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# Liquefaction Impacts on Pipe Networks

**Misko Cubrinovski**
**Matthew Hughes**
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**Rolando Orense**
**Thomas O'Rourke**

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# 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*



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# 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.

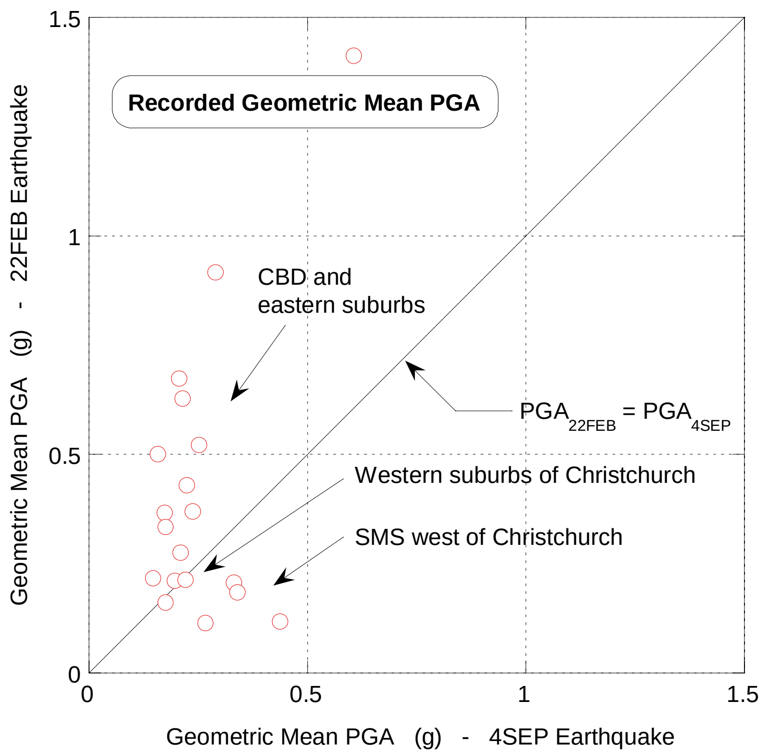
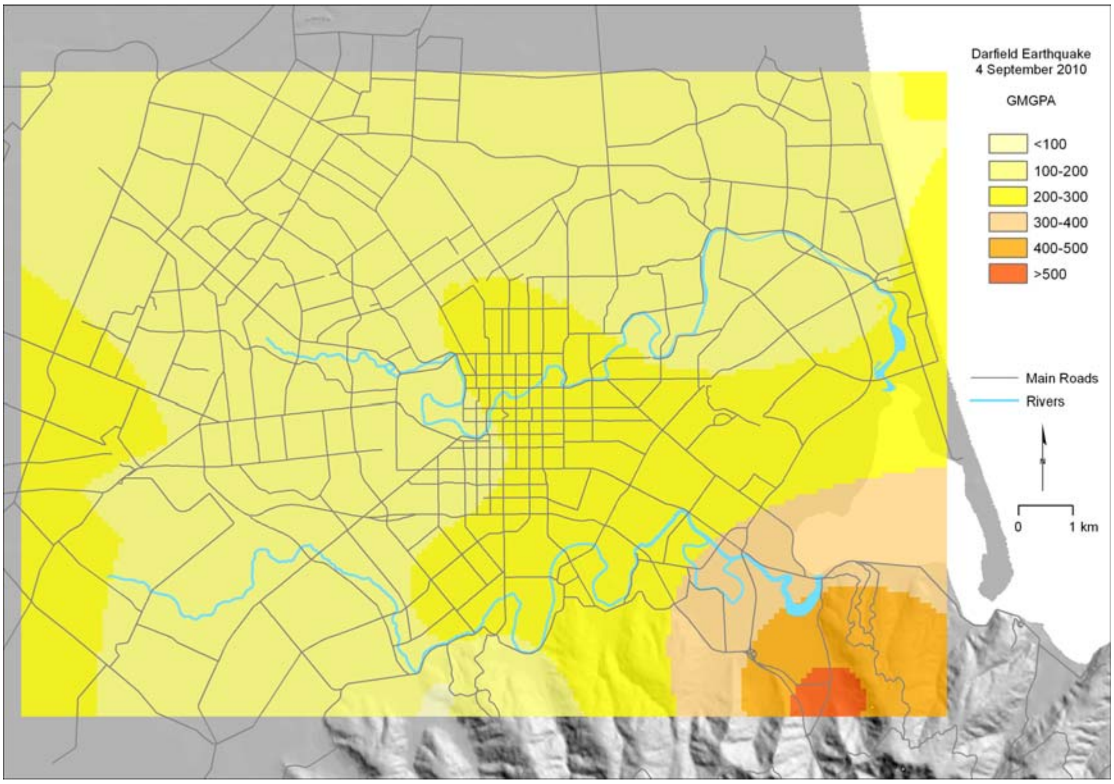
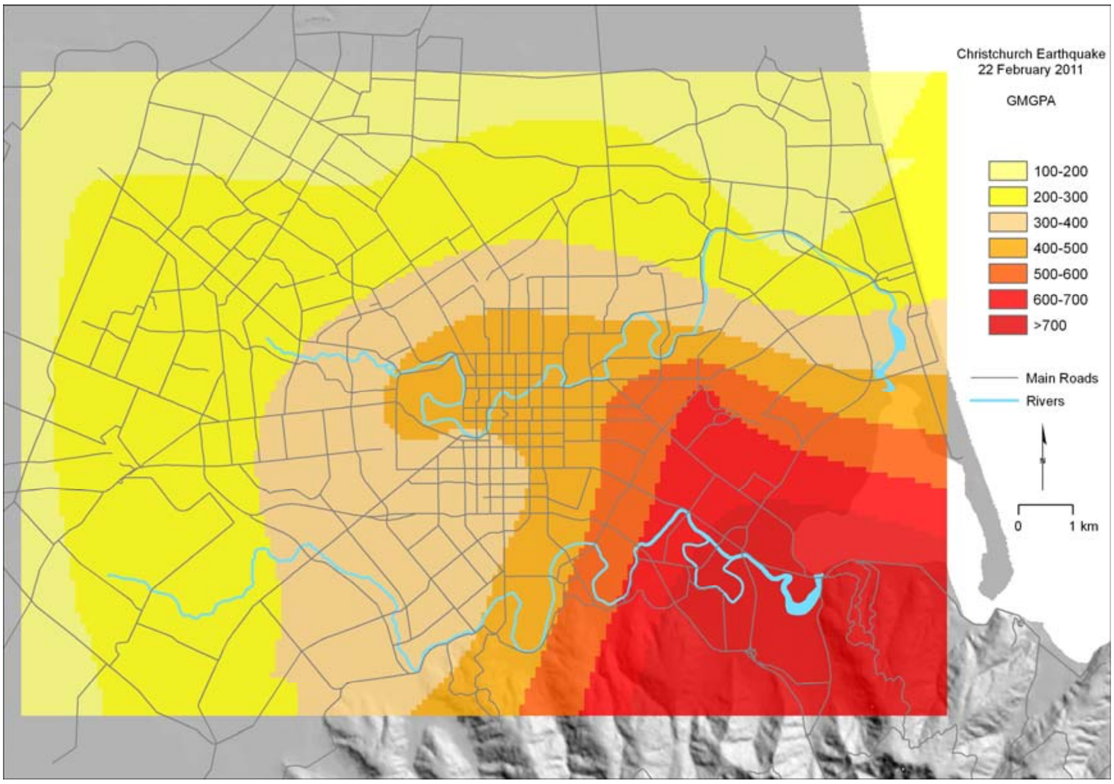


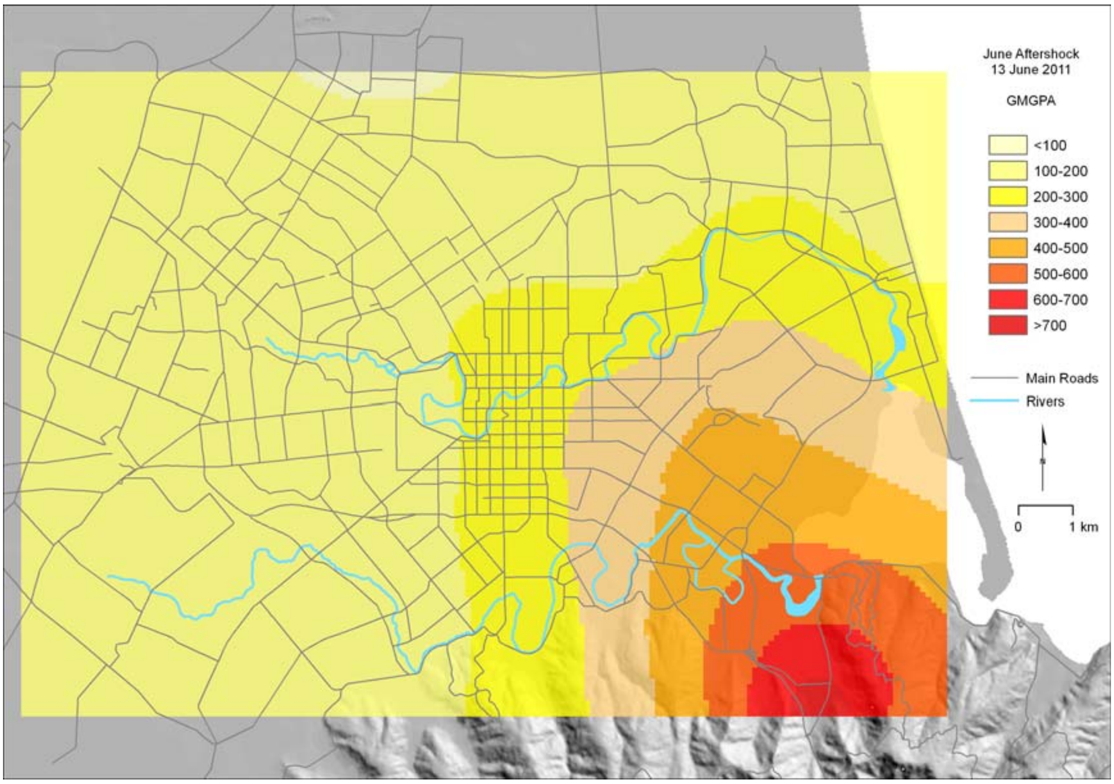

**Figure 1. Comparison of geometric mean PGAs recorded during the 2010 Darfield earthquake (horizontal axis) and 2011 Christchurch earthquake (vertical axis); if two horizontal acceleration records were obtained in the NS and EW directions respectively, then the geometric mean PGA was calculated as**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




