Key factors in the liquefaction-induced damage to buildings and infrastructure in Christchurch: Preliminary findings

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ABSTRACT: The paper presents preliminary findings from comprehensive research studies on the liquefaction-induced damage to buildings and infrastructure in Christchurch during the 2010-2011 Canterbury earthquakes. It identifies key factors and mechanisms of damage to road bridges, shallow foundations of CBD buildings and buried pipelines, and highlights the implications of the findings for the seismic analysis and design of these structures.

1 INTRODUCTION

Soil liquefaction and lateral spreading caused extensive and very severe damage to buildings and infrastructure in Christchurch in the 2010-2011 Canterbury earthquakes. The liquefaction affected nearly 60,000 residential buildings, many multi-storey CBD buildings and the horizontal infrastructure over approximately one third of the city area. One may argue that approximately half of the $30 billion dollars loss inflected by the earthquakes (or ~ $15 billion NZD) was directly caused by liquefaction.

While liquefaction has been widely recognized as one of the principal earthquake hazards, it is important to understand the particular features of the Christchurch liquefaction and identify key factors that contributed to the extensive damage. With this goal in mind, several research studies involving field surveys of damage, in-situ and laboratory testing of soils, and both simplified and advanced analyses have been initiated after the earthquakes. This paper presents some preliminary findings from these studies and focuses in particular on spreading-induced damage to road bridges, effects of liquefaction on shallow foundations of CBD buildings and controlling factors in the liquefaction-induced damage to water and wastewater pipelines. More details on these on-going studies can be found in the referenced papers, whereas herein key factors in the liquefaction-induced damage have been highlighted and presented in a succinct form.

2 SPREADING-INDUCED DAMAGE TO BRIDGES

2.1 Lateral spreading

While extensive liquefaction affected most of the eastern Christchurch, the liquefaction was particularly severe and damaging along the Avon River where it was accompanied by substantial lateral spreading. Ground surveying measurements of lateral spreading at approximately 80 locations along the Avon River (Robinson et al., 2011; 2013) have shown that the magnitude of permanent ground displacements and the width of the zone affected by spreading generally increased heading down the Avon River, from the CBD towards the estuary. As shown in Figure 1, maximum spreading displacements within the CBD were predominantly between 10 and 30 cm, and were confined within a distance of 40-50 m from the river. The first more substantial lateral spreads with displacements greater than 1 m were observed at the Avon Loop, in the northeast corner of the CBD (the meandering loop before the Fitzgerald Bridge). Downstream from the Avon Loop, the spreading was more substantial and very damaging with permanent displacements often exceeding 0.5 m or even 1 m, and the zone of spreading extending up to 150 m to 200 m from the river. The spreading displacements were highly variable in their magnitude and spatial distribution even within a small area reflecting the complexities of the lateral spreads in the meandering loops of the Avon River (Cubrinovski et al., 2012). The largest permanent displacements of ~ 3 m were measured at the Pleasant Point Yacht Club, near the South Brighton Bridge where the Avon River discharges into the estuary.

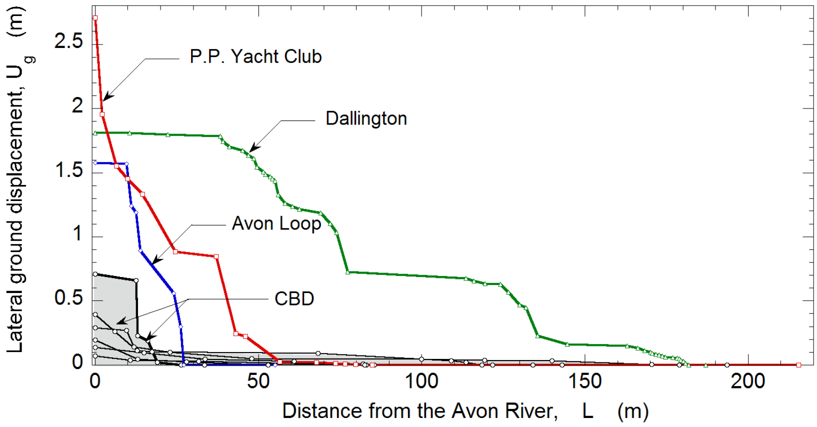

Figure 1. Lateral spreading displacements along the Avon River measured by ground surveying after the Christchurch earthquake (Cubrinovski et al., 2014)

2.2 Characteristic deformation (damage) mechanism

Practically all bridges on the Avon River from the CBD to the estuary were subjected to these large spreading displacements and exhibited a characteristic deformation (damage) mechanism governed by the spreading demand and a particular feature of the bridge structures.

