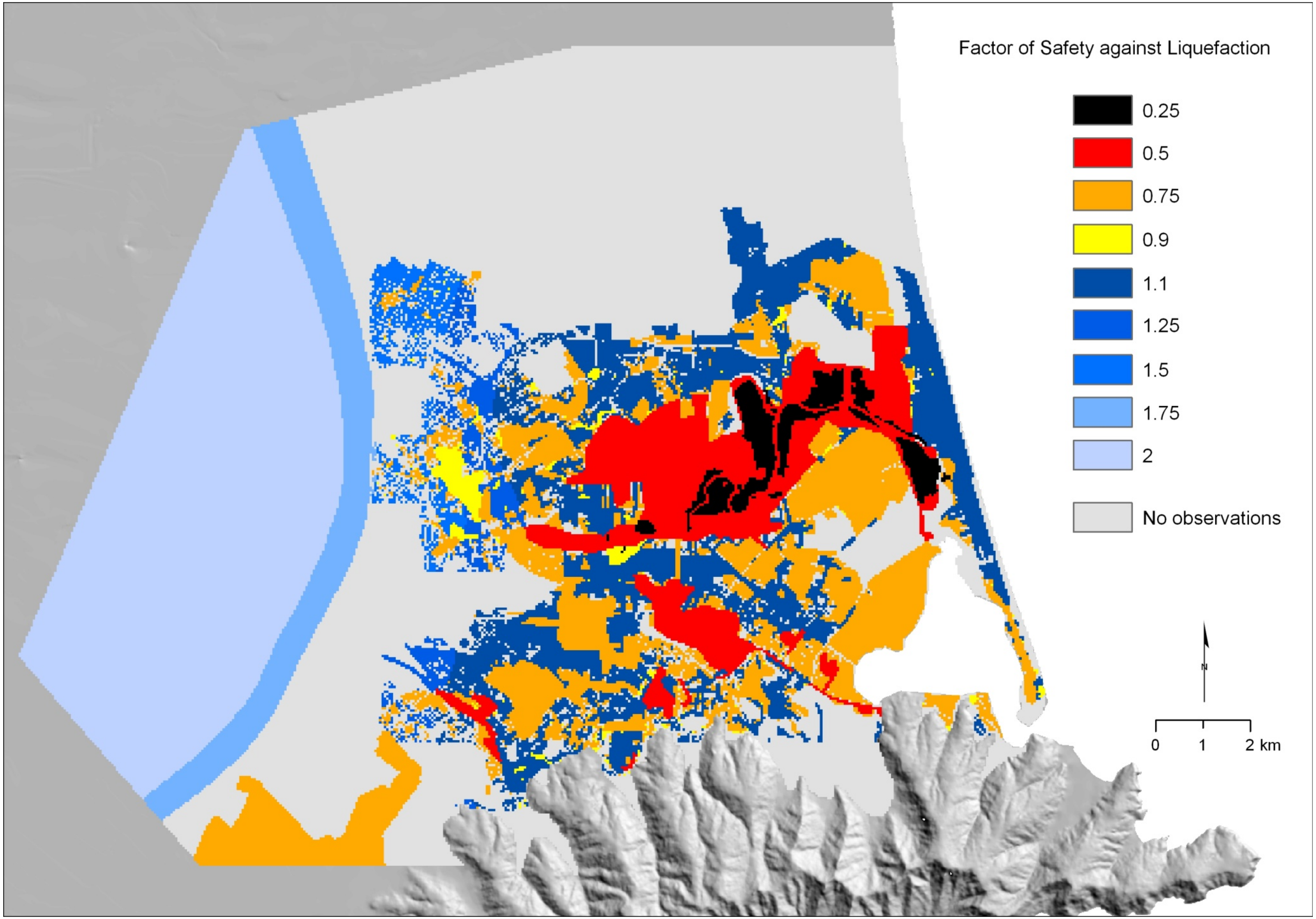


Figure 26. **Factors of safety (against liquefaction) map back-calculated based on liquefaction observations from the 2010-2011 earthquakes and water table depth information.**

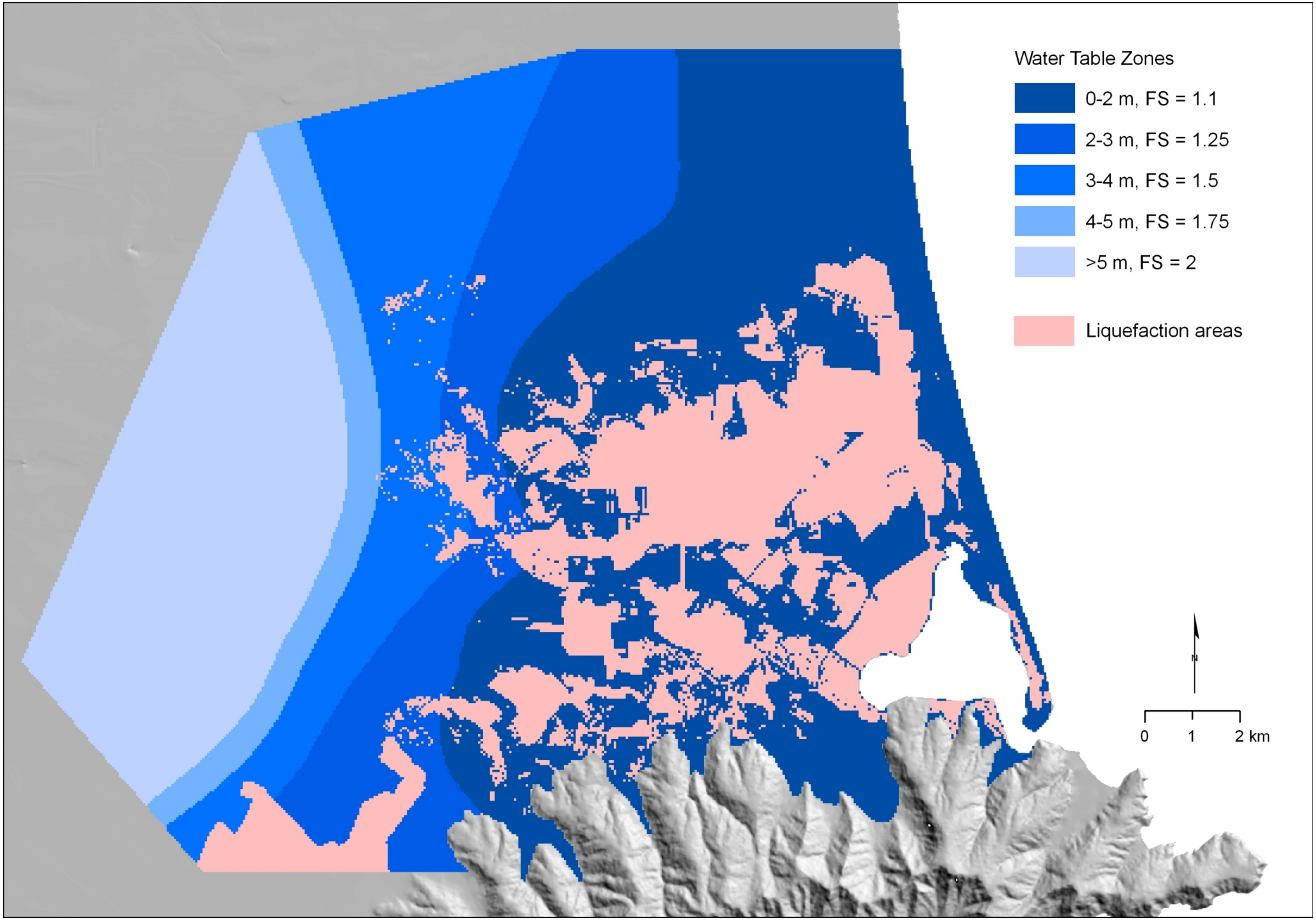


Figure 27. Water table depth contours modified from Brown and Weeber (1992), converted to a 50 m resolution raster with Factors of safety allocated to areas that did not liquefy during the 4 September 2010 and 22 February 2011 earthquakes

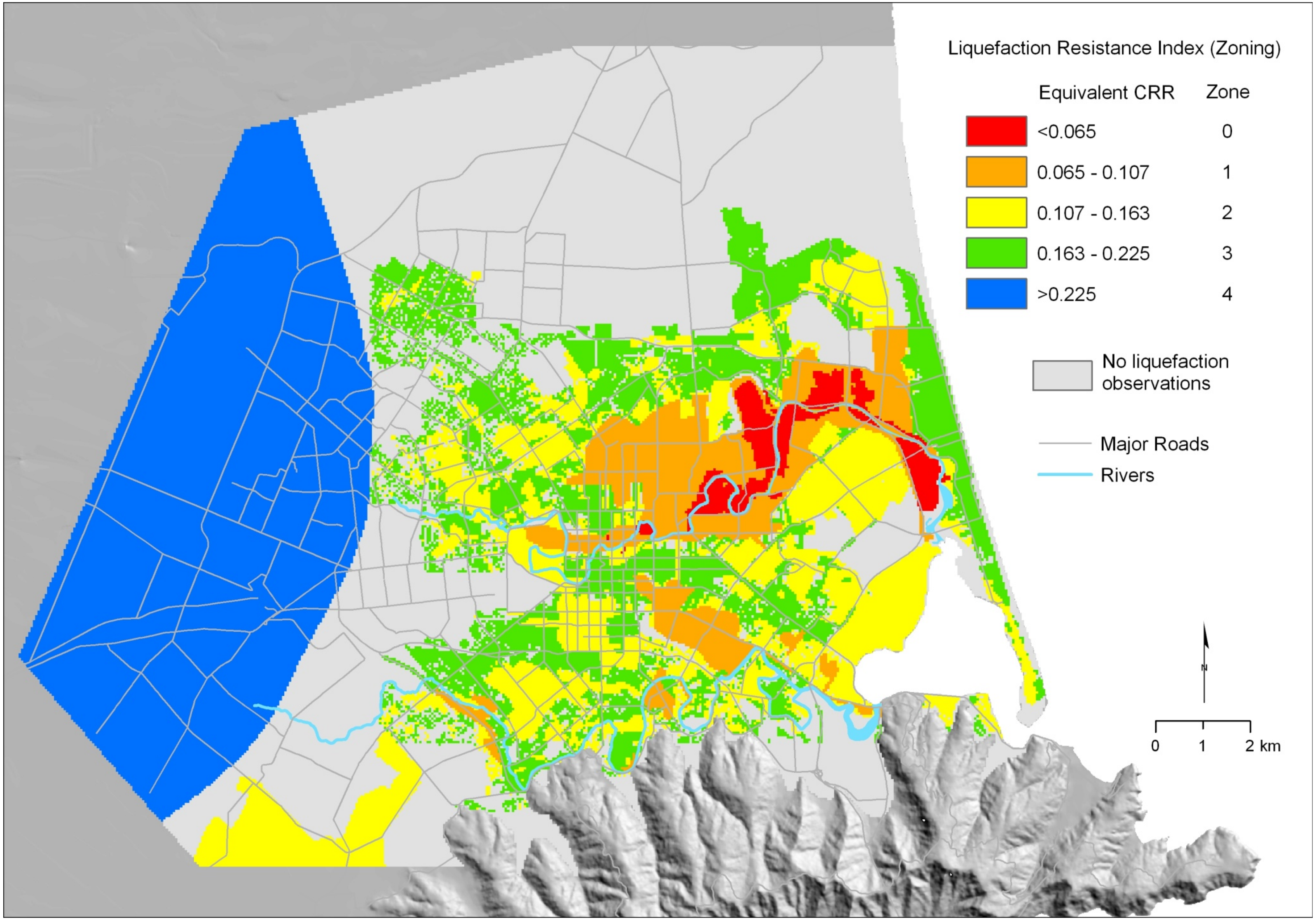


Figure 28. **Liquefaction Resistance Index (Zoning) of Christchurch at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information**

Table 5. LRI Zones and associated ground deformation (settlements, lateral displacements and strains)

Zone	Equivalent CRR (at water table)	Representative LRI (at water table)	Ground settlement (mm)	Lateral displacement (relative; transient) (mm)	Assumed ground strains and thickness of liquefied layer
0	< 0.065	-	> 500	> 400	$\varepsilon_v > 5\%$, $\gamma > 4\%$, H_L=5-10m
1	0.065 -0.11	0.065	250 – 500	200 - 400	$\varepsilon_v = 5\%$, $\gamma = 4\%$, H_L=5-10m
2	0.11 – 0.16	0.13	50 - 250	40 - 200	$\varepsilon_v = 3\%$, $\gamma = 2\%$, H_L=4-8m
3	0.16 – 0.23	0.195	20 - 50	20 - 40	$\varepsilon_v = 1\%$, $\gamma = 1\%$, H_L=2-4m
4	> 0.23	0.26	< 20	< 20	$\gamma < 0.5\%$, H_L=0m

- The ground displacement values exclude effects of lateral spreading
- Design should accommodate the higher value of displacement/deformation
- ε_v = volumetric strain, γ = shear strain
- The table and LRI map are for preliminary use and restricted to the water / wastewater systems of Christchurch

9 LITERATURE REVIEW

9.1 American Lifeline Alliance (ALA) Guidelines (Documents)

A set of guidelines for Water Systems and Wastewater Systems prepared by the American Lifeline Alliance (ALA) was reviewed during this project including advice and consultation with USA authorities and experts on lifelines. While further review and scrutiny of these guidelines is in progress, here we provide some excerpts from the ALA Wastewater Systems: Guideline (ALA, 2004).

ALA Wastewater Guideline Objective

The guideline provides minimum recommended requirements for evaluating wastewater systems to allow defensible answers to questions regarding system performance in natural hazards and human threat events.

Risk Based Assessment

The performance assessment approach presented in the guideline is, in general terms, developed to estimate the relative risk associated with each wastewater system component for each natural hazard or human treat.

The period on which to base the probability of exceedance for the hazard is often considered 50 years. Useful life of various components of lifeline systems:

- 50-year life: an average estimate of the useful life of various components
- 20-year life: mechanical components
- 50-year life: building components
- 100-year life: buried pipelines

Levels of Performance Assessment

Three levels of assessment are advocated:

(1) Simplified
(2) Intermediate
(3) Advanced

Simplified assessment is first carried out, to determine if Intermediate or Advanced Assessment is required. Several motivating factors for assessment have been identified including: *"direct loss – repair costs of facilities damaged in hazard events", public health and safety, level of service (outage time), societal cost/business interruption and loss of public confidence.* These are all relevant for the Christchurch wastewater system.

Performance Assessment Process

Step 1: Define the project objective and select the required level of assessment
Step 2: Select performance metrics
Step 3: Define the performance objectives
Step 4: Define the WS to be assessed
Step 5: Define relevant natural hazards and human threats
Step 6: Assess the vulnerability of system components
Step 7: Assess system performance under conditions of natural hazards
Step 8: Assess whether the performance objectives are met.

Overview of mitigation
The assessment is an iterative process in which the following are used as mitigation factors:

(1) Modification of the emergency response,
(2) Modification of component vulnerability, and
(3) Modification of target performance objective

Performance metrics
The originating purpose of providing sanitary sewers was to protect public health by transporting raw sewage away from population. The metric can be posed in terms of the success of achieving this objective.

Potential metrics recommended are:

1) public health/backup of raw sewage
2) discharge of raw/inadequately treated sewage (for some events that occur every 100 to 500 years, it is assumed that discharge of raw/inadequately treated sewage will occur; the intent of the metric is to quantify that discharge and the probability of its occurrence; provide different criteria as a function of the receiving water, e.g. stream, river, lake ocean).
3) direct damage/financial impact (Historically, property losses are an order of magnitude smaller than societal economic losses, and usually do not control; however, in some situations, direct damage to WS should be taken into account; significant earthquake damage is given as an example; potential secondary damage due to loss of wastewater service to commercial and industrial facilities).
4) security system performance

Performance objectives
Suggested starting points (performance objectives) are listed in the table below. Interestingly enough, the CCC assets management team arrived independently at similar performance objectives based on their analysis of the performance of the wastewater system in the 2010-2011 earthquakes including perceptions and reactions of the public and wastewater system managers.

Performance Objective Category	100-Year Return Event (40% in 50 years)	500-Year Return Event (10% in 50 years)
Public Health		
Backup of any raw sewage into buildings	Not acceptable (less than 1% probability of occurrence)	Not acceptable (less than 5% probability of occurrence)
Overflow of raw sewage into streets	Acceptable in localized areas; less than 24 hrs	Acceptable (treatment plant is inundated) less than 72 hrs
Environmental		
Discharge of raw sewage to stormwater system, ditch or stream	Acceptable in localized areas; less than 72 hrs	Acceptable less than 7 days
Discharge of raw sewage to lake or river	Acceptable in accordance with CSO/NPDES	Acceptable less than 30 days
Discharge of raw sewage to salt water	Acceptable in accordance with CSO/NPDES	Acceptable less than 90 days
Discharge of disinfected primary effluent	Acceptable less than 30 days	Acceptable less than 180 days
Discharge of disinfected secondary effluent (meet NPDES permit requirements)	Acceptable	Acceptable

Other suggestions/recommendations from the ALA guidelines

- Do not collect data for the sake of collecting data, but gather it with a specific need in mind.
- The performance assessment can focus on system operations or on infrastructure vulnerability / damage exposure (probable maximum losses).
- For single site facilities, hazard probabilities and associated intensities can be used directly (such as is done for building codes).
- However, for distributed lifeline systems, scenario events, (with a determined probability) must be used, to reflect the variation in hazard intensity across system in any given event.
- Dependent (earthquake shaking and liquefaction, lateral spreading) hazards. The probability of liquefaction in an earthquake is determined from the independent ground motion and the liquefaction susceptibility relationship. Hazard scenarios include both independent and dependent hazards.
- Advanced Hazard Assessment: An assessment of this level would probably never be warranted for a wastewater utility.
- Use caution when applying "water" experience database to wastewater systems due to inherent differences between the two systems.

  Gravity sewers differ from water pipelines as follows:
  - They are generally buried deeper
  - The pipe body/materials and joints are typically weaker as they are not designed for pressure
  - They are more buoyant because they are only partially filled with sewage
  - Sewer pipelines can generally withstand more damage and remain functional, relative to pressurized water pipelines.

- Pipeline damage relationships are usually developed in the form of failures per unit length.
- Failure of sewers can result in development of large sinkholes that result in damage to the utilities above.
- Failure to provide adequate treatment of wastewater before it is discharged will "contaminate" the receiving water.
- A Correlation Factor (dimensionless term) is added to the risk equation in the Simplified Assessment to take into account the number of system components a single hazard event will impact:

Relative Risk = Hazard x Vulnerability x Consequence x Correlation Factor

- Any assessment of WS performance only represents a snapshot at a particular point in time.

9.2 The 1995 Kobe (Hyogoken Nambu) Earthquake Experience

Background
At 5:46 AM on Tuesday January 17 1995, a powerful earthquake struck the Hanshin and Awaji area that includes Kobe city west of Osaka. Its epicentre was the northern end of Awajishima (or Awaji island) with magnitude M7.2 on the Richter scale (and moment magnitude, M_W=6.9). More than 5,500 people died and about 35,000 were injured. Some 320,000 survivors were left homeless and were forced to take shelter in school gymnasiums and other public facilities. The earthquake damaged many segments of the infrastructure, including water supply and wastewater facilities, mainly induced by the earthquake forces and ground displacements, notably due to widespread liquefaction of the ground.

Damage to water works was observed in over 68 municipal water utilities and 3 bulk water supply authorities, which cover 9 prefectures. The number of houses which suffered from water supply cutoff reached 1,200,000 immediately after the earthquake. Twenty water purification plants were heavily damaged in Kobe, Nishinomiya and Ashiya cities. Pumps and mechanical or electric installations placed in galleries were submerged and damaged. The most damaged parts of the water supply system were the pipelines where a great deal of leakage occurred in transmission lines and distribution networks. Leakage occurred at pipe breaks, joints or couplings, valves, air valves, fire hydrants and so on. Breakage such as pull-out or crushing at pipe joints was dominant, reaching nearly 50% of the total breakage in the distribution network. Forty three wastewater treatment plants out of 102 which were operating in Hyogo, Osaka and Kyoto prefectures were damaged. In eight of the plants, secondary treatment function was lost. The Higashinada Wastewater Treatment Plant in Kobe City was severely damaged due to liquefaction and lateral ground deformation. Sewer pipes were also damaged, with a total length of about 162 km or 1866 sites. Trunk sewers were only slightly damaged though some of them were broken resulting in temporary suspension of water transmission. Damage was found mainly at the connections of manholes and pipes, and the connections of lateral sewers and public inlets. Moreover, some damage in outlet bulkheads was also reported.

More detailed description of the degree of damage is provided in the *Appendix*.

Emergency Actions
In order to supply necessary drinking water to residents, many supply utilities and personnel were sent from all over Japan. An emergency water supply system was organized, with 1,027 tank trucks from 44 prefectures and several water tank boats were provided by the central government. Emergency restoration was executed on the water supply system through repair of the distribution network and installation of temporary water taps for damaged houses.

Long-term Restoration Methods
Following the reconnaissance works, the Japan Society of Civil Engineers (JSCE) put out a proposal for the earthquake resistant design of civil engineering structures. The proposal mentioned that trunk lines for lifeline systems (such as water, sewerage, electricity, gas, and telecommunications) must be designed to maintain functionality after a Level II earthquake (equivalent to a ULS event in NZ), taking into account the

topography, ground conditions, and the city layout in the vicinity. If this is difficult for economic reasons or because of ground conditions, continued functionality (or rapid restoration) after a disaster should be ensured by selecting the most appropriate route, adopting a multi-route system, using a block system, or implementing some alternative measures.

Repair of water supply pipes
The Kobe Municipal Waterworks Bureau adopted the following aseismic design guidelines for future earthquakes: (1) to localize earthquake damage as much as possible; (2) to easily repair damage; and (3) to provide measures to prevent secondary disasters following the earthquake.
Based on the experience from this earthquake, the use of earthquake-resistant pipe with excellent “earthquake-proofing” capability has been adopted (see *Appendix* for some examples). In addition to replacing old pipes, existing pipes were also made “earthquake-proof”. By considering the emergency water supply activity following the earthquake, the “earthquake-proof” pipes were laid out at spacing of 500 m along the route toward the designated disaster prevention centres.

In addition to two water supply tunnels that pass through Rokko Mountain, large-capacity transmission mains that pass through deep underground in urban areas were developed. The idea is that the seismic risk can be distributed by dividing the water supply route into the urban area and through Rokko Mountain. Moreover, because of their high earthquake resistance and large capacity for storing water, emergency water supply during and at the early stage of a disaster would not be a problem.

Repair of Sewage Facilities
Following the earthquake, "Sewer Ordinance Amendment” is now required for the earthquake resistant design of sewerage pipeline facilities. The Ministry of Land, Infrastructure and Transport (MLIT) notified all sewerage companies to implement the "Earthquake Emergency Sewer Improvement Project." In this project, for example, pipelines connecting the treatment plant to the refuge shelters and disaster prevention centres, as well as pipelines buried under emergency tracks and roads, should be earthquake-resistant within five years after implementation of the plan. The design concepts are described in the "Guide to Aseismic Emergency Sewer Improvement Plan (Draft)" published by MLIT, as well as in "Guidelines and Description for Aseismic Measures in Sewerage Facilities" and "Earthquake-resistant Sewer Manual" published by the Japan Sewage Works Association.

As a result of the damage observed during the Kobe earthquake, the Kobe City Construction Bureau recommended a set of design standards for sewer facilities. These standards were further refined, incorporating lessons learned from subsequent major earthquake events that occurred in other parts of the country. The current design standards, which were enforced in the city from 1 June 2011, are illustrated in the Kobe City website (http://www.city.kobe.lg.jp/life/town/waterworks/sewage/gesuidosekkeihyojunzu.html).

One of the problems seen during the Kobe earthquake was the buoyant rise of manholes, resulting in damage to sewer pipes. In order to prevent manholes from rising as a result of the liquefaction of foundation ground as well as damage to

connecting pipes, several methods have been developed, and some of these are shown in the *Appendix*. These methods are explained briefly below.

(1) Earth drain method: Artificial drain consisting of high permeability soil is placed around the manhole using a specialized machine.
(2) Anchor wing method: The manhole is anchored to the bottom unliquefied layer by a frame structure (called wing).
(3) LAM Method: The manhole is anchored to the bottom unliquefied layer by a single rod attached to the bottom of the structure.
(4) Safe Manhole Method: Tubes are installed within the manhole and near the joint to drain excess pore water pressure generated during liquefaction.
(5) Anti-float method: A heavy base plate is placed at the bottom of the manhole to prevent uplift.
(6) Aseismic method for existing manholes: A special cutting machine is used to cut the edge of the manhole and the existing pipe by cutting the manhole wall and flexible joint and elastic sealant are installed at the connection.
(7) Aseismic improvement method for existing pipe: Using a chainsaw-type cutting machine, the pipe joint is cut and a light fitting consisting of rubber and the steel-made is placed to make the joint flexible.
(8) Prevention of uplift using manhole flange: A convex-shape material is placed on the outer part of the manhole, and a weight is placed to increase resistance against uplift.
(9) Float-less method (non-excavation type): Excess pore water pressure generated by earthquake is drained out.
(10) Magma lock method: the impact of earthquake-induced displacement is decreased using a special flexible joint and magma lock.
(11) Hat ring method: A cylindrical ring block is placed on existing manhole to prevent uplift.
(12) Wide safety pipe method: Tubes installed inside the manhole dissipates the excess pore water pressure. Moreover, underground water is not taken by installing a reverse-action valve in the manhole pipe.
(13) “Mr. Aseismic” (Taishin-ippatsu kun) method: New pipes are installed to add seismic capacity to old structures with worn-out pipes and manholes.

9.3 The 2011 Great East Japan Earthquake Experience

In the 11 March 2011 earthquake (Great East Japan Earthquake; magnitude 9.0) extensive liquefaction was induced in suburbs of Tokyo (Urayasu) and residential areas immediately north-east of the capital (Itako and Kamisu). The impacts on residential areas and lifelines were in many ways similar to those in Christchurch (Towhata, 2011). In the further studies, it is intended to compile a brief summary and draw some conclusions based on these experiences as well.

10 CALCULATION OF LIQUEFACTION-INDUCED UPLIFT OF MANHOLES

10.1 Overview

The occurrence of liquefaction is often associated with significant ground deformations and settlements. However, where buoyant structures exist, the occurrence of liquefaction can lead to the uplift of such structures. Observations of unacceptably large uplifts to pipe networks and manholes have been observed in past earthquakes (e.g. Koseki et al (1997), and references therein) and hence require appropriate design consideration.

The primary reason for liquefaction-induced uplift of structures is the result of the increased uplift forces at the base of structures due to upward flow of excess pore-water pressures, as well as the loss of resistive friction forces along the embedded sides of structures due to the reduction in soil effective stress.

While uplift of structures has been well documented in past earthquakes, the complexity of the liquefaction phenomena and post-liquefaction deformations and pore-water flow mean simplified calculations for initial assessment of uplift hazard are still relatively imprecise. A potentially more precise estimation of uplift hazard can be obtained via effective stress analyses, but often such complex analyses are not viable for various reasons (personnel experience and time demands).

10.2 Factor of safety against uplift

The assessment of uplift hazard should be conducted within the 'impact on structures' step of a liquefaction hazard assessment. Specifically, before conducting an uplift hazard assessment one must first have assessed the potential for liquefaction to occur for the considered scenario.

The most common method for the simplified estimation of liquefaction induced uplift of structures is the factor of safety approach. The factor of safety against uplift can be represented as:

$$F_s = \frac{\text{resisting forces}}{\text{uplift forces}} = \frac{W + Q}{U_s + U_d + F}$$

where W is the weight of the structure (and possibly overlying soil); Q is the resisting side friction along the embedded surface of the structure (typically assumed to be zero if the surrounding soils are expected to liquefy); U_s is the (static) buoyancy force due to hydrostatic pressure; U_d is the (dynamic) uplift force due to excess pore pressures and F is the seepage force which can be calculated as per Figure 29:

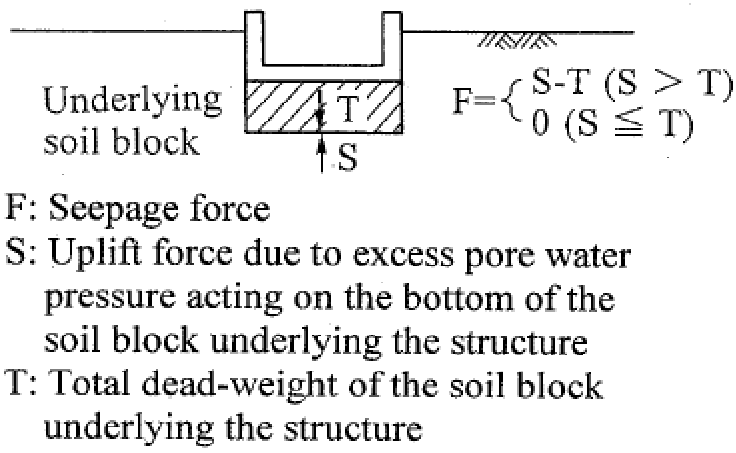


Figure 29: Calculation of seepage force, F (Koseki *et al.* 1997).

For the specific geometry of manholes in particular it is possible to semi-automate the calculation of the uplift factor of safety. The calculation requires several physical and geometrical parameters, as outlined in Figure 30.

The key geometrical parameters are:

- The manhole dimensions: diameter, B_0; and depth, h_0.
- The manhole weight, Mg
- The density of the soil above and below the WT, ρ_d and ρ_{sat}, respectively
- The depth of the WT, h_w, and depth from base of the manhole to non-liquefiable layer, h_{b0}.

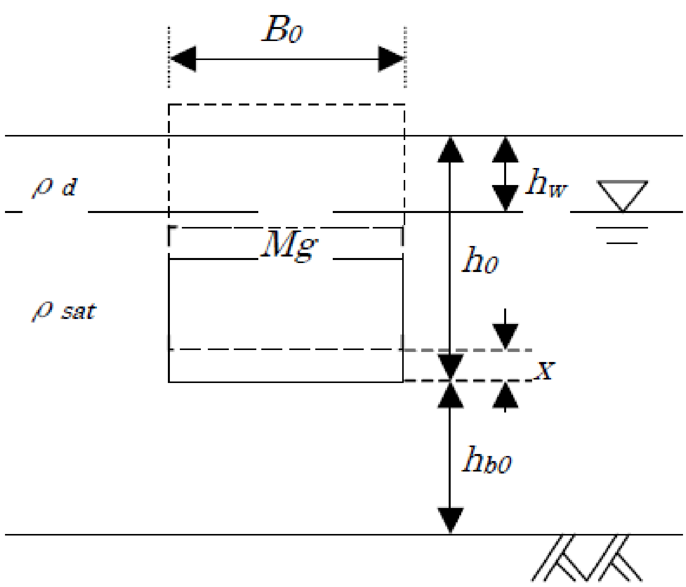


Figure 30: Notation used for manhole uplift calculations.

Note that no parameters for the uplift account for the soil strength etc. and hence as previously noted, the factor of safety against liquefaction is first required.

A spreadsheet was developed to carry out the discussed manhole uplift calculations. The spreadsheet provides the factor of safety against uplift (as well as liquefaction factor of safety for completeness), as well as determining the required manhole mass

in order to achieve neutral buoyancy (and hence for heavier masses uplift will not occur).

10.3 First-order estimation of uplift displacements

A limitation of the uplift factor of safety assessment is that it does not provide an assessment of the consequent uplift displacements which may occur. Unfortunately, at present, there is not a great body of knowledge on simplified methods by which to calculate such displacements. One such method is that of Sasaki and Tamura (2004), which is based on a simple differential equation with empirical parameters calibrated from several centrifuge tests. The method of Sasaki and Tamura is also implemented in the aforementioned spreadsheet. It is noted in particular, that the uplift displacements are dependent on the assessed duration of shaking. Based on judgement the duration of shaking should be the 5-95% significant duration parameter, which approximately represents the duration of the body wave and significant surface wave arrivals. Calculation of this significant duration as a function of earthquake magnitude and source-to-site distance can be calculated using empirical equations such as Bommer et al. (2009).

11 SUMMARY AND CONCLUSIONS

Seismic Demand

1) The ground motions produced by the 2010-2011 earthquakes within Christchurch and its surroundings were recorded by a dense array of strong motion instruments thus providing an excellent record and data for characterization of the ground motions and their distribution within the affected region.
2) The ground motions are characterized with high and damaging peak amplitudes, with horizontal (geometric mean) peak ground accelerations of 0.18-0.674 g and horizontal peak ground velocities of 27.6-72.8 cm/s.
3) When combining the peak amplitude with the number of significant cycles, moderate to high cyclic stress ratios ($CSR_{7.5(wt)}$) were obtained for depths two metres below the water table. These stress ratios increase with depth as indicated in the footnote of Table 1.
4) All intensity measures are consistent and indicate that the intensity of the ground motion in the CBD and eastern suburbs was on average 1.5 to 2.0 times that observed in the western suburbs during the 2011 Christchurch earthquake. Any comparison of the performance of the water or wastewater networks must account for this difference in the seismic demand (different inertial and kinematic loads imposed by the earthquake in different parts of the city).
5) The *PGA* is proxy for the inertial loads imposed on rigid structures above the ground, *PGV* is proxy for the kinematic loads on buried structures due to ground movement while the *CSR* is proxy for the intensity of the motion with respect to liquefaction triggering.
6) The observations and effects of the 2010-2011 earthquakes must be kept in the context of these very severe and damaging ground motions produced by these events.