**(a) 4 September 2010 (Darfield) earthquake**




**(b) 22 February 2011 (Christchurch) earthquake**

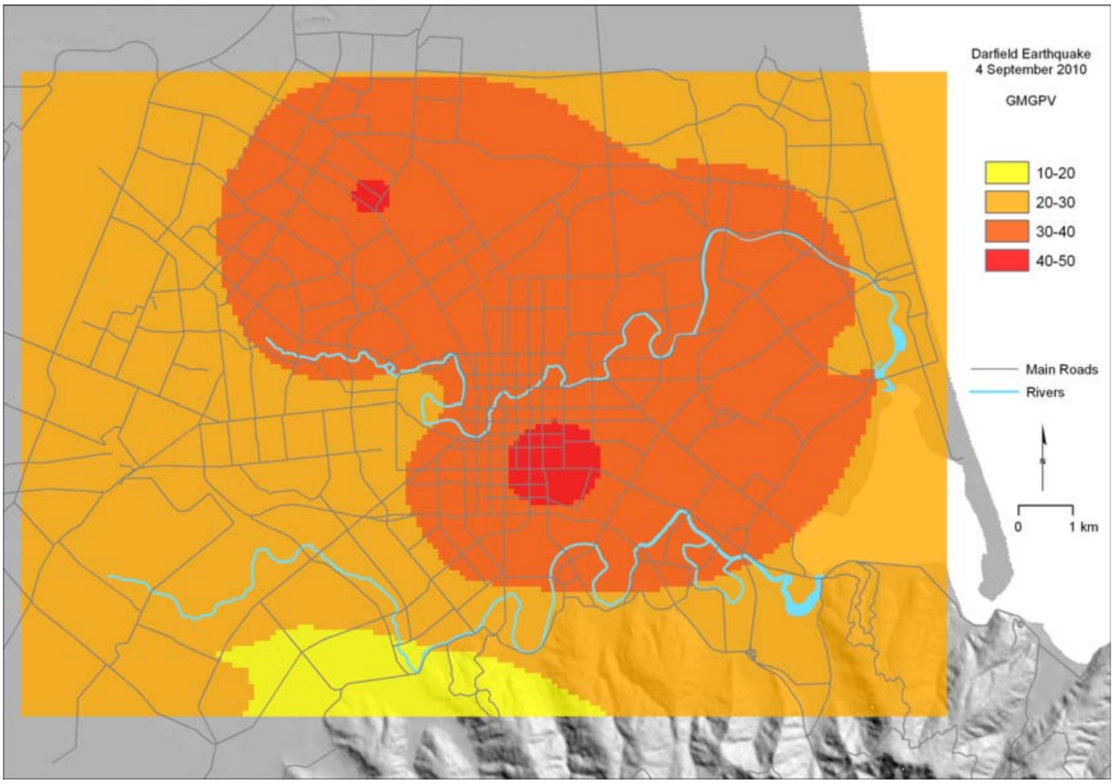



**(c) 13 June 2011 earthquake**

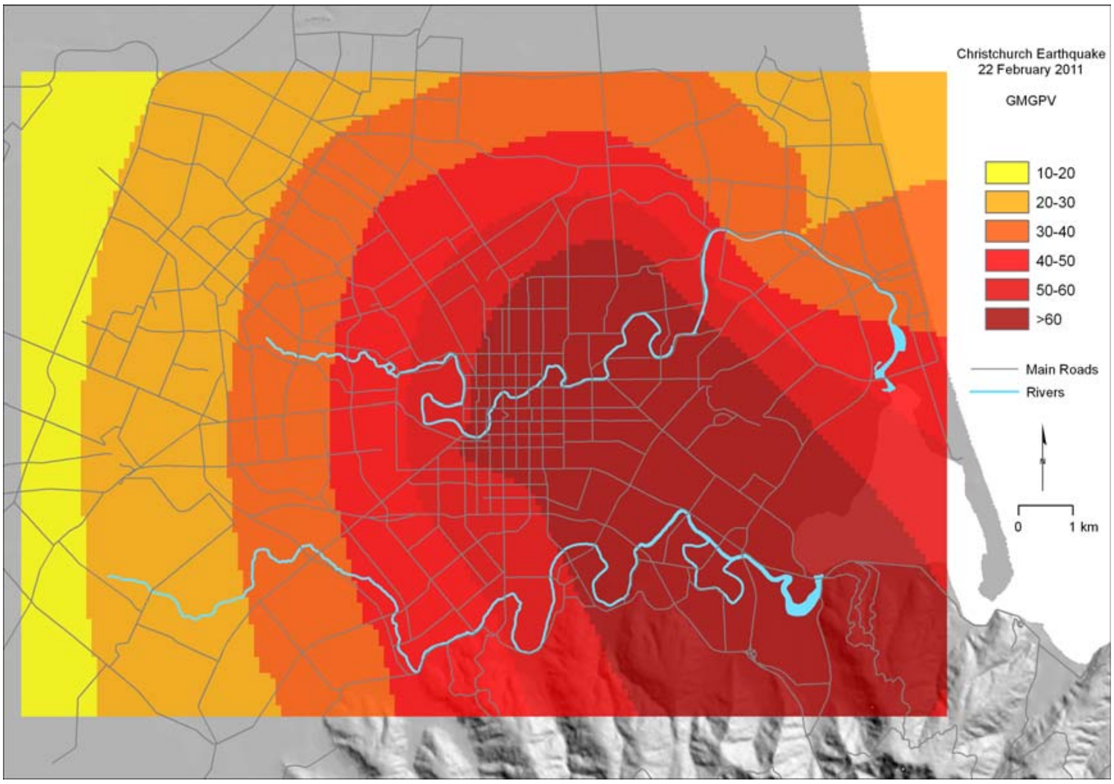
**Figure 2. Distribution of geometric mean PGAs obtained by ordinary krigging interpolation of recorded accelerations (O'Rourke and Milashuk, 2011)**

## *2.2 Peak Ground Velocities*

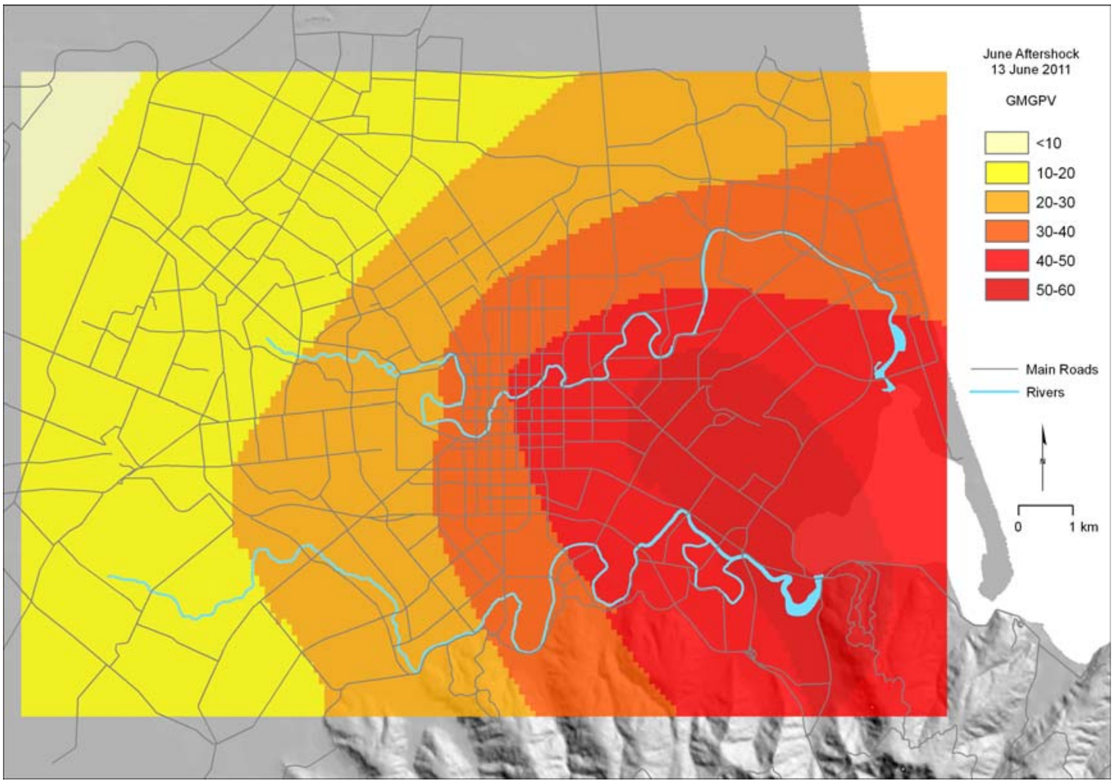
The peak ground velocity (PGV) is a better measure for the ground motion intensity and its effects on buried structures which are subjected to substantial ground movement and deformation than PGA (Bradley *et al.* 2009). This is why design codes often employ PGV rather than PGA when correlating ground 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 kriging 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

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

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.

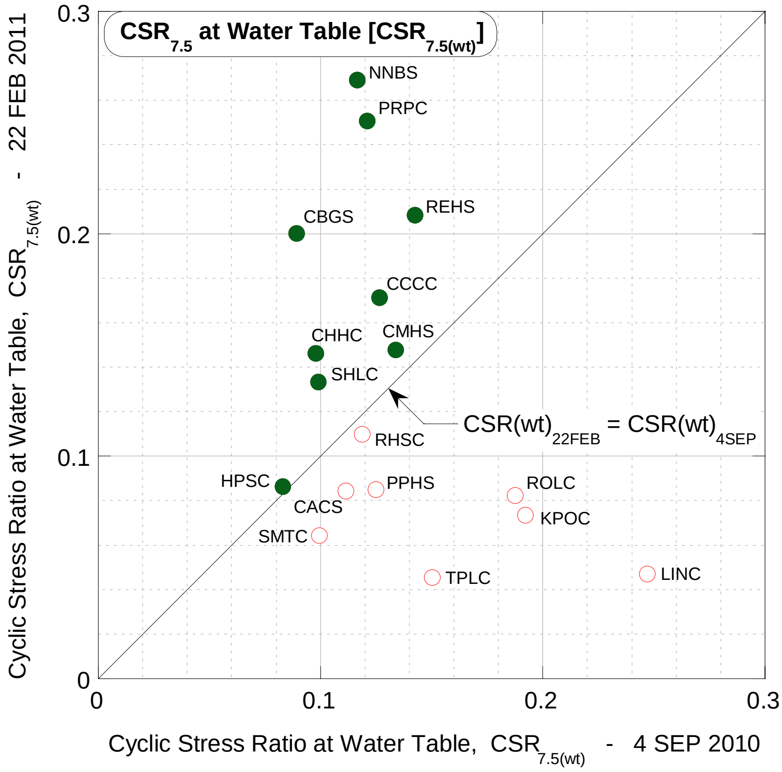



**Figure 4. Comparison of equivalent cyclic stress ratios at water table depth, $CSR_{7.5(wt)}$, induced by the 2010 Darfield earthquake (horizontal axis) and 2011 Christchurch earthquake (vertical axis) computed using geometric mean horizontal PGAs recorded at SMS (SMS acronyms shown next tot each symbol)**

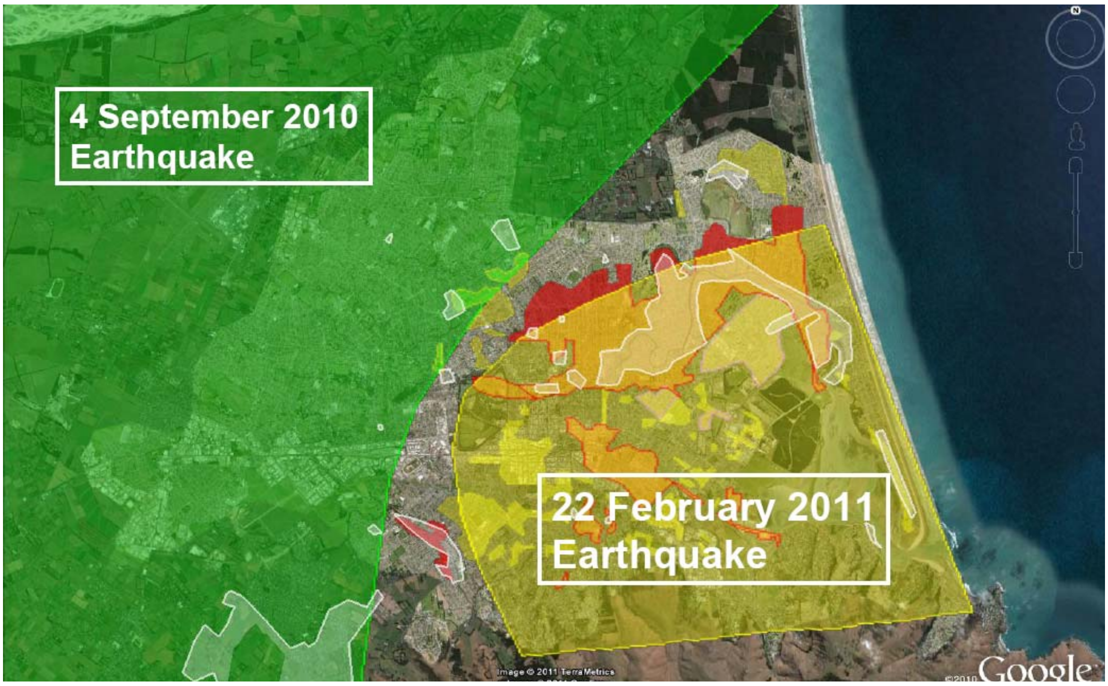



**Figure 5. Dominant earthquake event producing the maximum cyclic stress ratios or shaking intensity pertinent to liquefaction triggering; in the green area the highest CSRs were produced by the Darfield earthquake (4 SEP 2010) while the Christchurch earthquake (22 FEB 2011) produced the highest CSRs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

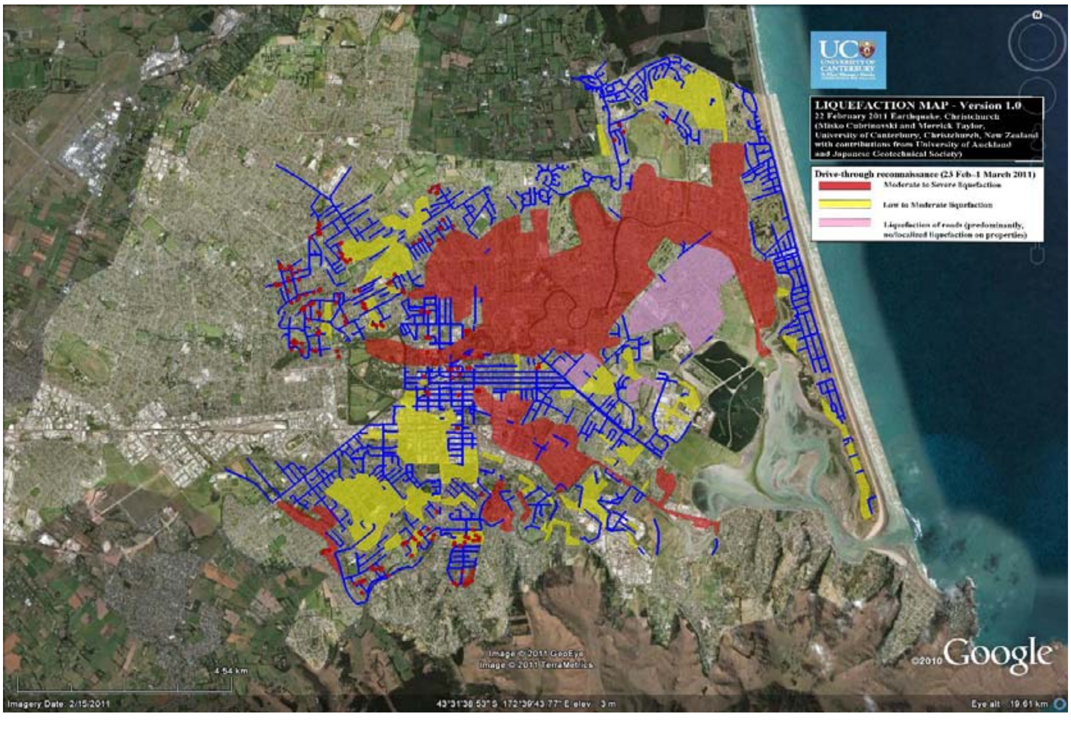



**Figure 6. Liquefaction map of Christchurch from drive-through reconnaissance (Cubrinovski and Taylor, 2011); the map is incomplete (i.e. does not cover all of Christchurch) and shows only general overlay of areas (i.e. it cannot be used on property basis)**

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.