The road bridges of Christchurch are short- to moderate-length bridges typically with two or three short spans. Older bridges are integral systems while recent bridges are commonly precast concrete structures with movable joints. They all have sturdy configurations with rigid deck-wall or deck-girder superstructure providing large stiffness and strength in the longitudinal direction. Hence, when subjected to large spreading displacements of the ground and closing of the banks towards the river, the bridge superstructure resisted the ground movement through a deck-pinning or deck-strutting mechanism, as illustrated in Figure 2. This mechanism involves resistance to ground movement by the superstructure through deck-pinning (strutting), consequent back-rotation of the abutments with permanent displacement of the abutment piles towards the river, and substantial slumping of the approaches resulting in large vertical offsets between the approaches and pile-supported deck of the bridge. The permanent tilt of the abutments often reached 5-8 degrees and induced lateral movement of the abutment piles towards the river on the order of 20-30 cm. This deformation pattern consistently damaged the piles at their top with horizontal bending cracks occurring on the river-side and concrete crushing due to compression on the land-side of the piles (Figure 3). Clearly, the large lateral resistance of the bridge superstructure in conjunction with deck-pinning (strutting) played a key role in the development of the characteristic deformation (damage) mechanism of bridges induced by spreading of liquefied soils.

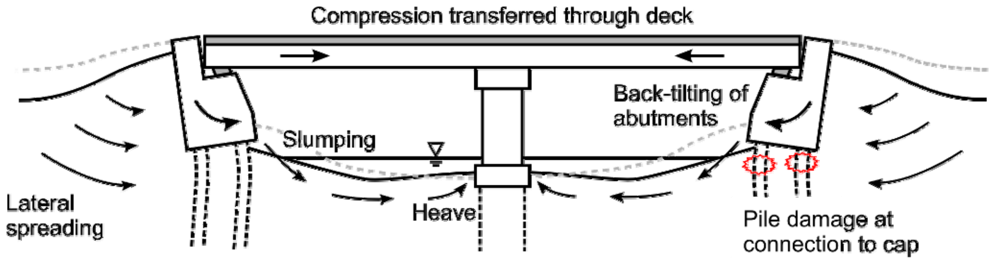

Figure 2. Schematics illustration of characteristic spreading-induced mechanism of deformation (damage) for short-span road bridges in Christchurch

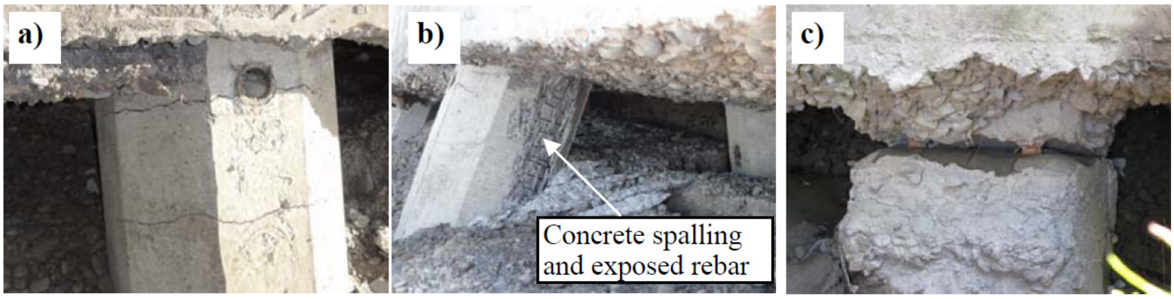

Figure 3. South Brighton Bridge: bending cracks at the top of abutment pile, on the river side (a) and concrete crushing on the land-side of abutment pile (b); Fitzgerald Bridge: failure of the northeast abutment pile (c)