Soil Liquefaction and Lateral Spreading

1) Widespread and severe liquefaction occurred in the suburbs of Christchurch and its CBD. Such extensiveness and severity of liquefaction in native soils is exceptional by international standards.
2) The repeated and very severe liquefaction particularly along the Avon River and in some other localized areas, clearly indicate that such soils have very low liquefaction resistance. There are several contributing factors to such low resistance:
   i) By composition and their plasticity, (non-plastic sands, silty sands, sandy silts, and silt-sand-gravel mixtures) the soils are highly susceptible to liquefaction.
   ii) Their in situ state (conditions) including full saturation (high water table), medium or loose to very loose density and fluvial deposits

fabric (granular structure of river deposits) all point out to a high liquefaction potential (or low liquefaction resistance).

iii) The soils are relatively young and apparently free of any serious aging effects, which again suggests a low liquefaction resistance.

iv) The groundwater regime of Christchurch is exceptional with significant groundwater flow through aquifers and many wells and natural springs in the area. The artesian pressure and upward water flow reduce the effective stress in the subsurface soils and reduce (eliminate) the possibility for soils to get stiffer and stronger due to aging effects. Liquefaction resistance is known to increase with the age of soils due to changes in their micro-structure and cementation (aging effect). We speculate that such aging effects on soils could not develop in the Christchurch groundwater environment.

v) Finally, we have to emphasize again that the severity of ground shaking together with the aforementioned factors played key role in the severity and extensiveness of the induced liquefaction.

3) Ground surveying measurements at approximately 80 locations along Avon River indicate maximum (relative) magnitudes of permanent lateral ground displacements due to spreading of liquefied soils on the order of 1.0 – 2.0 m. The spreading typically extended inland up to a distance of 100 m to 250 m from the waterway.

4) Different spreading patterns and distribution of lateral displacements with distance from the waterway were observed in North Kaiapoi, South Kaiapoi and along Avon River in Christchurch. In addition to the more conventional 'exponential decay' distribution where the spreading displacements rapidly decrease with the distance from the waterway, a block-mode failure was observed in South Kaiapoi with the largest and very damaging ground fissures opening at a distance of approximately 125-250 m from the waterway. The spreading along the meandering loops of Avon River showed very complex pattern and was affected by the interplay of soil conditions, topography, river geometry and local depositional environment.

5) The spreading induced very large and non-uniform ground deformation/displacements in the affected zone severely impacting infrastructure in the area. Road bridges suffered consistent spreading-induced damage and deformation mechanism including rotation of the abutments associated with deck pinning and damage at the top of the piles. Slumping of the approaches was also typical damage feature at locations of large lateral spreads. Loss of grade in gravity pipes, breakage of brittle pipes, failure of joints and connections were typical failures in potable water and wastewater pipe networks of Christchurch.

6) When evaluating lateral spreading one should carefully consider ground elevation (direction of sloping), river geometry (meandering, loops, cutbanks, point bar deposits), presence of weakened zones (old river channels, fills, etc.) and geotechnical conditions, then develop lateral spreading zoning and probable range of spreading displacements and their distribution, and assess the loads and deformation of the pipeline having in mind its particular layout relative to the direction of lateral spreading.

Performance of the Potable Water System - Watermains

1) For all pipe materials except PE pipes, there is a clear increase in the affected length (percentage of damage) with increasing liquefaction severity.
2) For steel (S), asbestos cement (AC) and other material pipes, the percentage of damaged pipes in areas of severe liquefaction was very high, between 15% and 22%.
3) PVC pipes suffered two to four times less damage than S, AC and other material pipes.
4) There is an indication that PE pipes performed well though the watermains sample is too small for any definitive conclusions. For the same reason, the "anomalous" result obtained (where the only damaged PE pipe is in 'no liquefaction' area) should be ignored until data and details of the failure/repair of the short pipe segment of PE pipe are available/clarified.
5) The level of pipe damage in no liquefaction and not inspected areas are similar indicating that ground displacements/performance were similar in these areas (with general absence of liquefaction manifestation). This fact together with the findings that the percentage of damage was linked to and increased with liquefaction severity provide an independent verification of the good quality and reliability of the generated liquefaction map shown in Figure 6.

Submains

6) GI pipes performed poorly with 17% damaged length in areas of low to moderate liquefaction and 26% damaged pipes in areas of severe liquefaction.
7) For PE submain pipes, the percentage of damaged length ranged between 1.4% (not inspected areas) and 5.2% (areas of severe liquefaction). Again, there was a clear increase in damage with liquefaction severity.
8) PE pipes suffered, on average, five to six times less damage than GI pipes.
9) For PVC pipes, the percentage of damaged length ranged between 2% and 3% (for non-liquefied and severe liquefaction areas respectively). However, the PVC pipes sample size was insufficient and hence the PVC submains results should be treated with caution.
10) Comparing the damage of watermains and submains, it appears that for each pipe material the damage to the submains was larger than the damage to the mains. It is important to understand what features/details contributed to this outcome. The total damaged length of submains was smaller however because over 80% of the submains were comprised of the well performing PE pipes.
11) Even though in the simplest form of the analysis the damage is always associated with certain pipe material, the nominally defined "failures" include (and probably are dominated at least for the PE pipes) by failures of particular components (joints, connections, fire hydrant details, crossovers, laterals) rather than pipe failures. *It is critically important therefore to discriminate between different types of failure and carry out a more rigorous second stage analysis, which will help us to identify key weaknesses and also "good design/construction details/characteristics" of the pre-earthquake potable water system.*

12) Having in mind the severity of ground shaking and failures caused by the earthquakes, as well as the reasonably quick restoration of potable water services throughout the city, one may argue that, by and large, the potable water system performed satisfactorily under the extreme seismic events.

Performance of the Wastewater System

1) The wastewater system of Christchurch was damaged severely by the series of strong earthquakes. This extensive damage is clearly related to a greater vulnerability of the wastewater pipe network to liquefaction and lateral spreading because of its larger depth of embedment, as compared to the potable water system.
2) Out of total pipe length of 1766 km shown in Figure 22, 542 km or 31% of the pipes had limited service and 142 km or 8 % had no service nearly one month after the 22 February earthquake. This clearly illustrates the severity of the impacts and damage level to the system.
3) By and large, the performance of the wastewater system was poor and not satisfactory (below desirable level/standard) despite the acknowledgement of the extreme severity of the earthquakes and liquefaction-induced ground failures.
4) Detailed information on the damage to the wastewater system was still not available because of the extensive damage and very difficult accessibility due to the large embedment depth. The following damage inspections and data collection efforts on the wastewater system are noted:
   - CCTV inspections have been completed for parts of the damaged network after the Darfield earthquake.
   - CCTV inspections have been also conducted for parts of the network after the 22 February earthquake.
   - Information on manholes including their movement relative to the adjacent road surface has been also compiled.

   It is critically important to systematically collect, interpret and analyse these data in order to find out what worked well and what didn't work in the existing wastewater system, and to identify key modes of failure/damage. In addition to the abovementioned data/information, the project team recommended and initiated:
   - Detailed case studies in selected areas in order to collect data and information on the performance of the network targeting specific sections/details of the system.
   - Development of systematic field inspection procedures/form for documenting relevant damage data during repairs over the period of reinstatement of the system.
5) Further studies and analyses of the wastewater network are required and strongly recommended (at all three levels of analyses and both in its simplified form and also by discriminating different failure types and damage contributions. While desk-top studies/analyses should accompany such efforts, they cannot provide on their own good quality information that will feed recovery decisions, more robust design solutions or better long-term resilience of the system.

CCC Issues and Considerations

1) The 2010-2011 Christchurch earthquakes affected the pipe networks in various ways and impacted their ability to provide adequate service to large areas of Christchurch. This was demonstrated in the following ways, amongst others:
   - Gravity reticulation experienced reduced capacities.
   - Potable water networks and pressure sewers experienced leakage and loss of pressure.
   - Wastewater reticulation had increased flows.
   - Pump stations were incapacitated.
   - All networks experienced damage leading to blockage or leakage at the interface with structures.

   A significant effect on the gravity pipe network has been the movement of pipe sections relative to each other, the surrounding ground and/or structures. This movement is evidenced by changes in grade, varying grades along a pipe length, or joint dislocation either within the pipe length, or at connections to structures. Effects included:
   - The loss of grade, which has reduced capacity particularly in gravity lines.
   - Inconsistencies in the invert level, which has encouraged deposition and reduced capacity.
   - Partial or complete blockage of main lines or laterals.
   - Joint damage or movement, allowing silt and groundwater infiltration or discharge of wastewater to groundwater.
   - Depressions in carriageways caused by infiltration of subgrade materials into the gravity system (or removal of liquefied subsoils through groundwater flow towards the surface).

   Pressure networks have also experienced
   - Leakage and loss of pressure due to pipe and joint damage.
   - Blockage and damage to pumps and treatment facilities from ground movement including settlement, uplift and lateral displacements; and from mobilised silt in the wastewater stream.

   Some portions of Christchurch's gravity reticulation network are installed at depths exceeding 2.5m. Damage to these lines has meant that there have been:
   - Large numbers of repairs at depth.
   - Delays when repairing, and high reinstatement costs, due to the required dewatering and trench support.

2) In order to inform and improve the design and construction of replacement infrastructure and to ensure future assets perform to acceptable levels of service, current Christchurch City Council material, design and construction specifications were reviewed. The standard CCC details were reviewed with respect to concept, geofabric selection and grading of the backfill and foundation material. Matters considered included a review of current best practice, an examination of alternative design and construction options, and possible opportunities to utilise these to increase the resilience of the built infrastructure. Overseas experience was also canvassed and the proposals from such experiences were examined in light of Christchurch conditions for their applicability. A set of construction alternatives were also considered/ developed including:

- Pipe haunching or surround and backfill details
- PE pipe construction
- Manhole construction
- Pipe material selection
- Joint details
- Sewer depths and grades
- Performance-based design concepts and objectives

Liquefaction Resistance Map

1) Liquefaction Resistance Index (LRI) map of Christchurch was developed providing liquefaction zoning for the purpose of design/reinstatement/recovery of the potable water and wastewater systems.
2) The map is based solely on observations from the 2010-2011 earthquakes and uses actual acceleration records to quantify the seismic demand (severity of ground shaking) and observed liquefaction manifestation (liquefaction maps) to quantify the severity of land damage.
3) The LRI map (Figure 28) shows the LRI at water table depth. The intent of the map is to show the liquefaction resistance in relative terms (between different areas of Christchurch), though absolute resistance could be also easily inferred.
4) Four zones are defined and quantified in the map: Zone 1 to Zone 4. They indicate relative liquefaction resistance, where the lower bound value of 0.065 of Zone 1 provides the reference resistance; for example, Zone 3 has a liquefaction resistance three times that of Zone 1.
5) Zone 0 (red area) identifies areas with resistance equal to or lower than the reference resistance.
6) There was no sufficient evidence for zoning of the grey areas. It us recommended to use the performance of the water and wastewater networks in these areas as gauge of the ground performance (liquefaction resistance), for provisional classification, since clear link between the performance of the potable water and severity of liquefaction has been established. One should be rigorous when comparing different parts of the network (and apply this to network segments with similar design/material characteristics) and also reasonably conservative when adopting this approach of provisional zoning of the uninspected grey areas.
7) Table 5 provides estimates of ground deformation (settlement, lateral displacement, strains) for each of the zones. These estimates are based on expert judgement and should be applied within the restrictions stated in the footnote.
8) The map should be considered as a provisional tool until more robust and better zoning map/information is made available. It does have however an inherent quality in that it provides actual evidence of ground performance during these earthquakes, while accounting for the different levels of ground shaking severity across Christchurch.
9) The map is based on general liquefaction map and does include significant variation even within a single zone; we assume that most of the estimates are on the conservative side, but this is not necessarily always the case.

12 ACKNOWLEDGEMENTS

This project was funded by the Natural Hazard Research Platform (NHRP) within the framework of the Short Term Recovery Projects in an effort to provide immediate technical support and expert advice to the Christchurch City Council in their efforts to reinstate and recover the potable water and wastewater systems following the extensive damage due to the 2010-2011 Canterbury earthquakes. The authors would like to particularly acknowledge the support of Dr. Kelvin Berryman, NHRP manager.

A large number of people form the Christchurch City Council and associated organizations contributed to this project. We would like to acknowledge in particular the contributions of John Walter, Senior Environmental Engineer, Aylwin Sim, Asset Management Analyst and Gordon Taylor, Contracts Engineer, from the Christchurch City Council.

The first author would like to express special thanks to his postgraduate students Jawad Arefi, Sophie Bainbridge, Simona Giorgini, Duncan Henderson, Kelvin Loh, Kun Ma, Masoud Moghaddasi, Kelly Robinson, Merrick Taylor, Anna Winkley for the great work in the reconnaissance, data compilation and analysis of the geotechnical impacts of the earthquakes.

Special thanks and acknowledgement of a great collaboration and overall support of our USA colleagues and in particular Thomas O’Rourke (Cornell University), Jonathan Bray (University of California, Berkeley) and Russell Green (Virginia Tech) in the reconnaissance work and analysis of the geotechnical aspects of the earthquakes. Prof. O’Rourke made particularly great contribution to the presented study on pipe networks and lifelines in general.

The University of Canterbury, the New Zealand Earthquake Commission (EQC) and the Environment Canterbury (ECan) provided significant funding support in these efforts.

Finally, the first author would like to acknowledge the JSPS (Japan Society for Promotion of Science) Fellowship and the support of the host academic Prof. Ikuo Towhata, University of Tokyo, during the preparation of this report, in Tokyo, November-December 2011.

13 REFERENCES

American Lifelines Association (ALA) (2004) Wastewater System Performance Assessment Guideline, Part 1: Guideline, June 2004.

American Lifelines Association (ALA) (2004) Wastewater System Performance Assessment Guideline, Part 2: Commentary, June 2004.

American Lifelines Association (ALA) (2005) Seismic guidelines for water pipelines: March 2005.

Archives New Zealand (2011). Black Map of Christchurch, March 1850, Sheet 3. Downloaded from **http://archives.govt.nz/gallery/v/Online+Regional+Exhibitions/Chregionalofficegallery/sss/Black+Map+of+Christchurch/CH1031-180+273+Black+Map+Christchurch+1850+Copy+2+2.JPG.html**. (Accessed 22 July 2011).

Aydan O, Hamada M 2011. Aerial photogrammetric interpretation of lateral spreading displacements due to 2010 Darfield earthquake. Personal communication.

Bartlett SF, Youd TL 1995. Empirical prediction of liquefaction-induced lateral spread. Journal of Geotechnical Engineering, 121(4): 316-329.

Bommer, J. J., Stafford, P. J., Alarcon, J. E., (2009). Empirical Equations for the Prediction of the Significant, Bracketed, and Uniform Duration of Earthquake Ground Motion*, Bulletin of the Seismological Society of America*, **99**, 3217-3233.

Bradley, B. A., Cubrinovski, M., Dhakal, R. P., MacRae, G. A., (2009). Intensity measures for the seismic response of pile foundations*, Soil Dynamics and Earthquake Engineering*, **29**, 1046-1058.

Brown L.J. and Weeber J.H. (1992). *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences, p.103.

Christchurch City Council, (2010a). Infrastructure Design Standard (IDS), Part 7: Water Supply, p. 46.

Christchurch City Council, CSS, (2010b). Construction Standard Specification (CSS), Part 4: Water Supply, p. 59.

Christchurch City Council, (2011). Personal communication.

Cubrinovski M, Uzuoka R, Sugita H, Tokimatsu K, Sato M, Ishihara K, Tsukamoto Y and Kamata T 2008. Prediction of pile response to lateral spreading by 3-D soil-water coupled dynamic analysis: shaking in the direction of ground flow. Soil Dynamics and Earthquake Engineering, 28(6): 421-435.

Cubrinovski M, Green RA eds. 2010. Geotechnical reconnaissance of the 2010 Darfield (Canterbury) earthquake. Bulletin of the New Zealand Society for Earthquake Engineering 43: 243–320.

Cubrinovski M, Hughes M 2011. Liquefaction map of Christchurch based on drive-through reconnaissance after the 13 June 2011 earthquake, University of Canterbury.

Cubrinovski M, McCahon I, Hughes M, McDonald Y, Simpson H, Cameron R, Christison M, Henderson B, Bradley B, Orense R 2011. Liquefaction impacts on pipe networks. Short Term Recovery Project No. 6, Natural Hazards Research Platform, University of Canterbury.

Cubrinovski M, Taylor M 2011. Liquefaction map of Christchurch based on drive-through reconnaissance after the 22 February 2011 earthquake, University of Canterbury.

Cubrinovski M, Robinson K, Taylor M, Hughes, M and Orense R (2012). Lateral spreading and its impacts in urban areas in the 2010-2011 Christchurch earthquakes. Special Issue on Canterbury Earthquakes, *New Zealand Journal of Geology and Geophysics (submitted).*

Fukuda, H. (2002). Damages in the Kobe earthquake & Japanese seismic design guidelines of drinking water supply facilities, *International Course on Microzonation and its Applications in Disaster Mitigation*, Lima Peru.

Hamada M, Yasuda S, Isoyama R, Emoto K. 1986. Study on liquefaction induced permanent ground displacements. Monograph, Association for the Development of Earthquake Prediction in Japan.

Haskell JJM, Madabhushi SPG, Cubrinovski M 2012. Boundary conditions in physical model tests – the influence of deck pinning on the response of piled bridge abutments in laterally spreading soils. GeoCongress 2012, Oakland 25-29 March 2012 (in print).

Ishihara K, Yoshida K, Kato M 1997. Characteristics of lateral spreading in liquefied deposits during the 1995 Hanshin-Awaji earthquake. Journal of Earthquake Engineering. 1 (1): 23-55.

Itoh M. (1995). "Hyogo Nanbu Earthquake - damage to waterworks and its restoration," *INTEP Newsletter*, No. 8, June issue (in Japanese).

Japan Society of Civil Engineers (1996). *Proposal on Earthquake Resistance for Civil Engineering Structures*, Special Task Committee of Earthquake Resistance of Civil Engineering Structures.

Kameda, H. (2000). "Engineering management of lifeline systems under earthquake risk," *Proc. 12th World Conference on Earthquake Engineering*, 17pp.

Koseki, J., Matsuo, O., Koga, Y., (1997). Uplift behaviour of underground structures caused by liquefaction of surrounding soil during earthquake, *Soils and Foundations*, **37**, 97-108.

Landcare Research (2011). Digital elevation model of Christchurch. http://lris.scinfo.org.nz/#/layer/187-christchurch-15m-dem-height-corrected/.

Miyane, K. and Kawahara, D. (2010). "Restoration of water in the Great Hanshin earthquake," *January 17th – Keep the Hanshin-Awaji Earthquake in Our Memories* (in Japanese)

Mizukuchi, K. (2008). "Aseismic measures for water network," *8th Annual General Meeting of the Institution of Professional Engineers*, Japan Kinki Chapter (in Japanese).

Newmark (1965). Effects of earthquakes on dams and embankments. Geotechnique, 15: 139-160.

New Zealand Government (2011). http://www.beehive.govt.nz/release/govt-outlines-next-steps-people-canterbury. Last accessed 18 July 2011.

O'Rourke T. and Milashuk S. (2011). Spatial distribution of ground motion during earthquakes affecting Christchurch, New Zealand.

Robinson K, Cubrinovski M, Kailey P, Orense R 2010. Field measurements of lateral spreading following the 2010 Darfield earthquake. Proc. 9th Pacific Conf. on Earthquake Engineering, 14-16 April, Auckland 2011, Paper No. 52: 1-8.

Sasaki, T., Tamura, K., (2004). *Prediction of Liquefaction-Induced Uplift Displacement of Underground Structures*, in *36th Joint Meeting US-Japan Panel on Wind and Seismic Effects*. 191-198.

Seed HB and Idriss IM (1971). Simplified Procedure for Evaluating Soil Liquefaction Potential, *J. Soil Mechanics and Foundations Div.*, ASCE, 97:SM9, 1249-1273.

Shioji, K. (1995). "Hyogo Nanbu Earthquake - damage to sewerage systems," *INTEP Newsletter* No. 8, June issue (in Japanese).

Tokimatsu, K, Asaka, Y 1998. Effects of liquefaction-induced ground displacements on pile performance in the 1995 Hyogoken-Nambu earthquake, Special Issue of Soils and Foundations, September 1998: 163-177.

Tonkin and Taylor 2011. Personal communication.

Towhata I (2011). Personal Communication.

Waimakariri District Council 2010a. Personal communication.

Waimakariri District Council 2010b. Lateral spreading displacements from geodetic re-surveying. Personal Communication.

Wotherspoon LM, Pender, MJ, Orense RP 2011. Relationship between observed liquefaction at Kaiapoi following the 2010 Darfield earthquake and former channels of the Waimakariri River. Engineering Geology (in print).

Youd, T. L., Member, C., ASCE, Idriss, I. M., Fellow, C.-C., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., Dobry, R., Finn, W. D. L., Jr., L. F. H., Hynes, M. E., Ishihara, K., Koester, J. P., Liao, S. S. C., III, W. F. M., Martin, G. R., Mitchell, J. K., Moriwaki, Y., Power, M. S., Robertson, P. K., Seed, R. B., II, K. H. S., (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils*, Journal of Geotechnical and Geoenvironmental Engineering*, **127**, 817-833.

Youd TL, Hansen KM, Bartlett SF 2002. Revised multilinear regression equations for prediction of lateral spread displacement. Journal of Geotechnical and Geoenvironmental Engineering, 128(12): 1007-1017.

Appendix A. Lateral Spreading and Its Impacts in the 2010-2011 earthquakes (paper submitted to NZJGG)

Lateral spreading and its impacts in urban areas in the 2010-2011 Christchurch earthquakes

M Cubrinovski[a*], K Robinson[a], M Taylor[a], M Hughes[a] and R Orense[b]

[a] *Department of Civil and Natural Resources Engineering, University of Canterbury, Christchurch, NZ*
[b] *Department of Civil and Environmental Engineering, University of Auckland, Auckland, NZ*
[*] *Corresponding author. Email: misko.cubrinovski@canterbury.ac.nz*

Abstract

In the 4 September 2010 (M_w=7.1) and 22 February 2011 (M_w=6.2) earthquakes, widespread liquefaction and lateral spreading occurred throughout Christchurch and the town of Kaiapoi. The severe soil liquefaction and lateral spreading in particular caused extensive and heavy damage to residential buildings, CBD buildings, bridges, and water supply and wastewater systems of Christchurch. After the earthquake, comprehensive field investigations and inspections were conducted to document the liquefaction-induced land damage, lateral spreading displacements and their impacts on buildings and infrastructure. Results of ground surveying measurements of lateral spreads at approximately 120 locations along Avon River, Kaiapoi River and streams in the affected area reveal permanent lateral ground displacements at the banks of up to 2-3 metres that progressed inland as far as 200-250 m from the waterway causing significant damage to structures located within the spreading zone. Different spreading patterns were identified, which were often affected by a complex interplay of ground conditions, topography, meandering river geometry and local depositional environment. The spreading was characterized by very large and highly non-uniform deformation causing stretching of building foundations and the buildings themselves. Road bridges suffered a characteristic spreading-induced damage mechanism including rotation of the abutments associated with deck pinning and damage at the top of the abutment piles. The wastewater system of Christchurch was hit particularly hard by the liquefaction and lateral spreading, and approximately 60% of the damaged pipes of the potable water system were located in areas affected by severe liquefaction and lateral spreading.

Keywords: 2010 Darfield earthquake, 2011 Christchurch earthquake, earthquake damage, ground displacements, lateral spreading, liquefaction

Introduction

In the period between September 2010 and June 2011, Christchurch and its surroundings were hit by a sequence of strong earthquakes including three significant events, all generated by local faults in proximity to the city: 4 September 2010 (M_w=7.1), 22 February 2011 (M_w=6.2), and 13 June 2011 (M_w=5.3 and M_w=6.0) earthquakes. The earthquakes caused tremendous physical damage and impacts on the people, natural and built environments of Christchurch. The 22 February 2011

earthquake was particularly devastating. This earthquake was caused by a local fault located practically within the city boundaries, beneath the Port Hills and the estuary in the south-east part of the city. Hence, the ground motions generated by the earthquake were intense and in many parts of Christchurch substantially above the ground motions used to design the buildings in Christchurch. The earthquake caused 182 fatalities, collapse of two multi-storey reinforced concrete buildings, collapse or partial collapse of many unreinforced masonry structures including the historic Christchurch Cathedral. The Central Business District (CBD) of Christchurch, which is the central heart of the city just east of Hagley Park, was practically lost with majority of its 3,000 buildings being damaged beyond repair. Widespread liquefaction in the suburbs of Christchurch, as well as rock falls and slope/cliff instabilities in the Port Hills affected tens of thousands of residential buildings and properties, and shattered the lifelines and infrastructure over approximately one third of the city area. The total economic loss caused by the 2010-2011 Christchurch earthquakes is estimated to be in the range between 25 and 30 billion NZ dollars.

Figure 1 indicates areas within Christchurch that liquefied during the 4 September 2010 earthquake (white contours/shaded area), 22 February 2011 earthquake (red = moderate to severe liquefaction; yellow = low to moderate liquefaction; magenta = moderate liquefaction predominantly on roads with some on properties; Cubrinovski and Taylor, 2011) and 13 June 2011 earthquakes (dark grey contours/shaded area; Cubrinovski and Hughes, 2011). The liquefaction was particularly extensive and damaging along the meandering loops of Avon River, from the CBD to the estuary, where multiple episodes of severe liquefaction occurred during the earthquakes. In areas close to waterways (rivers, streams), the liquefaction was often accompanied by lateral spreading, a particular form of ground movement/failure associated with liquefaction that produces excessive lateral ground displacements (from tens of centimetres to several metres), and hence, is very damaging for buildings and infrastructure. In addition to the areas along Avon River shown in Figure 1, substantial damage to residential buildings and lifelines due to liquefaction and lateral spreading also occurred in North Kaiapoi, South Kaiapoi, Brooklands and parts of Spencerville during the 4 September 2010 earthquake. Lateral spreading was consistently associated with the largest and most severe damage, and many of the nearly 6,000 residential properties that will be abandoned (New Zealand Government, 2011) were affected by spreading of liquefied soils.

This paper focuses on the characteristics of lateral spreading and its impacts on buildings, lifelines and infrastructure during the 2010-2011 earthquakes. Evidence on lateral spreading displacements including their magnitude, spatial distribution and modes of failure is first presented based on detailed ground surveying measurements undertaken at over 120 locations along rivers and streams in Christchurch and Kaiapoi. Typical damage patterns and impacts on residential buildings, bridges, buried pipe networks and CBD multi-storey buildings are then described with reference to the lateral spreading mechanism and kinematic loads imposed by ground displacements/movement. Finally, the key lateral spreading features and observations from the 2010-2011 Christchurch earthquakes are briefly discussed in the context of further development and improvement of predictive models for lateral spreading and quantification of its impacts on buildings and infrastructure.

Mechanism of lateral spreading

Lateral spreading typically occurs in sloping ground or level ground close to waterways/open face (e.g. river banks, streams, in the backfills behind quay walls). Even a very gentle slope in the ground (of just few degrees) will create a bias in the cyclic loads acting on the soil mass during earthquakes which will drive the soil to move in the down-slope direction. If the underlying soils liquefy then the liquefied soil mass ('heavy liquid') will naturally move down-slope and will continue this movement until equilibrium is re-established (or resisting forces reach the level of driving forces). The process of spreading in backfills behind retaining walls is similar, with large ground shaking first displacing the retaining structure outwards (e.g. towards the waterway), which is then followed by lateral spreading in the backfills.

The temporal evolution of lateral spreading is closely related to the development of excess pore water pressures and soil liquefaction in the spreading deposit. While spreading due to the biased seismic loads might be initiated at the early stages of strong shaking during the pore pressure build-up, the magnitude of lateral spreading displacements will increase substantially once the soils liquefy, because soil liquefaction will dramatically reduce the stiffness and strength of soils and will reduce their shear resistance to levels below the amplitude of the driving shear stresses. The spreading may continue even after the strong shaking has diminished and is influenced by a number of factors such as the available soil resistance (soil properties and in-situ state), driving stresses (topography, and ground motion characteristics), dissipation of excess pore water pressures (dynamic permeability and water flow conditions) and magnitude of lateral displacements (change of overall soil volume during spreading). The last factor may require some clarification. A large magnitude of lateral spreading displacements often implies lateral expansion of the spreading soils, as the ground moves laterally and expands in the outward direction towards the waterway. This lateral expansion is usually associated with slumping (or large settlement) of the spreading soils which compensates, at least partially, for the volume change due to the outward movement. The lateral expansion and resulting cracks and fissures in the surface layer, in turn, affect the development of excess pore water pressures and their dissipation.

Clearly a complex interplay of the topography, soil characteristics and conditions, temporal and spatial development of pore pressures and strains in the ground during cyclic loading, residual strength and stiffness of liquefied soils, conditions for dissipation of excess pore water pressures, mode of deformation and characteristics of ground motion (earthquake loads) is affecting lateral spreading. For this reason, its modelling, interpretation and analysis is neither simple nor straightforward. Three types of models are commonly used as predictive/analysis tools for lateral spreading: empirical models (e.g. Hamada et al. 1986; Bartlett and Youd, 1995; Youd et al. 2003), simplified physics-based models (e.g. Newmark, 1965) and advanced numerical analyses (e.g. Cubrinovski et al. 2008).

Evaluation of lateral spreading displacements by ground surveying

Following the 4 September 2010 earthquake and 22 February 2011 earthquake, field measurements of lateral spreading were conducted by means of the ground surveying method (Ishihara et al., 1997) at approximately 120 locations along Avon River,

Kaiapoi River and streams in the affected area. This surveying method is reasonably simple and allows us to estimate the magnitude of permanent lateral spreading displacements and their spatial distribution in relative terms, with respect to a reference point of measurement. At a selected location of lateral spreading measurement (transect), the measurement was conducted along a straight line perpendicular (as much as possible) to the river bank (waterway). Along the transect alignment, the width of cracks and fissures were systematically measured and their location (distance from the river bank) was recorded. Vertical offsets at cracks/fissures and the slope of the ground were also recorded along the transect alignment. Large continuous cracks were traced with GPS and then imported into a GIS framework to facilitate analysis of crack patterns and modes of movement. The measurements were carried out using simple tape measures, GPS units and rangefinders, and generally extended 200-300 metres along the transect alignment. Details of the applied methodology can be found in Robinson et al. (2010).

Characteristics of lateral spreading displacements

Figure 2 shows aerial photographs of Kaiapoi (A), Bexley (B), and Avonside and Dallington (C) where ground surveying measurements of lateral spreads were conducted after the 4 September earthquake, in the period September – November 2010. Red solid lines indicate the transect alignments while yellow solid lines indicate major cracks and fissures caused by spreading. As illustrated in this figure, areas along waterways (rivers, streams, lakes and wetlands) were targeted in the lateral spreading measurements.

Results of measured permanent lateral spreading displacements in South Kaiapoi (A) and North Kaiapoi (B) after the 2010 Darfield earthquake are shown in Figure 3. The locations of measurements (transect alignments) are those indicated in Figure 2A. In South Kaiapoi, the most dominant ground failure feature was the massive lateral spread along the Courtenay Stream and Courtenay Lake which caused substantial damage to the residential properties along the eastern branch of Courtenay Drive. The area affected by lateral spreading was approximately 1-km long in the north (northeast) - south (southwest) direction and extended up to 200-300 m inland from the Courtenay Stream/Lake. The lake was artificially created during the construction of the northern end of Courtenay Dr. Borrow material was removed from the area where the lake is presently located and used as hydraulic fill (about 1 m thick) for the northern branch of Courtenay Dr (Waimakariri District Council, 2010a). Swedish Weight Sounding (SWS) penetration tests and Spectral Analysis of Surface Waves (SASW) measurements of shear wave velocities consistently indicated a loose to very loose liquefiable layer from 1 m to 5-6 m depth in this area (Cubrinovski et al., 2010).

Each relationship between the lateral ground displacements (U_G) and distance from the waterway *(L)* shown in Figure 3 was obtained by summing up the width of the cracks measured along a particular transect alignment. Strictly speaking, each line indicates permanent horizontal displacements relative to a reference point, which is located approximately at the furthest lateral-spreading crack or the right-most point of the line where the displacement is practically zero. A long horizontal section of the U_G-L line indicates that there were no cracks/fissures on the ground surface within that range of distances, while a steep (nearly) vertical line indicates a large crack at the particular distance, the size of which is readily available from the difference in

displacements between the high and low points of the vertical-line segment. With this in mind, the following observations can be made from the results shown in Figure 3:

- The magnitude of spreading displacements was largest near the waterway (at the banks) and reached between 0.5 m and 3.0 m.
- The spreading extended inland up to a distance of 100 m to 250 m from the waterway.
- In North Kaiapoi, the spreading along Kaiapoi River (Pegasus Bay Walkway) followed more usual pattern of ‘exponential decay’ where the largest permanent ground movement is at/near the banks (waterway), and the magnitude of spreading displacements diminishes with the distance from the waterway. The observed spatial distribution of spreading displacements is similar to that observed in the 1995 Kobe earthquake (Ishihara et al., 1997) and recommended by the empirical model of Tokimatsu and Asaka (1998). The stopbank and infill in the park along Kaiapoi River, as well as the widespread inland liquefaction along Charles and Sewell streets influenced the lateral spread at North Kaiapoi (Cubrinovski et al., 2010).
- In South Kaiapoi, a very different pattern of ground movement was observed, indicative of a ‘block-mode’ failure. The soils from 0 m (at the banks) to approximately 125-200 m inland from the waterway moved as a unit (block) opening large cracks/fissures at these distances. In essence, all of the spreading cracks (deformation) that are usually distributed over 50-100 m distance, in this case were lumped together and concentrated within a short span of 10-20 m, and occurred in the form of large ground fissures at a distance of approximately 125 m to 250 m from the waterway. Unfortunately, these large cracks run right through the houses/properties along Courtney Drive causing tremendous damage in this neighbourhood. Wotherspoon et al. (2011) showed evidence from old maps of Kaiapoi that in the 1865 the Waimakariri River channel was located underneath the present day Courtenay Drive area where severe damage to residential properties occurred due to lateral spreading. The location of this old river channel appears to have contributed significantly both to the block-mode failure and substantial damage due to concentrated fissuring.

Table 1 summarizes measured permanent horizontal displacements at the banks (open face) or maximum displacements due to lateral spreading caused by the 2010 Darfield earthquake. Despite the fact that the measurements were taken at particular discrete locations, they are representative of the general spreading features in the area since over 80 locations distributed along Avon River, Kaiapoi River and streams were systematically investigated. The largest lateral spreading displacements of up to 3.0 - 3.5 m occurred in South and North Kaiapoi followed by Dallington, Avonside and Avondale areas along Avon River where the maximum spreading movements reached 1.8 m. There were relatively significant but localized lateral spreads also in Spencerville and Bexley. It is important to note that a wide range of maximum lateral spreading displacements was observed in any given area in spite of the ‘apparently similar’ conditions within the area.