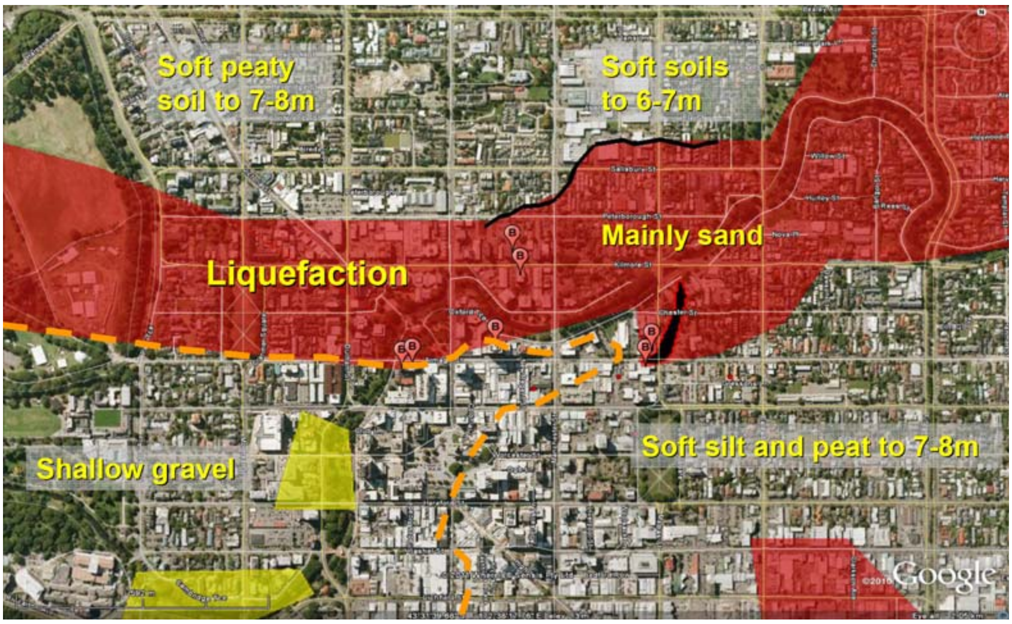



**Figure 7.** **Liquefaction map indicating zones (in general terms, not on property basis) within the CBD affected by liquefaction in the 22 February earthquake (Cubrinovski and Taylor, 2011); predominant soils in the top part of the deposits are also indicated (Cubrinovski and McCahon, 2011)**

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

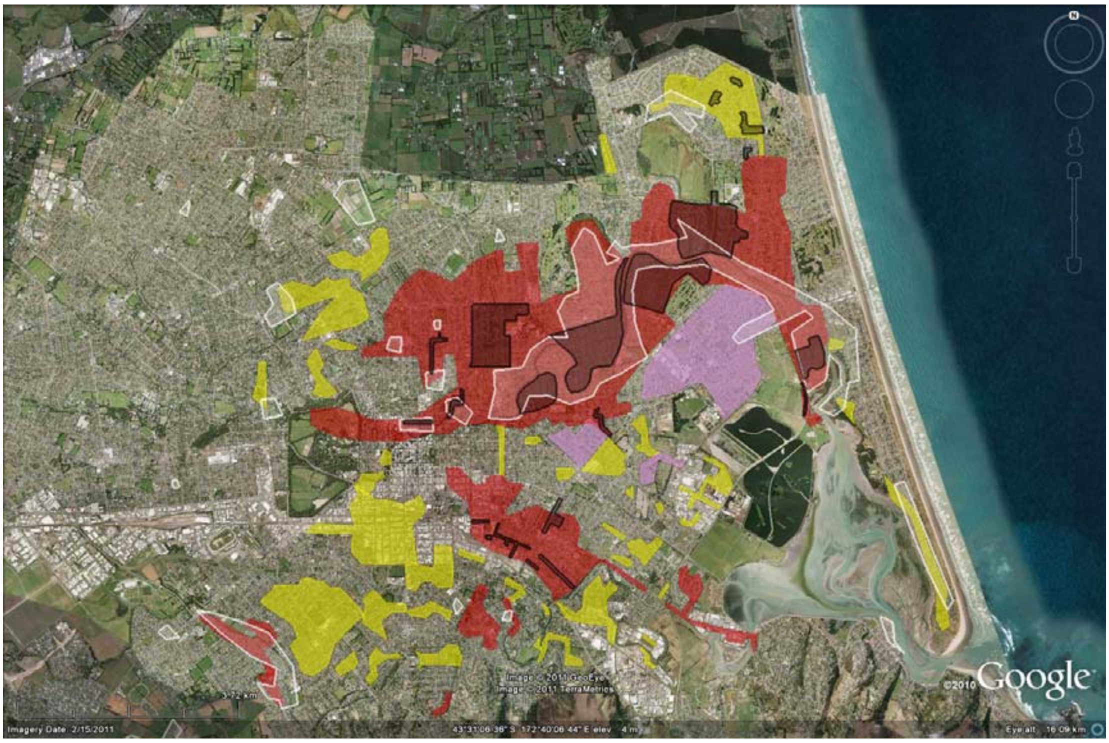



**Figure 8.** **Liquefaction maps documenting areas of observed liquefaction in the 4 September 2010 (white contours), 22 February 2011 (red, yellow, magenta areas; Cubrinovski and Taylor, 2011), and 13 June 2011 (black contours; Cubrinovski and Hughes, 2011) earthquakes; note that only parts of Christchurch 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.

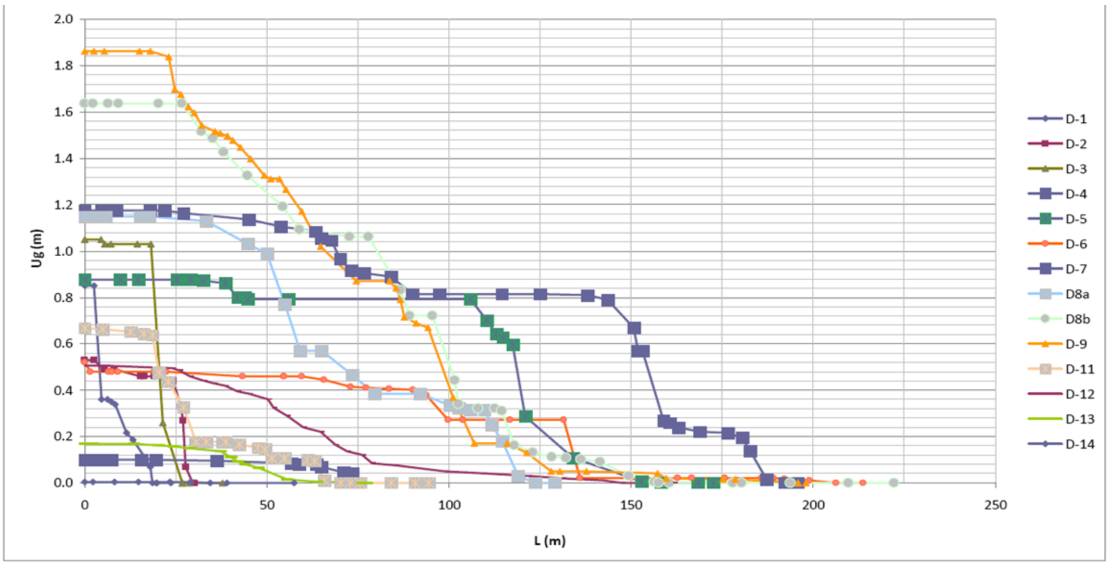



**Figure 9.** **Permanent lateral ground displacements due to spreading along Avon River in Dallington caused by the 4 September 2010 earthquake (plotted as a function of the distance from the waterway)**

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.

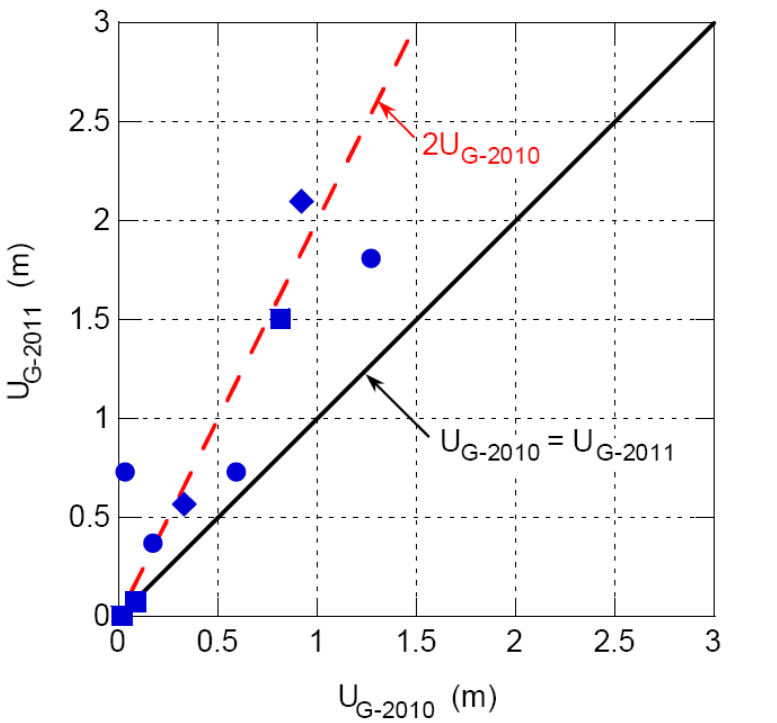



**Figure 10. Comparison of permanent lateral ground displacements along Avon River (Avonside, Dallington, Bexley) caused by the 4 September 2010 earthquake (horizontal axis) and cumulative displacements due to the 4 September 2010 and 22 February 2011 earthquakes (vertical axis)**

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).