2.3 Key observations relevant for seismic analysis and design

The ANZAC Bridge on the Woolston-Burwood expressway also exhibited the characteristic spreading-induced mechanism outlined above (as did all other bridges for that matter) and provided important details on the permanent ground movement in the foundation soils of the bridge abutments. Massive liquefaction occurred in the area of the bridge during the Christchurch earthquake with large volumes of sand ejecta seen particularly on the south side of the bridge. Figure 4a shows an aerial view of the bridge in which the solid lines indicate alignments of lateral spreading measurements (Robinson et al., 2013) with the numbers at the tip of the transects indicating maximum displacements at/near the river banks measured after the Christchurch earthquake. It is apparent that the south banks of the river moved approximately 1 m towards the river, in the free field. However, the large resistance to the ground movement provided by the bridge superstructure, as described earlier, reduced the movement of the foundations soils. The ANZAC bridge case history provides an excellent evidence for quantifying this reduction in movement of the foundations soils in relation to the permanent free field ground displacements.

Figure 4b illustrates the permanent movements at the south abutment of the bridge with the red arrow indicating an inward permanent tilt of the abutment of about 6 degrees. The horizontal arrows and associated displacements indicate permanent lateral displacements of ~ 20 cm at the bottom of the abutment (i.e. top of the abutment piles) resulting from the rotation of the abutment, lateral offset of the precast concrete underpass of 40 cm to 80 cm, and lateral movement of the surrounding free field soil of ~ 100 cm. These displacements infer a horizontal displacement at the top of the piles of about 20 cm (i.e. 16 cm due to abutment rotation, and 3-4 cm displacement required to close the gap between the abutment and the deck-girder). The pedestrian underpass was founded on independent foundations consisting of 6 m-long rigid RC (reinforced concrete) piles that effectively floated in the liquefied soil and moved together with the surrounding foundation soils. The lateral displacements of the south underpass relative to the abutment were about 20 cm at its edges and 50-60 cm in its central part (Cubrinovski et al., 2013). From these displacements, one can infer that lateral displacements of the foundations soils of the abutment were about 50 cm to 60 cm or approximately 50% of the displacements of the riverbanks in the free field.

These findings have an important implication for the analysis and design of piles based on the equivalent static or pseudo-static analysis in which ground displacements are applied to the piles as an input in the analysis. The common practice of applying full free field displacements to the piles is clearly overly conservative for bridges with a capacity to reduce ground movements through a deck-pinning or deck-strutting mechanism. Moreover in such cases, it is important to account for the global bridge response and effects of deck-strutting using a whole bridge model. Such model and analysis for the ANZAC bridge is shown in Figure 5 where maximum ground displacements of 66 cm and 36 cm (best estimates) were simultaneously applied to the south and north bridge abutments respectively, while smaller spreading displacements were applied to the pier piles. Details of these analyses are given in Cubrinovski et al. (2014) and are beyond the scope of this paper, however two key findings from the analyses are important to emphasize: (i) the whole bridge model (analysis) captured the characteristic spreading-induced mechanism and damage to the piles (bridge), and clearly demonstrated the need to consider the response of the bridge and its components using a global interaction model, and (ii) the analyses confirmed that lateral movements of the foundation soils were substantially smaller than (or approximately 50% of) the free field displacements of the riverbanks.

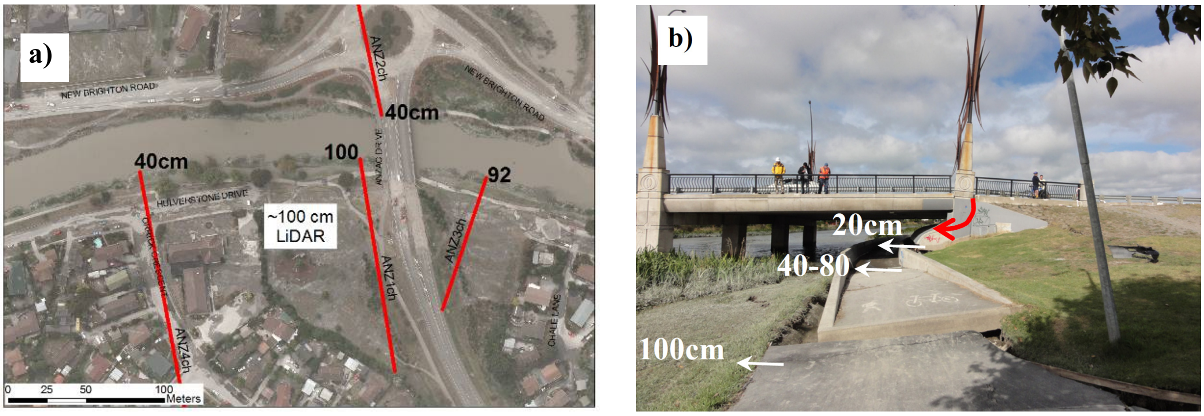

Figure 4. ANZAC Bridge: (a) aerial view showing transects and measured lateral spreading displacements in the free field; (b) side view of the south abutment indicating back-rotation of the abutment (red arrow), and permanent lateral displacements of the abutment base, adjacent pedestrian underpass and free field soils

