Probabilistic Assessment of Increased Flooding Vulnerability in Christchurch City after the Canterbury 2010-2011 Earthquake Sequence, New Zealand

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ABSTRACT: Major earthquakes can extensively transform the above and below ground natural and built environments of cities, leading to decreased drainage system capacity and, ultimately, to Increased Flooding Vulnerability (IFV). This has been the case for Christchurch city in New Zealand, which experienced the 2010 to 2011 Canterbury Earthquake Sequence (CES). These seismic events were followed by extreme rainfall in March-April 2014, with much of the city experiencing damaging flooding. This paper uses data from a Christchurch case study to extend a recently-developed infrastructure damage simulation tool to the probabilistic assessment of earthquake-altered flood risk in a built environment. In particular, the focus is on the IFV caused by the earthquake-induced damage to the pipeline component of Christchurch's storm water system, which was analysed at both connectivity and capacity levels. The probabilistic analysis was carried out via a plain Monte Carlo simulation, enabling the uncertainty affecting several key parameters to be taken into account. Final analysis results are presented spatially and in the form of cumulative distribution of flood height, the latter being an impact metric of great interest for infrastructure owners and emergency managers.

INTRODUCTION

High magnitude earthquakes can significantly alter urban land and built environments, with effects such as ground subsidence and uplift, alterations of river channel capacity, slope and elevation, and damage to storm water systems. The altered urban environment may be exposed to Increased Flooding Vulnerability (IFV), including a higher probability for Critical Infrastructure (CI) systems such as buildings and lifelines to suffer greater flood depths and/or extents in response to future rainfall events compared to pre-quake scenarios. The case study city, Christchurch, New Zealand, was hit by the Canterbury Earthquake Sequence (CES), a series of strong earthquakes between September 2010 and December 2011 (Hughes at al., 2015). The IFV phenomenon post the CES was revealed when the city experienced extreme flooding across large areas in August 2012, June 2013 and March-April 2014 (EQC, 2014; Christensen and Gillooly, 2014; Allen et al., 2014), due to three flooding mechanisms: pluvial, fluvial and tidal (Fisher et al., 2014).

Published literature to date features few studies of IFV following significant earthquakes (e.g. Hughes at al., 2015; Tonkin & Taylor, 2014b). The goal of this paper is to provide another contribution to this emerging research field.

The paper presents an extension of a recently developed civil infrastructure simulation tool, namely Object-Oriented Framework for Infrastructure Modelling and Simulation (OOFIMS, 2010-2014) (see details in Franchin, 2014), to the probabilistic assessment of the earthquake-altered flooding risk on the built environment. The paper focuses, in particular, on IFV caused by the earthquake-induced damage to the pipeline component of the storm water system. The case study area was identified as the part of Christchurch that experienced flooding mainly due to storm water overflow during the March-April 2014 events.

Final analysis results are presented in terms of flood height, an impact metric that is of great interest to infrastructure owners and emergency managers, since it can be easily correlated to network performance metrics, such as road functionality levels (open, open for emergency, closed).

1. CHRISTCHURCH STORM WATER PIPELINE NETWORK

Figure 1a) shows the Christchurch's storm water pipeline network (light grey) as well as the sub-catchments' boundaries (dark grey). The system is made up of a number of hydraulic pieces of equipment, including: 1) pipelines (which can be gravity or pressure); 2) manholes (for maintenance access and replacements); 3) sumps (where ground/road surface water discharges into a pipeline); 4) pipe outfall structures or discharge locations (where a pipeline discharges into an open waterway, such as the Heathcote River, Avon River and Avon-Heatcote Estuary), with flap gates at the outlets.

Figure 1: a) Christchurch storm water pipeline network (sketched in light grey) and sub-catchments' boundaries (dark grey); the Heathcote and Avon rivers are also indicated in blue. b) Close-up on the network portion inside the selected sub-catchment.

If seen as a tree-like graph, the network consists of around 64,600 circular section pipes and 71,600 nodes. The diameters of most pipes range from 100 to 375 mm, while about 16% of the pipes have sizes in the 400 to 3000 mm range. The most common materials used for pipes are the following: Rubber Ring Joint Reinforced Concrete (RRJRC); Concrete (CONC); Asbestos Cement (AC); Corrugated Aluminium (AL); Cement Lined Steel (CLS); Cast Iron (CI); Polyvinyl Chloride (PVC), Modified Polyvinyl Chloride (mPVC), and Unplasticized Polyvinyl Chloride (uPVC); Polyethylene (PE) and High Density Polyethylene (HDPE). Other materials used are Earthenware (EW), Brick Barrel (BB), Cured In Place Pipe (CIPP) liner, Vitrified Clay Pipe (VCP) and galvanized corrugated metal (CORRGALV).