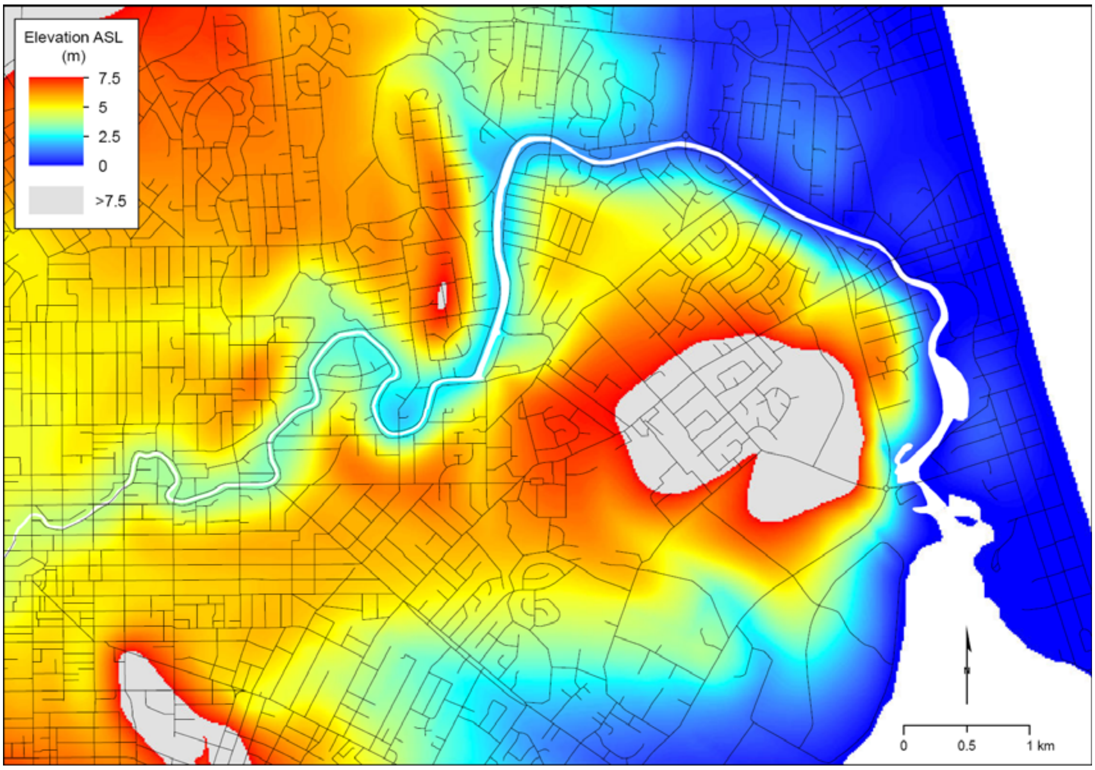



**Figure 11. Pre-earthquakes ground elevation above sea level along Avon River, from the CBD to the estuary (derived from the Digital Elevation Model of Christchurch, Landcare Research, 2011)**

### *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.

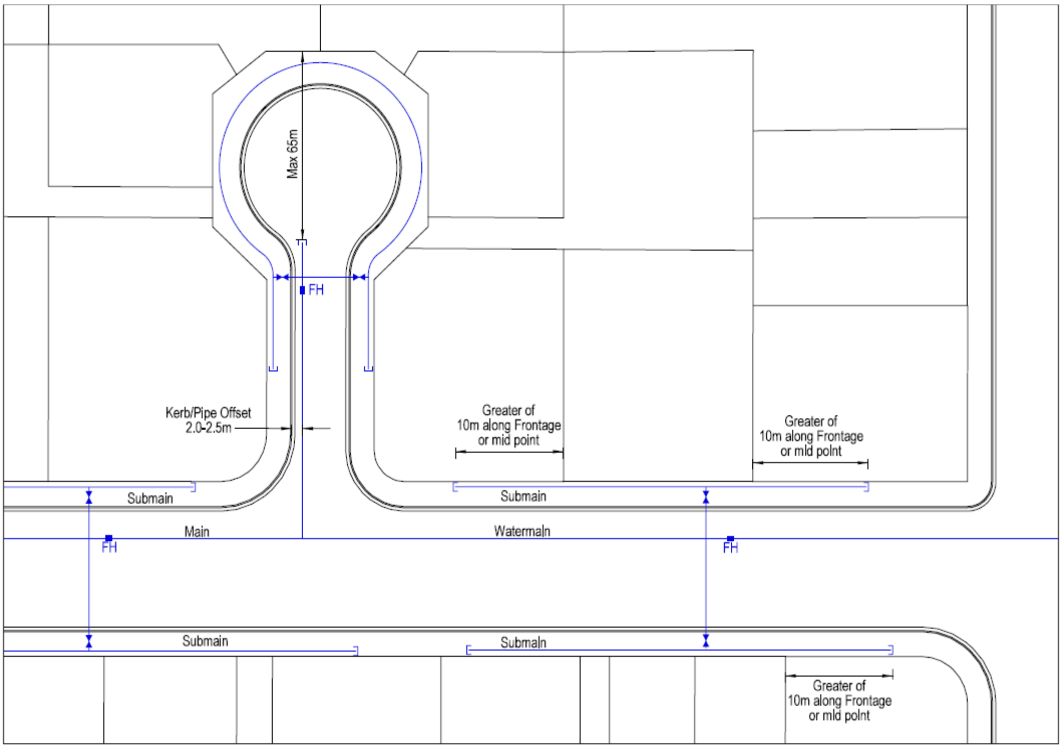
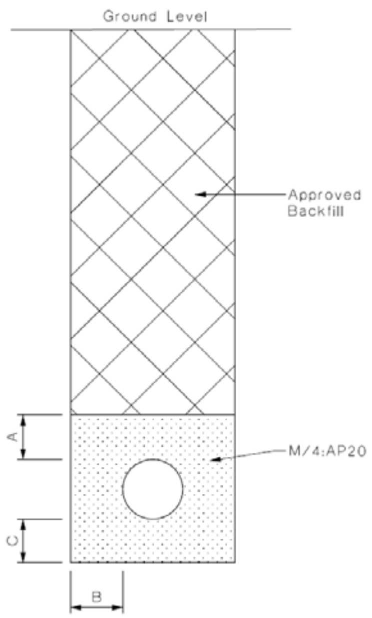



**Figure 12. Typical layout of watermains and submains in the carriageways and footpaths of Christchurch streets (CCC, 2010a)**

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)**

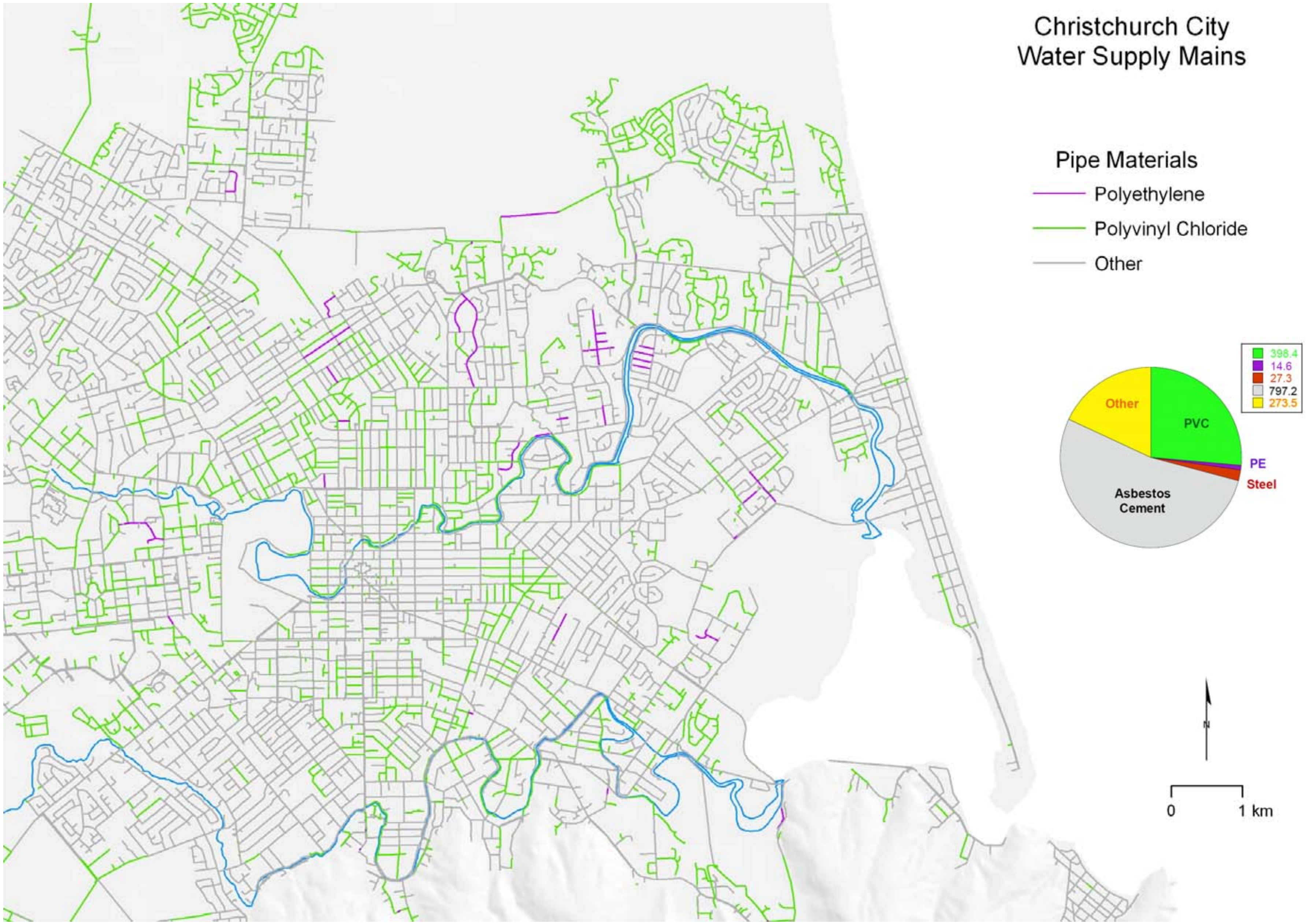



**Figure 14. Christchurch watermains network indicating different pipe materials (location and length of prevalent or relevant materials from a resilience perspective)**