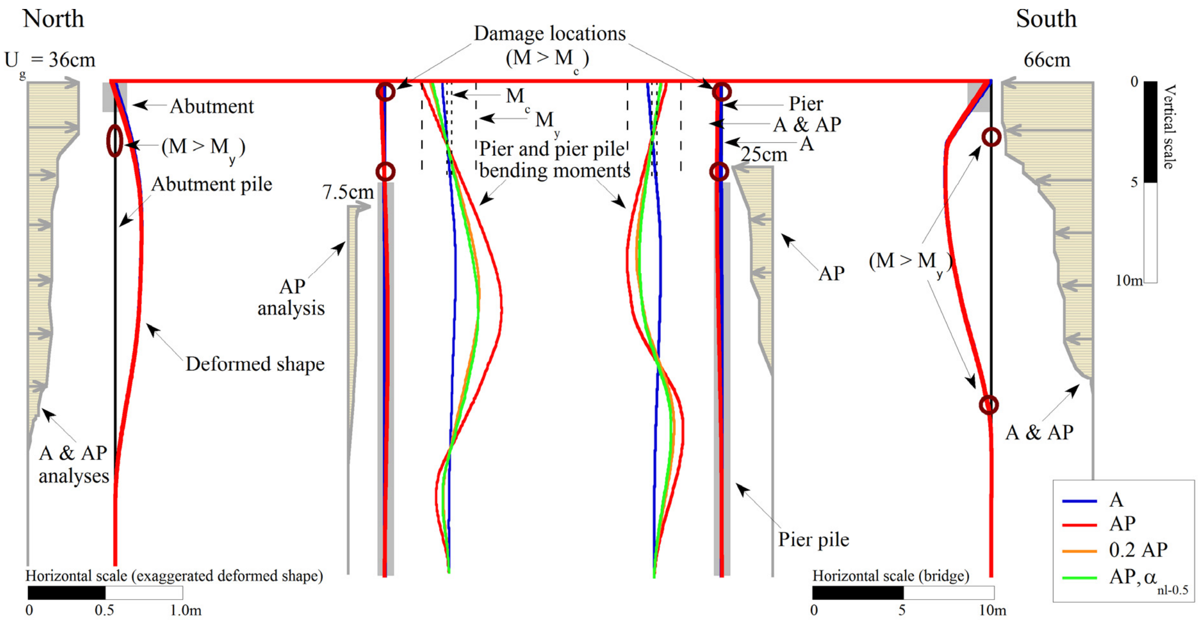

Figure 5. Schematic plot of global bridge model analysis showing applied ground displacement at the abutment piles, ground displacements applied along the pier piles, deformed shape of the bridge, damage locations where M > My, and bending moments along the piers and pier piles (Cubrinovski et al. 2014); A = Analysis in which ground displacements were applied to the abutment piles only; AP = Analysis in which ground displacements were applied to the abutment piles and pier piles.

3 EFFECTS OF SHALLOW LIQUEFACTION ON CBD BUILDINGS

3.1 Liquefaction within the CBD

Soil liquefaction adversely affected the performance of many multi-storey buildings in the CBD resulting in total and differential settlements, lateral movement of foundations, tilt of buildings, and bearing failures. In this section, one case history is presented to illustrate a characteristic liquefaction-induced damage to shallow foundations of CBD buildings with further details given in Cubrinovski et al. (2011a) and Bray et al. (2014).