After the 22 February 2011 earthquake, the ground surveying measurements were repeated at 25 locations to quantify the additional lateral ground movements due to spreading induced by the February event. Figure 4 comparatively shows the spreading

displacements induced by the two earthquakes at nine locations along Avon River. Here the horizontal axis indicates the displacement induced by the 4 September 2010 earthquake while the vertical axis indicates the cumulative displacements due to both earthquakes. The contribution of all aftershocks preceding the 13 June earthquakes to the cumulative displacements shown in the vertical axis is considered negligible. The results indicate that the lateral spreading displacements along Avon River induced by the 22 February 2011 earthquake where of similar magnitude with those induced by the 4 September 2010 earthquake.

Impacts on buildings and infrastructures

As described in the previous sections, lateral spreading involves large lateral ground movement at/near open faces (banks of waterways), but also substantial differential displacements in the direction of spreading resulting in large extensional deformation and ground fissuring. Figure 3 implies, for example, that the equivalent extensional strains in the zone of largest ground cracks were on the order of 5-10% while the 'average' extensional strains in the area affected by spreading were approximately 0.5-1.0%. The spreading was often accompanied by slumping of the soils near the banks (large settlement/subsidence), which was particularly noticeable at the approaches of bridges. Both lateral and vertical ground displacements induced by spreading where spatially non-uniform resulting in large localized deformation, stretching, tensile cracking and shearing of the ground. The non-uniformity of the ground deformation was further exacerbated by the spatial variability in the severity of liquefaction as well as the influence of soil-structure interaction on the movement, deformations and flow of water during dissipation of excess pore water pressures. Finally, as implied in the description of the mechanism of lateral spreading, spreading-induced loads involve combined inertial (due to accelerations) and kinematic (due to ground movement) effects, the characteristics of which depend on the evolution of the lateral spreading process in relation to the particular acceleration time history (ground motion at the site) and the site/soil response. Clearly, engineering structures that were located within the spreading zone were subjected to very large and highly non-uniform (both spatially and temporally) ground deformation and seismic forces which were more often than not above the available capacity to sustain such movements/loads, hence resulting in substantial and widespread damage to buildings and infrastructure.

Residential buildings

Figure 5(A) shows an aerial view of the lateral spread at North Kaiapoi caused by the 4 September 2010 earthquake, with large fissures in the ground near the stopbank and massive sand boils in the park area. The large volumes of silt/sand piled up in the cleanup of properties and streets from the silt/send ejecta indicate the high severity of liquefaction in the inland soils away from the stopbanks (along Charles and Sewell streets). A large crack on the ground surface can be seen affecting a house about 50 m from the Kaiapoi River, shown enlarged in Figure 5(B). Such major spreading-induced cracks were very damaging both for the foundations and building itself. This was most amply demonstrated in South Kaiapoi along Courtenay Drive where 0.5 - 1.5 m wide cracks opened up and run through the properties and buildings. As shown in Figure 6, in this area single storey and two storey residential buildings suffered

very severe damage including excessive tilt, loss of foundation support and severe structural damage (near collapse state in some cases). Despite the extreme lateral stretching of the foundations and the buildings, the houses showed large ductile deformation capacity and continued to carry gravity loads despite literally being ripped in half in some cases. This good feature in the performance of residential buildings was consistently found in timber-framed houses.

The two most commonly used foundations for the residential buildings in the area were the slab on grade and concrete perimeter footing. The slab on grade is typically unreinforced (except for the thickened perimeter) approximately 100 mm thick concrete slab for one storey houses, and it is reinforced by a relatively low capacity wire-mesh for two-storey houses. The concrete perimeter foundations range from unreinforced concrete filled with loose bricks (older foundations) to reinforced concrete foundations (newer foundations). Some typical spreading-induced damage to these foundations is illustrated in Figure 7 where large cracks in an unreinforced concrete slab, separation of poor quality unreinforced perimeter footing and rupture of the reinforced perimeter of a slab on grade are shown. These foundations could not resist the large lateral loads imposed by the spreading resulting in splitting/rupturing of the foundations and subsequent upward propagation of the rupture/cracks through the building and roof structure. In the case of more robust and stiffer foundations the effects of spreading were manifested predominantly through differential settlements and tilt of buildings, with less structural damage.

CBD buildings

The effects of lateral spreading within the CBD were localized within relatively narrow zones along Avon River. After the 22 February 2011 earthquake, ground surveying measurements were conducted at ten locations along Avon River within the CBD. The maximum spreading displacements were predominantly between 10 cm and 30 cm, except at three locations where the displacements reached 50-70 cm. The zone affected by spreading was relatively narrow and usually confined within a distance of 50 m from the river. Even in the cases when the spreading displacements were larger and extended up to 150 m from the banks, the permanent lateral displacements beyond the distance of 50 m were less than 10 cm. Structures and foundations within the spreading zone were greatly impacted by the horizontal ground strains causing stretching of the ground, foundations and then the building itself. Typical stretching of the foundations resulting in damage of the structure (opening of expansion joints) is shown in Figure 8.

Bridges, roads and pipe networks

Starting from the Colombo St Bridge, practically all bridges downstream Avon River were severely impacted by lateral spreading. Rotational movements of abutments, damage to foundation piles, subsidence of approaches to bridges and in some cases structural damage were the most typical spreading-induced damage to bridges. The images of the Bridge on the Bridge St shown in Figure 9 will be used to illustrate the typical deformation mechanism. As described earlier, the spreading induced large lateral displacements of the banks towards the river. This movement was resisted by the stiff and strong upper structure of the bridge (girders and deck) causing a pinning effect and rotation of the abutments, as illustrated in Figure 9(B), because the large

lateral movement of the foundation soils could not be resisted/prevented by the foundation piles. The rotation of the abutments was quite large (over 5 degrees) and imposed substantial additional stresses at the top of the piles causing damage, as indicated in Figure 9(C). Haskell et al. (2012) reproduced this deformation mechanism due to spreading and deck-pinning effects in seismic centrifuge tests providing additional details on the abutment rotation and damage to the piles. In cases when the lateral displacements were very large, the spreading was accompanied with slumping of the approaches, which produced large vertical offsets between the road surfaces of the approaches and the bridge itself.

The roads and lifelines within the spreading zone (especially at distances within 50 m from the waterway) were subjected to extreme effects of spreading and ground failures, as illustrated in Figure 10(A). Preliminary GIS analyses of the performance of the potable water system of Christchurch show clear link between the damage to the network and occurrence/severity of liquefaction (Cubrinovski et al., 2011). As shown with the pie chart in Figure 10(B), approximately 80% of the water mains breaks (repairs) occurred in areas affected by liquefaction, and nearly 60% of the damage was in areas of moderate to severe liquefaction where lateral spreading was the key contributing factor to the damage. The impacts of lateral spreading were even more pronounced on the wastewater system because its network of pipes is laid at larger depths (2.0-3.5 m), and hence is more susceptible to damage and also more difficult to access during repair/reinstatement works. Loss of grade, breakage of brittle pipes, failure of joints and connections (laterals) were typical failure types of the potable water and wastewater pipe networks.

Discussion

Estimates of permanent ground displacements due to lateral spreading at South Kaiapoi induced by the 4 September 2010 earthquake have been also made using detailed optical geodetic surveying (Waimakariri District Council, 2010b). These results were used to rigorously scrutinize the accuracy of the ground surveying measurements employed in this study. It was found that over the entire range of examined spreading displacements (from 0.05 m to 2.5 m), the displacements estimated from the ground surveying measurements were very similar and within 20% difference of those determined by the more accurate geodetic measurements. Hence, the ground surveying method provides accurate estimate of lateral spreading displacements, relative to a reference point usually at a distance of 200-300 m from the waterway.

In addition to the local ground and geodetic surveying measurements, estimates of the global lateral spreading movements have been carried out by aerial surveying using LiDAR (Tonkin and Taylor, 2011) and high-resolution aerial photographs (Aydan and Hamada, 2011). These aerial surveying methods are particularly useful in identifying global movement and more complex patterns of lateral spreads. For example, it was illustrated with the data presented herein that the lateral spreads at South and North Kaiapoi were very different and influenced by local factors (old river channel at South Kaiapoi, reasonably thick random fill with stopbank and severe liquefaction in inland soils at North Kaiapoi). Along Avon River, the meandering loops (present and past) and topography were affecting the spreading in a very complex way. This is apparent in the pre-earthquake ground elevation map shown in Figure 11 (Landcare Research 2011) where elevation above sea level is indicated. Clearly, the ground is sloping

towards the Avon River, though the slopes are very gentle and generally less than 1.5 degrees (2.6 %), and hence the bias in the cyclic load due to the sloping ground is relatively small.

A wide range and variation of maximum spreading displacements was measured even within a given area illustrating the complex influence and interplay of various factors affecting lateral spreading. The spreading was often very pronounced and of large magnitude in point bar deposits, but much smaller at the cut-banks on the opposite side of the river. Old river channels, streams and gullies, and artificial infilling of wetland areas during the European settlement have also contributed to the variability and complexity in the manifestation of lateral spreading. Field penetration tests such as CPT and SPT conducted after the 4 September 2010 earthquake (Tonkin and Taylor 2011) as well as SWS tests conducted by the University of Canterbury consistently show loose to very loose liquefiable sands and silty sands typically 4 m to 8 m thick dominating the areas along Avon River affected by spreading. Preliminary analyses clearly indicate a high potential for severe liquefaction and lateral spreading in these deposits.

It was illustrated through the presented case studies that liquefaction-induced spreading imposes very large and non-uniform loads throughout the footprint/foundations of a building, along the segment of a pipe network or on the piled foundations of the abutments and central piers of a bridge. Hence, in the evaluation of the seismic performance of these structures, it is critically important to understand and quantify the spatial distribution of ground deformation and associated kinematic loads. The surveying method and results presented herein contribute particularly to this aspect of lateral spreading and its impacts on structures, whereas the aforementioned aerial surveying techniques will provide better understanding and interpretation of the global patterns of lateral spreads induced in the areas affected by the 2010-2011 earthquakes. Further detailed studies using all these methods are currently under way.

Conclusions

The characteristics of lateral spreading and its impacts in urban areas of Christchurch in the 2010-2011 earthquakes could be summarized with the following conclusions.

1. Ground surveying measurements at approximately 120 locations along Avon River and Kaiapoi River indicate maximum (relative) magnitudes of permanent lateral ground displacements due to spreading of liquefied soils on the order of 2.0 – 3.5 m. The spreading typically extended inland up to a distance of 100 m to 250 m from the waterway.

2. Different spreading patterns and distribution of lateral displacements with distance from the waterway were observed in North Kaiapoi, South Kaiapoi and along Avon River in Christchurch. In addition to the more conventional ‘exponential decay’ distribution where the spreading displacements rapidly decrease with the distance from the waterway, a block-mode failure was observed in South Kaiapoi with the largest and very damaging ground fissures opening at a distance of approximately 125-250 m from the waterway. The spreading along the meandering loops of Avon River showed very complex pattern and was affected by the interplay of soil conditions, topography, river geometry and local depositional environment.

3. The spreading induced very large and non-uniform deformation causing stretching of the ground, foundations and then the building itself. The conventional slab on grade and concrete perimeter foundations of residential buildings could not resist such loads/deformation and failed in tension/shear. Road bridges suffered consistent spreading-induced damage and deformation mechanism including rotation of the abutments associated with deck pinning and damage at the top of the piles. Slumping of the approaches was also typical damage feature at locations of large lateral spreads. Loss of grade in gravity pipes, breakage of brittle pipes, failure of joints and connections were typical failures in potable water and wastewater pipe networks of Christchurch, with approximately 60% of the damaged water mains pipes being located in areas of severe liquefaction and lateral spreading.

Acknowledgements

The ground surveying work for this study was conducted in two phases. The first phase (September-October 2010) was conducted by the University of Canterbury and was financially supported by the New Zealand Earthquake Commission (EQC). The second phase of the field work (November 2010-February 2011) was a joint summer research program of the University of Canterbury (UC) and the University of Auckland (UA), which was partially funded by the Environment Canterbury (ECan). The authors would like to acknowledge the contributions of Yasuyo Hosono (UC), Liam Wotherspoon (UA), Mick Pender (UA) and UC postgraduate students Catherine Tatarniuk, Kim Rait, Masoud Moghaddasi, and particularly Patrick Kailey, for his able support in the early days of the reconnaissance work, and the summer research students Mohammed Al-Kubaisy (UA), Pak To Tang (UA), Maggie Yan (UA), Michelle Wang (UA), Kiran Rai (UA), Duncan Henderson (UC), Hayden Nikolaison (UC), John Wong (UC), Kun Ma (UC), Anna Winkley (UC), and Rachel Bennett (UC). The first author would like to acknowledge the JSPS (Japan Society for Promotion of Science) Fellowship and the support of the host academic Prof. Ikuo Towhata, University of Tokyo, during the preparation of the manuscript.

References

Aydan O, Hamada M 2011. Aerial photogrammetric interpretation of lateral spreading displacements due to 2010 Darfield earthquake. Personal communication.

Bartlett SF, Youd TL 1995. Empirical prediction of liquefaction-induced lateral spread. Journal of Geotechnical Engineering, 121(4): 316-329.

Cubrinovski M, Uzuoka R, Sugita H, Tokimatsu K, Sato M, Ishihara K, Tsukamoto Y and Kamata T 2008. Prediction of pile response to lateral spreading by 3-D soil-water coupled dynamic analysis: shaking in the direction of ground flow. Soil Dynamics and Earthquake Engineering, 28(6): 421-435.

Cubrinovski M, Green RA eds. 2010. Geotechnical reconnaissance of the 2010 Darfield (Canterbury) earthquake. Bulletin of the New Zealand Society for Earthquake Engineering 43: 243–320.

Cubrinovski M, Hughes M 2011. Liquefaction map of Christchurch based on drive-through reconnaissance after the 13 June 2011 earthquake, University of Canterbury.

Cubrinovski M, McCahon I, Hughes M, McDonald Y, Simpson H, Cameron R, Christison M, Henderson B, Bradley B, Orense R 2011. Liquefaction impacts on pipe networks. Short Term Recovery Project No. 6, Natural Hazards Research Platform, University of Canterbury.

Cubrinovski M, Taylor M 2011. Liquefaction map of Christchurch based on drive-through reconnaissance after the 22 February 2011 earthquake, University of Canterbury.

Landcare Research (2011). Digital elevation model of Christchurch. http://lris.scinfo.org.nz/#/layer/187-christchurch-15m-dem-height-corrected/.

Hamada M, Yasuda S, Isoyama R, Emoto K. 1986. Study on liquefaction induced permanent ground displacements. Monograph, Association for the Development of Earthquake Prediction in Japan.

Haskell JJM, Madabhushi SPG, Cubrinovski M 2012. Boundary conditions in physical model tests – the influence of deck pinning on the response of piled bridge abutments in laterally spreading soils. GeoCongress 2012, Oakland 25-29 March 2012 (in print).

Ishihara K, Yoshida K, Kato M 1997. Characteristics of lateral spreading in liquefied deposits during the 1995 Hanshin-Awaji earthquake. Journal of Earthquake Engineering. 1 (1): 23-55.

Newmark (1965). Effects of earthquakes on dams and embankments. Geotechnique, 15: 139-160.

New Zealand Government (2011). http://www.beehive.govt.nz/release/govt-outlines-next-steps-people-canterbury. Last accessed 18 July 2011.

Robinson K, Cubrinovski M, Kailey P, Orense R 2010. Field measurements of lateral spreading following the 2010 Darfield earthquake. Proc. 9th Pacific Conf. on Earthquake Engineering, 14-16 April, Auckland 2011, Paper No. 52: 1-8.

Tokimatsu, K, Asaka, Y 1998. Effects of liquefaction-induced ground displacements on pile performance in the 1995 Hyogoken-Nambu earthquake, Special Issue of Soils and Foundations, September 1998: 163-177.

Tonkin and Taylor 2011. Personal communication.

Waimakariri District Council 2010a. Personal communication.

Waimakariri District Council 2010b. Lateral spreading displacements from geodetic re-surveying. Personal Communication.

Wotherspoon LM, Pender, MJ, Orense RP 2011. Relationship between observed liquefaction at Kaiapoi following the 2010 Darfield earthquake and former channels of the Waimakariri River. Engineering Geology (in print).

Youd TL, Hansen KM, Bartlett SF 2002. Revised multilinear regression equations for prediction of lateral spread displacement. Journal of Geotechnical and Geoenvironmental Engineering, 128(12): 1007-1017.

TABLES

Table 1 Range of measured permanent lateral ground displacements induced by lateral spreading at/near the waterway (at the banks) after the 2010 Darfield earthquake (except for the CBD where the measurements were conducted after the 22 February 2011 earthquake)

Location	Permanent lateral ground displacement, U_G (m)
South Kaiapoi	0.5 - 3.5
North Kaiapoi	0.2 – 3.1
Spencerville	0.6 - 1.5
Bexley	0.3 - 0.9
Burwood	0.1 - 0.9
Dallington, Avonside, Avondale	0.5 - 1.8
Central Business District (CBD)	0.04 – 0.7 *)

*) Displacements measured after the 22 February 2011 earthquake

FIGURES

Figure 1 Preliminary liquefaction maps documenting areas of observed liquefaction in the 4 September 2010 (white contours), 22 February 2011 (red, yellow, magenta areas), and 13 June 2011 (black contours) earthquakes; note that only parts of Christchurch were surveyed and that the aim of the surveys was to capture only general features and severity of liquefaction manifestation as observed from the roads, hence, the zoning is not applicable to specific properties. Normalized cyclic stress ratios at water table depth, $CSR_{7.5(wt)}$, which were calculated using the recorded geometric mean peak ground accelerations and respective earthquake moment magnitude are also shown. For the Strong Motion Stations (SMS) denoted with green symbols, the 22 February 2011 produced the highest $CSR_{7.5(wt)}$ value whereas the 4 September 2010 earthquake produced the highest $CSR_{7.5(wt)}$ value at the SMS depicted with blue symbols. The corresponding maximum cyclic stress ratio at Kaiapoi was $CSR_{7.5(wt)}$ = 0.19 (produced by the 4 September 2010 earthquake)

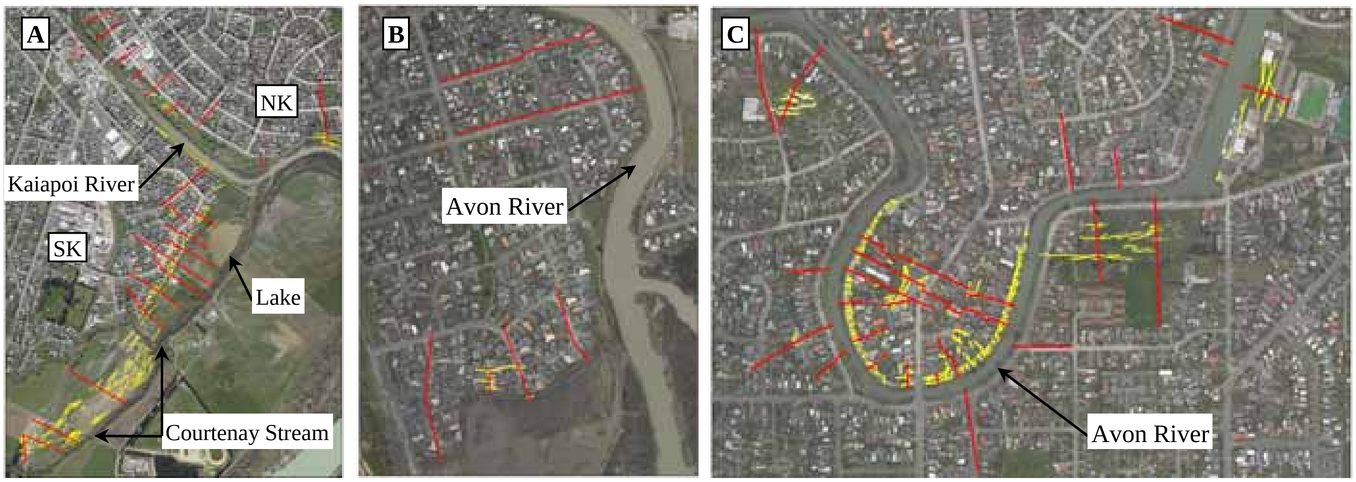


Figure 2 Aerial photographs indicating transect locations (red lines) and major ground cracks/fissures (yellow lines) in areas of ground surveying measurements of lateral spreading in **A,** Kaiapoi; **B,** Bexley; **C,** Avonside and Dallington.

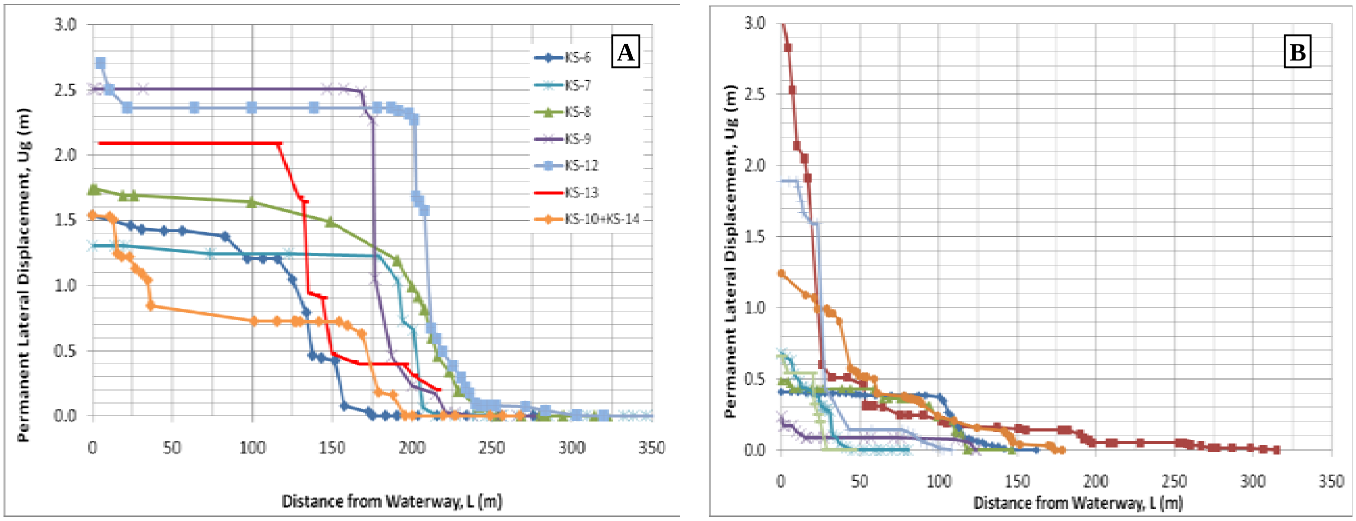


Figure 3 Permanent lateral ground displacements plotted against the distance from the waterway, measured in lateral spreading ground surveying in **A,** South Kaiapoi; and **B,** North Kaiapoi.

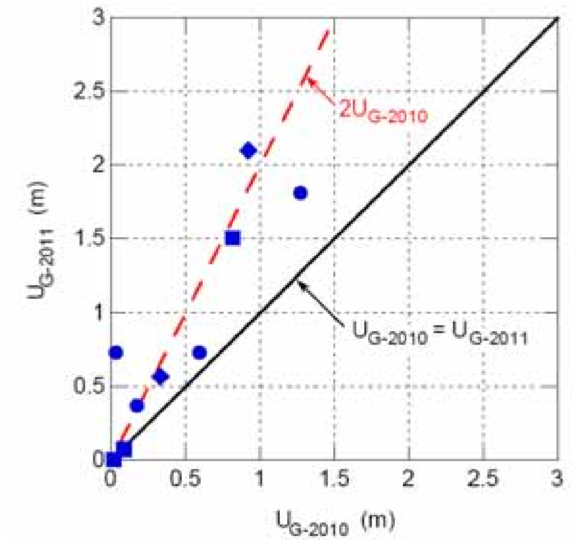


Figure 4 Comparison of permanent lateral ground displacements along Avon River (Avonside, Dallington, Bexley) caused by the 4 September 2010 earthquake (horizontal axis) and cumulative displacements due to the 4 September 2010 and 22 February 2011 earthquakes (vertical axis)

Figure 5 A, Aerial view of North Kaiapoi after the 4 September 2010 earthquake showing impacts of liquefaction and lateral spreading **B,** Lateral spreading cracks affecting residential properties approximately 50 m away from the Kaiapoi River

Figure 6 Spreading-induced damage to residential buildings in South Kaiapoi during the 4 September 2010 earthquake

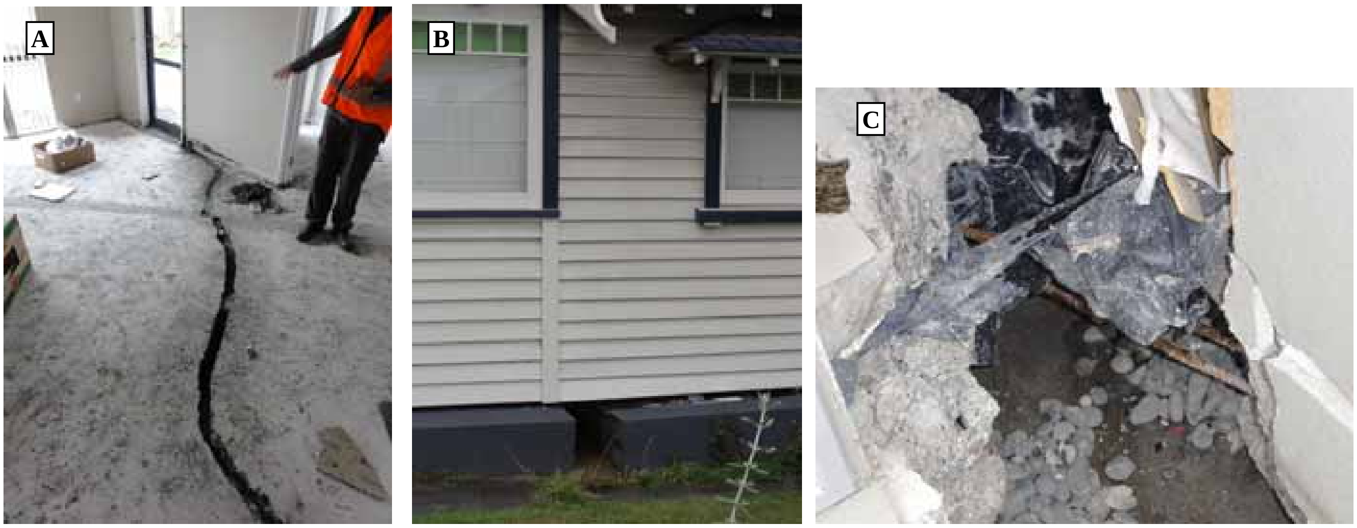


Figure 7 Damage to foundations of residential buildings due to liquefaction and lateral spreading: **A,** Unreinforced slab on grade, **B,** unreinforced perimeter footing, **C,** reinforced perimeter of a concrete slab

Figure 8 Stretching of foundations due to lateral spreading **(A)** resulting in opening of the expansion joints of a CBD building **(B)**

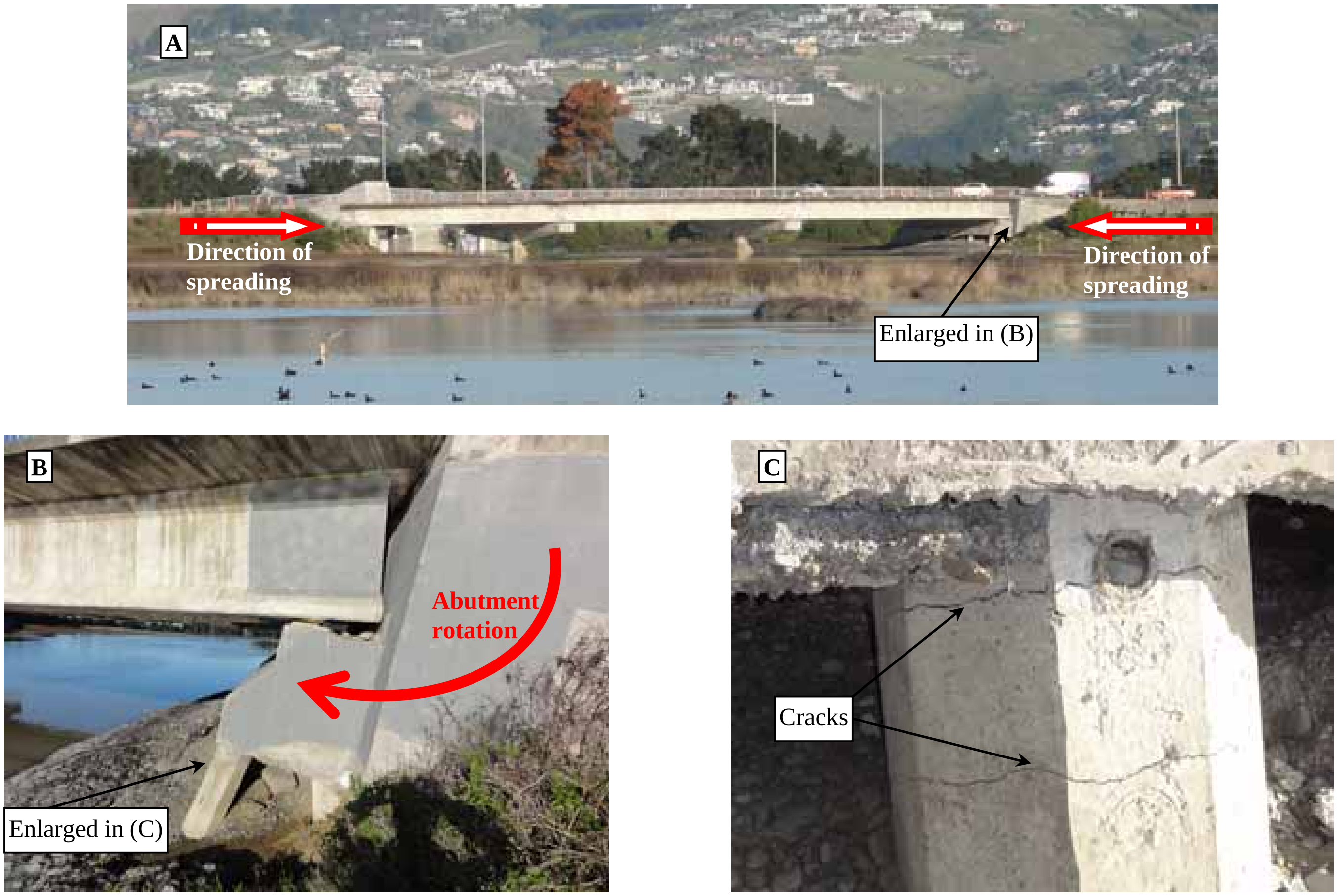


Figure 9 Typical spreading-induced damage to bridges (Bridge St Bridge): **A,** Lateral ground displacement is resisted by the upper bridge structure, **B,** Abutment rotation due to pinning effect and lateral displacements of foundation soils, **C,** Spreading-induced damage (bending cracks) at the top of abutment piles

Figure 10 Ground cracks and distortion of road surface due to lateral spreading towards Avon River (A); pie chart indicating the distribution of damage for the water mains network and its relation to the occurrence and severity of liquefaction (B)

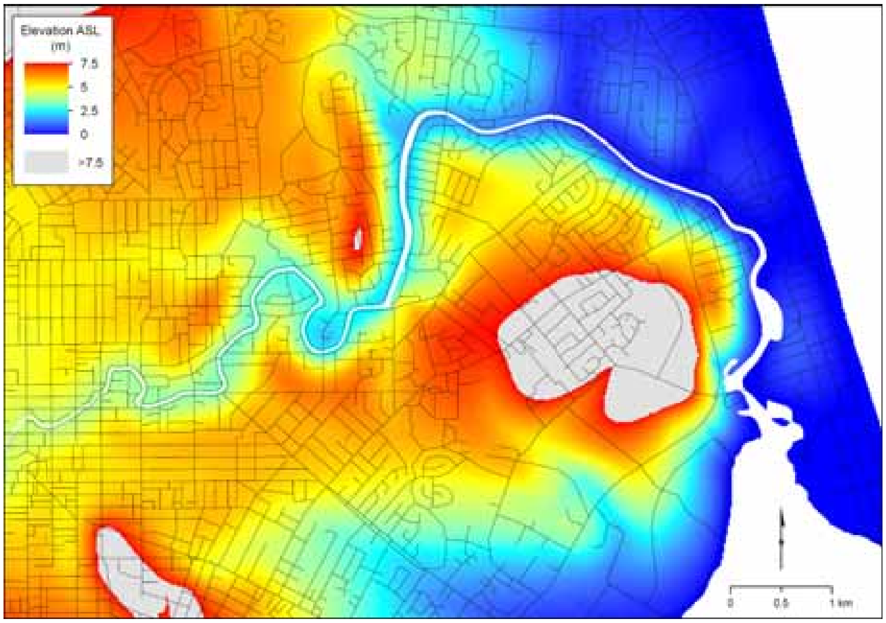


Figure 11 Pre-earthquakes ground elevation above sea level along Avon River, from the CBD to the estuary (derived from the Digital Elevation Model of Christchurch, Landcare Research, 2011)

Appendix B. Water Supply Analysis

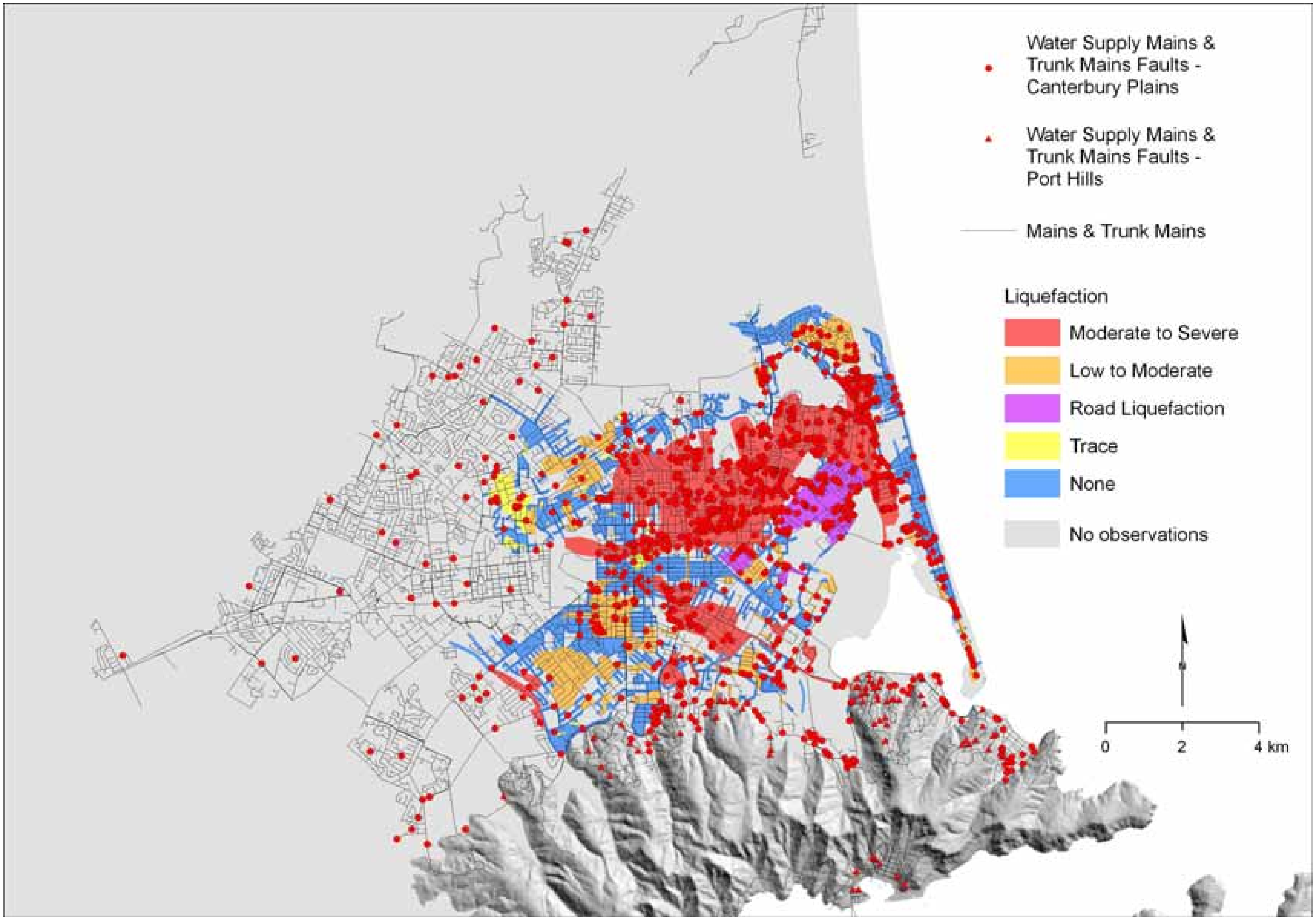


Figure 1. Christchurch City water supply mains and trunk mains system, showing in-service pipes as of 21 February 2011. Water supply pipe fault locations on the Canterbury Plains and Port Hills are shown, with 22 February 2011 liquefaction zones as mapped by University of Canterbury.