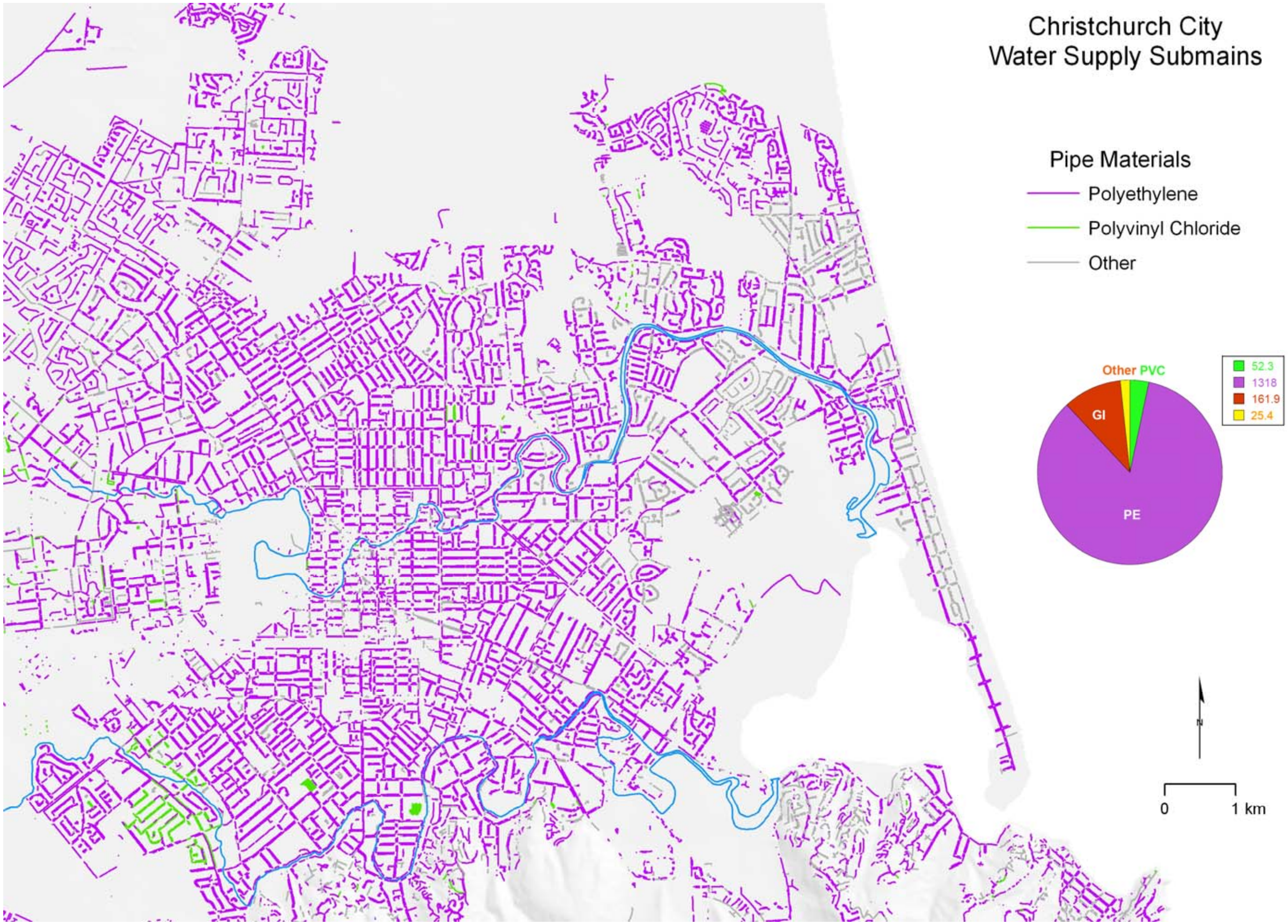



**Figure 15. Christchurch submains network (PE pipes cover approximately 85% of pipe length)**

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

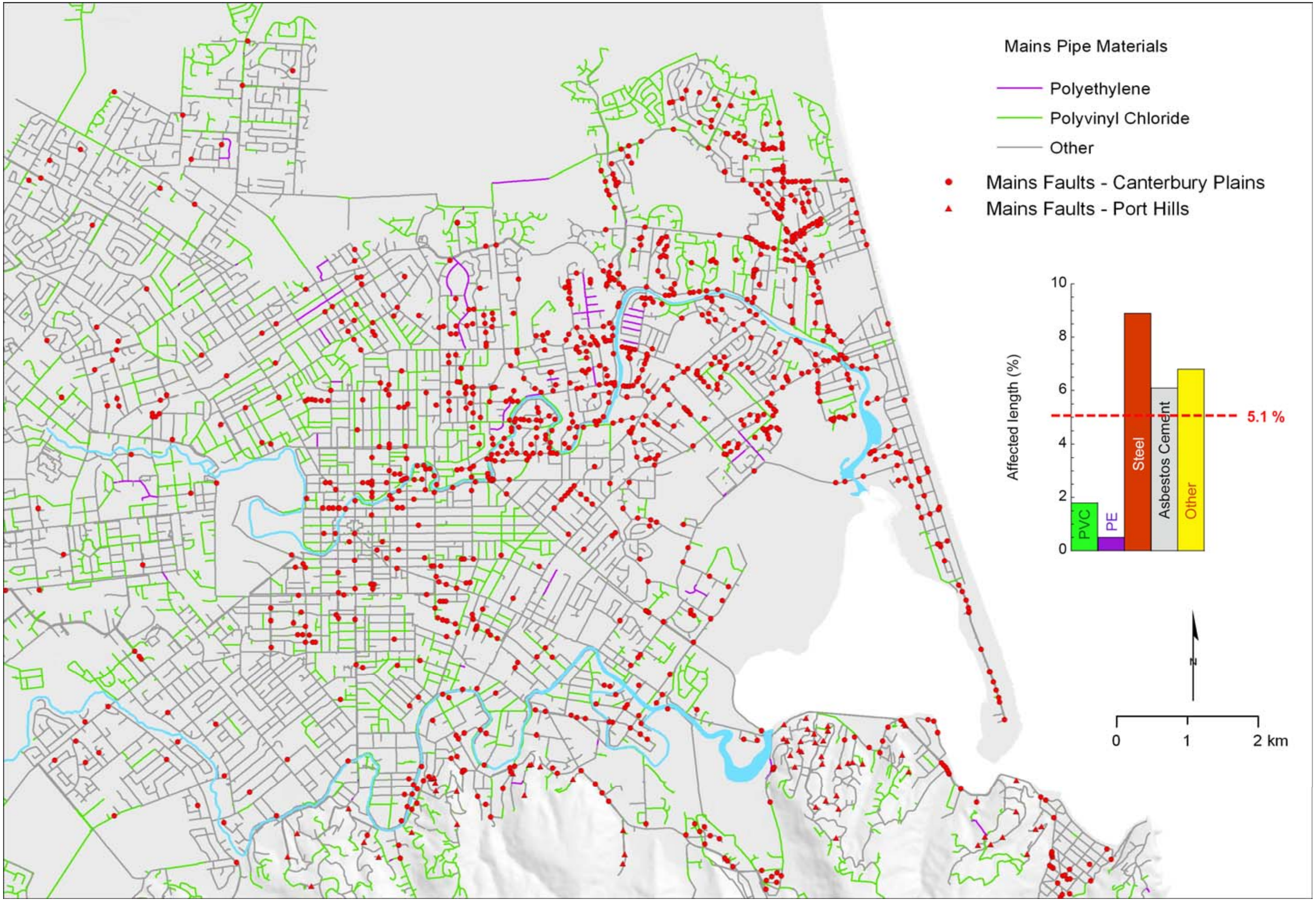



**Figure 16. Locations of repairs/faults (indicated by red symbols) on the Christchurch watermains network following the 22 February 2011 earthquake**

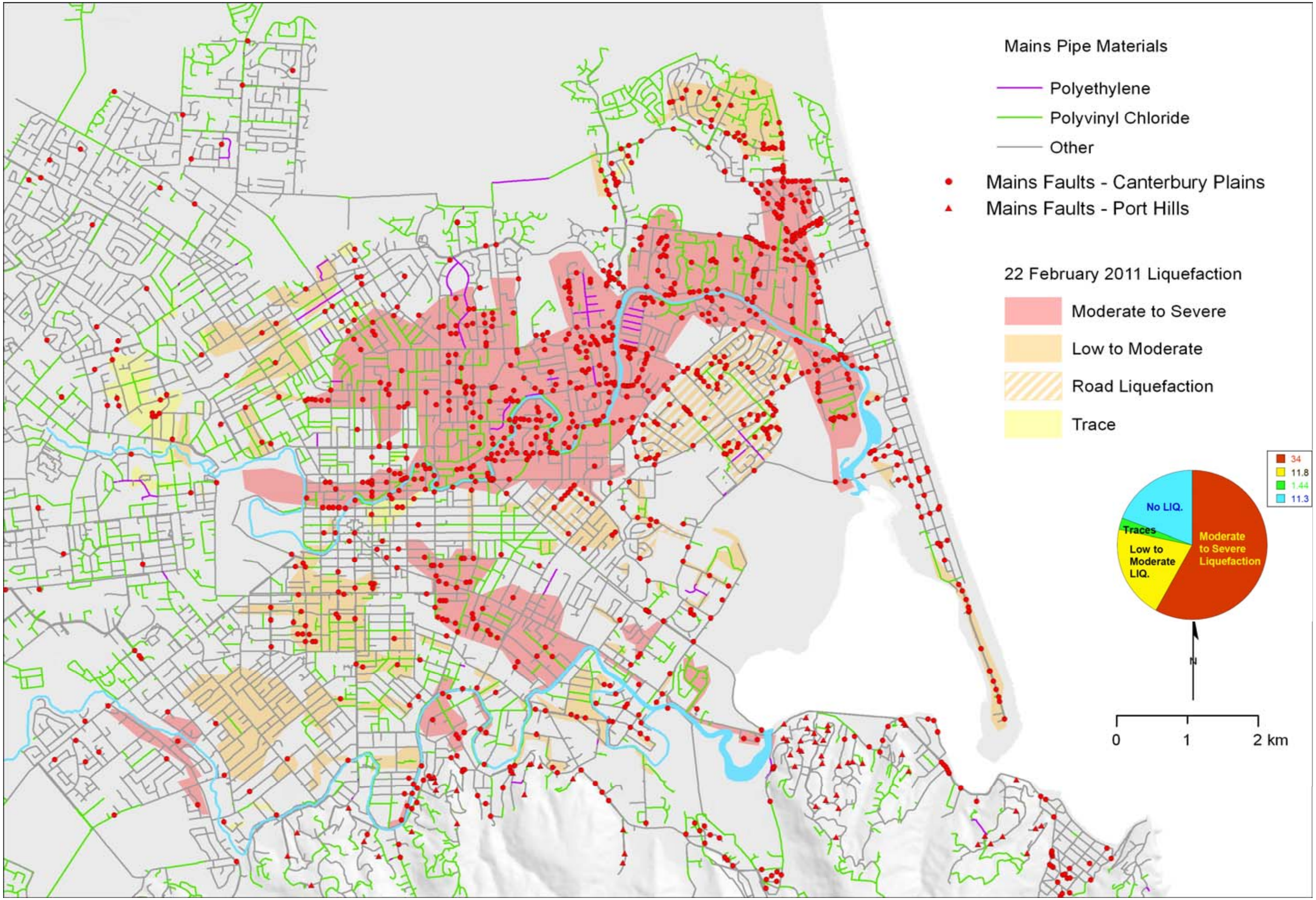



**Figure 17. Locations of repairs/faults (red symbols) on the Christchurch watermains network and areas of liquefaction following the 22 February 2011 earthquake**

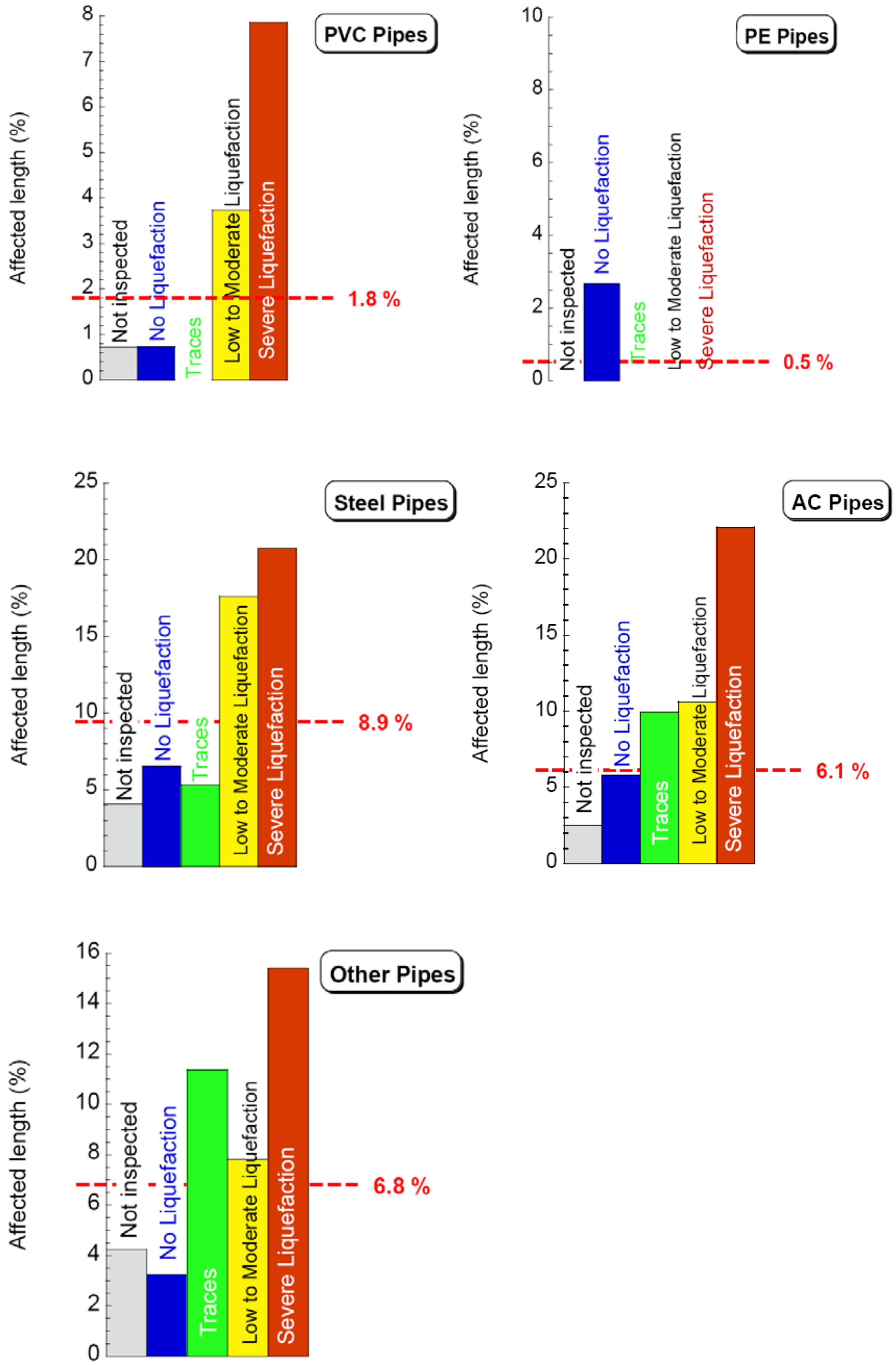



**Figure 18. Percentage of length damaged for water mains due to the 22 February earthquake, for different pipe materials and liquefaction severity (dashed red lines represent the average affected length across all ground damage levels).**