Figure 6a shows the observed liquefaction in the CBD as documented 10 days after the Christchurch earthquake. The principal zone of moderate-to-severe liquefaction (red area) 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 Avenue Bridge. This zone is of particular interest because moderate to severe liquefaction affected many high-rise buildings on shallow foundations and deep foundations in different ways. Even though the map shown in Figure 6a distinguishes the zone most significantly affected by liquefaction, the severity of liquefaction within this zone was not uniform. The manifestation of liquefaction was primarily of moderate intensity with relatively extensive areas and volumes of sediment ejecta. There were 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. This non-uniformity in liquefaction manifestation reflects the complex and highly variable soil conditions even within the principal liquefaction zone in the CBD and the fact that this zone largely coincides with the path of the Avon River and network of old streams. The solid black lines (areas) indicate zones of pronounced ground distress caused by liquefaction. Within the area affected by liquefaction in the CBD, detailed investigations have been conducted in eight zones in a collaborative research study between the University of Canterbury and University of California, Berkeley (Cubrinovski et al., 2011a; Bray et al., 2014).

3.2 Effects of liquefaction in foundation soils

The case history presented herein is in the investigation Zone 8 at the intersection of Madras and Armagh Streets. At this site, the liquefaction was manifested by a well-defined, narrow zone of surface cracks, fissures, and depression of the ground surface about 50 m wide, as well as water and sand ejecta (Cubrinovski et al., 2011a). This liquefaction feature stretched from the Avon River towards south, and affected the southeast corner of the 6-storey CTUC building, shown in Figure 6b. The CTUC building was on shallow foundations consisting of 2.4 m x 2.4 m isolated footings (approximately 0.6 m deep) connected with tie beams and perimeter grade beams. A side view of the building looking towards the west, shown in the figure, indicates differential settlement of the southeast corner of the building of approximately 25 cm, which led to serious foundation and structural damage. Sand ejecta surrounded the southeast corner of the building presenting clear evidence that liquefaction was the most likely cause of the differential settlements.

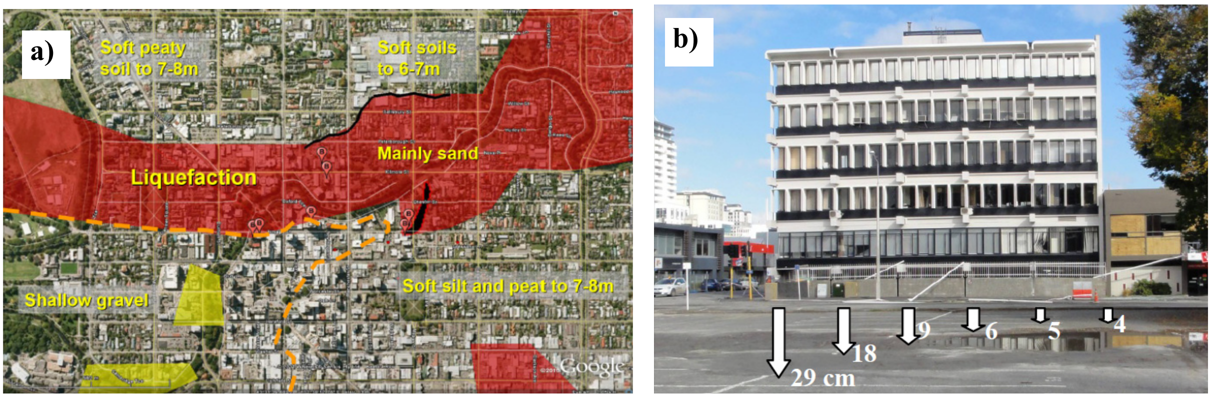

Figure 6. (a) Liquefaction map for the CBD (Christchurch earthquake) indicating zones of moderate-to-severe liquefaction (red) and zones of pronounced ground distress (black); yellow areas indicate zones of low (traces) to moderate liquefaction; (b) liquefaction-induced settlements of CTUC building

To further investigate the direct cause of the observed differential settlements, detailed CPT investigations were performed including 6 CPTs at the CTUC building site, and 15 CPTs at the nearby Armagh-Madras parking lot site (Bray et al., 2014). The latter involved closely spaced CPTs at 3m to 5m intervals aiming at depicting details of the ground profile across the previously mentioned liquefaction feature and its boundaries. Bray et al. (2014) used these data to develop detailed soil profile and investigate the variability of soil conditions over relatively short distances. They also conducted liquefaction triggering analyses based on the CPT data for the four most significant earthquakes affecting Christchurch in the 2010-2011 Canterbury earthquake sequence. Results of the triggering analyses are illustrated in Figure 7 where layers that have liquefied according to the triggering analysis are indicated in orange and red. The footprint of the CTUC building is also shown in the figure.