Table 1. Summary data for Canterbury Plains water supply mains and trunk mains system performance on 22 February 2011, classified according to liquefaction zone as mapped by University of Canterbury.

Moderate to Severe Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	48.888	41	3.841	7.86	0.84
Polyethylene	5.151	0	0.000	0.00	0.00
Steel	4.945	15	1.026	20.75	3.03
Galvanised Iron	-	-	-	-	-
Asbestos Cement	91.540	307	20.192	22.06	3.35
Concrete	-	-	-	-	-
Other	58.684	123	9.020	15.37	2.10
Total	**209.207**	**486**	**34.078**	**16.29**	**2.32**
Low to Moderate Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	26.571	11	0.992	3.73	0.41
Polyethylene	1.044	-	-	-	-
Steel	2.626	3	0.463	17.62	1.14
Galvanised Iron	-	-	-	-	-
Asbestos Cement	53.520	65	4.775	8.92	1.21
Concrete	-	-	-	-	-
Other	23.956	25	1.644	6.86	1.04
Total	**107.717**	**104**	**7.874**	**7.31**	**0.97**
Road Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	3.539	1	0.056	1.58	0.28
Polyethylene	0.830	-	-	-	-
Steel	0.410	1	0.046	11.19	2.44
Galvanised Iron	0.006	-	-	-	-
Asbestos Cement	21.086	60	3.159	14.98	2.85
Concrete	-	-	-	-	-
Other	6.230	11	0.718	11.52	1.77
Total	**32.102**	**73**	**3.978**	**12.39**	**2.27**
Trace Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	4.091	-	-	-	-
Polyethylene	0.287	-	-	-	-
Steel	0.710	1	0.038	5.32	1.41
Galvanised Iron	-	-	-	-	-
Asbestos Cement	10.581	14	1.049	9.92	1.32
Concrete	-	-	-	-	-
Other	3.592	6	0.408	11.37	1.67
Total	**19.262**	**21**	**1.496**	**7.76**	**1.09**

Table 1-cont. Summary data for Canterbury Plains water supply mains and trunk mains system performance on 22 February 2011, classified according to the liquefaction zones as mapped by University of Canterbury.

No Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	65.152	3	0.478	0.73	0.05
Polyethylene	2.572	1	0.069	2.67	0.39
Steel	4.973	3	0.325	6.53	0.60
Galvanised Iron	0.224	-	-	-	-
Asbestos Cement	137.508	120	7.974	5.80	0.87
Concrete	0.054	-	-	-	-
Other	76.059	44	2.479	3.26	0.58
Total	**286.542**	**171**	**11.324**	**3.95**	**0.60**
No observations					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	250.237	14	1.790	0.72	0.06
Polyethylene	4.774	-	-	-	-
Steel	13.743	8	0.560	4.07	0.58
Galvanised Iron	1.670	-	-	-	-
Asbestos Cement	482.958	163	12.017	2.49	0.34
Concrete	0.000	-	-	-	-
Other	105.119	55	4.457	4.24	0.52
Total	**858.500**	**240**	**18.824**	**2.19**	**0.28**

Table 2. Summary data for Port Hills water supply mains and trunk mains system performance on 22 February 2011, classified according to the liquefaction zones as mapped by University of Canterbury.

Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	89.064	8	0.282	0.32	0.09
Polyethylene	2.229	-	-	-	-
Steel	8.991	2	0.115	1.28	0.22
Galvanised Iron	0.457	3	0.230	50.35	6.57
Asbestos Cement	71.075	43	3.858	5.43	0.60
Concrete	0.292	-	-	-	-
Other	41.666	21	1.194	2.86	0.50
Total	**213.774**	**77**	**5.679**	**2.66**	**0.36**

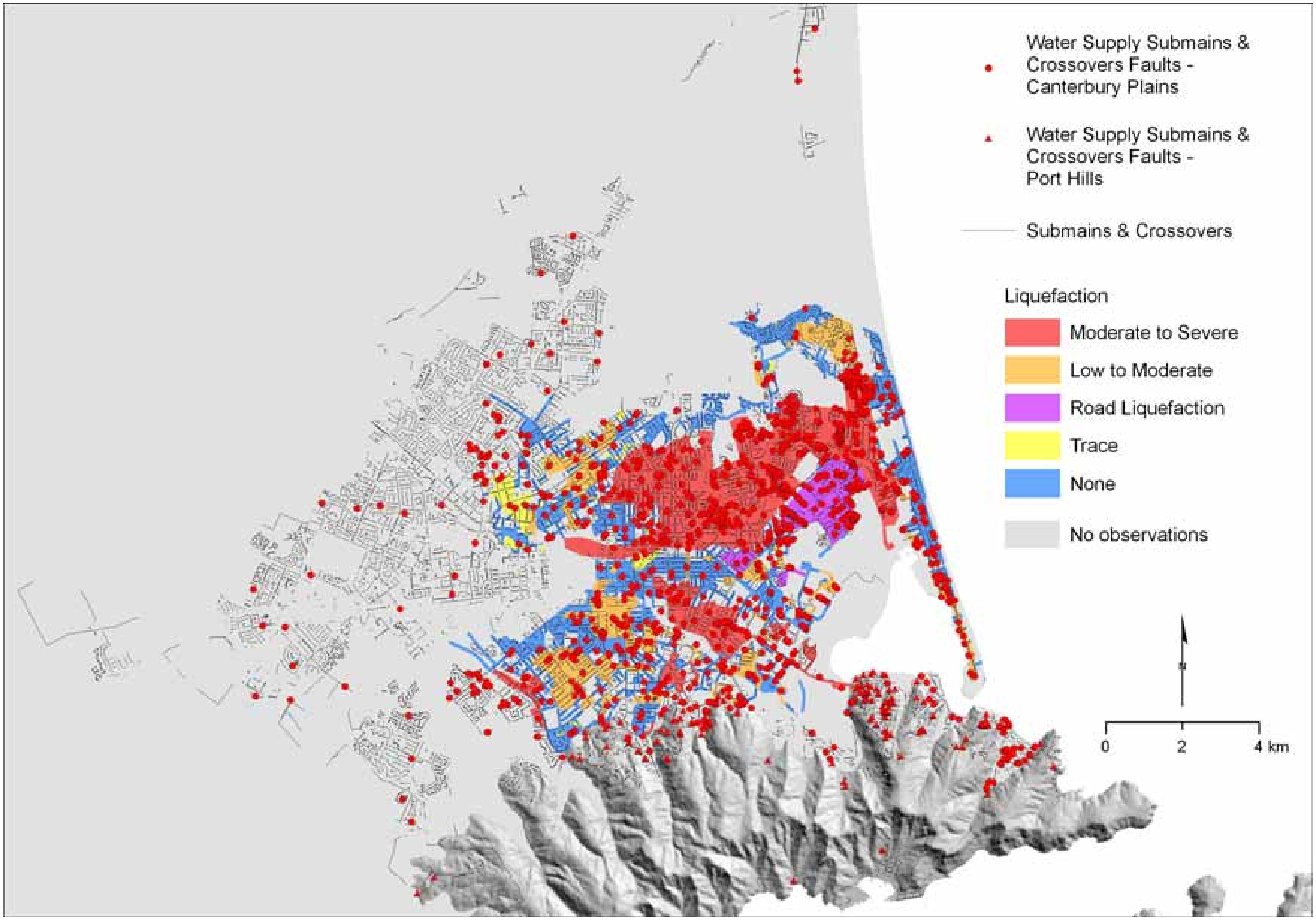


Figure 2. Christchurch City water supply submains and crossovers system, showing in-service pipes as of 21 February 2011. Water supply pipe fault locations on the Canterbury Plains and Port Hills are shown, with liquefaction zones as mapped by University of Canterbury.

Table 3. Summary data for Canterbury Plains water supply submains and crossovers system performance on 22 February 2011, classified according to liquefaction zone as mapped by University of Canterbury.

Moderate to Severe Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	4.098	2	0.121	2.95	0.49
Polyethylene	216.782	195	11.345	5.23	0.90
Steel	-	-	-	-	-
Galvanised Iron	35.591	235	9.269	26.04	6.60
Asbestos Cement	0.120	1	0.018	14.90	8.37
Concrete	-	-	-	-	-
Other	1.318	3	0.216	16.39	2.28
Total	**257.908**	**436**	**20.969**	**8.13**	**1.69**
Low to Moderate Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	1.396	1	0.037	2.65	0.72
Polyethylene	115.113	76	5.206	4.52	0.66
Steel	-	-	-	-	-
Galvanised Iron	9.534	44	1.858	19.49	4.62
Asbestos Cement	-	-	-	-	-
Concrete	-	-	-	-	-
Other	1.318	1	0.017	1.25	0.76
Total	**127.360**	**122**	**7.117**	**5.59**	**0.96**
Road Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	0.300	-	-	-	-
Polyethylene	21.673	10	0.472	2.18	0.46
Steel	-	-	-	-	-
Galvanised Iron	18.813	82	2.952	15.69	4.36
Asbestos Cement	0.064	-	-	-	-
Concrete	-	-	-	-	-
Other	0.500	1	0.069	13.77	2.00
Total	**41.350**	**93**	**3.494**	**8.45**	**2.25**
Trace Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	0.286	-	-	-	-
Polyethylene	18.023	7	0.444	2.46	0.39
Steel	-	-	-	-	-
Galvanised Iron	0.914	3	0.051	5.55	3.28
Asbestos Cement	0.143	-	-	-	-
Concrete	-	-	-	-	-
Other	0.024	-	-	-	-
Total	**19.389**	**10**	**0.495**	**2.55**	**0.52**

Table 3-cont. Summary data for Canterbury Plains water supply submains and crossovers system performance on 22 February 2011, classified according to liquefaction zone as mapped by University of Canterbury.

No Liquefaction					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	3.442	1	0.054	1.57	0.29
Polyethylene	263.804	115	6.806	2.58	0.44
Steel	0.011	-	-	-	-
Galvanised Iron	38.569	107	4.507	11.69	2.77
Asbestos Cement	0.250	-	-	-	-
Concrete	-	-	-	-	-
Other	0.698	1	0.020	2.84	1.43
Total	**306.774**	**224**	**11.387**	**3.71**	**0.73**
No observations					
Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	42.843	9	0.953	2.23	0.21
Polyethylene	682.351	134	9.458	1.39	0.20
Steel	-	-	-	-	-
Galvanised Iron	58.530	123	5.380	9.19	2.10
Asbestos Cement	14.804	3	0.375	2.53	0.20
Concrete	-	-	-	-	-
Other	6.219	1	0.078	1.25	0.16
Total	**804.746**	**270**	**16.243**	**2.02**	**0.34**

Table 4. Summary data for Port Hills water supply mains and trunk mains system performance on 22 February 2011, classified according to the liquefaction zones as mapped by University of Canterbury.

Material	**Total Length (km)**	**Segments Affected**	**Length Affected (km)**	**% Affected**	**Faults / km**
Polyvinyl Chloride	25.829	1	0.210	0.81	0.04
Polyethylene	99.774	45	2.508	2.51	0.45
Steel	0.232	-	-	-	-
Galvanised Iron	29.682	77	3.430	11.55	2.59
Asbestos Cement	3.842	-	-	-	-
Concrete	-	-	-	-	-
Other	6.493	5	0.229	3.53	0.77
Total	**165.851**	**128**	**6.376**	**3.84**	**0.77**

Appendix C. Liquefaction Resistance Index Calculations

Simplified Procedure for Liquefaction Assessment

The simplified procedure (Seed and Idriss, 1971; Youd et al., 2001; Idriss and Boulanger, 2008) compares the liquefaction resistance (cyclic strength) of the soil with the cyclic shear stresses in the soil induced by an earthquake (seismic load/demand). The liquefaction resistance is expressed as a Cyclic Resistance Ratio (CRR), while the seismic demand is expressed in terms of Cyclic Stress Ratio (CSR), and a factor of safety against triggering of liquefaction (in a free field level ground deposit) is calculated as:

$$FS = \frac{CRR}{CSR} \tag{1}$$

If $FS \leq 1.0$, then liquefaction will be triggered (will occur) for the considered ground motion (earthquake). In the simplified procedure, the peak ground acceleration (PGA) is used as a measure for the amplitude of ground shaking while the earthquake (moment) magnitude (M_w) is used as a proxy for the duration of shaking (i.e. number of significant stress cycles), as described below.

Because of the empirical basis of the simplified procedure in which a magnitude 7.5 earthquake is used as a reference, a specific form of Equation (1) is used to account for earthquakes of different magnitudes:

$$FS = \frac{CRR_{7.5}}{CSR} MSF \tag{2}$$

where $CRR_{7.5}$ is the cyclic resistance ratio for a M_w=7.5 earthquake and MSF is the Magnitude Scaling Factor which depends on the earthquake magnitude. In essence, the demand normalized to a magnitude 7.5 earthquake is represented by

$$Demand = CSR_{7.5} = \frac{CSR}{MSF} = \frac{f(PGA)}{f(M_w)} \tag{3}$$

and combines the effects of the peak ground acceleration (PGA) or amplitude of ground shaking, and the duration of shaking (or number of significant stress cycles) using the moment magnitude.

The CSR produced at any given depth of the deposit (z) is calculated using (e.g. Youd et al., 2001)

$$CSR = 0.65 \frac{a_{\max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \tag{4}$$

where a_{max} is the peak ground acceleration (PGA) at the ground surface, σ_{vo} and σ'_{vo} are the total and effective overburden stresses at depth (z) respectively, and r_d is a stress reduction factor as a function of depth, given by (e.g. Youd et al., 2001)

$$r_d = 1.0 - 0.00765z \qquad \text{for } z \leq 9.15 \text{ m} \tag{5}$$

The MSF can be calculated, for example (Youd et al., 2001), using

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \tag{6}$$

Substituting Equations (4) to (6) into Equation (2) yields the following expression for the factor of safety against liquefaction triggering:

$$FS = \frac{CRR_{7.5}}{0.65 \frac{a_{\max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} \left(1.0 - 0.00765z\right)} \frac{10^{2.24}}{M_w^{2.56}} \tag{7}$$

<u>The Inverse Problem: Back-Calculating $CRR_{7.5}$ from Earthquake Observations</u>

The simplified procedure has been developed for evaluating liquefaction triggering as a predictive tool. Using penetration resistance or shear wave velocity data for the site of interest, the liquefaction resistance $CRR_{7.5}$ is first estimated using empirical charts, and then design *PGA* values and representative moment magnitudes are used from site-specific or regional seismic hazard analyses to calculate the factor of safety, and estimate whether liquefaction will occur or not at the site when shaken with the adopted ground motion (a_{max}- M_w combination defining the amplitude and the number of loading cycles).

In cases where the ground motion and the severity of liquefaction induced by an earthquake have been recorded and documented, one may approximate the liquefaction resistance at the site by back calculating $CRR_{7.5}$ by expressing Equation (2) as

$$CRR_{7.5} = \frac{CSR \cdot FS}{MSF} \tag{8}$$

Here, *CSR* and *MSF* can be easily calculated using recorded PGA (a_{max}) at the site/area and the moment magnitude of the earthquake via Equations (4) and (6) respectively. The calculation of *FS* is less straightforward, but still possible at least in approximate terms.

The key objective of such calculation is to quantify the liquefaction resistance of soils using actual earthquake observations and hence provide liquefaction map based on field evidence and records of performance. This will provide alternative and complementary information that could be particularly useful for liquefaction mapping/zoning purpose. A direct back-calculation of the absolute liquefaction resistance using this approach could be difficult, however the liquefaction resistance can be reasonably well defined in relative terms (as a first objective) based on observed differences in the ground performance and liquefaction manifestation in the areas of concern. Thus, the keywords associated with the proposed method are *actual earthquake observations* and *relative liquefaction resistance*.

This method was employed to back-calculate the liquefaction resistance and produce a Liquefaction Resistance Index map of Christchurch, based on ground motion records and liquefaction observations from the 4 September 2010 and 22 February earthquakes.

<u>Average Factor of Safety $\overline{FS}$</u>

In the original definition, *FS* defines the factor of safety at a particular depth of the deposit, whereas in the inverse problem when back-calculating $CRR_{7.5}$ from earthquake observations, the average factor of safety for a representative portion (thickness/depth) of the deposit should be used instead. Hence Equation 8 is expressed as

$$CRR_{7.5} = \frac{CSR \cdot \overline{FS}}{MSF} \tag{9}$$

where $\overline{FS}$ is the average factor of safety over a relevant depth of the deposit (which correlates best with the severity of liquefaction manifested/observed on the ground surface). The thickness of the soil layer (depths) over which $\overline{FS}$ should be averaged depends on the particular structure considered and depth of influence. Thus, larger range of depths and thickness of the deposit should be considered for large high-rise buildings as compared to small one or two storey dwellings. Similarly, smaller depth range and shallow depths should be considered when evaluating the average factor of safety $\overline{FS}$ for buried lifelines such as water and wastewater pipe networks.

Correlation between the factor of safety $\overline{FS}$ and severity of liquefaction

To use effectively Equation (9), the factor of safety $\overline{FS}$ has to be related to the liquefaction severity at least in some general way. By definition, $\overline{FS}$ = 1.0 denotes the liquefaction triggering criterion, and hence

$$\overline{FS} > 1.0 \rightarrow \quad \text{No Liquefaction} \tag{10a}$$

$$\overline{FS} \leq 1.0 \rightarrow \quad \text{Liquefaction} \tag{10b}$$

Once the liquefaction triggering is satisfied, i.e. $\overline{FS}$ < 1.0, again by definition a lower factor of safety implies more severe effects of liquefaction and greater liquefaction-induced damage.

Based on the well-documented observations of liquefaction manifestation and liquefaction-induced damage in the 22 February 2011 Christchurch earthquake, the following correlation between the Factor of Safety $\overline{FS}$ and liquefaction severity was developed, as summarised in Table 1.

Table 1. Anticipated correlation between $\overline{FS}$ and liquefaction severity

Factor of Safety $\overline{FS}$	**Liquefaction Severity**	**Typical Manifestation and Damage to Structures**	**Estimated ground settlement**
0.90	Traces of liquefaction	Some evidence of liquefaction, but very limited both in extent and impacts, judged non-damaging for structures	< 50 mm
0.75	Low to moderate	Clear evidence of liquefaction, with scattered sand boils (sand ejecta) and limited ground distortion; low damage to residential buildings and buried pipe networks.	50 – 200 mm
0.50	Moderate to severe	Very large, continuous and thick sand ejecta, severe ground distortion (undulations, fissures) and substantial total and differential settlements; moderate to severe damage to residential buildings and buried pipe networks.	200 – 400 mm
0.25	Very severe (extreme)	Extreme manifestation of liquefaction with excessive ground distortion including very large total and differential settlements, vertical offsets and ground fissures, often accompanied with severe effects of lateral spreading; excessive (most often beyond repair) damage to residential buildings and buried pipe networks.	> 400 mm

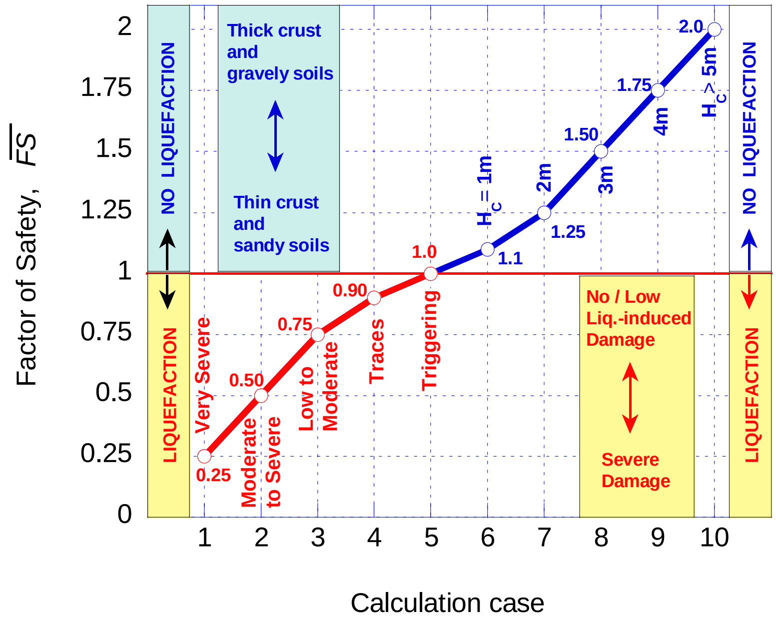


Figure 1. Adopted factors of safety (against liquefaction) in areas of manifested liquefaction (*FS* < *1.0*) and areas of no liquefaction (*FS* > 1.0) with different water table depth

For the Christchurch potable water system, for example, the pressurised pipe network is typically at depths of 0.6 m to 0.8 m, while the wastewater pipes are predominantly at depths from 1.5 m to 3.5 m, with occasional deeper segments of up to 4.0 m – 4.5 m depth. In most of the area/suburbs that suffered liquefaction in Christchurch, the water table was very high, about 1 m to 1.5 m from the ground surface. Thus, for the liquefaction mapping of pipe networks one would focus on the shallow part of the deposit. Hence, Liquefaction Resistance Index (LRI) values were calculated for depths at the water table and 2m below the water table. The LRI zoning map for the water table depth shown in figure 28 and the accompanying Table 5 are recommended for preliminary estimates of the liquefaction resistance of soils and associated ground deformation/damage level.

For the liquefied areas, the factor of safety was defined based on the severity of manifested liquefaction in the field, as summarized in Table 1. Since triggering of liquefaction yields by definition *FS* = 1.0, traces of liquefaction, low to moderate liquefaction and moderate to severe liquefaction were given *FS* values of 0.9, 0.75 and 0.50 respectively. In other words, *FS* decreases with increased severity of liquefaction manifestation. An *FS* of 0.5 indicates that the available cyclic strength of the soil was half of the seismic load induced by the earthquake. For cases of extreme or very severe effects of liquefaction, an *FS* value of 0.25 was adopted.

In the non-liquefied areas, it was conservatively adopted that in areas where the water table was at 1m or 2m depth, that $\overline{FS}$ was slightly above the threshold triggering value or 1.1 and 1.25, respectively. Then $\overline{FS}$ was increased with the water table depth since it is well known that a thick crust decreases the likelihood of occurrence and surface manifestation of liquefaction. Thus, $\overline{FS}$ = 1.5, 1.75 and 2.0 was adopted for areas with water table depth of 3.0, 4.0 and 5.0m. The adopted $\overline{FS}$ values across different severity of liquefaction and water table depths are depicted in Figure 1.

Pictorial description of all stages in the development of the LRI zoning map is given in Appendix D where all major steps including assumptions made in the calculations and resulting outputs are provided in the form of GIS maps.

Appendix D. Christchurch Liquefaction Resistance Map

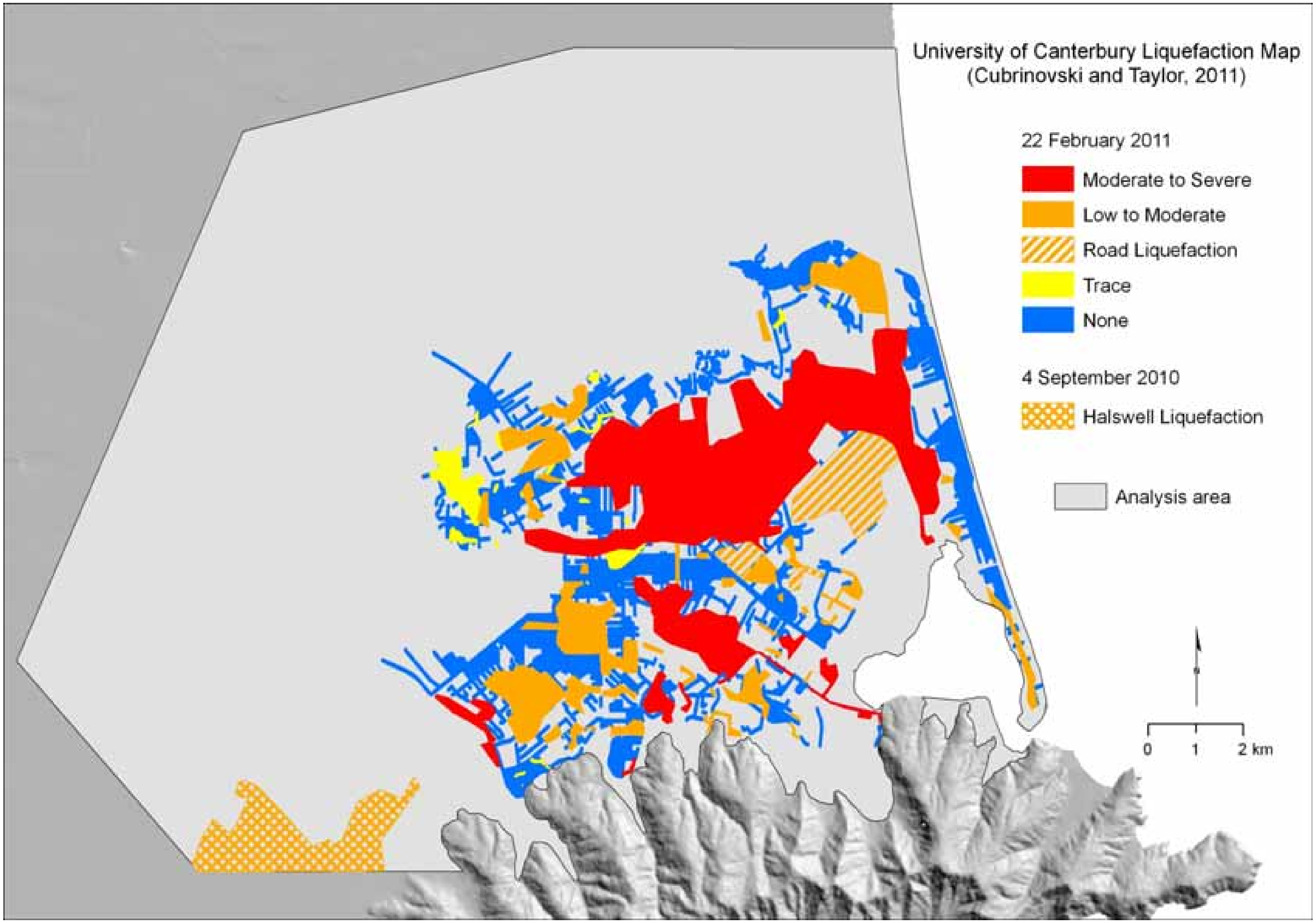


Figure 1. University of Canterbury liquefaction map from 22 February 2011 earthquake. Note that the Halswell area that liquefied in the 4 September 2010 earthquake is also shown. Data were polygons digitised originally in Google Earth and then converted to ArcGIS shapefiles.

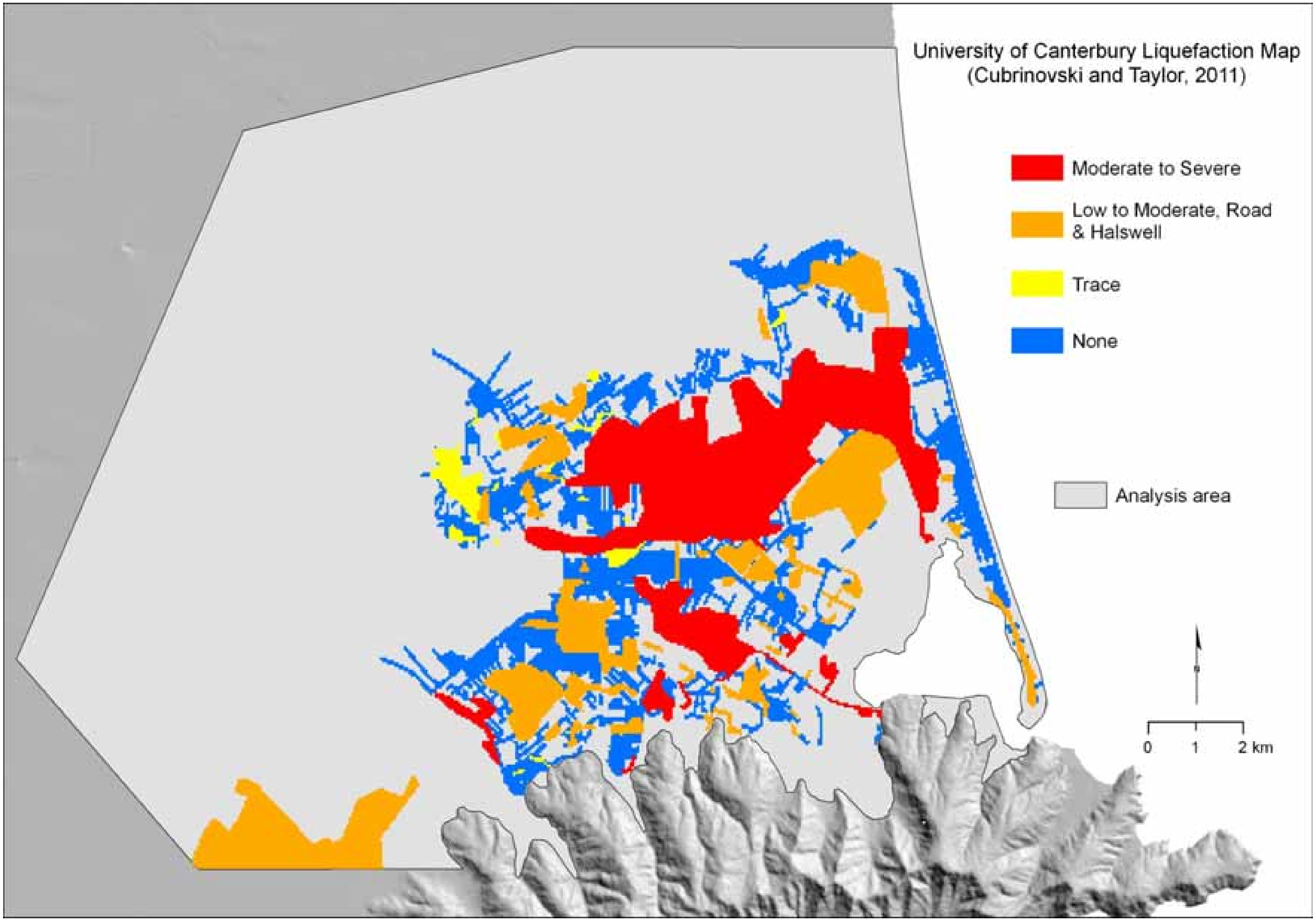


Figure 2. University of Canterbury liquefaction map from 22 February 2011 used in the LRI analysis, converted to a 50 m-resolution raster.