*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

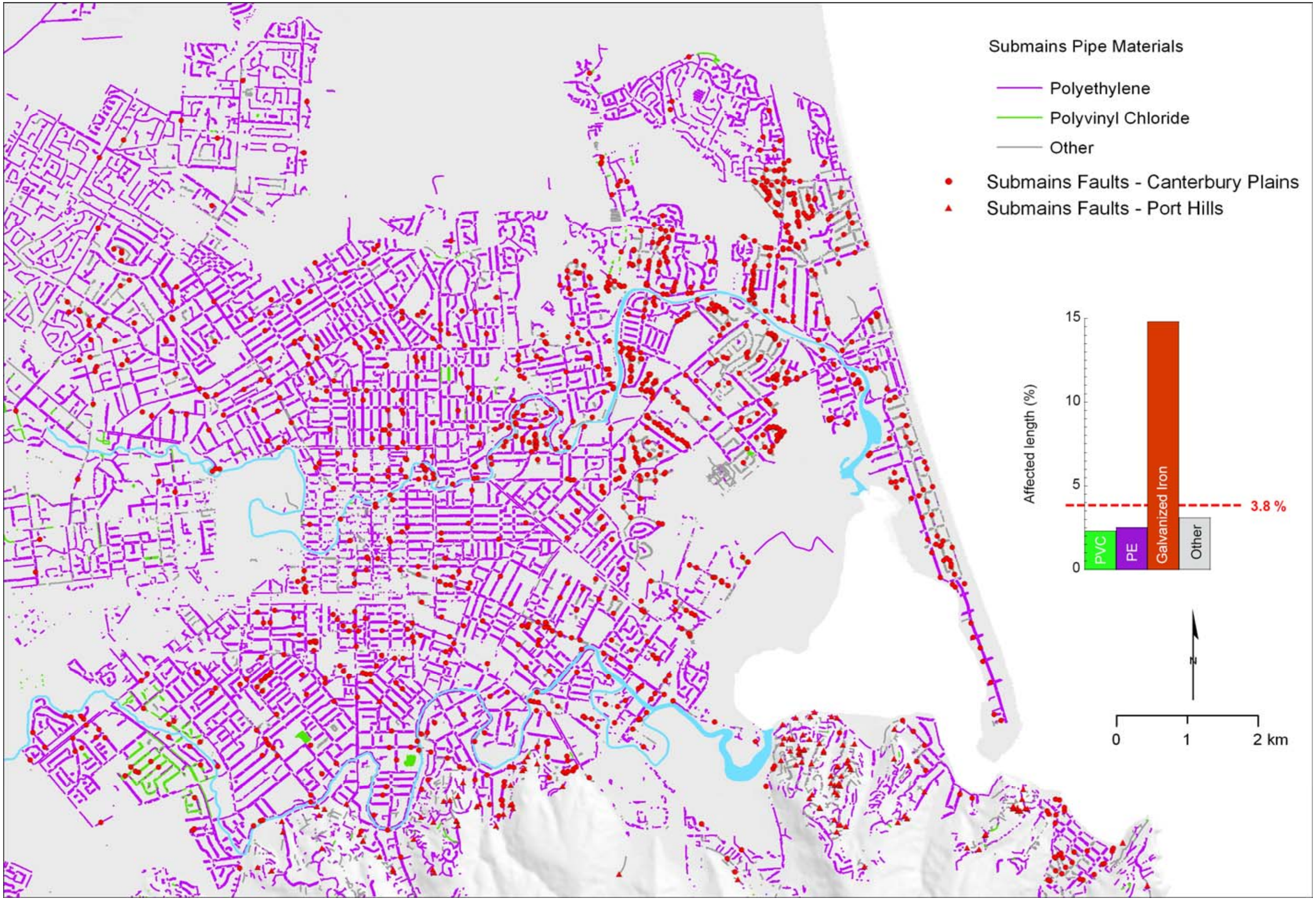



**Figure 19. Locations of repairs/faults (red symbols) on the Christchurch submains network following the 22 February 2011 earthquake**

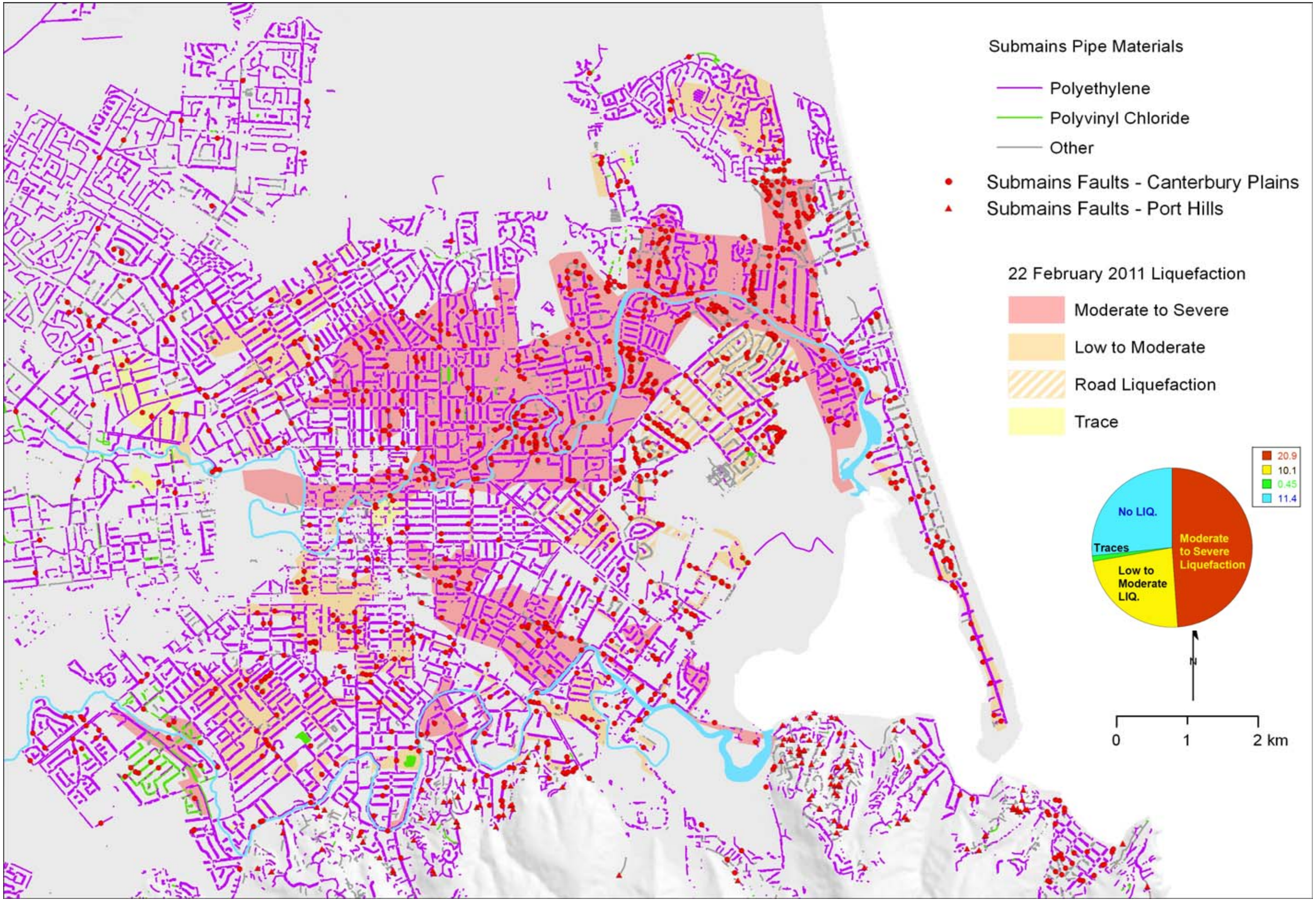



**Figure 20. Locations of repairs/faults (red symbols) on the Christchurch submains network and areas of liquefaction following the 22 February 2011 earthquake**

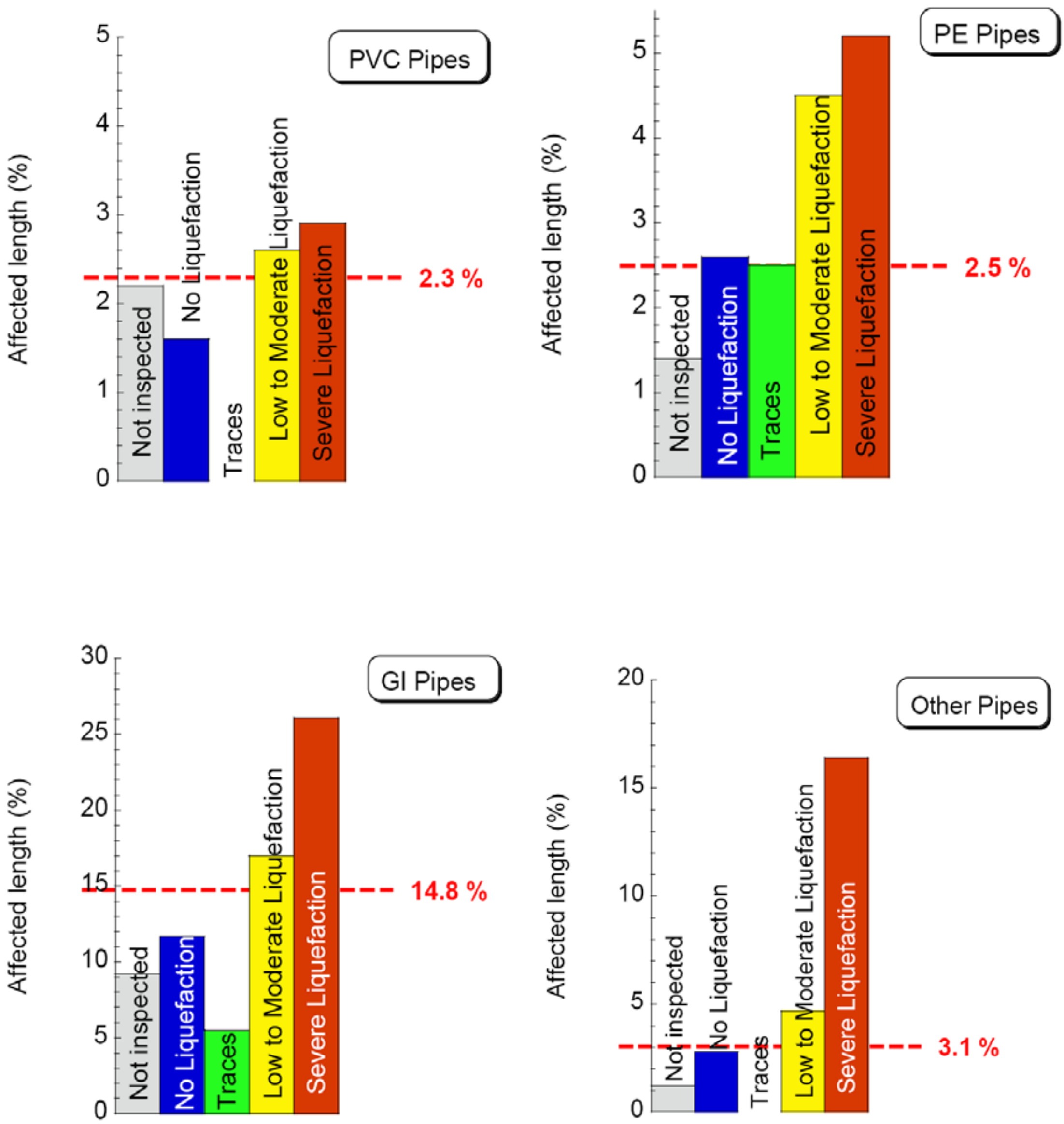



**Figure 21. Percentage of length damaged for submains due to the 22 February earthquake, for different pipe materials and liquefaction severity (dashed red lines represent the average affected length across all ground damage levels).**

# 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).*

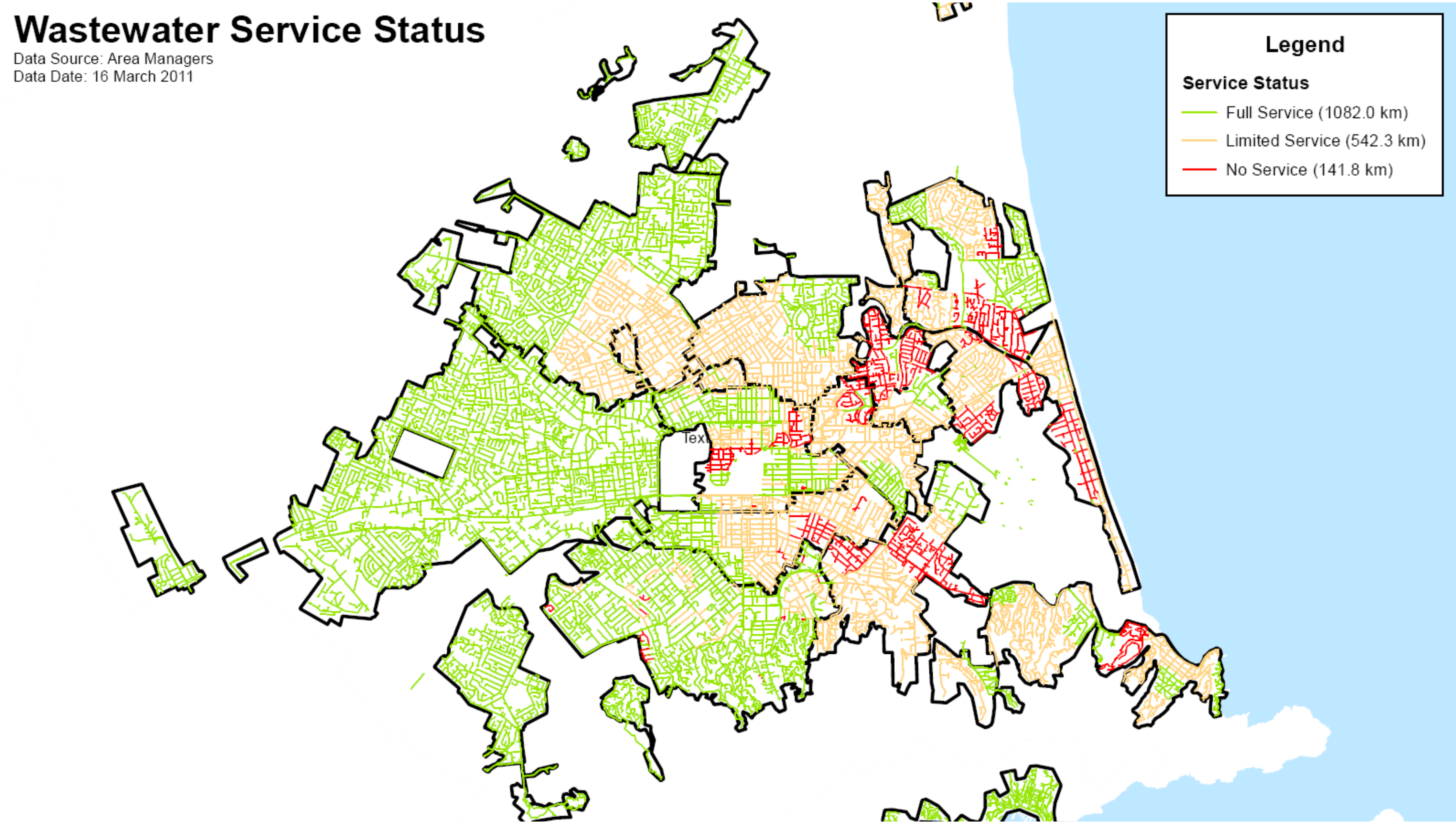



**Figure 22. Wastewater network status as of 16 March 2011 illustrating distribution and severity of damage inflicted by the 22 February 2011 earthquake**