It is evident from Figure 7 that triggering of liquefaction is predicted at depths greater than 10 m across the whole area of interest. The distinguishing difference however is the presence of shallow liquefiable soils just beneath the foundations at the southeast corner of the building (CPT Z4-5 in Figure 7). This layer is not continuous however, and it was not present near the centre of the east side of the building or at the northeast corner of the building. This drastic change in the shallow soil conditions from the building's north end, which did not contain shallow liquefiable soils, to its south end, which contained shallow liquefiable soils, led to significant differential settlement over the southernmost spans of the building frame. Liquefaction of shallow soils just beneath the foundations of buildings was identified as a direct cause for excessive differential and total settlements of several other CBD buildings that were also subject of our detailed investigations clearly identifying the shallow liquefaction as a key factor in the poor performance of many CBD buildings on shallow foundations. Such damaging effects of shallow liquefaction for the buildings on shallow foundations were largely due to the direct disturbance of the foundation soils and loss of foundations soils due to sediment ejecta.

The shallow soils at the southeast corner of the building that were responsible for the shallow liquefaction and consequent large differential settlements were sandy silt to silty sands, with nonplastic fines of up to 50%, soil behaviour type index of I_c = 2.0 – 2.5, and cone tip resistance of about 3 MPa to 5 Mpa. This information is important in the context of our on-going research efforts to identify the composition and conditions of the soils that were responsible for the extensive damage caused by liquefaction in Christchurch.

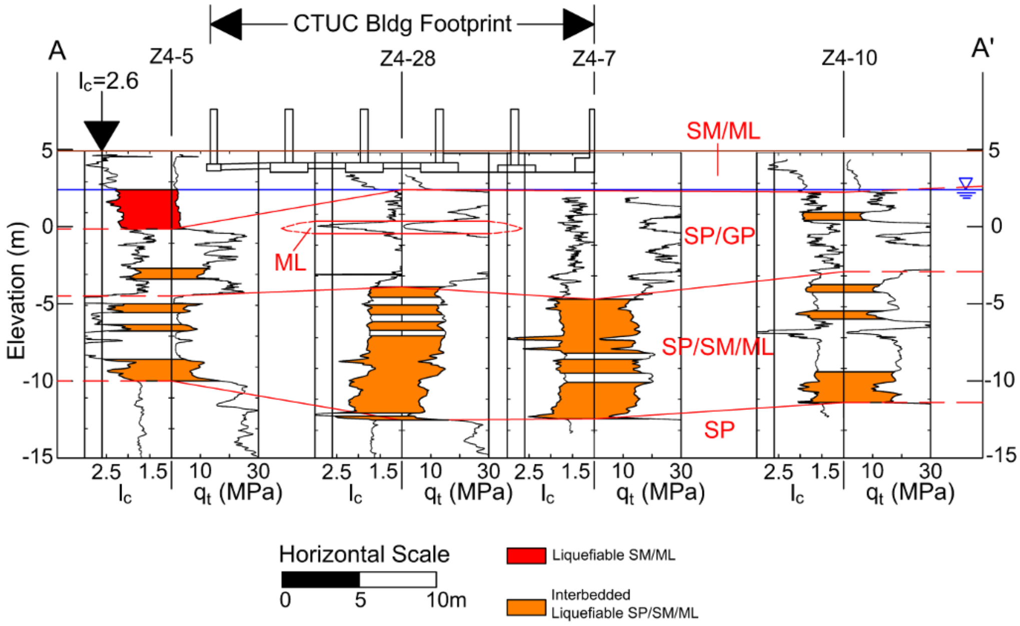

Figure 7. Subsurface conditions at CTUC building site indicating layers that have liquefied in the Christchurch earthquake according to CPT-based triggering analysis; note the presence of shallow liquefiable soils just beneath the foundations at the southeast corner where the largest settlement of ~30 cm was observed (Figure 6b)

4 LIQUEFACTION-IMPACTS ON BURIED PIPE NETWORKS

4.1 Correlation between liquefaction and damage to pipes

Buried pipe networks suffered extensive liquefaction-induced damage in the 2010-2011 Christchurch earthquakes over approximately one third of the city area. The wastewater system of Christchurch was hit particularly hard resulting in numerous failures and loss of service to large areas. Out of the 1766 km long wastewater pipe network, 142 km (8%) were out of service and 542 km (31%) were with limited service nearly one month after the February earthquake. Typical damage to the wastewater 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). A number of pump stations were taken out of service and the wastewater treatment plant suffered serious damage and barely remained in operation though with significantly diminished capacity.