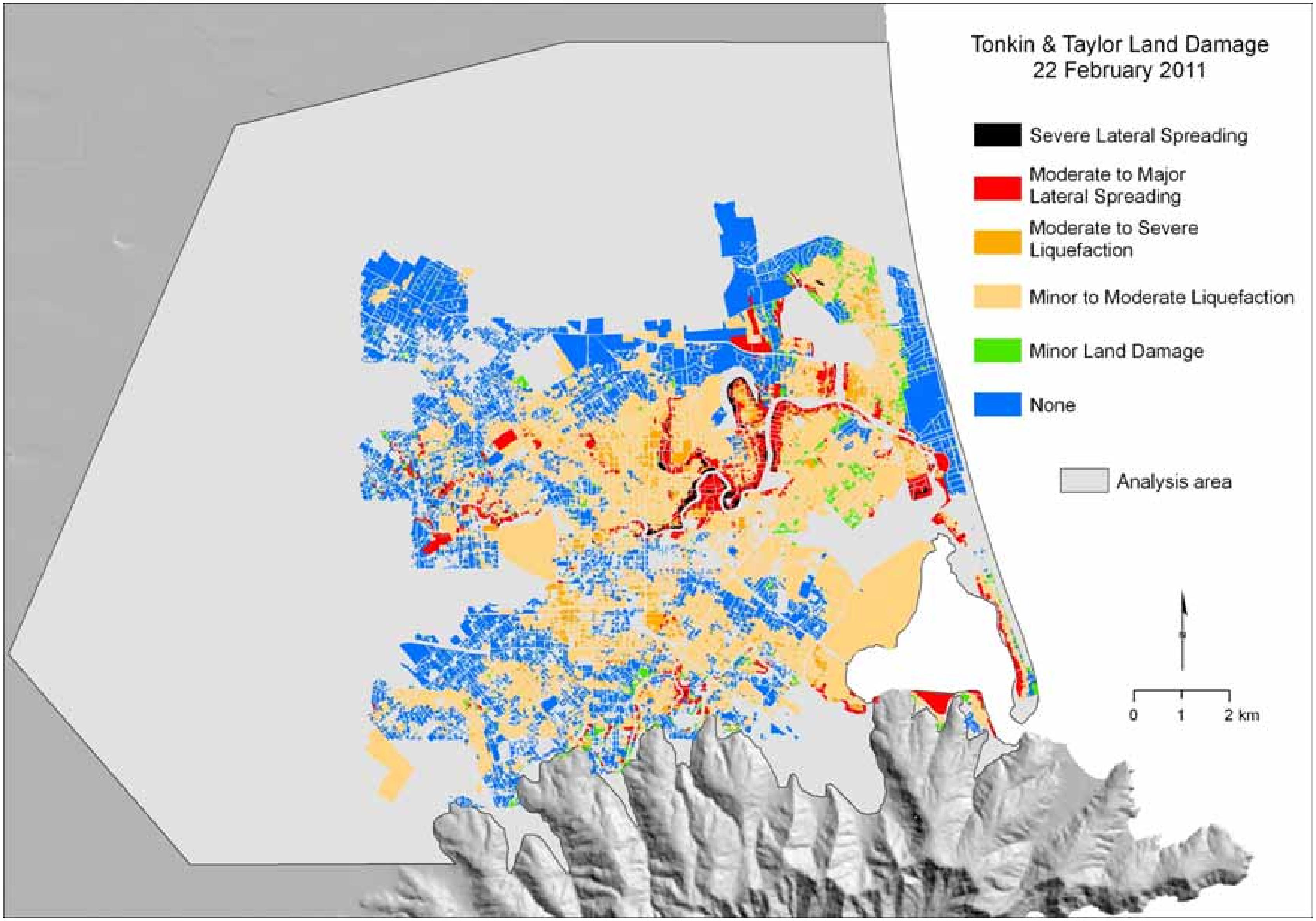


Figure 3. Tonkin and Taylor land damage data from 22 February 2011. Data are polygons downloaded from the Project Orbit website as Google Earth .kml files, converted to ArcGIS shapefiles.

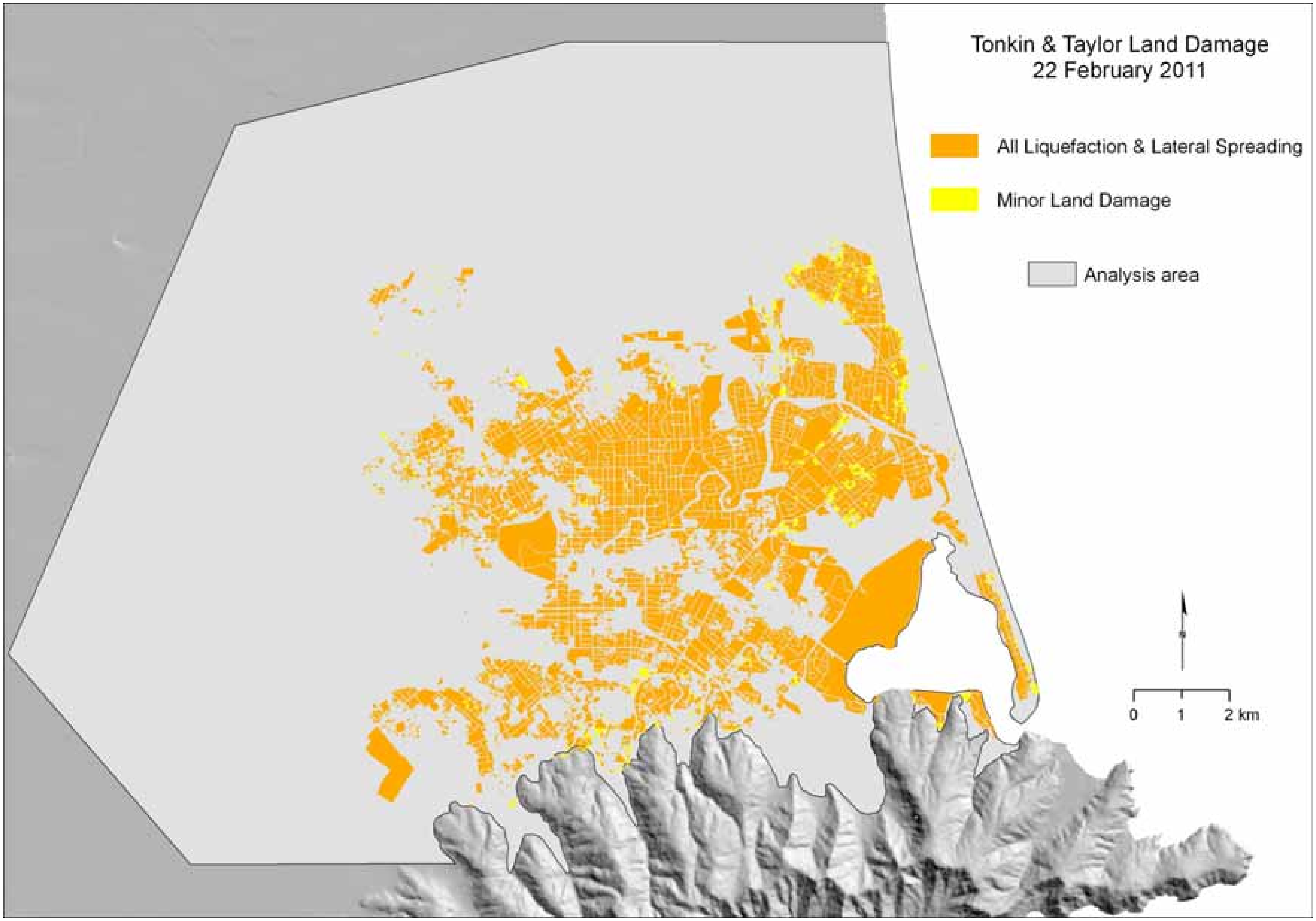


Figure 4. Tonkin and Taylor land damage data from 22 February 2011, with all areas of liquefaction and lateral spreading aggregated. Areas of no liquefaction not shown.

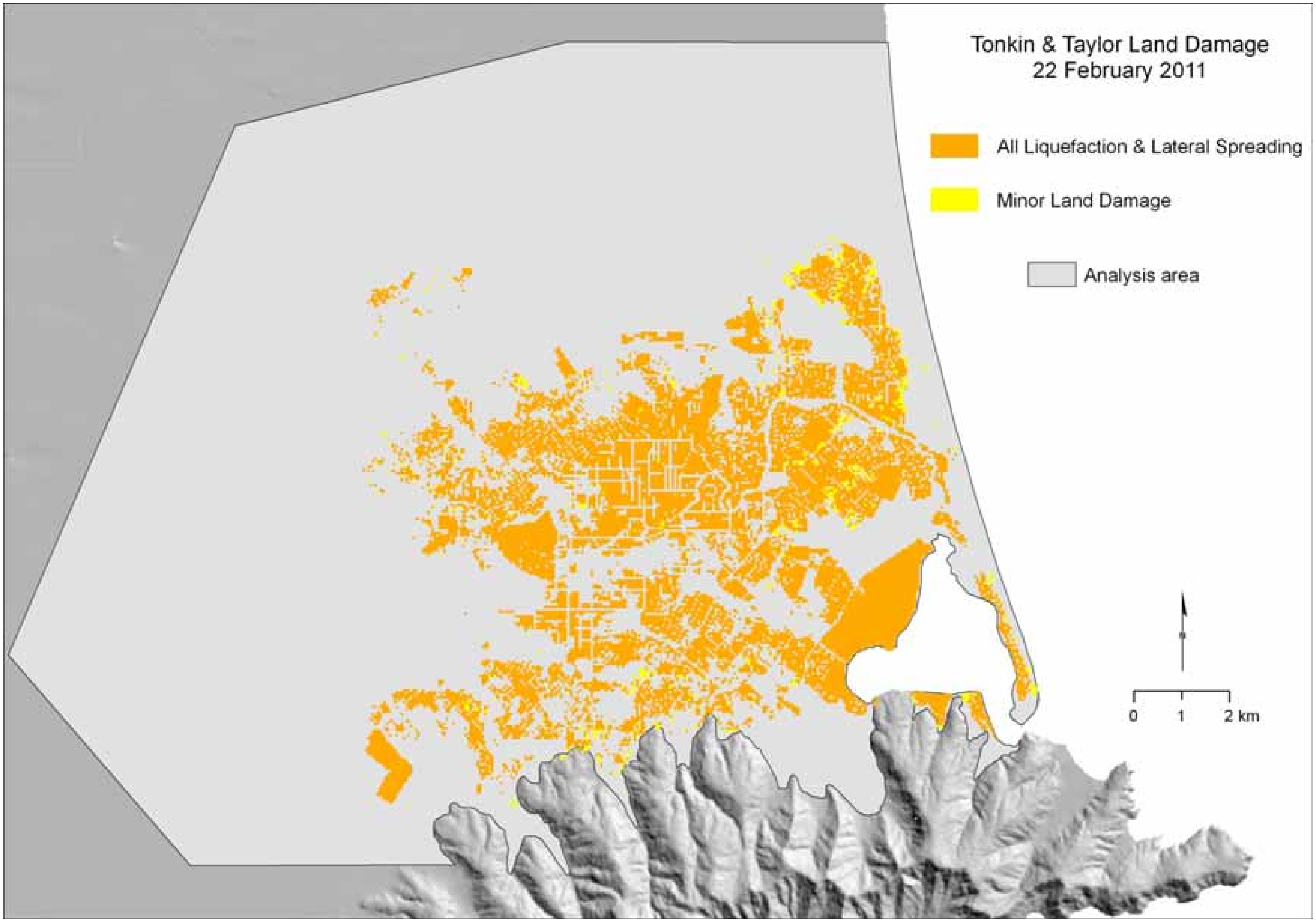


Figure 5. Tonkin and Taylor land damage data from 22 February 2011, with aggregated areas of liquefaction, lateral spreading and minor land damage converted to a 50 m-resolution raster.

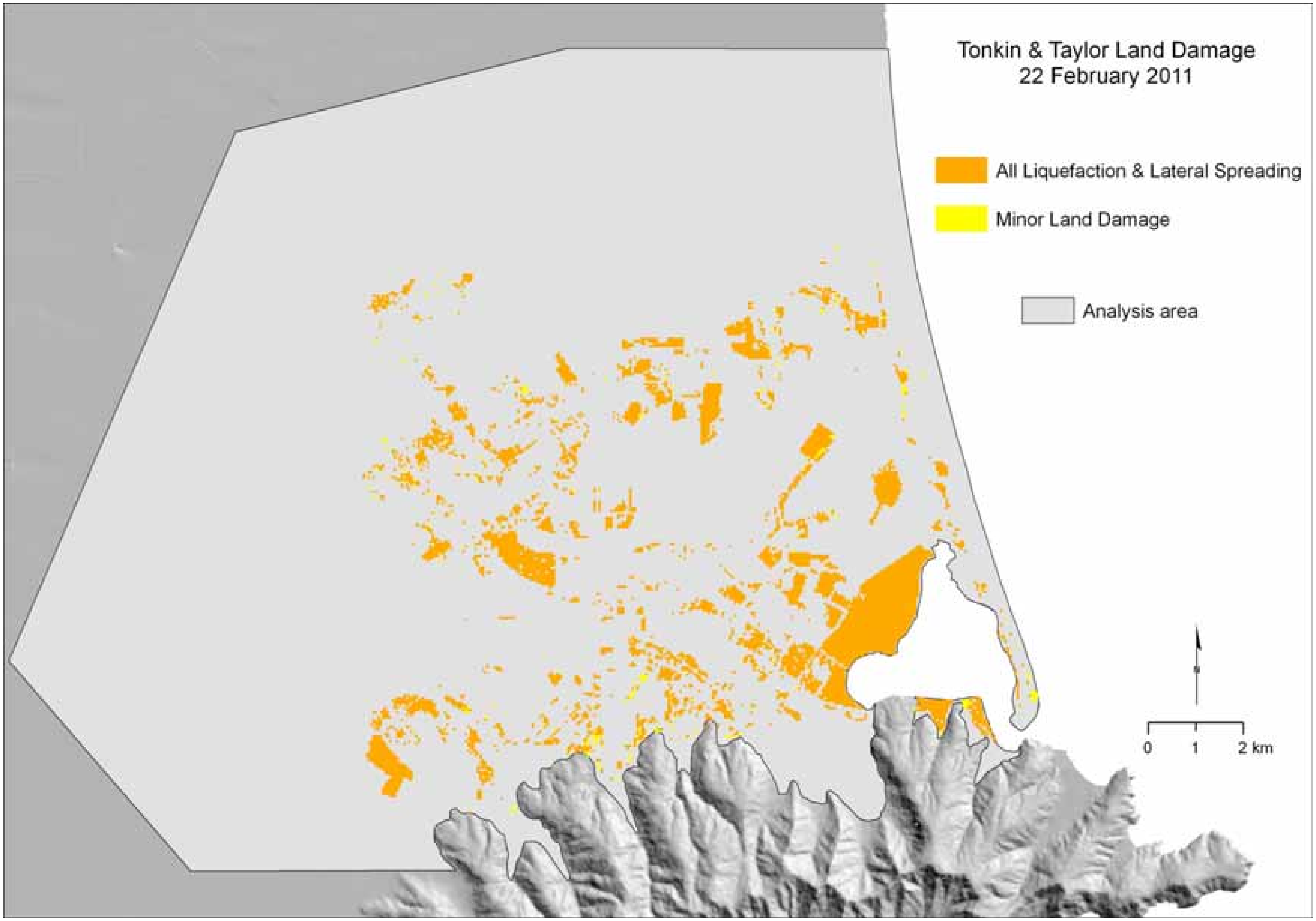


Figure 6. Tonkin and Taylor generalised raster land damage map from 22 February 2011 used in the LRI analysis, the areas outside those mapped by University of Canterbury.

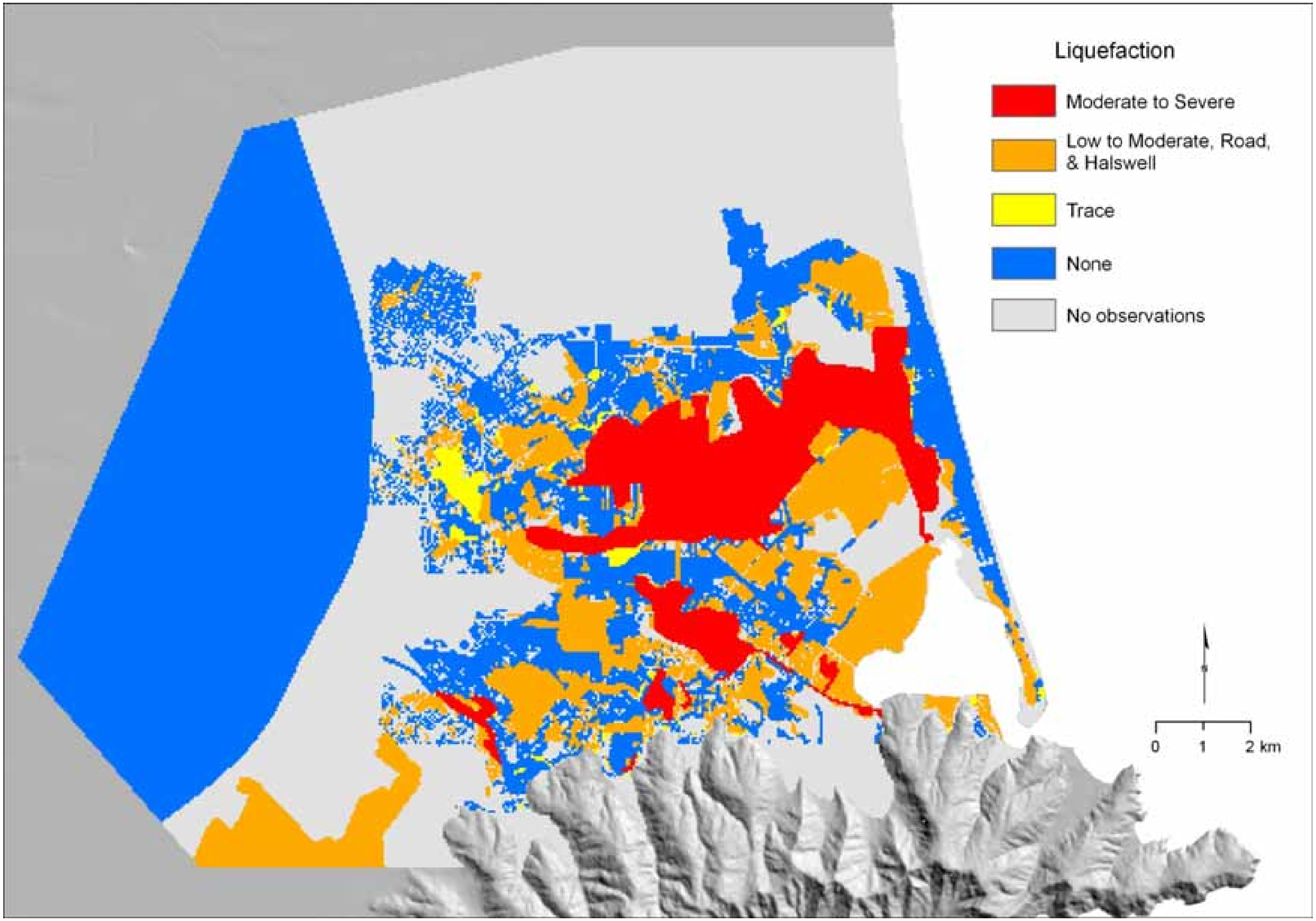


Figure 7. Integrated University of Canterbury and Tonkin and Taylor liquefaction map from 22 February 2011 earthquake. Note that the Halswell area that liquefied in the 4 September 2010 earthquake is also shown. Areas of no observation conducted also shown.

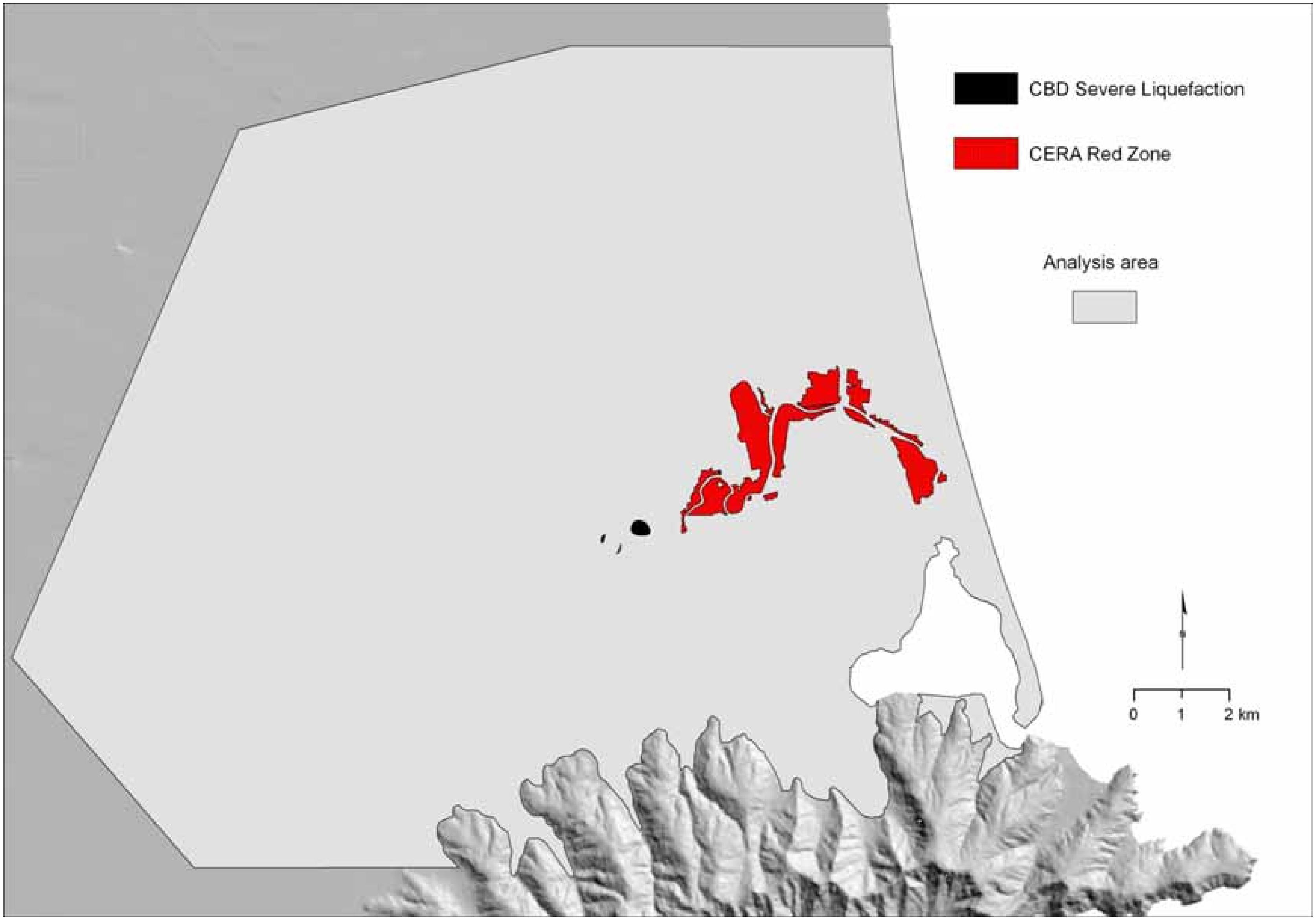


Figure 8. Additional liquefaction data used in the LRI analysis. CERA abandoned areas are the residential red zone adjacent to the Avon River. CBD severe liquefaction is additional areas observed by the University of Canterbury.

Figure 9. Severe CBD and CERA liquefaction areas converted to a 50 m-resolution raster.

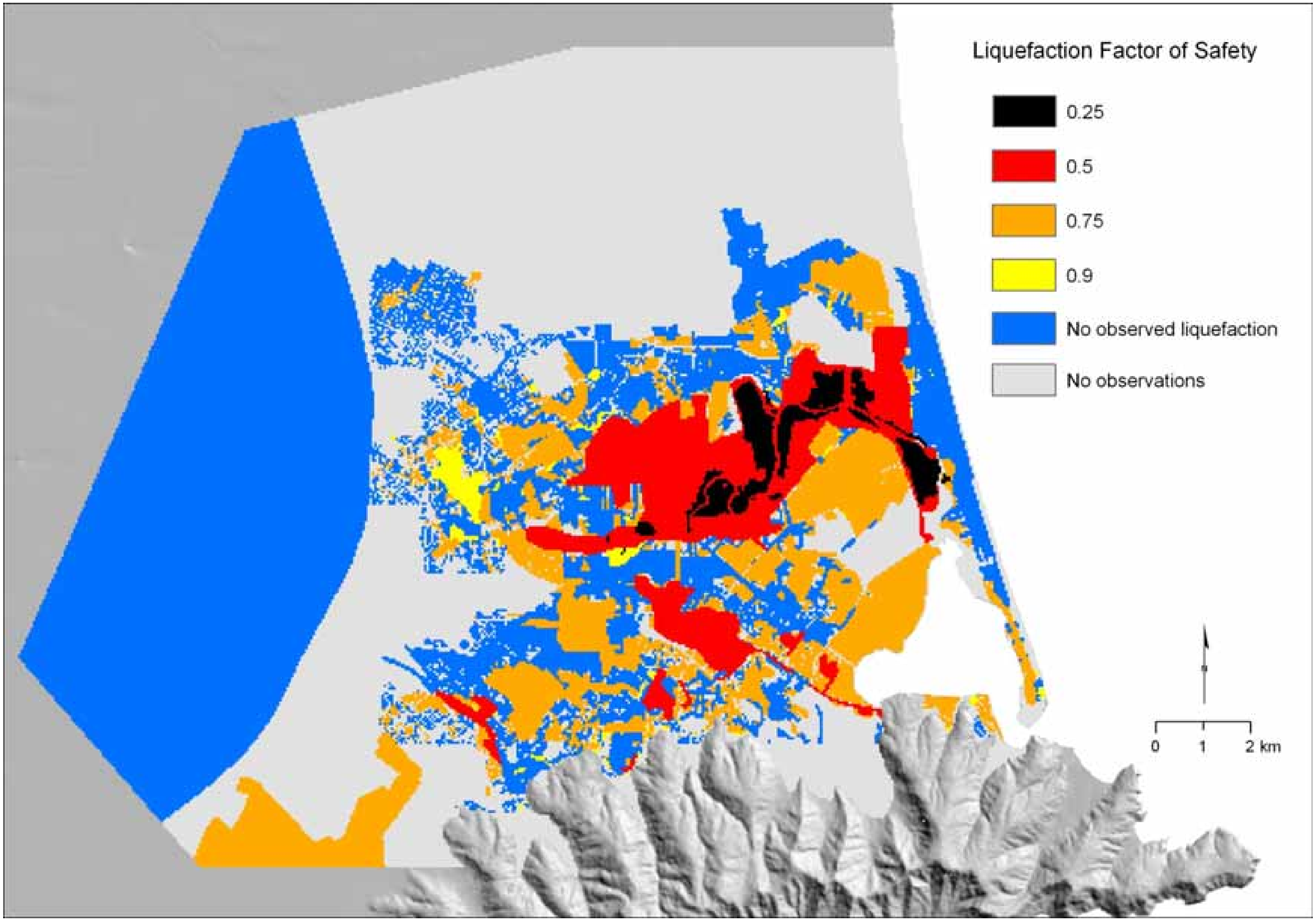


Figure 10. Factor of Safety (FS) against liquefaction used in the LRI analysis for areas that liquefied during the 4 September 2010 and 22 February 2011 earthquakes.

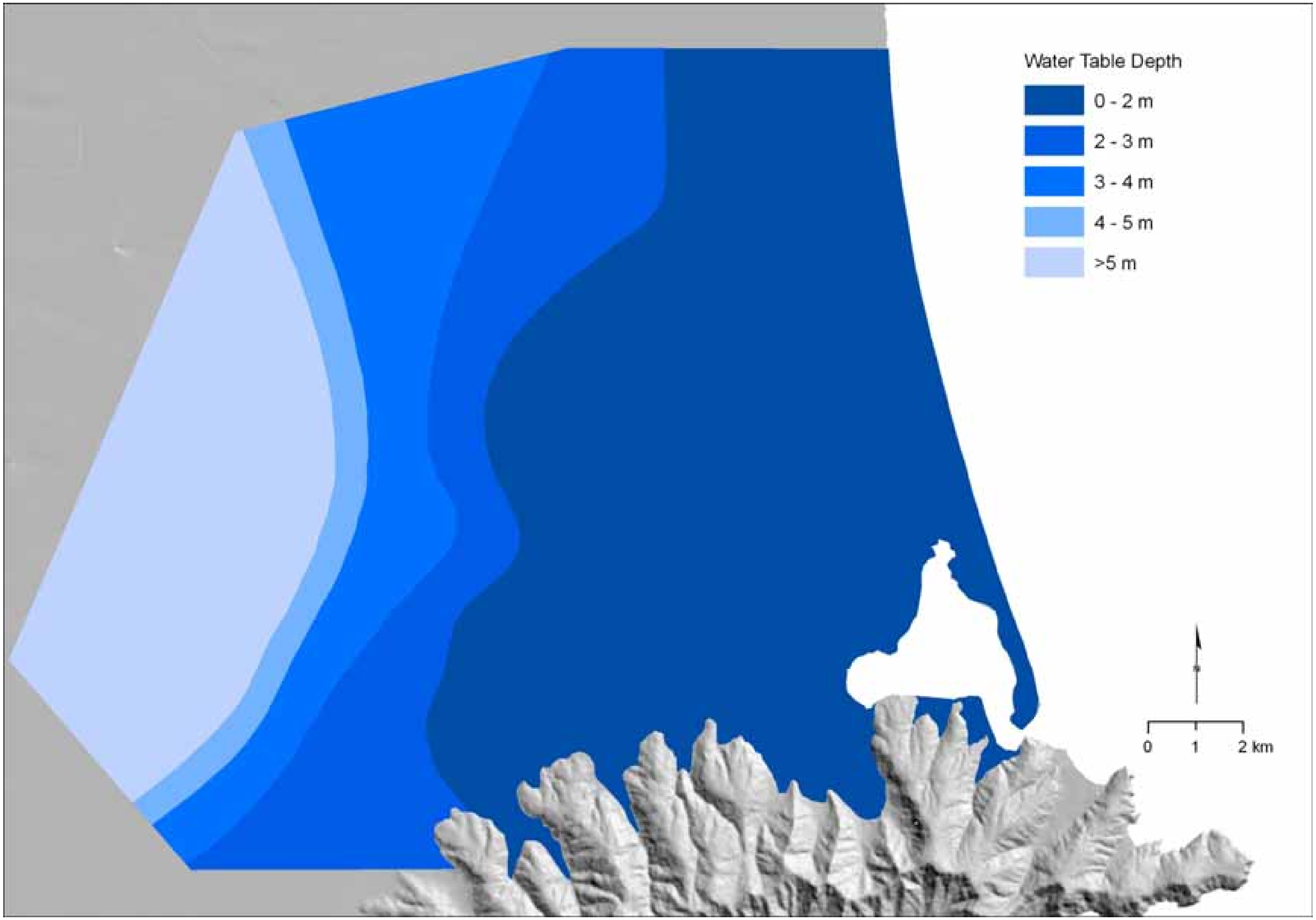


Figure 11. Water table depth contours modified from Brown and Weeber (1992).