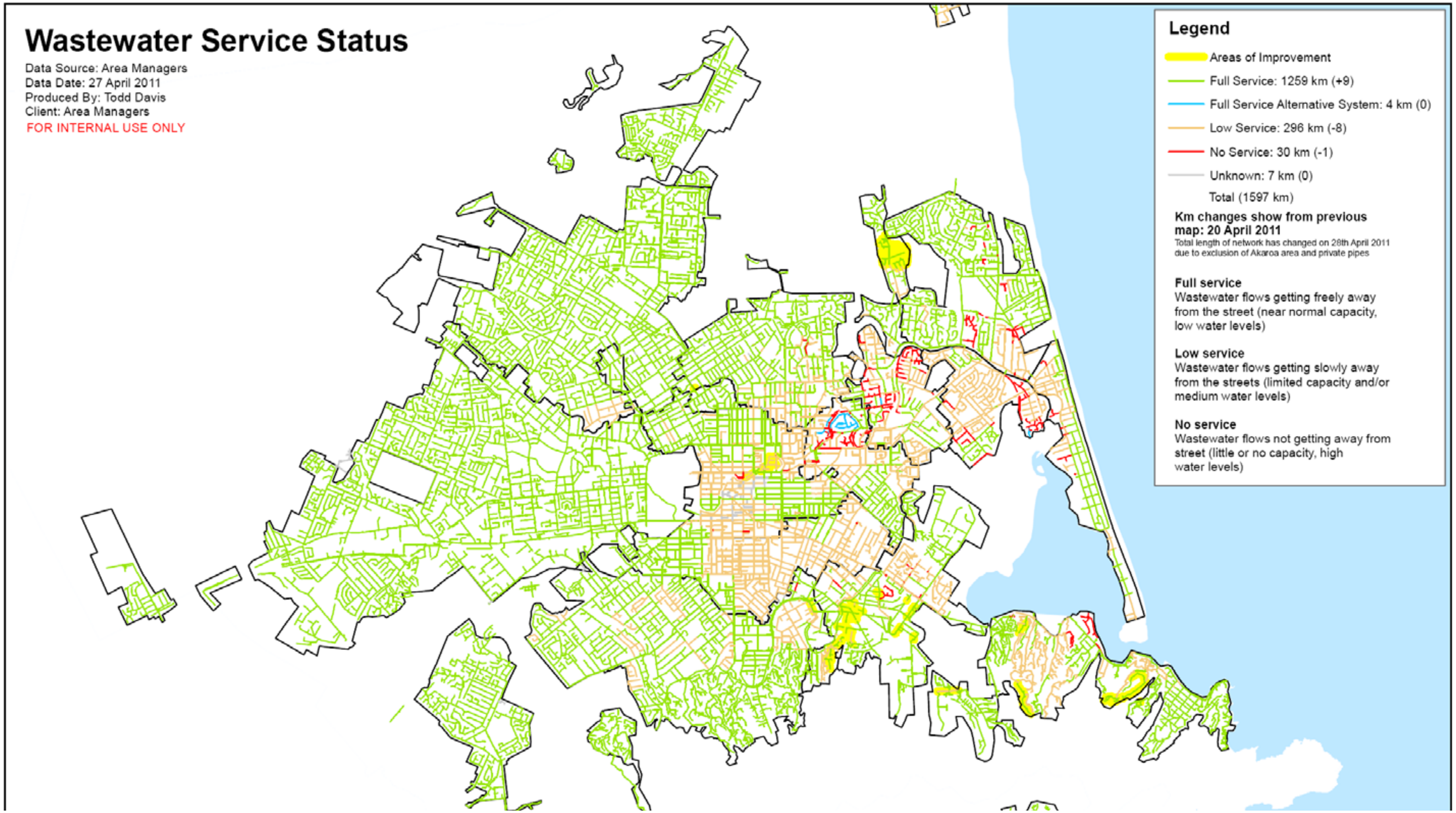



**Figure 23. Wastewater network status as of 27 April 2011 indicating the speed of reinstatement of services in conjunction with the status map shown in Figure 22**

# 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.<br>✖ 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<br>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<br>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<br>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<br>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<br>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<br>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<br>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<br>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<br>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).

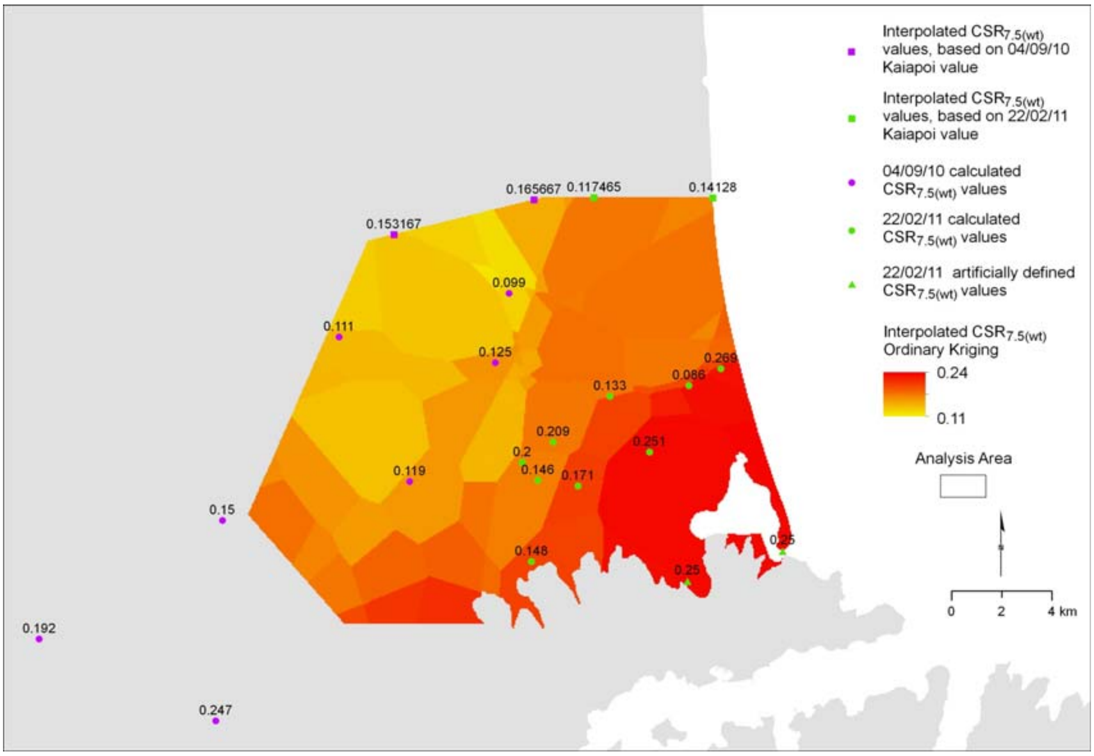



**Figure 24. $CSR_{7.5(wt)}$ values produced in Christchurch by the 4 September 2010 and 22 February 2011 earthquakes (ordinary krigging interpolation with a spherical variogram model; note that the CSR-values at the north boundary and the area at the HVSC station were constrained to eliminate spurious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

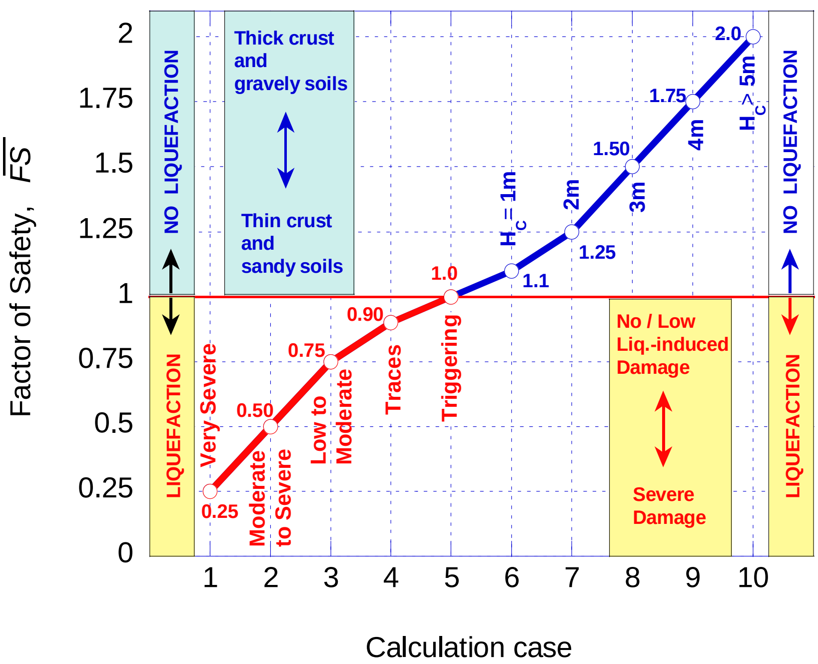



**Figure 25. Adopted factors of safety (against liquefaction) in areas of manifested liquefaction ($FS$ < *1.0*) and areas of no liquefaction ($FS$ > 1.0) with different water table depth**