The potable water system was proven to be much more resilient. Even though a large number of breaks/repairs of the water pipes have been reported, the water supply service was quickly restored. In order to examine the relation between the repairs (faults) of the potable water system and observed liquefaction severity, GIS analyses were performed using the pipe network damage data and liquefaction observation maps (Cubrinovski et al., 2011b). Figure 8 shows the location of repairs/faults on the watermains network (red symbols) following the 22 February 2011 earthquake. Superimposed in the background of the figure (with red, orange and yellow colours) is the liquefaction map indicating the severity of liquefaction (and associated land damage) induced by this earthquake. According to the GIS analyses approximately 80% of the breaks occurred in areas affected by liquefaction.

The analyses also revealed that more ductile pipelines (e.g. polyethylene - PE and polyvinyl chloride - PVC pipelines) suffered significantly less damage (three to five times less on average) than asbestos cement (AC), galvanized iron (GI) and other brittle material or ‘rigid’ pipelines. Figures 9a and 9b summarize the performance of different pipe materials (PVC and AC pipelines) and clearly indicate the difference in the performance of different pipe materials, and also the increase in the watermains damage with increasing liquefaction severity.

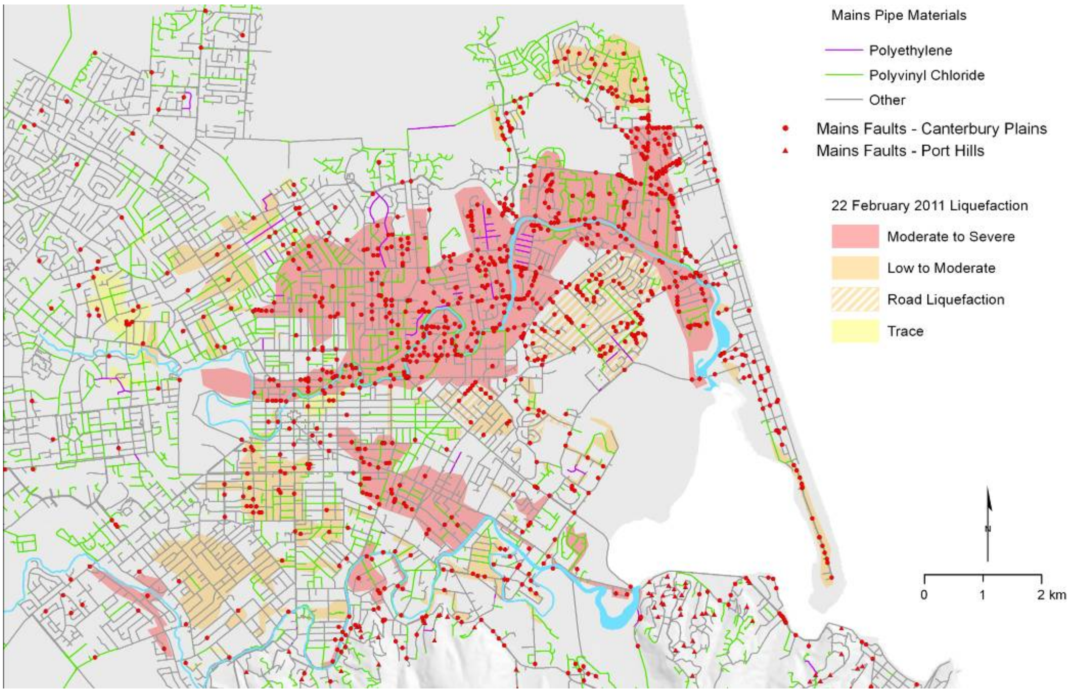

Figure 8. Locations of breaks (repairs) to the watermains after the Christchurch earthquake; liquefaction map for the same event is shown in the background

4.2 Ground deformation: key damage intensity measure

O'Rourke at al. (2014) performed more detailed analysis quantifying the ground distortion and strains in liquefied areas using LiDAR data (CERA, 2012). Using vertical ground surface movement from LiDAR surveys, differential vertical displacements were first defined between two adjacent points, and then angular distortion was calculated by dividing the differential vertical movement by the horizontal distance between the two points. In a similar fashion, lateral movement derived from the LiDAR surveys was used to calculate spatial distribution of lateral ground strains in areas affected by liquefaction.