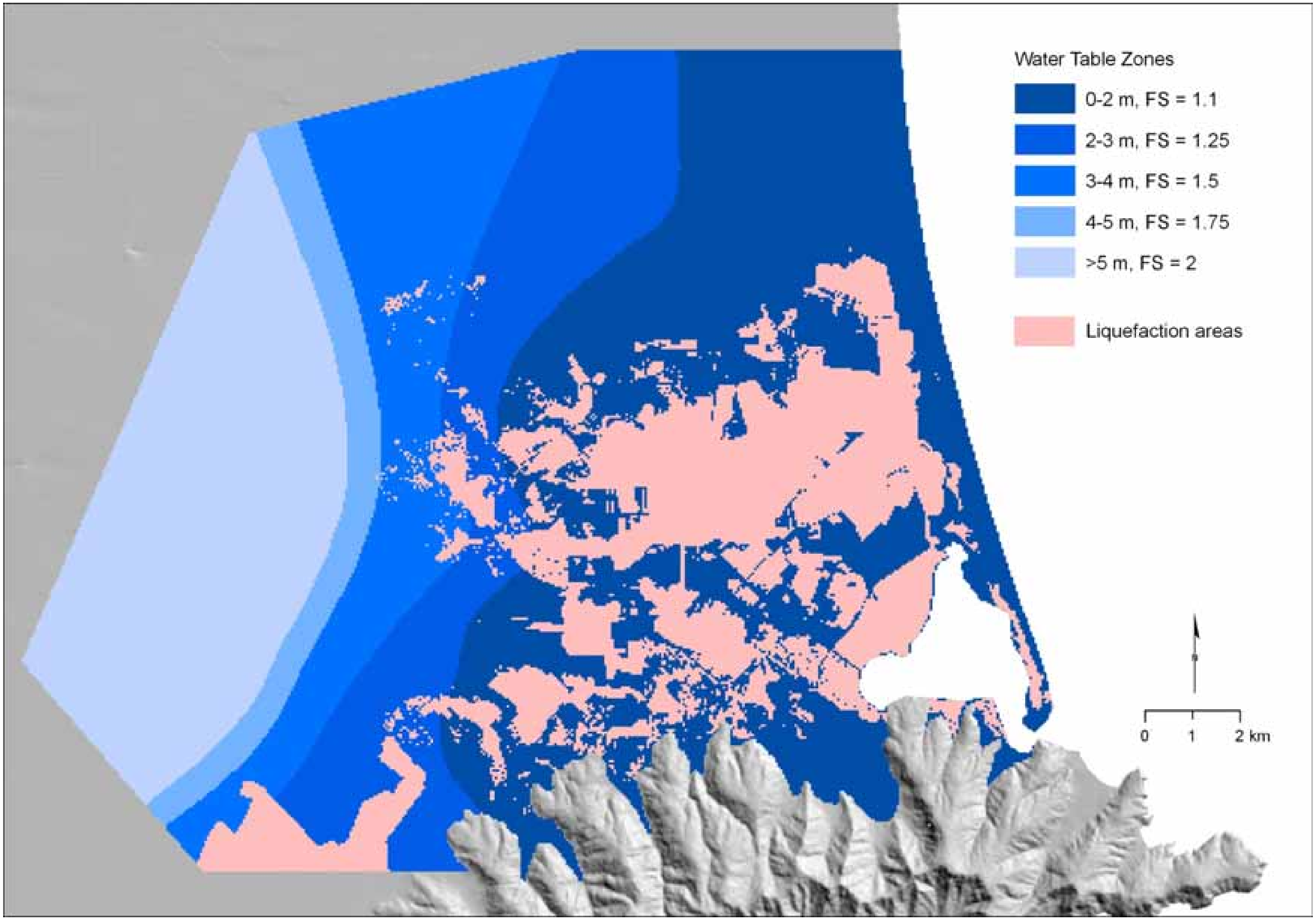


Figure 12. Water table depth contours modified from Brown and Weeber (1992), converted to a 50 m resolution raster with Factors of safety allocated to areas that did not liquefy during the 4 September 2010 and 22 February 2011 earthquakes

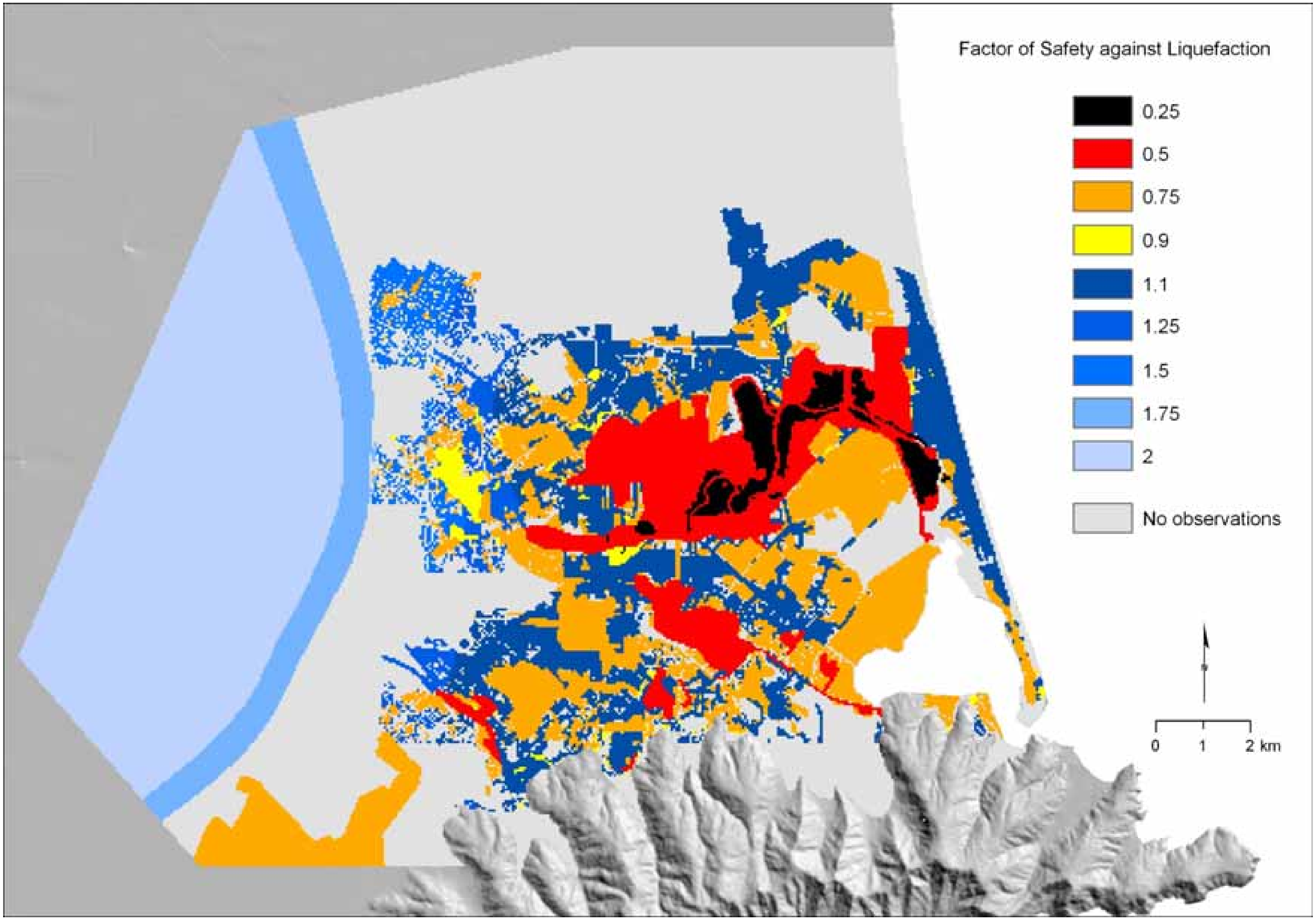


Figure 13. Factors of safety (against liquefaction) map back-calculated based on liquefaction observations from the 2010-2011 earthquakes and water table depth information.

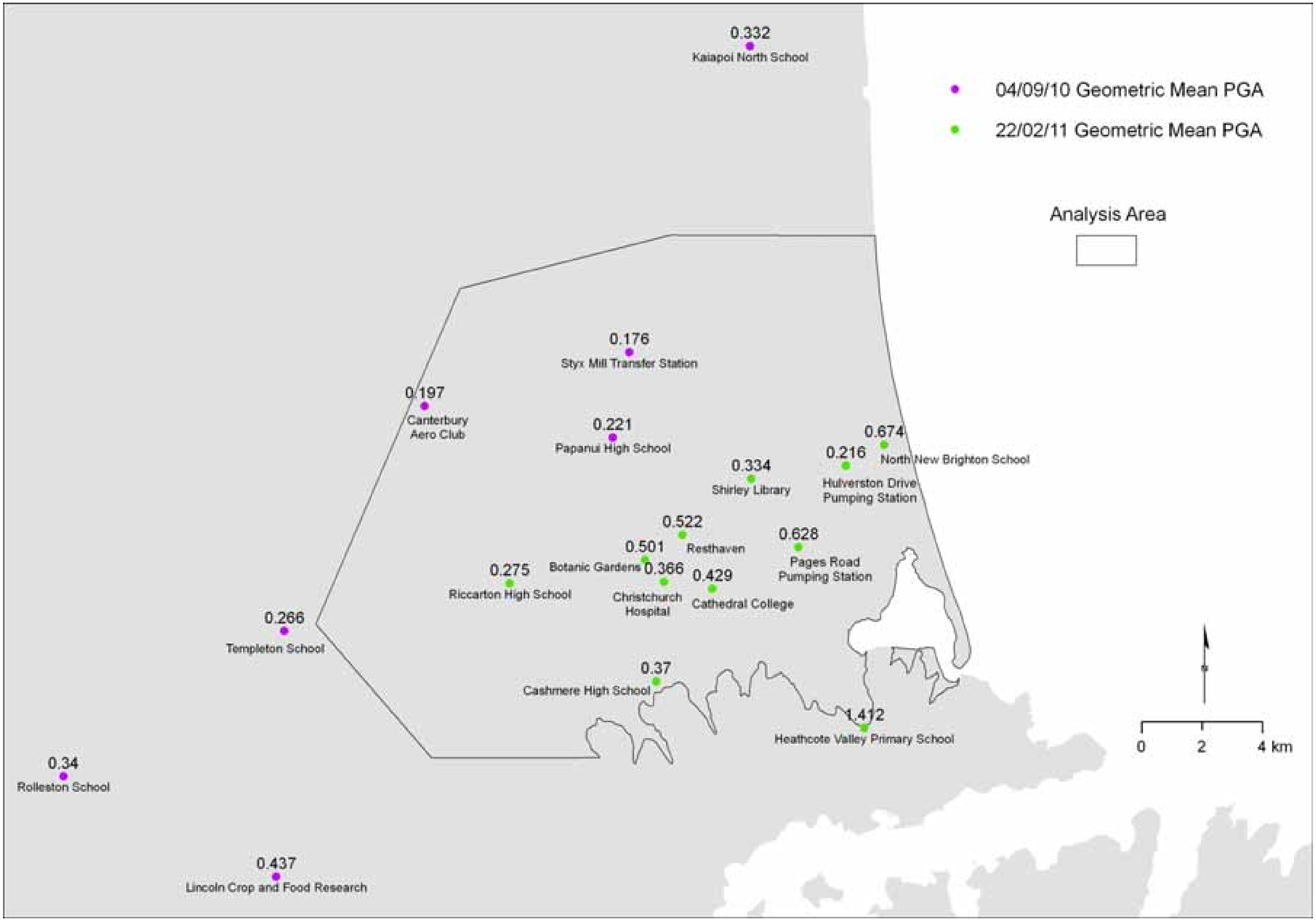


Figure 14. Geometric mean PGA for the 4 September 2010 (magenta symbols) and 22 February 2011 (green symbols) earthquakes.

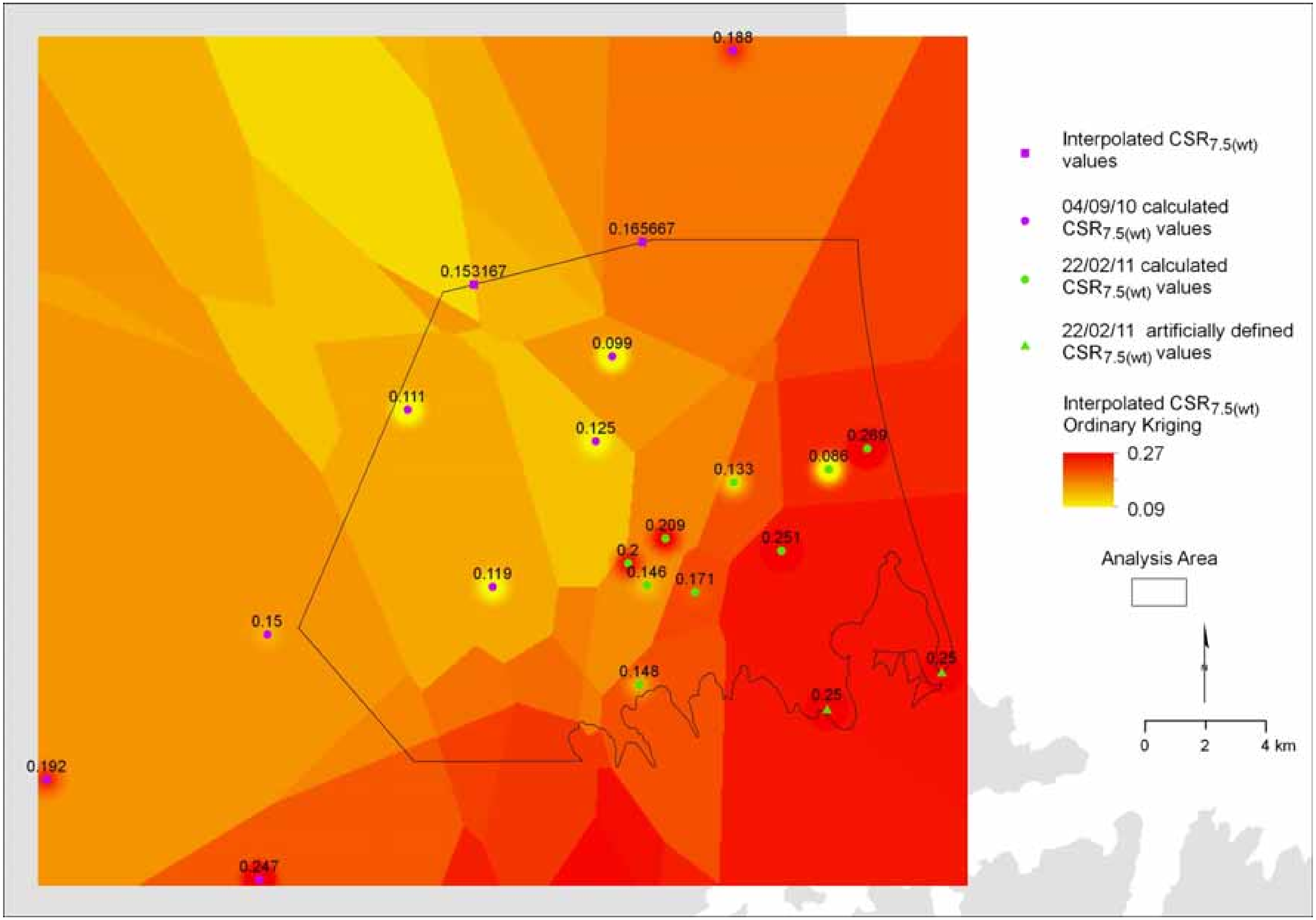


Figure 15a. Interpolated $CSR_{7.5(wt)}$ values in the northwest sector of the analysis area using 4 September 2010 $CSR_{7.5(wt)}$ = 0.188 for Kaiapoi North School, and $CSR_{7.5(wt)}$ = 0.25 for the point 1 km northwest of Heathcote Valley Primary School and the southern tip of Brighton Spit.

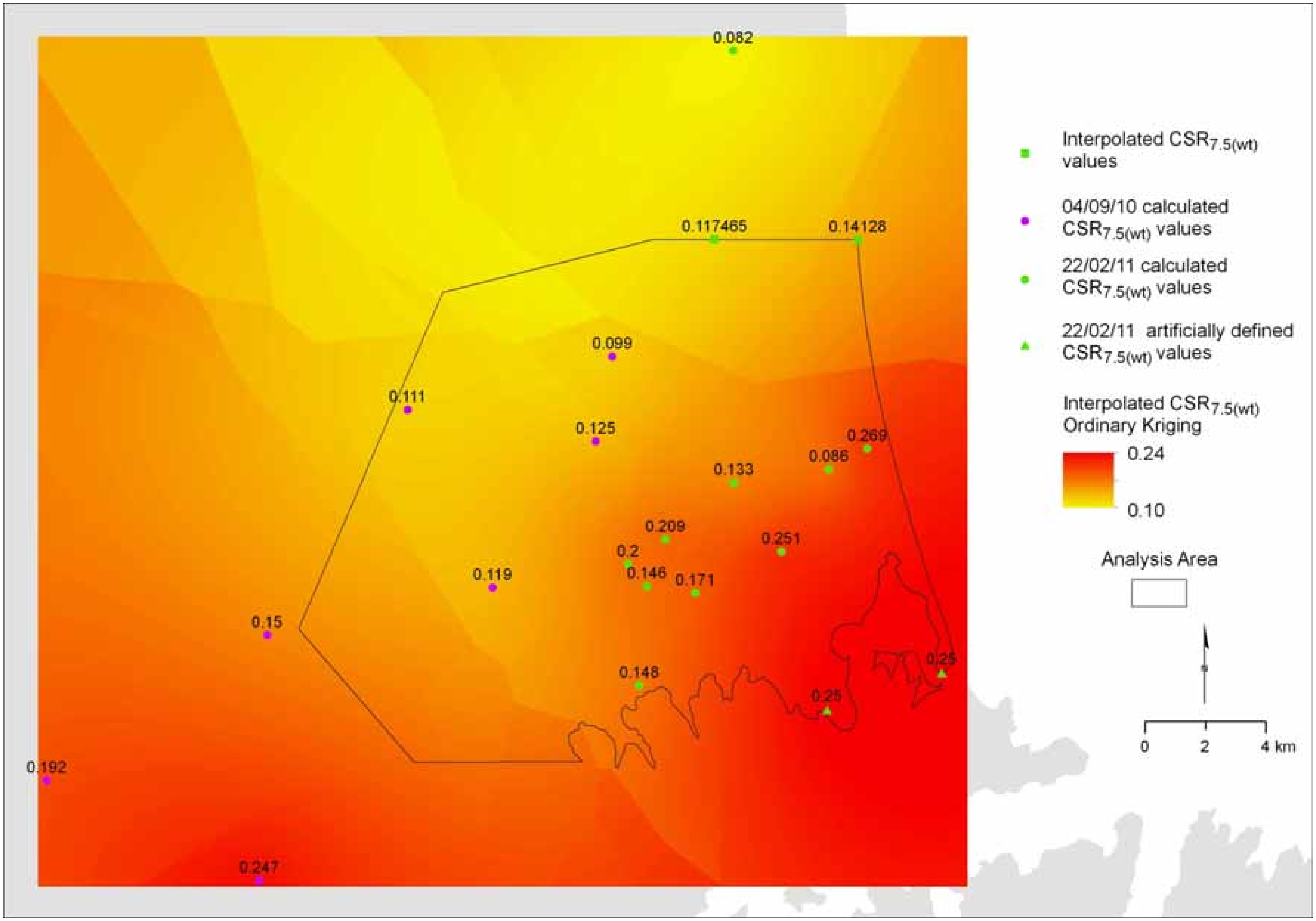


Figure 15b. Interpolated $CSR_{7.5(wt)}$ values in the north sector of the analysis area using 22 February 2011 $CSR_{7.5(wt)}$ = 0.082 for Kaiapoi North School, and $CSR_{7.5(wt)}$ = 0.25 for the point 1 km northwest of Heathcote Valley Primary School and the southern tip of Brighton Spit.

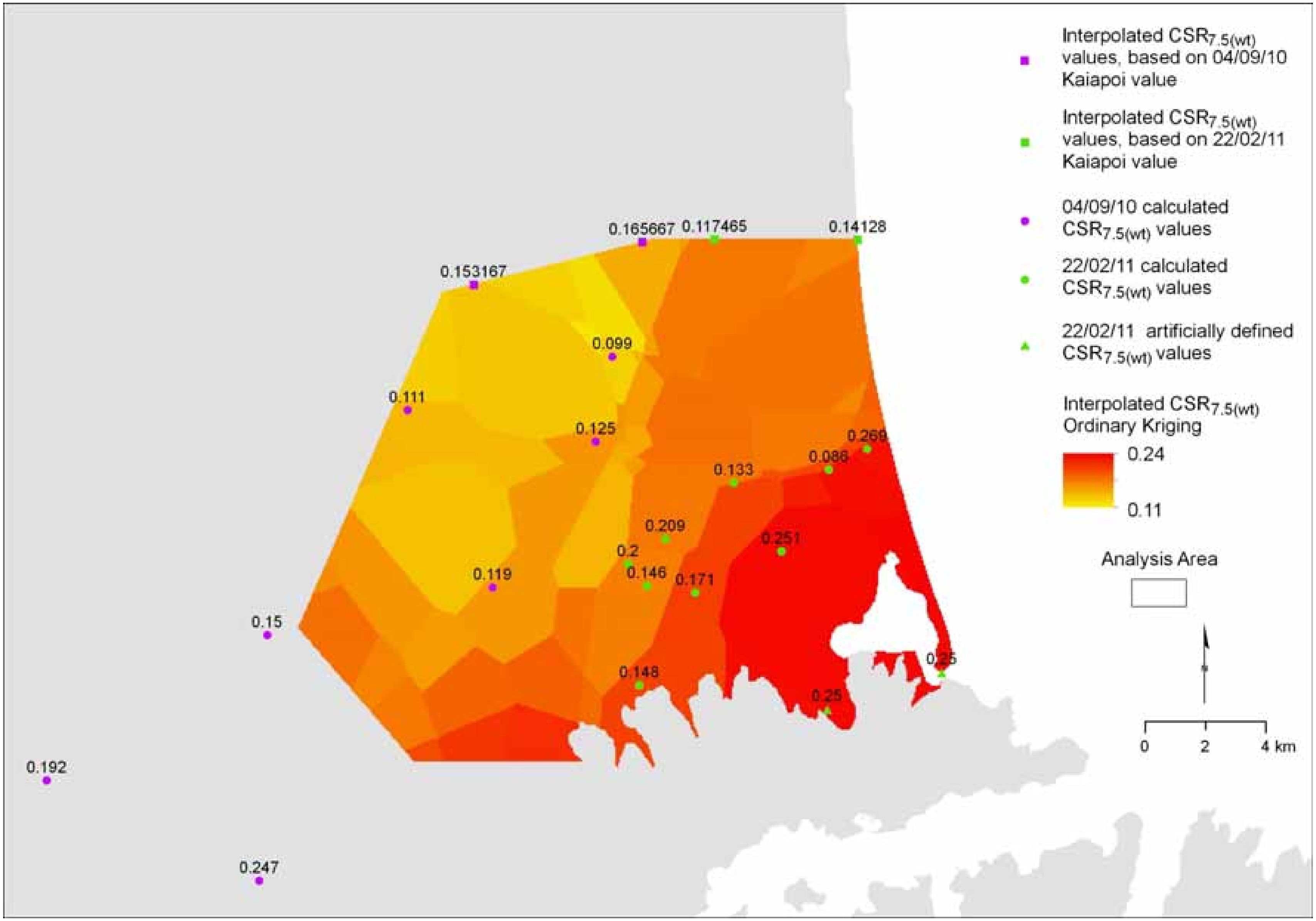


Figure 16. Interpolated $CSR_{7.5(wt)}$ values for the analysis area, using ordinary kriging with a spherical variogram model.

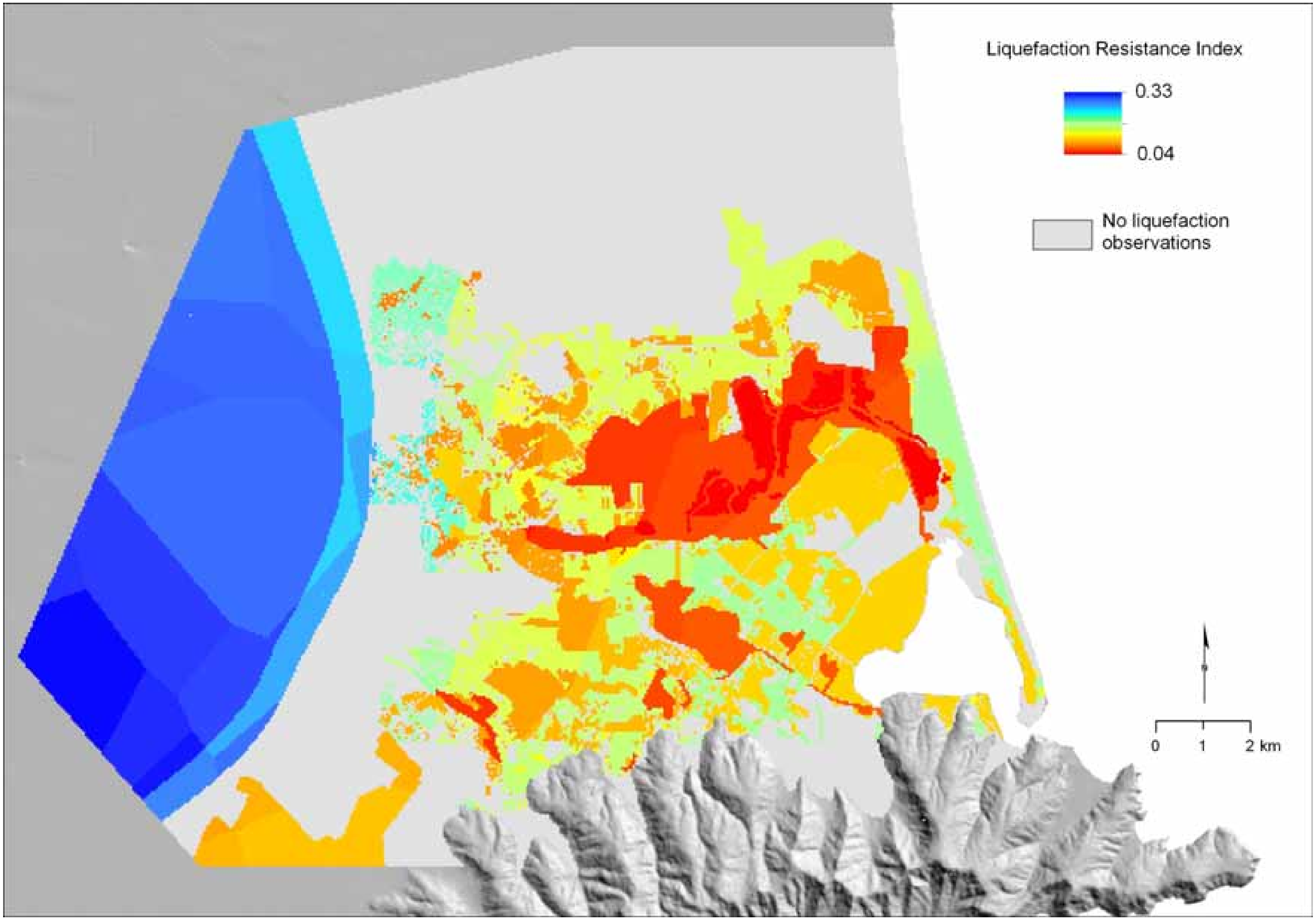


Figure 17. Computed Liquefaction Resistance Index map at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information; derived by multiplying interpolated $CSR_{7.5(wt)}$ values (Figure 16) with the factor of safety (FS) values (Figure 13).

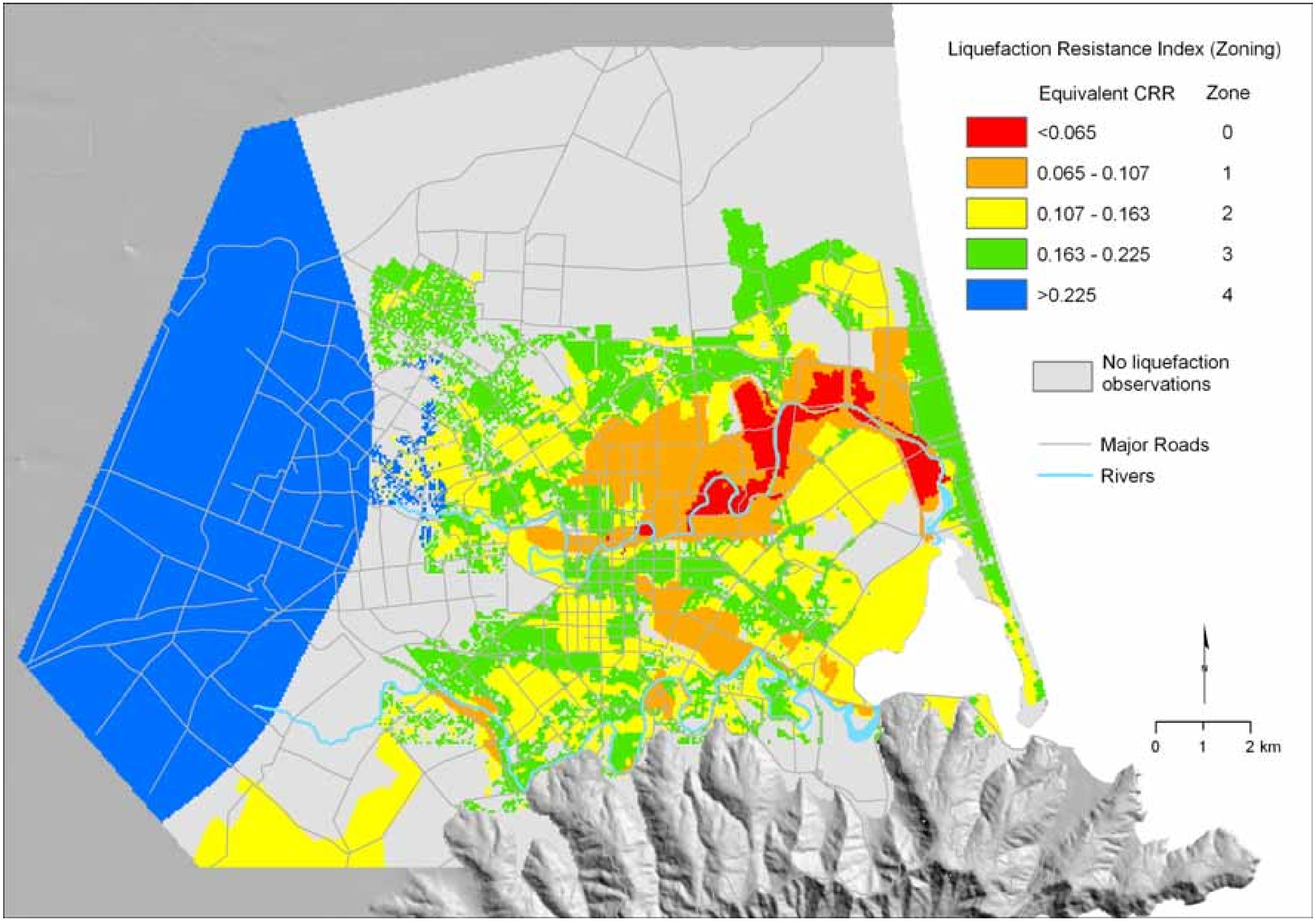


Figure 18. Computed Liquefaction Resistance Index (Zoning) of Christchurch at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information.

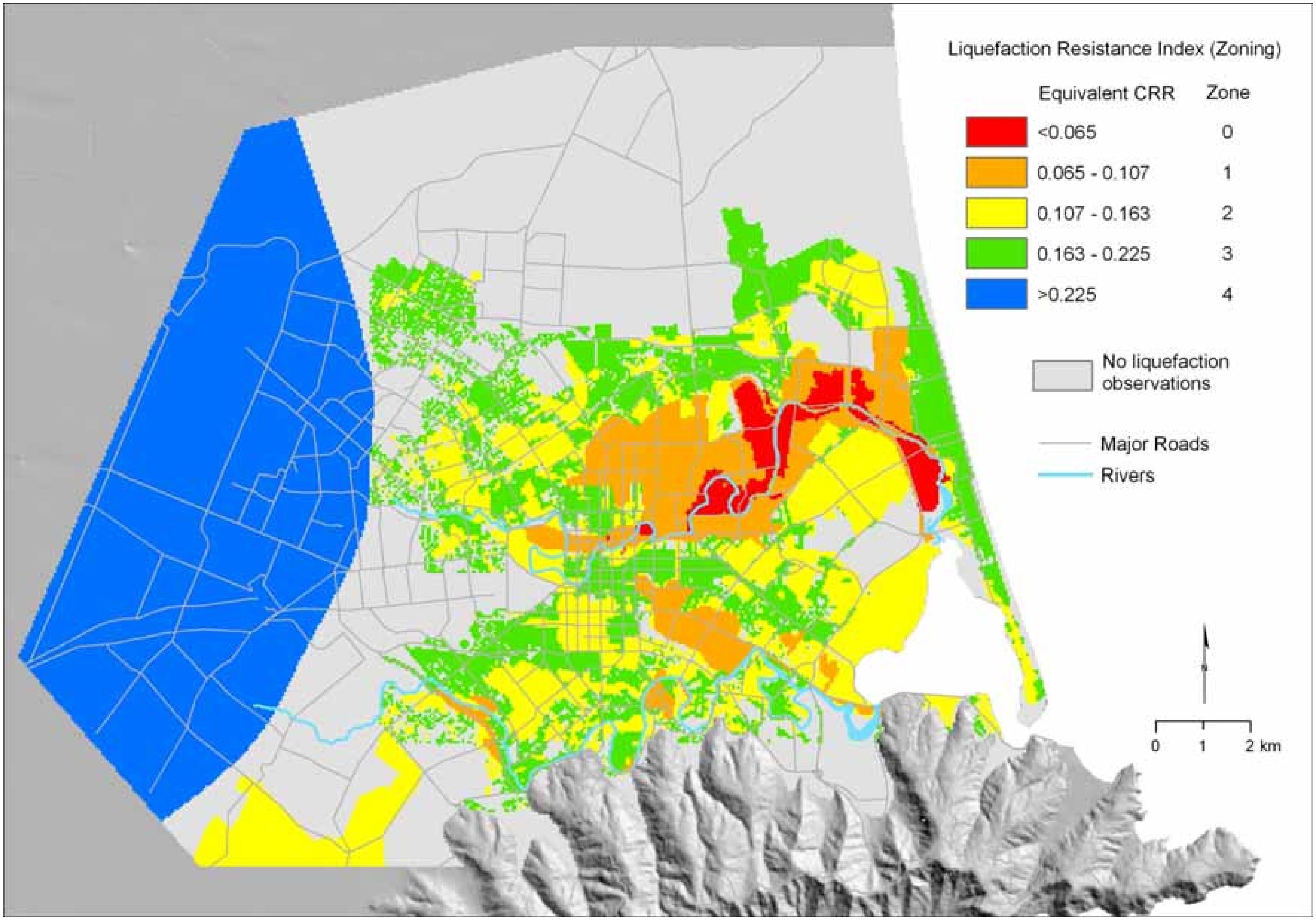


Figure 19. Liquefaction Resistance Index (Zoning) of Christchurch at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information.