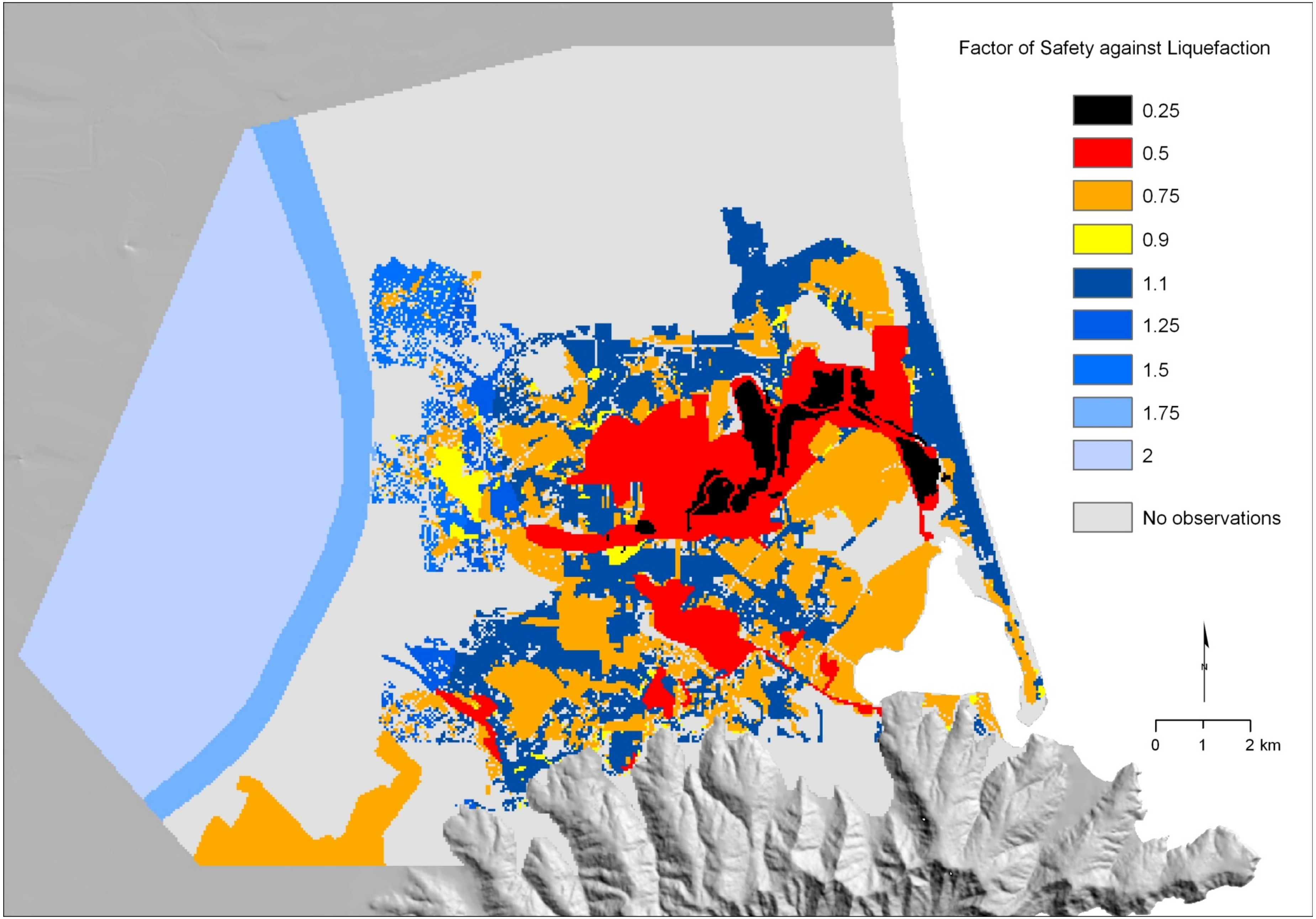



**Figure 26.** **Factors of safety (against liquefaction) map back-calculated based on liquefaction observations from the 2010-2011 earthquakes and water table depth information.**

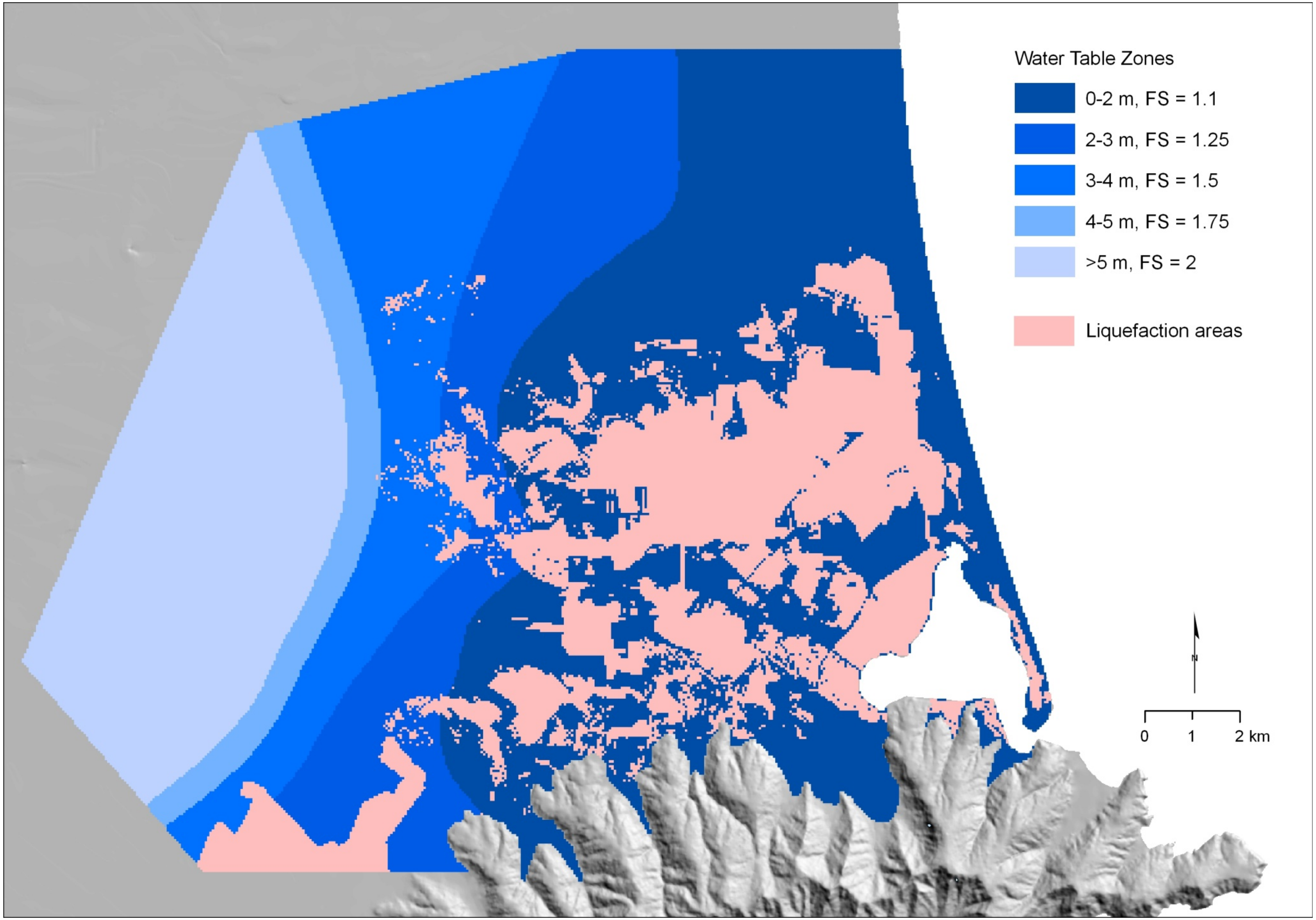



**Figure 27.** **Water table depth contours modified from Brown and Weeber (1992), converted to a 50 m resolution raster with Factors of safety allocated to areas that did not liquefy during the 4 September 2010 and 22 February 2011 earthquakes**

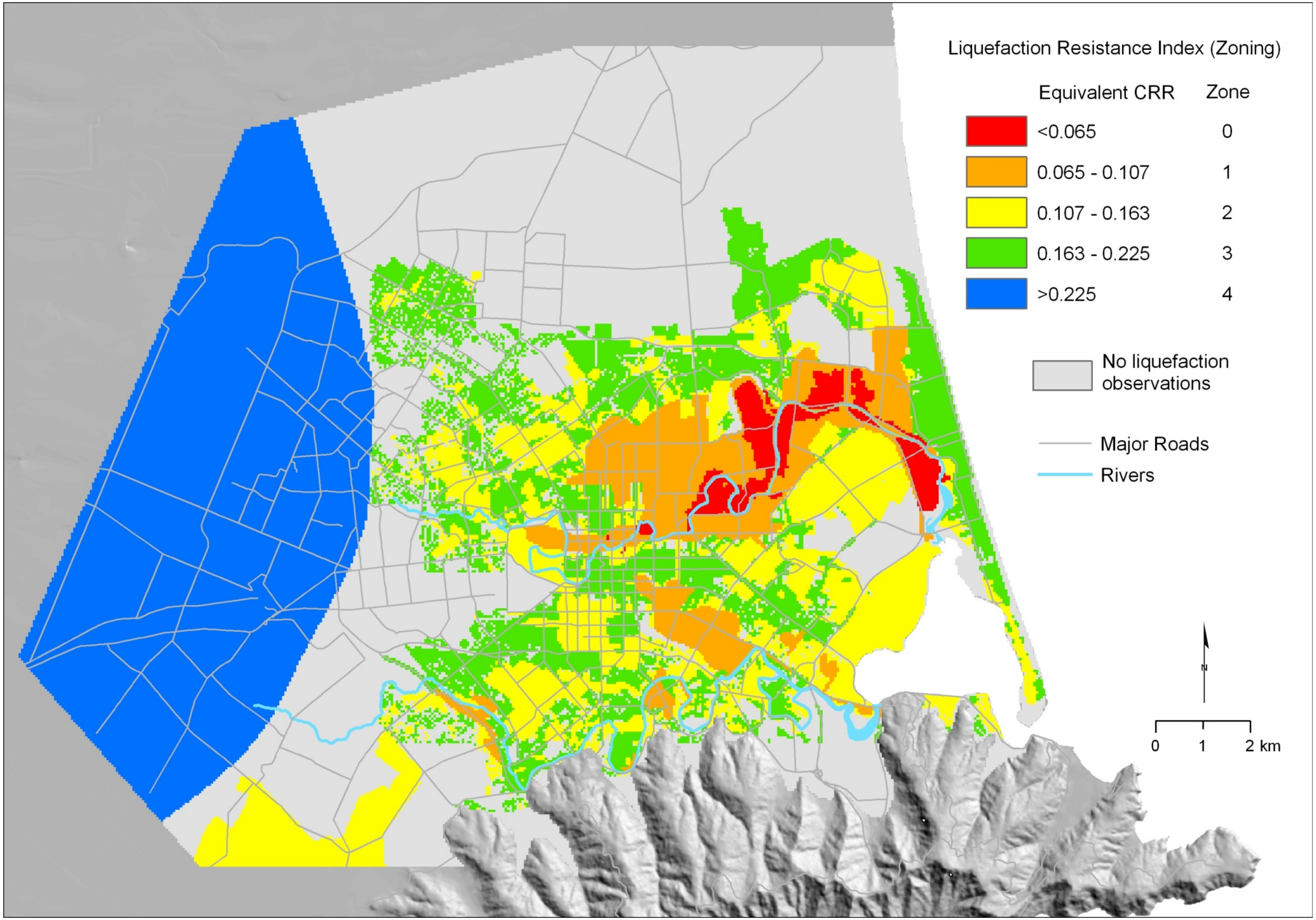



**Figure 28. Liquefaction Resistance Index (Zoning) of Christchurch at water table depth based on liquefaction observations from the 2010-2011 earthquakes and water table depth information**

**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
- $\varepsilon_v$ = volumetric strain, $\gamma$ = 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:

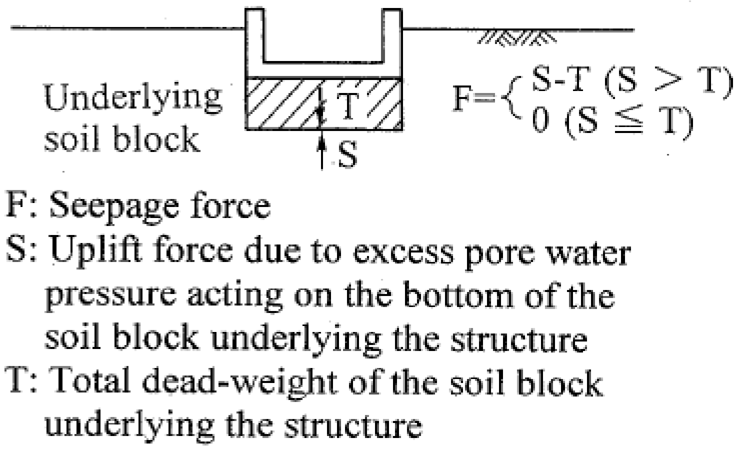


Figure 29: Calculation of seepage force, F (Koseki *et al.* 1997).

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, $\rho_d$ and $\rho_{sat}$, respectively
- The depth of the WT, $h_w$, and depth from base of the manhole to non-liquefiable layer, $h_{b0}$.

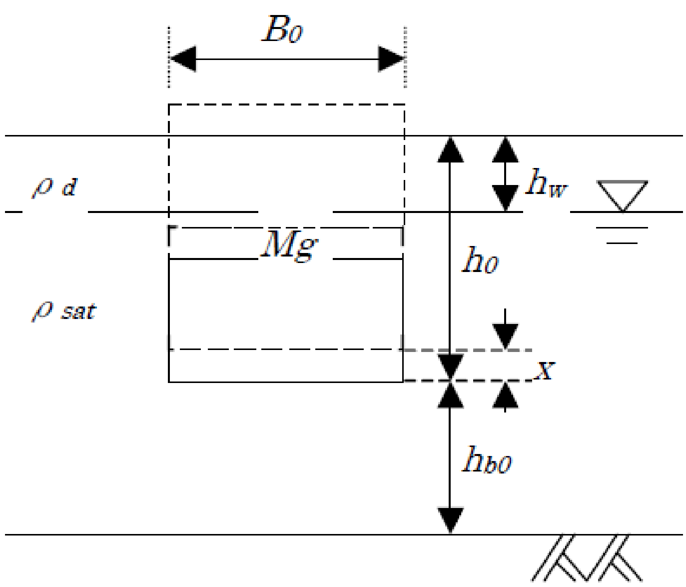


Figure 30: Notation used for manhole uplift calculations.

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.

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