Figures 9c and 9d show the correlation between the damage to the pipes (in terms of repairs per kilometer) and ground distortion, expressed in terms of angular distortion and lateral ground strain as summarized by O'Rourke at al. (2014). The approximated linear relationships show clear increase in the damage to both water (AC, CI, PVC) and wastewater pipelines (EW, RCRR, CONC) with the angular distortion and lateral strain of the ground. Figure 9c shows that AC pipelines are especially vulnerable to differential vertical movement, with repair rates 1.5 to 3 times higher than those for CI pipelines at comparable levels of β. O'Rourke et al. (2014) attributed this vulnerability in part to the use of a relatively weak AC collar to join adjacent lengths of pipe which is susceptible to cracking in response to relative rotation caused by differential settlement.

The evidence on the performance of buried pipelines suggests that soil liquefaction was the principal factor for the extensive damage to the pipelines, and that the assessment of earthquake-induced damage to buried pipelines should be based on ground deformation/distortion parameters.

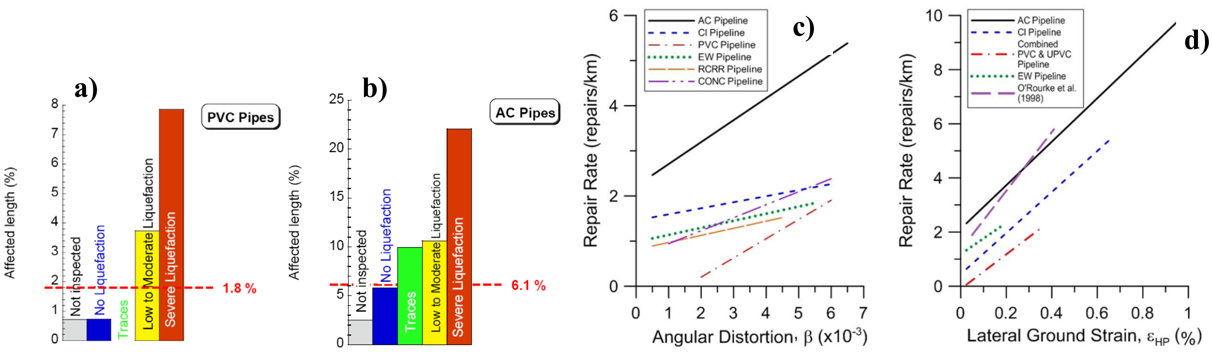

Figure 9. (a,b) Summary of damage to PVC and AC watermains due to the Christchurch earthquake (Cubrinovski et al., 2011b); (c, d) Repair rates for pipelines of different materials as a function of angular distortion and lateral ground strain (O'Rourke et al., 2014)

5 CONCLUSIONS

When subjected to large spreading displacements of liquefied soils, the short-span bridges with rigid superstructures resisted the ground movement through a characteristic mechanism involving deck-strutting and back-rotation of the abutments, which led to permanent displacement (damage) of the abutment piles and slumping of the approaches. All bridges affected by lateral spreads exhibited this damage/deformation mechanism. The ground displacements in the foundation soils of the abutments were substantially smaller or ~50% of the spreading displacements in the free field. Thus, reduced free field displacements and whole bridge model (analysis) should be employed when evaluating the effects of spreading for such bridges using an equivalent static analysis (PSA).

Liquefaction of shallow soils just beneath the foundations of buildings has been identified as the direct cause for excessive differential settlement of multi-storey CBD buildings on shallow foundations. In the particular case study, the critical layer responsible for the shallow liquefaction was silty sand to sandy silt, with nonplastic fines of up to 50%, soil behaviour type index I_c = 2.0 – 2.5, and cone tip resistance of 3-5 Mpa.

Nearly 80% of the damage to the water pipelines was caused by liquefaction, with clear trends in the correlations for an increase in the damage to pipelines with increasing liquefaction severity. Moreover, the damage to the buried pipelines directly correlates with the magnitude of ground distortion (deformation) as expressed by the angular distortion and lateral ground strain which in turn explains why liquefaction is the principal seismic-hazard for buried pipelines.

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