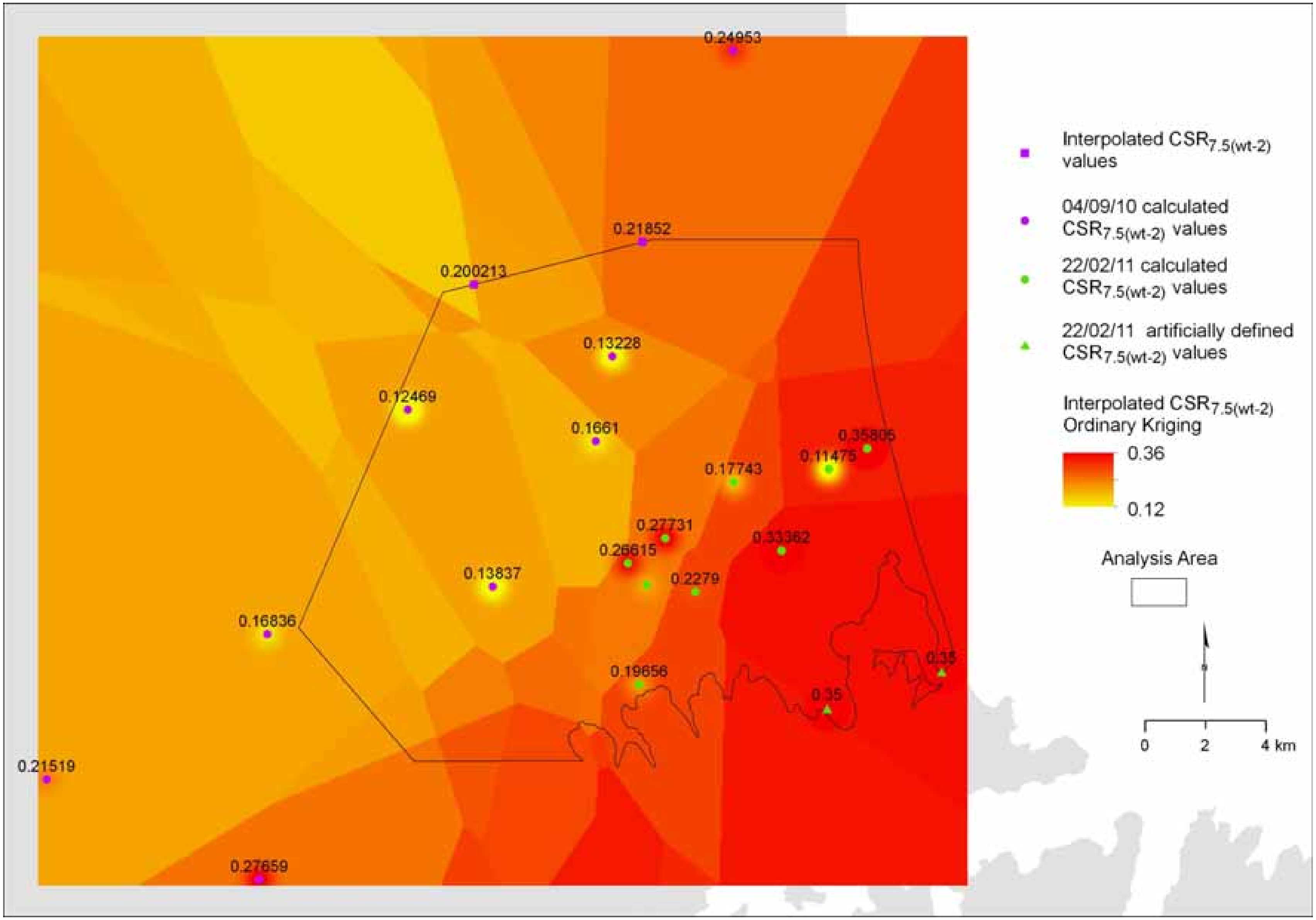


Figure 20a. Interpolated $CSR_{7.5(wt-2)}$ values in the northwest sector of the analysis area using 4 September 2010 $CSR_{7.5(wt-2)}$ = 0.24953 for Kaiapoi North School, and $CSR_{7.5(wt-2)}$ = 0.35 for the point 1 km northwest of Heathcote Valley Primary School and the southern tip of Brighton Spit.

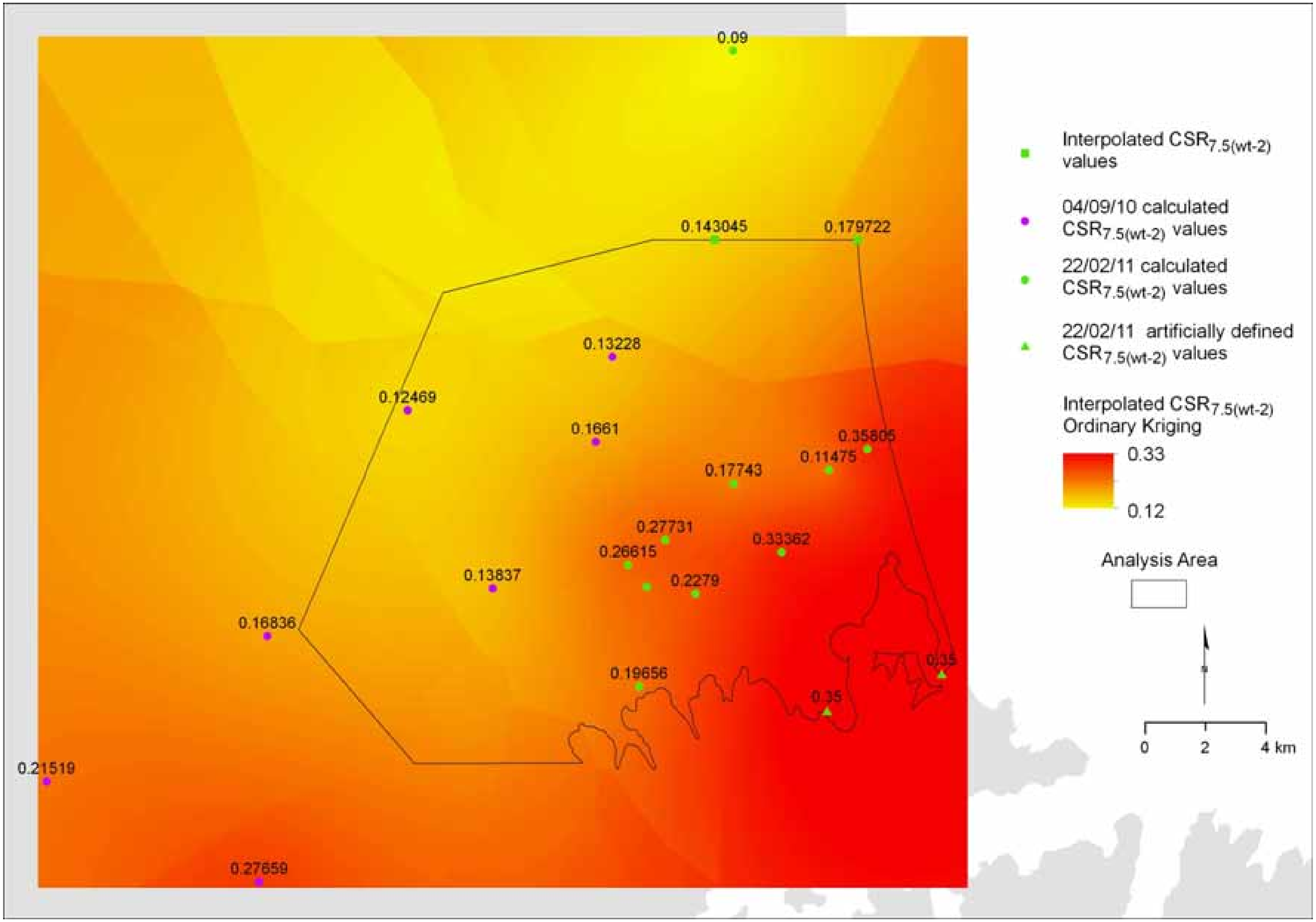


Figure 20b. Interpolated $CSR_{7.5(wt-2)}$ values in the north sector of the analysis area using 22 February 2011 $CSR_{7.5(wt-2)}$ = 0.09 for Kaiapoi North School, and $CSR_{7.5(wt-2)}$ = 0.35 for the point 1 km northwest of Heathcote Valley Primary School and the southern tip of Brighton Spit.

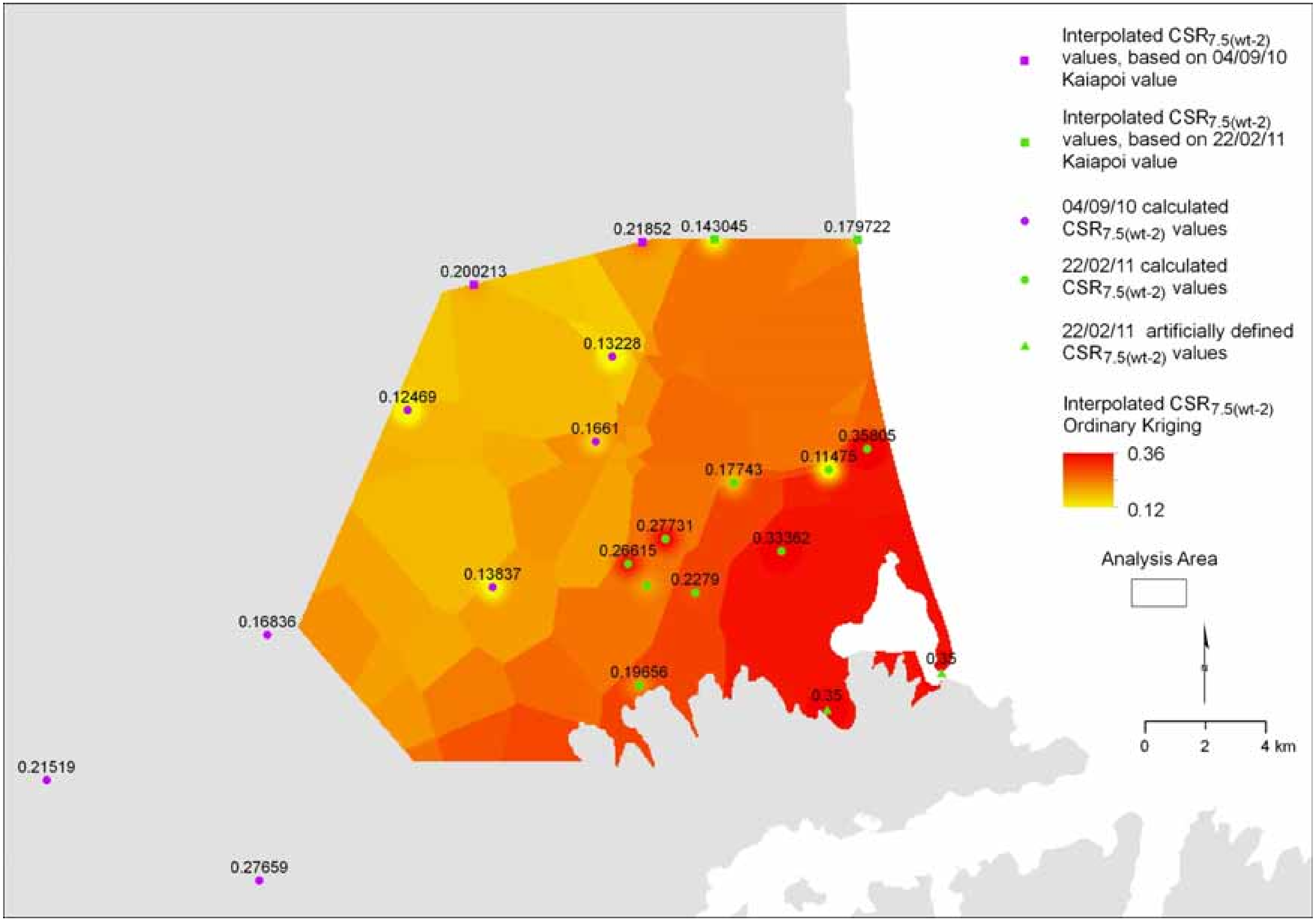


Figure 21. Interpolated $CSR_{7.5(wt-2)}$ values for the analysis area, using ordinary kriging with a spherical variogram model.

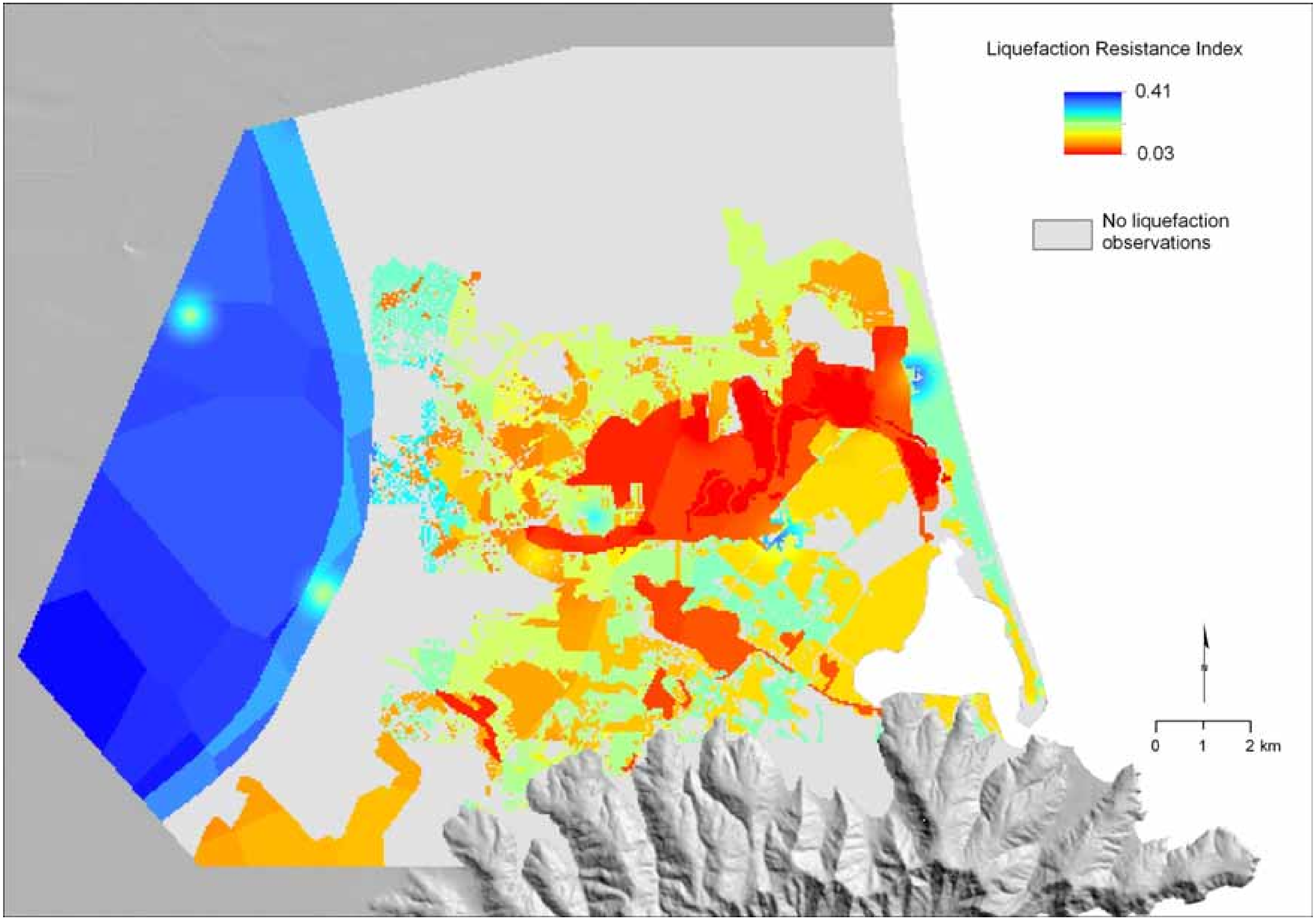


Figure 22. Computed Liquefaction Resistance Index map at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information; derived by multiplying interpolated $CSR_{7.5(wt-2)}$ values (Figure 21) with the factor of safety (FS) values (Figure 13).

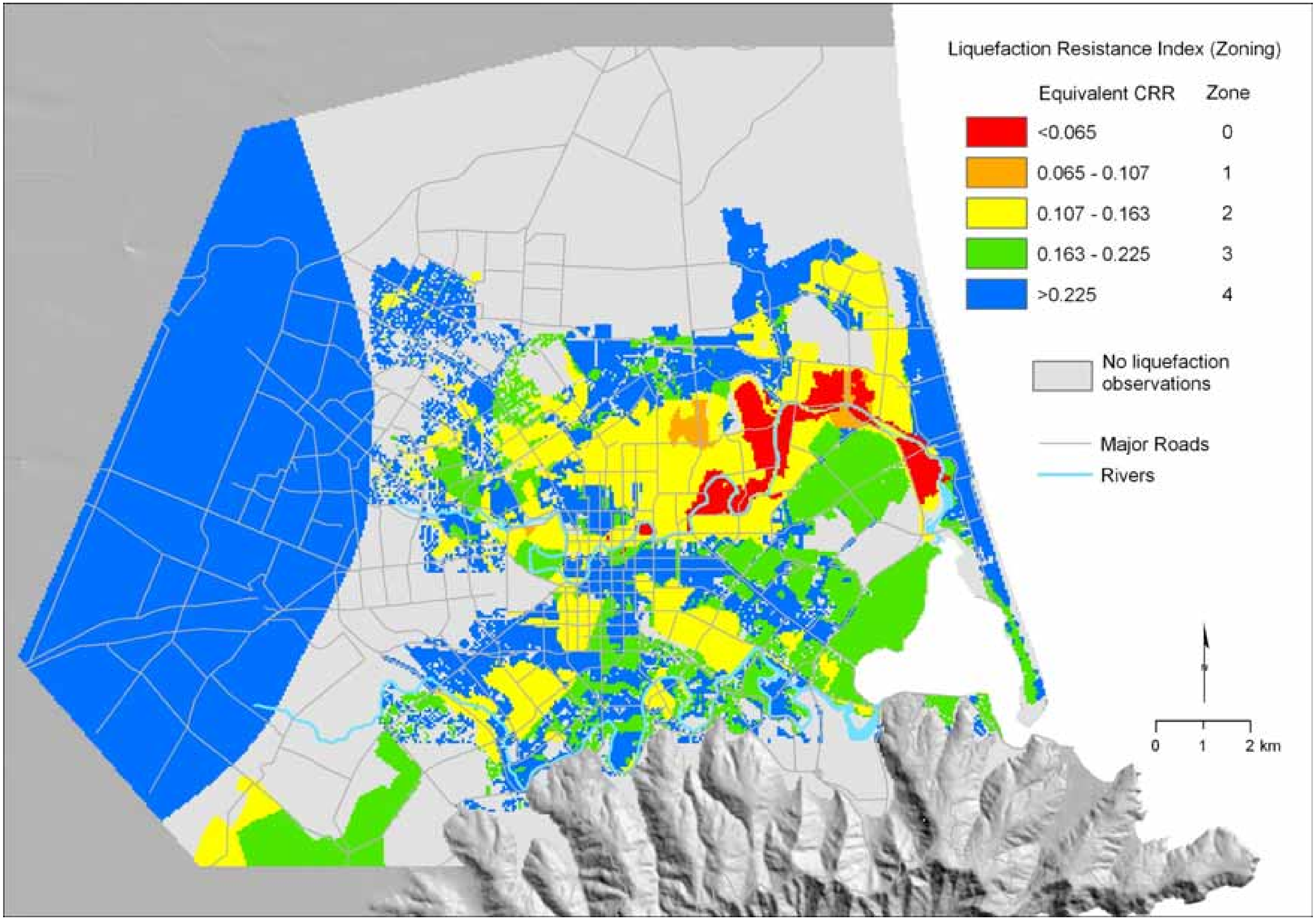


Figure 23. Computed Liquefaction Resistance Index (Zoning) of Christchurch at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information. Derived using interpolated $CSR_{7.5(wt\text{-}2)}$ values.

Appendix E. Pipe Networks Resilience Level of Service (Discussion Document)

Christchurch City Council
City Environment Group

Memorandum

Date: 2nd May 2011 (Adjusted Sept 2011)

From: **BRUCE HENDERSON (TEAM LEADER ASSET MANAGEMENT)**

To: **Mark Christison**
Terry Howe

COPY

Simon Collins
John Noonan
Daniela Murugesh
Howard Simpson
John Walter
Bryan Hickling
Mike Bourke
Yvonne McDonald

Background

As most of you are aware a series of working parties have been set up to support a Government funded research programme on the impact and mitigation of earthquake events. The research has a focus on both long term learnings for the wider benefit, and shorter term improvements that can / should be incorporated into this city's rebuild.

Two subgroups I have become involved with are related to design standards for piped Network infrastructure (retic and pumping stations), i.e. what changes should / could be made to the Councils IDS and CSS documents. Clearly there is a balance between the time to undertake sound research, and the need to incorporate adjustments ASAP so that the rebuild occurring now has the benefit of desired improvements. Thus an iterative process is likely where by some relatively "quick and dirty" changes will be made and then these modified as research and experience outcomes become known further down the track.

The question has been asked, and it is fundamental to design / construction standards, is "what are the Councils acceptable Levels of Service for Major events such as these earthquakes". For example what is a acceptable % of the population to be without service for what period of time"? Indeed what, in such an event, is meant by "the provision of service" (e.g. wastewater removed from properties / streets, or removed and disposed via a treatment plant.)?

Once a level of service for these events is agreed, then the infrastructure can (in theory) be designed and build to achieve this. Clearly there is balance between effort & cost (capital and operational), and benefit to acceptable levels.

The purpose of this memo is to raise this issue and seek feedback so that “feels in the right order” can be established as a starting point.

Community Reaction / feedback.

Post September 2010 Earthquake. Generally the impression gained was that the community was very pleased with the less than a week it took to restore water supply to virtually every property in the city and the boil water notices being removed shortly after that.

For wastewater there was general acceptance that some small areas were still not receiving service (i.e. having to use portaloos) after 8 weeks, but there was also dissatisfaction with the actual residents involved that more permanent conventional solutions were not being implemented about that time.

There was evidence of wider community disquiet that the infrastructure rebuild was not yet in full swing as of February 2011. Inference, perhaps, being that full restoration of service to virtually all properties should be nearing completion by winter of 2011.

Welfare issues appeared to be reasonably able to be catered for (welfare centres, schools, care of elderly etc)

Post February 2011 Earthquake.

The greater impact of this event caused a number of further issues to appear. For example Welfare had to shut Cowles Stadium due to sanitation concerns (lack of water and wastewater service) and struggled to find a satisfactory alternative site near the worst affected areas.

Probably general acceptance that restoration of water supply to virtually all properties occurred within 4 weeks. Some frustration that it took (approx) 10 weeks for the boil water notices to be lifted. Similarly some on going negative feedback with the use of chlorination was being experienced.

Although water supply to properties was restored within a month, the fragile nature of the network required restrictions on usage to be imposed. These definitely included a domestic irrigation ban, but at the same time confusion occurred as to the situation in respect to industrial and commercial (such as car washes) activities resuming normal business. Such limitations have direct effect on business viability, employment, and providing a community a sense of security and return to normality.

There appeared to be general acceptance, and ability, for individuals and small groups of neighbours etc to “make do” and cope for the initial days / weeks. Collecting and carrying water from water tanks etc, and need to make and utilise “make do” toilets for a week or two appeared to be OK and worked.

There has been considerable disquiet over the need for such wide use of portaloos and chemical toilets, particularly at the prospect of them being required for a number of months / years into the future. There is anecdotal feedback that many households are not using chemical toilets (i.e. using their flush toilets) despite specific publicity to do so.

Appears to have been general acceptance of the need to spill raw wastewater into rivers / onto beaches, but equally some concerns that this has restricted the recreational options for children (and

others), especially at a time when "time out to let off steam" was important. The reduced level of treatment at the Treatment Plant appears to have been accepted because of the Ocean Outfall operating and disposing of the effluent a considerable distance off shore. Not sure that this level of acceptance would have occurred if discharge to the Estuary was occurring.

Important to note is that discharging wastewater to rivers / beaches, and the reduced ability to treat wastewater impeded many industries and commercial operations to resume business, with the resultant financial and social impacts. Management of Tradewaste discharges has been a particular issue.

Discussion.

Considerable effort will be expended in the next few years to rebuild these damaged networks.

We have also been advised (verbally) that there is a very high probability that Christchurch will suffer another significant earthquake with liquefaction damage in the next 50 years. This could be from any of the known faults (Southern Alps, Greendale, Port Hills etc) or from yet unidentified faults. Each individual fault has a reasonably low probability (less than 6%) and thus the actual severity and impact of the event on Christchurch's liquefiable soils cannot be precisely determined. However the Sept and Feb events can be used to benchmark the possible or higher end level of impact. And thus be used to review design standards to reasonably mitigate the effects of these future events.

Levels of service can (perhaps) be partitioned as follows:

Water Supply

- ***Domestic Continuity*** - ability to directly deliver (piped) domestic water to Properties.
- ***Quality*** **-** of water delivered (potable or boil water)
- ***Business Continuity*** **-** of water (normal domestic or restricted irrigation and/ or commercial use).

Wastewater

- ***Domestic Continuity*** **-** ability for individual premises, to undertake normal indoor household operations (use flush toilets, shower, use laundry etc) and not result in overflows onto other properties or roadways.
- ***Quality of Service*** – ability to convey wastewater to treatment facilities and treat without environmental impacts (overflows to rivers , beaches etc)
- ***Business Continuity*** **-** ability for Industry and commercial activity to continue or resume normal business.

Given the amount of rebuild work that will occur in the next few years, it would unwise to not consider building further resilience into the networks.

Equally if a significant earthquake event was to reoccur in living memory (say next 30 years) and the city experience similar levels of damage , then the community (arguably) would not be as tolerant or accepting of the service disruptions as they have been with these 2 events.

While the ideal would be to continue without (or very minimal) disruption to provide full normal water supply and wastewater service in an event such as the February 22nd earthquake, this is probably unrealistic, and if possible, would likely come with a significant cost. However it is well worth exploring the development of Levels of Service for recovery that are more likely to be acceptable in the future, and that are achievable. When considering such Levels of Service (LoS) ,

it must been borne in mind that this discussion is centred on LoS for rebuild infrastructure and not for the overall network (LTP) Levels of Service. A LTP LoS would need to be influenced by the remaining existing infrastructure in the network which will continue to be in service through its normal life cycle.

Resilience Levels of Service would however drive the development of operational and Asset Management practices and decision making beyond the initial rebuild. They would also reinforce, or seek review, of the Asset Management Project "Standards Work" such as criticality of classes of assets (e.g. Terminal pump stations, Normal P/S, and Small Catchment P/S, - Trunk mains, Collector mains and local reticulation). These criticality groupings, combined with the Levels of Service, may well result in differing design/ build standards for different asset groups and sub groups.

Draft proposed levels of service.

Given the commentary above the following "straw man" Levels of Service for future network resilience are offered for discussion. Please bear in mind these are design criteria for the network, and not LTP LoS. LTP LoS will need to be more forgiving given the amount of existing infrastructure that will remain. Over time the two could/ should merge to be the same.

Water Supply.

- ***Domestic Service***–90% of properties to have water supply restored to their street connection in quantities sufficient for basic domestic indoor activity (food preparation, showers, toilet flushing, laundry) within 48 (72) hours, 95% within 4 (7) days, 99.5% within 7 (14) days. 95% of (designated) critical community facilities such as hospitals (72) hours.
  Note1: *This LoS is for the public network and does not include supply beyond the network connection point on private land. This is the individual property owners responsibility.*
  Note 2: *Will need to develop definitions or list for these critical facilities*

- ***Quality*** - Boiled water notices not required for 95% of the city (population) after 1 month of initiating event. Ie water testing complies with NZ drinking water standards.

- ***Business Continuity*** - 95% of Industry / commercial activity able to resume normal business activity within 1 month of the initiating event**.**

- ***Other usage constraints*** - (eg irrigation, lower than normal pressure) imposed / caused by event removed within 2 years of the initiating event.

Need to differentiate between natural disaster type events and "Normal" Operation / resource imposed restrictions.

Waste Water

- ***Domestic Service*** - 85% of individual premises, have service restored to their street boundary to enable basic indoor household operations (use flush toilets, shower, use laundry etc) within 48 (72) hours, and not result in overflows onto others properties or roadways. 90% within 7 (14) days, and 99.5% within one (two) month. 95% of (designated) critical community facilities such as hospitals, hospital laundries , rest homes, welfare centres have sufficient supply for their basic needs within 48 (72) hours.
  Note1: *This LoS is for the public network and does not include the discharge pipework on private property before the network point of connection. This is the individual property owners responsibility.*
  Note 2: *Will need to develop definitions or list for these critical facilities*

- ***Quality of Service*** – 80% of collected wastewater is conveyed to the Treatment Plant by the network (without spills to rivers etc) within 2 weeks of event. 90% within 1 month and 99% within 3(6) months. Wastewater treatment and disposal returned to a standard that does not cause significant adverse environmental impact (odour, beach contaminants etc) within 3 months. Full resource consent compliance within 6 (12) months.
- ***Business Continuity*** - 90% of Industry / commercial activity able to resume normal business activity within 1 month of initiating event. 95% within 3 months. 99% within 6 months.

Domestic Service - Disaster Recovery- Design level of Service		
	Water Supply	**Waste Water**
48 (72)hours	90% of Premises	85% of Premises
48 (72) hours	95% Critical facilities	95% Critical Facilities
4 (7) days	95% of premises	n/a
7 (14) days	99.5% of premises	90% of Premises
1 (2) month	n/a	99.5% of Premises

Quality -Disaster Recovery -Design Levels of Service		
	Water Supply	**Waste Water**
2 weeks	n/a	80% of effluent reaches treatment plants
1 month	90% of city receives water conforming to NZDWS	90%
3 (6)months	n/a	99%
3 months	n/a	Treatment (or lack of) not causing significant adverse environmental impacts
6 months	n/a	Full Consent Compliance

Business Continuity -Disaster Recovery -Design Level of Service		
	Water Supply	**Waste Water**
1 month	95% of Industry/ commercial activity able to resume normal business	90% of Industry/ commercial activity able to resume normal business
3 months		95%
6 (12) months		99%

BRUCE HENDERSON

Appendix F. Lessons from 1995 Kobe Earthquake

1. INTRODUCTION

At 5:46 AM on Tuesday January 17 1995, a powerful earthquake struck the Hanshin and Awaji area that includes Kobe city west of Osaka. Its epicenter was the northern end of Awajishima (or Awaji island) with magnitude M7.2 on the Richter scale. More than 5,500 people died and about 35,000 were injured. Some 320,000 survivors were left homeless and were forced to take shelter in school gymnasiums and other public facilities. The earthquake damaged many segments of the infrastructure, including water supply and wastewater facilities, mainly induced by the earthquake forces and ground displacements, notably due to widespread liquefaction of the ground.

2. DAMAGE TO WATER SUPPLY AND WASTEWATER FACILITIES

2.1 Water Supply Facilities (Itoh, 1995).

The damage observed on waterworks and sewage system following the Kobe earthquake was the most severe that Japan has ever experienced in recent years. Damage to water works was observed in over 68 municipal water utilities and 3 bulk water supply authorities, which cover 9 prefectures. The cost of restoring the water supply facilities was about 60 billion yen (roughly NZ$ 1 billion). The number of houses which suffered from water supply cutoff reached 1,200,000 immediately after the earthquake. In Kobe City, 595,000 houses out of 650,000 (or 92%) had their water supply cut off immediately after the earthquake and there were still 41,000 houses which could not get water through their taps by the end of February that year.

2.1.1 *Damage to Water Purification Plants*

Twenty water purification plants were heavily damaged in Kobe, Nishinomiya and Ashiya cities. Out of these, five plants have filtration capacity of more than 100,000 m^3/d. Many open-top basins, like sedimentation basins, suffered damage like opening of expansion joints causing leakage. There was also differential displacement at many expansion joints and cracks were observed at the bottom of some basins. Leakage caused secondary damage. Pumps and mechanical or electric installations placed in galleries were submerged and damaged. Inclined tubes or plates in settling basins fell or were destroyed or displaced by sloshing and many inlet-outlet channels were also damaged. Damaged expansion joints were temporarily mended and pumps and other installations were repaired. Those purification plants supplied water again about a week after the earthquake.

The reasons for damage were classified according to: problems with the structures themselves, earthquake-induced forces and ground displacement. The latter includes sliding at faults, collapse of slopes or landslides, earth slides, liquefaction, ground settlement and others. Generally, the structures were designed based on different design criteria depending on the time they were constructed.

2.1.2 *Damage to Pipelines and Distribution System*

The most damaged parts of the water supply system were the pipelines. A great deal of leakage occurred in transmission lines and distribution networks. In Kobe City, there were 1,962 leakage points in the distribution network and 62,651 leakage points at service pipes as of the end of March 1995. Leakage occurred at pipe breaks, joints or couplings, valves, air valves, fire hydrants and so on. Breakage such as pull-out or crushing at pipe joints was dominant, reaching nearly 50% of the

total breakage in the distribution network. Breakage outbreak per unit pipe length was calculated to range between 0.45 to 1.42 breakage/km in Kobe, Nishinomiya, Ashiya cities and in areas operated by the Hanshin Water Supply Authority. These figures show higher leakage levels as compared to those recorded during past earthquakes in Japan.

Lead-caulked bell-and-spigot connections and mechanical couplings without stops to limit lateral movement caused pull-out breakage, but mechanical couplings with stops with seismic-resistant joints had little damage. Although older concrete or cast iron pipes suffered damage to the pipes themselves and joints, there was no leakage from ductile iron pipes with earthquake-proof connections laid in reclaimed land or supported under bridges. Generally, damage to pipes was concentrated in the geological or topographical conditions of alluvium, artificial islands, reclaimed land, near a fault and an unstable foundation like on fill or near the toe or shoulder of a slope.

Kameda (2000) noted that modern lifeline engineering technologies proved to be promising in various ways. High-performance pipes including steel pipes with modern butt welded joints and ductile iron pipes with seismic joints performed well with no substantial leaks even in areas where extensive liquefaction induced permanent ground displacement occurred, while old cast iron pipe broke easily even under relatively firm soil condition.

2.2 Damage to Wastewater Facilities (Shioji 1995)

2.2.1 *Wastewater Treatment Plants*

Forty three wastewater treatment plants out of one hundred and two which were operating in Hyogo, Osaka and Kyoto prefectures were damaged. In eight of the plants, secondary treatment function was lost. Some plants discharged wastewater with primary treatment, while others did with no treatment. Most plants, however, recovered full scale operation in a short time. Because of water supply suspension, the flow rate of wastewater in these plants was low for a while after the quake, and there was probably no adverse effect on any bodies of water. It was also consoling to note that the earthquake happened during the dry season.

The Higashinada Wastewater Treatment Plant in Kobe City was severely damaged. The plant is located on reclaimed land next to a canal. Because of liquefaction and lateral ground deformation, the buildings and facilities, as well as pressure pipes which connect the treatment plant and the pumping station on the other side of the canal, were damaged leading to loss of treatment functions. Because it was expected that repair work would take a long time, authorities partitioned the canal so that the canal serves as a sedimentation tank. From February 6 to the end of April 1995, wastewater was treated in this temporary sedimentation tank. On March 19, partitions were added to prevent short-circuit. A facility for coagulation was added on March 28 in order to improve effluent quality. On the first of May, the plant recovered secondary treatment functions (activated sludge treatment) through temporary repair work.

An intensive investigation was done on Higashinada Wastewater Treatment Plant. It has been found that pile foundations were broken under several buildings including the administration building and part of the wastewater treatment facilities. In one part of the treatment facility in particular, 40% of the pile foundations were broken.

The characteristics of damage in the wastewater treatment plants are:
- irregular settlement of the channel that seemed to be caused by liquefaction of the ground
- rupture of the pipeline and floating of the pipeline base
- damage in the foundation pile caused by lateral ground deformation
- rupture and cracking of joints of conduit and tanks caused by lateral ground deformation

- rupture of tanks and the piping gallery caused by the lack of elasticity in the joints.

On the other hand, there was only slight damage in wastewater treatment plants which were strengthened against liquefaction, even though other buildings in the area were more severely affected by liquefaction.

2.2.2 *Damage to Sewer Pipes*

Just after the quake, the authorities sent inspectors to check sewer pipes by opening manhole covers and observing the situation inside as the first stage of investigation. This stage was done for several days and it was reported that pipes were broken at about 1600 sites. Emergency countermeasures were taken in order to maintain sewer function where the pipes were jammed by debris, etc.

For the second stage of investigation, TV cameras were used to check the inside of pipes which were so narrow that investigators could not go inside. The total length of damaged pipes was about 162 km or 1866 sites. Investigation with TV cameras took a long time, and where the pieces of destroyed buildings still cover the streets, the investigation was impossible. Therefore, the investigation was not totally completed.

It was reported that most broken pipes were lateral pipes which generally have a shallow covering. Trunk sewers were only slightly damaged though some of them were broken resulting in temporary suspension of water transmission. Damage was found mainly at the connections of manholes and pipes, and the connections of lateral sewers and public inlets. Moreover, some damage in outlet bulkheads was also reported.

3. EMERGENCY ACTIONS

3.1 Emergency Water Supply

In order to supply necessary drinking water to residents, many supply utilities and personnel were sent from all over Japan. An emergency water supply system was organized in cooperation with the Ministry of Public Health and the Japan Water Works Association. 1,027 tank trucks were used from 587 water supply utilities from 44 prefectures including the stricken cities and Self Defense Forces. For the immediate supply of water, more than 300,000 twenty liter plastic containers, 500,000 one to two liter bottles of water and 210,000 polyvinyl bags were distributed. The Marine Self Defense Force and the Maritime Safety Agency offered water tank boats for emergency water supply besides tank trucks.

3.2 Emergency Restoration of Water Supply System

Emergency restoration was executed with nationwide cooperation through the Japan Water Works Association and other related organizations similar to those involved with the emergency water supply. 40 municipalities in 42 prefectures sent 6,208 personnel including engineers, technicians and workers with trucks and spare parts and tools. The distribution network was repaired and installation of temporary water taps for damaged houses was finished within months after earthquake but complete restoration of service pipes needed more time because there were still many damaged houses left as they were.

3.3 Restoration Works for Waterworks

The Ministry of Health and Welfare organized a study team and a committee, and surveyed waterworks in the stricken area. After the survey the committee proposed some items which should be considered during long-term restoration planning. Those are summarized as follows.

1) Because the water supply is a lifeline for citizens, a minimum amount of water should be secured at appropriate sites.
2) Since the water supply is an important function for urban activity, an earthquake-proof water supply system should be reconsidered according to the city restoration program.
3) Telemetering and a remote control system are effective when traffic is congested or information is unavailable. Information transmission, the remote control system, and the power supply system for plant operation should function even during an earthquake.
4) Restoration planning should not only include reconstruction to the original form but the improvement of earthquake resistance e.g. installing flexible joint, anti-pullout joints or reinforcement of concrete structures to make them water-proof.

4. LONG-TERM RESTORATION METHODS

Following the reconnaissance works, the Japan Society of Civil Engineers (JSCE) put out a proposal for the earthquake resistant design of civil engineering structures (JSCE, 1996). The proposal mentioned that trunk lines for lifeline systems (such as water, sewerage, electricity, gas, and telecommunications) must be designed to maintain functionality after a Level II earthquake, taking into account the topography, ground conditions, and the city layout in the vicinity. If this is difficult for economic reasons or because of ground conditions, continued functionality (or rapid restoration) after a disaster should be ensured by selecting the most appropriate route, adopting a multi-route system, using a block system, or implementing some alternative measures.

4.1 Repair of water supply pipes

The Kobe Municipal Waterworks Bureau adopted the following aseismic design guidelines for future earthquakes (Fukuda 2002):

- to localize earthquake damage as much as possible
- to easily repair damages
- to provide measures to prevent secondary disasters following the earthquake.

4.1.1 *Earthquake-resistant design of pipes*

Based on the experience from this earthquake, the use of earthquake-resistant pipe with excellent, earthquake-proofing capability has been adopted. In addition to replacing old pipes, the existing pipes were also made earthquake-proof. By considering the emergency water supply activity following the earthquake, the earthquake-proof pipes were laid out at spacing of 500 m along the route toward the designated disaster prevention centres. Currently, the percentage of earthquake proof pipes has increased from 9% before the quake to 20.5% (Miyane and Kawahara, 2002).

After the Great Hanshin earthquake, many types of earthquake-resistant pipes have been developed. Some of these are shown in Figures 1-5. It should be mentioned that after discussion with several geotechnical researchers in Japan, it appears that no change was adopted in terms of laying out the pipes to prevent liquefaction-induced damage. Attempts to confirm this with the water supply authorities failed; however, there seems to be a consensus that because of the wide network of pipes that are being laid out and the severity of liquefaction observed during the Great Hanshin Earthquake, there is not much that can be done to improve pipe layout procedures.

Figure 1. Use of reinforced-cement asbestos pipe *(*http://www.sakatsuru-suido.or.jp*)*

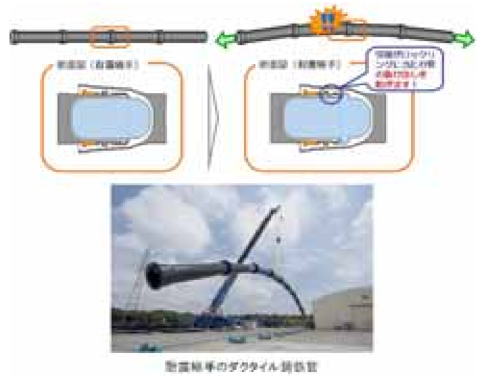


Figure 2. Seismic resistant joint for ductile iron pipe or welded steel pipe. *(*http://www.city.osaka.lg.jp*)*

.

Figure 3. Epoxy resin powder coating the inner surface of a pipe *(*http://www.nichu.co.jp*)*

.

Figure 4. Joint-protected coupling of cast-iron pipe *(*http://www.nichu.co.jp*)*

.

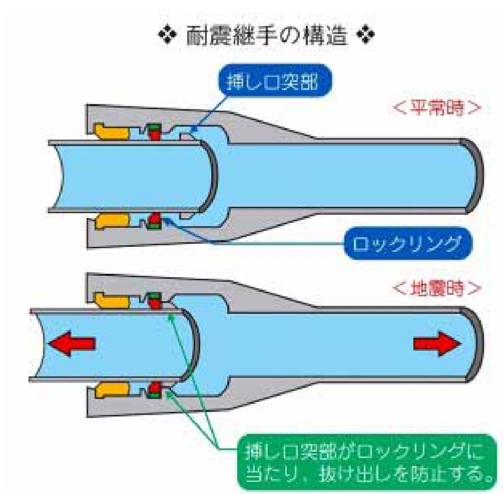
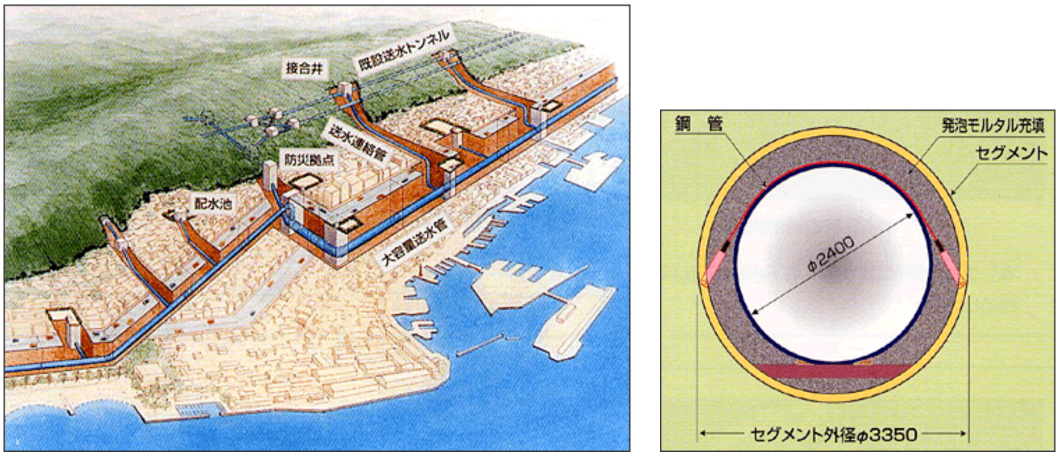


Figure 5. Structure of joint in an earthquake-proof water supply pipe. *The joint will be difficult to be pulled out during earthquakes (*http://www.city.hachioji.tokyo.jp*).*

4.1.2 *Large-Capacity Transmission Main*

In addition to two water supply tunnels that pass through Rokko Mountain, large-capacity transmission mains that pass through deep underground in urban areas was developed (see Figure 6). The idea is that the seismic risk can be distributed by dividing the water supply route into the urban area and through Rokko Mountain. Moreover, because of their high earthquake resistance and large capacity for storing water, emergency water supply during and at the early stage of a disaster would not be a problem (Miyane and Kawahara, 2002).

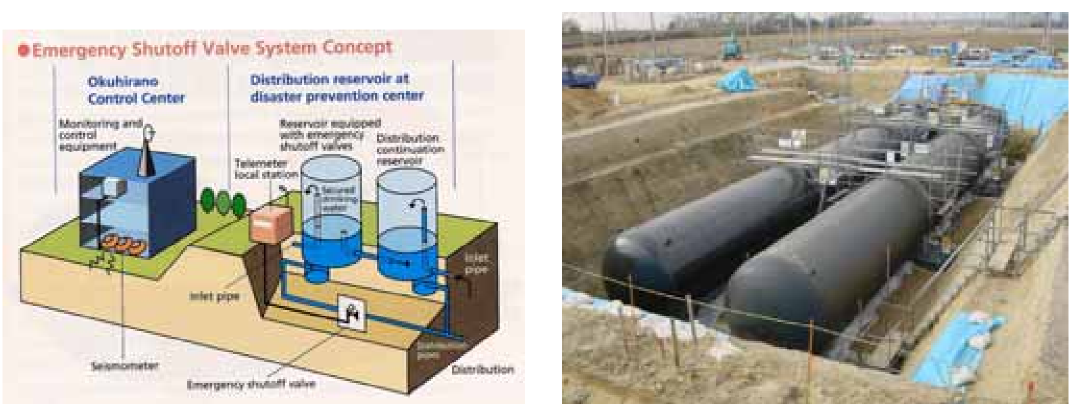


(a) (b)

Figure 6. (a) Schematic diagram of the large-capacity transmission main; (b) cross-section of the transmission main *(*http://www.city.kobe.lg.jp*)*.

4.1.3 *Emergency storage system*

Availability of drinking water is ensured by closing the outlet in one of two distribution reservoirs during a disaster, and using it as depot for water supply trucks and as temporary water supply base (see Figure 7). Supply of water is kept through the other distribution reservoir, and it will be used for extinguishing fires etc. As of the end of Fiscal Year 2000, 30 systems have been completed out of the 47 locations scheduled to be maintained in the whole city (Miyane and Kawahara, 2002).



(a) (b)

Figure 7. (a) Schematic diagram of the emergency storage system; (b) view of storage facility *(Mizukuchi 2008).*

4.2 Repair of Sewage facilities

Following the earthquake, "Sewer Ordinance Amendment" is now required for the earthquake resistant design of sewerage pipeline facilities. The Ministry of Land, Infrastructure and Transport (MLIT) notified all sewerage companies to implement the "Earthquake Emergency Sewer Improvement Project." In this project, for example, pipelines connecting the treatment plant to the refuge shelters and disaster prevention centers, as well as pipelines buried under emergency tracks and roads, should be earthquake-resistant within five years after implementation of the plan. The design concepts are described in the "Guide to Aseismic Emergency Sewer Improvement Plan (Draft)" published by MLIT, as well as in "Guidelines and Description for Aseismic Measures in Sewerage Facilities" and "Earthquake-resistant Sewer Manual" published by the Japan Sewage Works Association. The second publication was re-edited in 1997 following the damage caused by the Hyogoken-Nambu earthquake, and again revised in 2006.

4.2.1 *Design Standards for Sewer Facilities*

As a result of the damage observed during the Hyogoken Nambu earthquake, the Kobe City Construction Bureau recommended a set of design standards for sewer facilities. These standards were further refined, incorporating lessons learned from subsequent major earthquake events that occurred in other parts of the country. The current design standards, which were enforced in the city from 1 June 2011, are illustrated in the Kobe City website (http://www.city.kobe.lg.jp/life/town/waterworks/sewage/gesuidosekkeihyojunzu.html). The documents consist of the following

- Cover page
- Symbol / legend used
- Standard reference
- Pipe laying method
- Manhole installation
- Frame building
- Standard diagram for pump installation
- Pipe mounting installation
- Industrial pipe rehabilitation
- Temporary works
- Supplementary information
- Manhole cover
- Appendix (reference)

A typical example of pipe laying standard is shown in Figure 8, while that for manhole is illustrated in Figure 9.

4.2.2 *Pipe Connection and Manholes*

One of the problems seen during the Kobe earthquake was the buoyant rise of manholes, resulting in damage to sewer pipes. In order to prevent manholes from rising as a result of the liquefaction of foundation ground as well as damage to connecting pipes, several methods have been developed, and some of these are shown in Figure 10. These methods are explained briefly below.

(14) Earth drain method: Artificial drain consisting of high permeability soil is placed around the manhole using a specialized machine.

(15) Anchor wing method: The manhole is anchored to the bottom unliquefied layer by a frame structure (called wing).

(16) LAM Method: The manhole is anchored to the bottom unliquefied layer by a single rod attached to the bottom of the structure.

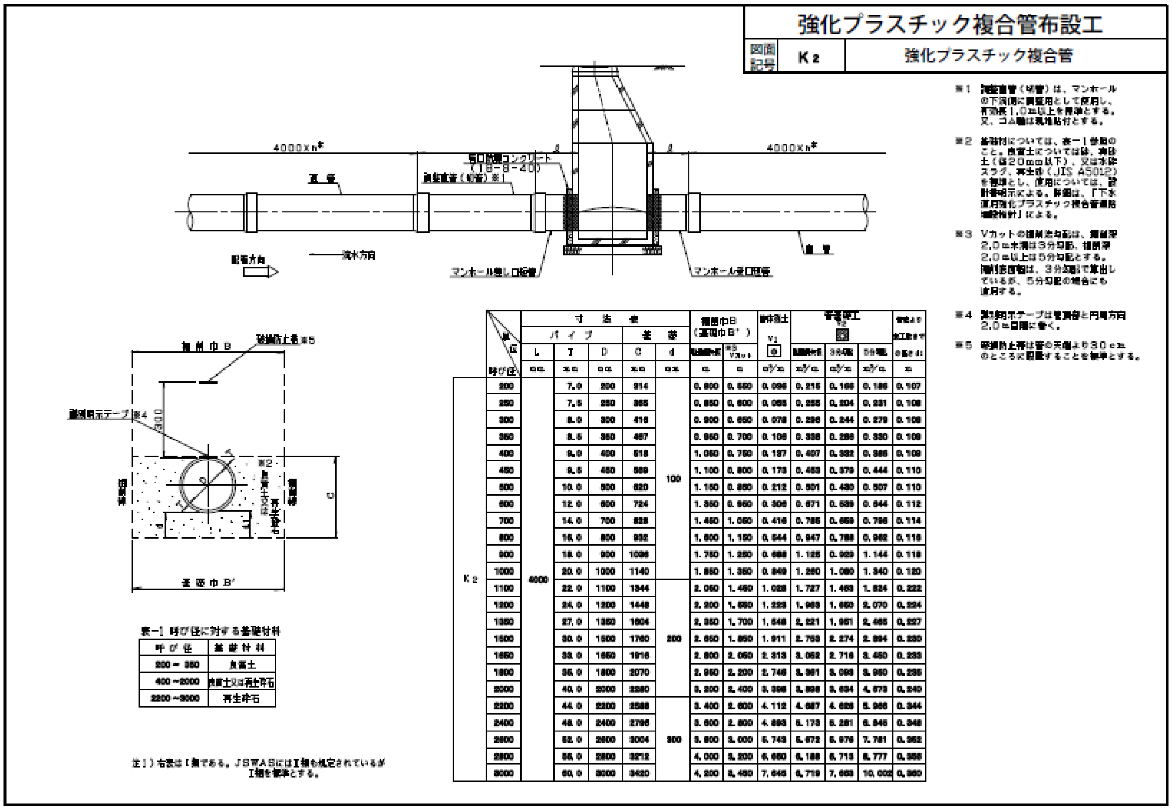


Figure 8.Typical example of design standard for laying sewer pipes (http://www.city.kobe.lg.jp).

.

(17) Safe Manhole Method: Tubes are installed within the manhole and near the joint to drain excess pore water pressure generated during liquefaction.

(18) Anti-float method: A heavy base plate is placed at the bottom of the manhole to prevent uplift.

(19) Asesimic method for existing manholes: A special cutting machine is used to cut the edge of the manhole and the existing pipe by cutting the manhole wall and flexible joint and elastic sealant are installed at the connection.

(20) Aseismic improvement method for existing pipe: Using a chainsaw-type cutting machine, the pipe joint is cut and a light fitting consisting of rubber and the steel-made is placed to make the joint flexible.

(21) Prevention of uplift using manhole flange: A convex-shape material is placed on the outer part of the manhole, and a weight is placed to increase resistance against uplift.

(22) Float-less method (non-excavation type): Excess pore water pressure generated by earthquake is drained out.

(23) Magma lock method: the impact of earthquake-induced displacement is decreased using a special flexible joint and magma lock.

(24) Hat ring method: A cylindrical ring block is placed on existing manhole to prevent uplift.

(25) Wide safety pipe method: Tubes installed inside the manhole dissipates the excess pore water pressure. Moreover, underground water is not taken by installing a reverse-action valve in the manhole pipe.

(26) “Mr. Aseismic” (Taishin-ippatsu kun) method: New pipes are installed to add seismic capacity to old structures with worn-out pipes and manholes.

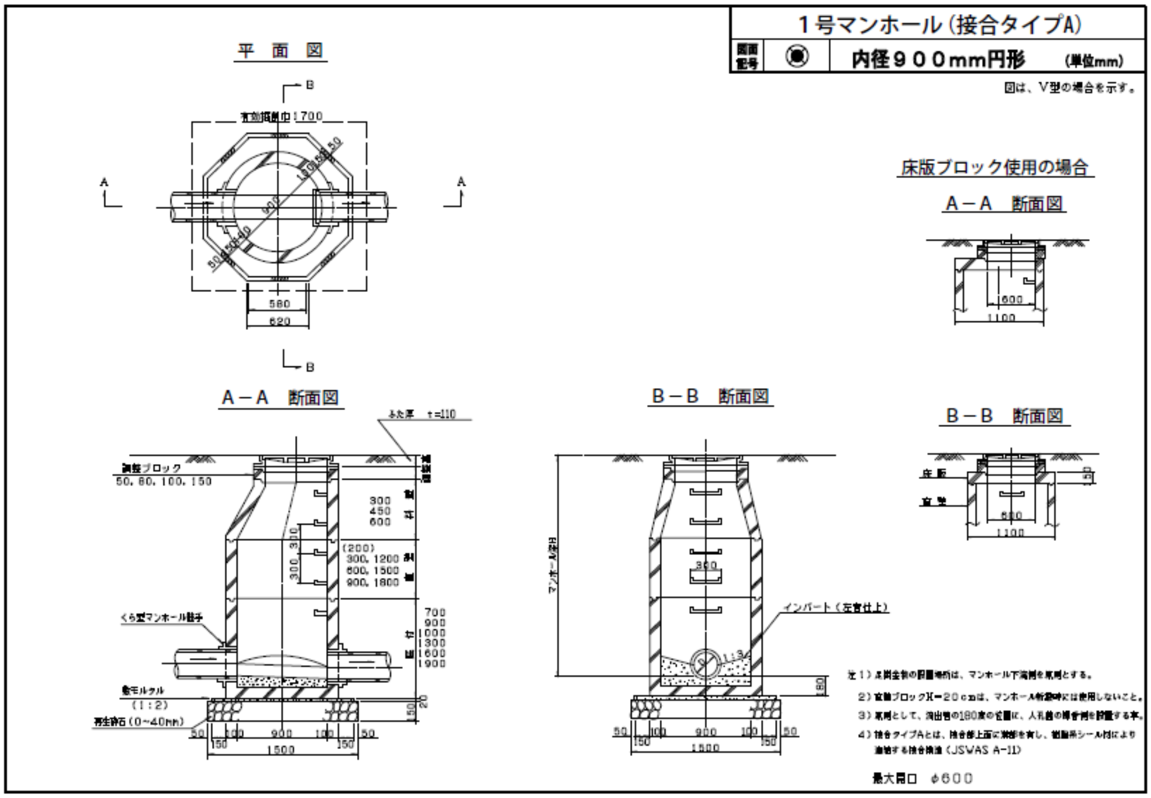


Figure 9.Typical example of design standard for installing manhole (http://www.city.kobe.lg.jp)

.

REFERENCES

Fukuda, H. (2002). "Damages in the Kobe earthquake & Japanese seismic design guidelines of drinking water supply facilities", *International Course on Microzonation and its Applications in Disaster Mitigation*, Lima Peru.

Itoh, M. (1995). "Hyogo Nanbu Earthquake - damage to waterworks and its restoration," *INTEP Newsletter*, No. 8, June issue (in Japanese).

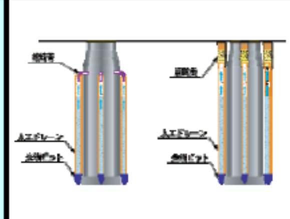
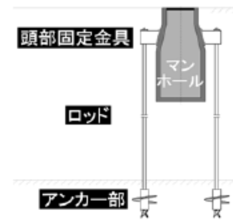
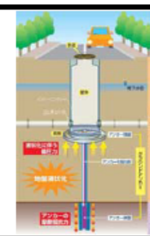
Japan Society of Civil Engineers (1996). *Proposal on Earthquake Resistance for Civil Engineering Structures*, Special Task Committee of Earthquake Resistance of Civil Engineering Structures.

Kameda, H. (2000). "Engineering management of lifeline systems under earthquake risk," *Proc. 12th World Conference on Earthquake Engineering*, 17pp.

Miyane, K. and Kawahara, D. (2010). "Restoration of water in the Great Hanshin earthquake," *January 17th – Keep the Hanshin-Awaji Earthquake in Our Memories* (in Japanese)

Mizukuchi, K. (2008). "Aseismic measures for water network," *8th Annual General Meeting of the Institution of Professional Engineers*, Japan Kinki Chapter (in Japanese).

Shioji, K. (1995). "Hyogo Nanbu Earthquake - damage to sewerage systems," *INTEP Newsletter* No. 8, June issue (in Japanese).

項目 ＼ 工法名	アースドレーン工法	アンカーウイング工法	LAM（アンカー定着）工法
工法の概略図			
工法概要	マンホール周囲に、透水性の高い人工ドレーンを専用機械にて地中に埋設することにより、マンホール周面地盤の液状化を防ぎマンホールの浮上を抑制する工法である。	定着層(非液状化層)に根入れした翼付アンカー体の引き抜き抵抗力によりマンホールの浮上を物理的に拘束する。1号マンホールに限らず大型マンホールや場所打ちマンホールへも対応可能。	液状化によりﾏﾝﾎｰﾙに作用する浮き上がり力に対して、非液状化層中に造成するｱﾝｶｰ体部の付着抵抗と、ｱﾝｶｰﾃﾝﾄﾞﾝの引張り力により対抗し、ﾏﾝﾎｰﾙの浮上りを防止する。
メカニズム	マンホール周囲に埋設された人工ドレーンにより、地震時に発生する液状化現象の原因である過剰間隙水圧を速やかに消散させ、地盤の液状化を防ぎマンホールの浮上を抑制する。	マンホールに生じた浮上力が頭部固定金具、ロッドを介して翼付アンカー体に伝わり、翼付アンカー体の引き抜き抵抗力でマンホール浮上力に抵抗する。	ｸﾞﾗｳﾝﾄﾞｱﾝｶｰは、ﾏﾝﾎｰﾙ床版部を頭部とし、液状化層を貫通し、非液状化層にｱﾝｶｰ体部を造成する。地盤の液状化による浮上り力に対して、ｱﾝｶｰ体部の付着抵抗力とｱﾝｶｰﾃﾝﾄﾞﾝの引張り力により対抗する。

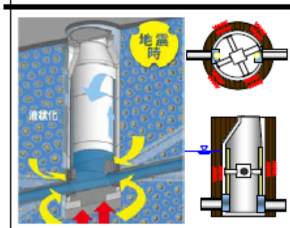
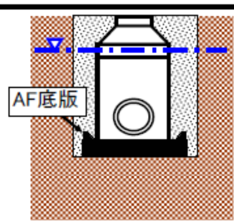
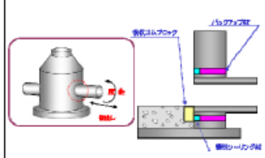
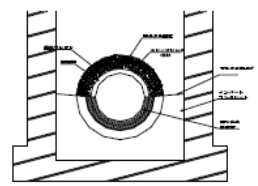
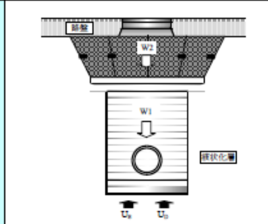
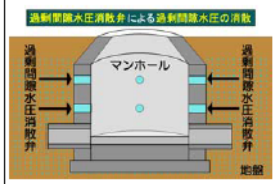
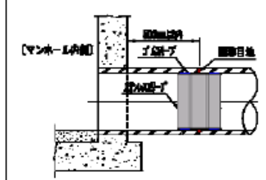
安心マンホール工法	アンチ・フロート工法	既設人孔耐震化工法（ガリガリ君）	既設管の耐震改良工法
			
ﾏﾝﾎｰﾙと本管接合部にろ過器及びその上部に誘導管を設置し、液状化に伴う過剰間隙水圧をﾏﾝﾎｰﾙ内に排水するとともに、ろ過装置の工夫により管口の可能性を確保するものである。	AF底版は、液状化によるマンホールの浮上を抑制する新設マンホール用浮上対策製品である。標準掘削幅内に設置が可能で、マンホールの底版をAF底版に置き換えるだけのとてもシンプルな方法である。	専用の切削機により、マンホールの壁を管外周部に沿って切削することでマンホールと既設管の縁を切り、切削した溝に土砂の流入を防止するバックアップ材、屈曲性に対応する弾性シーリング材を充填し、マンホール接続部の耐震化を行う工法です。	既設管の耐震改良工法は、人孔内面にチェーンソー式切断機を設置し、既設管外周部人孔壁を切断削孔後、人孔壁間隙にゴム及び鋼製スリーブからなる耐震用可とう継手を設置することにより、人孔と管との接続部の耐震化を図る工法である。
揚圧力が(自重+周面摩擦力)より大きくなると浮上が発生する。本工法は、ろ過器により揚圧力を低減するとともに、マンホール躯体を起振することで周面摩擦力の増強を図る。	底版重量の増加及び底版張出部に載荷する土荷重により、マンホールの重量化を図り、浮上を抑制する。また、底版形状をテーパー形状としたことで、くさび作用により周辺地盤の変形を抑え液状化を抑制する。	マンホールと既設管の接続部(剛接続)の縁を切り、接続溝に柔軟な材料を挿入・充填し、接続部を柔軟な構造に改良する。接続部を柔軟な素材に改良することにより、地震時の屈曲や突出しに対応(屈曲角:1.0°、抜出し(突出し)40mm)	人孔と管渠との接続部にレベル2地震動に対応する耐震用可とう継手を設置することにより地震時における管の水平移動・屈曲に追随し管口の破損を防ぐ。

Figure 10. Methods to prevent liquefaction uplift of manholes and to secure connecting pipes (http://www.suikon.or.jp)

項目 ＼ 工法名	浮上防止マンホールフランジ工法	フロートレス工法	マグマロック工法
工法の概略図			
工法概要	マンホールの外周部に凸型形状の部材を設け、浮上抵抗の増加と同時に重量体金枠を設け、内部に重量体を充てんしてマンホール底面に作用する揚圧力と吊り合せ、浮上防止を図る工法です。	フロートレス工法(非開削による既設マンホール浮上抑制工法)とは、地震時における地盤の液状化によるマンホールの浮上を抑制し、下水の流下機能を確保するとともに被災地での応急対策活動を行う緊急車両等の道路通行を確保することを目的とした工法である。	管きょに誘導目地とマグマロック(φ700以下は「ミニマグマ」と云う)を取付けておき、地震等の衝撃を受けた時に、その衝撃を誘導目地に誘導しひび割れを発生させることにより他への影響を最小限に抑える。誘導目地に沿って、ひび割れが発生した箇所にはマグマロック(ミニマグマ)が管内への地下水や土砂の流入を防止する。
メカニズム	揚圧力に対してマンホール本体および付加した重量体の重量によって抵抗する。	地震時に間隙水圧が上昇(過剰間隙水圧の発生)することで、地盤が液状化する。この過剰間隙水圧を瞬時にを消散させることで、液状化現象の発生を抑え、地盤とマンホールのせん断抵抗を保持させることによりマンホールの浮き上がりを抑制する。	誘導目地にひび割れが生じることによりマンホールと管きょの挙動が分離され、損壊の範囲を低減させる。

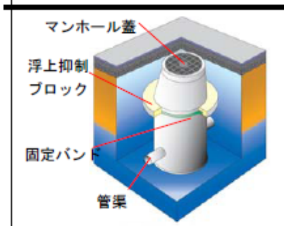
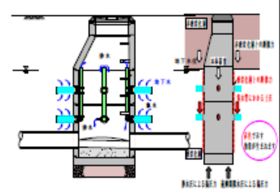
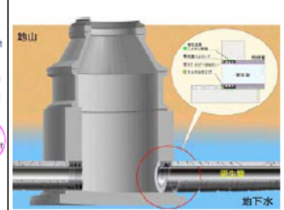
ハットリング工法	WIDEセフティパイプ工法	耐震一発 くん
		
円形の浮上抑制ブロックを既設マンホール外側に設置して、マンホールの浮上を抑制する。ブロックとブロック上の土砂重量は、常時はマンホールに荷重を与えず浮上時のみ作用	マンホール内部から設置した集水管によって，マンホール周囲の過剰間隙水圧を消散する。 また、マンホール内部には逆止弁と誘導パイプを設けることで、常時は地下水を取り込まず、地震時にのみ地下水を排水する構造としてある。	老朽化した既設管を更生した管とマンホールとの接続部を対象とし，更生管の抜出し・突出し・屈曲等に対応する耐震性能を付加させるために，更生前に非開削でフレキシブルな構造に改造を行う耐震化工法である。
○新設、既設どちらでも施工可能 ○マンホールに影響を与えない ○地震動による慣性力の増大はない ○簡単施工、しかも低コスト	地震によって発生したマンホール周辺地盤の過剰間隙水圧を消散し，マンホール内部に排水する。その結果，液状化によるマンホール周辺の摩擦力低下を抑制することと集水管に作用する土圧により，マンホールの浮上を抑制する。	マンホール内から，マンホール壁厚内の既設管の一部（管厚部のみ）を，専用の切削機により地山に貫通させることなく切削除去を行う。当該部分に，弾性と水密性を有した耐震ゴムリングを設置し，耐震ゴムリングの周囲空隙部に弾性湿潤エポキシ樹脂を充填する。 管きょの上下流のマンホール接続部に本技術を施工した後，更生管を構築することにより，更生管とマンホール接続部の耐震化を可能とする。

Figure 10. Methods to prevent liquefaction uplift of manholes and to secure connecting pipes (continued) (http://www.suikon.or.jp)

Appendix G. Manhole Uplift Calculations

Appendix: Liquefaction-induced uplift assessment

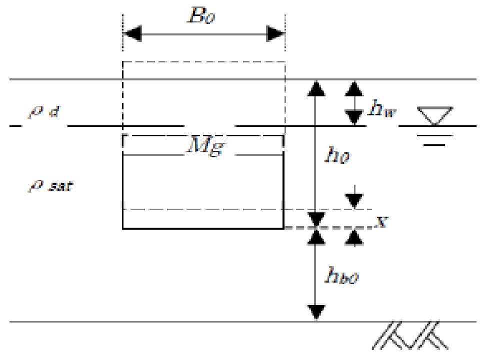
The liquefaction-induced uplift assessment documented in the report text has been implemented in an Excel spreadsheet for design.

The input requirements for this spreadsheet comprise soil material and geometrical parameters, as well as ground motion parameters. These required parameters are denoted by yellow boxes, while all other boxes contain parameters calculated from the input parameters.

The spreadsheet has a total length of three pages. The first page contains the input parameters and results of the assessment calculations, while the second and third pages contain the calculations. An example output of the first page of the spreadsheet is shown below.

Simplified Calculation of liquefaction uplift

Based on: 1) Koseki et al "Uplift behaviour of underground structures caused by liquefaction of surrounding soil during earthquake" Soils and Foundations (1997)
2) Sasaki T and Tamura "Prediction of Liquefaction-Induced Uplift Displacement of Underground Structures"



Input variables of soil:

ρt = ρd =	1.7	density of soil above the WT (t/m3)
ρsat =	1.8	density of saturated soil (t/m3)
hw =	0	depth of WT (m)
h0 =	1.5	depth to bottom of structure (m)
hb0 =	3.5	depth between base of structure and bottom of liq layer (m)
B0 = b =	1.3	width of structure (m)
M =	1	mass of structure + overburden soil per unit length (in t/m)

	Lower	Upper	
qc (MPa)	2	4	Lower and upper values of CPT qc value at base of structure

Input variables of earthquake:

Mw =	7	earthquake moment magnitude
Rrup =	20	distance from fault to site (km)
amax =	0.4	Peak acceleration (g)

M_neutral	2.7	Required mass for FS(uplift)=1
g =	9.81	acceleration of gravity (m/s2)

Results:

	Lower	Upper
Factor of safety against liquefaction :	0.21	0.49
Factor of safety against uplift :	0.37	
Expected post-liq duration of shaking:	17.2	11.1
Expected uplift displacement (m):	0.50	0.05