The pipeline storm water system was largely functional after the CES: Closed Circuit

Television (CCTV) camera investigations indicated no significant damage to pipes citywide. Most damage was localised in the central and eastern city areas, with cracking of roadside guttering, joint breakages, loss of gradient, pipe collapses (particularly for older EW pipes) and cracks in pipe walls, allowing groundwater and liquefaction silt to infiltrate into the pipes (liquefaction-induced blockage). Even though the system has not been assessed and repaired for damage in all its extent, in the recent years after the earthquake sequence it has been performing acceptably, with only moderate impacts from the damaged parts (SCIRT, 2014).

The case study area was selected as a part of the network within one sub-catchment, that during the March-April 2014 events was affected only by light pluvial flooding (fluvial and tidal flooding did not occur here). Figure 1b) shows the location and a close-up of the analysed network portion (light grey), of which only the pipes forming a connected graph were kept. The final system, sketched e.g. in Figure 2a), is composed of 224 gravity edges and 216 nodes. The prevailing materials are RRJRC and AC, but pipes made up of CONC, CI, PVC, uPVC, BB, EW and VCP are also present. Diameters range from 150 to 750 mm, while pipe slopes are in the [1.8,6.8] ‰ range.

2. METHODOLOGY

The assessment of urban flooding vulnerability in a seismically active area presents several input uncertainties as follows:

- *rainfall* (random rainfall durations, depths, and corresponding rainfall intensities).
- *surface runoff*, allowing rainwater to enter the storm water network;
- *regional seismicity* (randomised event magnitude and location, and corresponding local intensities at vulnerable components' sites);
- *network components fragility* as a function of local intensities (fragility functions);
- *functional consequences of physical damage* (network connectivity / flow analyses).

Some of the listed uncertainties (typically rainfall and seismicity) are *aleatory*, whereas the others are *epistemic*. The epistemic uncertainty on the parameters (or even the form) of the models for all of the above should also be taken into account.

In order to perform a probabilistic assessment of earthquake-induced increased flooding risk for the built environment, the OOFIMS tool (recently developed in an European project: see SYNER-G, 2012), was extended to allow the analysis of a pipeline storm water system subjected to earthquake and rainfall effects. The next section presents how the uncertainties for the fields listed above are treated in the tool.

2.1. Modelling and treatment of uncertainties

The Christchurch City Council (CCC, 2014) published a correlation analysis between rainfall intensity and known flooding events since 1950. It was found that urban flooding tends to occur during events when a threshold of 75 mm of rain per 40 hours is exceeded, corresponding to an average rainfall intensity of 1.875 mm/h. This Christchurch rainfall-induced-flooding threshold was chosen for the case study application. To gain further insight into the selected event, a depth-duration-frequency table for the area of interest was generated using the High Intensity Rainfall Design System (HIRDS) tool by NIWA (2014). It was seen that a rainfall intensity of 75 mm per 40 hours occurs at an Average Recurrence Interval (ARI) of two years, corresponding to a 39% Annual Exceedance Probability (AEP).

During a rain event, water falls on impervious and pervious urban surfaces. In the latter case, some rainfall infiltrates the soil and eventually the groundwater, while the rest becomes surface runoff. Both infiltration and surface runoff will eventually flow into an engineered open channel, watercourse or receiving waterbody. By contrast, for an impervious area most rainfall becomes runoff. Modern urban areas, such as that studied here, are characterised by concentrated human activity and extensive impervious surfaces and, hence, high runoff volumes and flooding vulnerability. Among the different approaches available in the literature to model surface runoff (Zoppou, 2001), a simple empirical approach was adopted - the *rational method*, which models peak runoff volumes without drawing the actual hydrograph (conservative assumption) as follows:

$$Q_{in,k} = \varphi_k \cdot i \cdot A_k \tag{1}$$

where $Q_{in,k}$ is the ingoing runoff flow for pipe k due to rainfall intensity i in the pipe's catchment of area A_k, while φ_k is the runoff coefficient, defined as the ratio between total runoff depth and total rainfall depth. This coefficient is a function of perviousness and topography, and its actual value for single pipes and catchments is affected by uncertainty (Dhakal et al., 2011). For the case at hand (a flat urban environment with private and, sometimes large, public green areas) it was considered uniformly distributed between 0.5 and 0.9, with each pipe catchment assigned a randomly generated value in this range.

Concerning the seismic hazard, rather than sampling values of intensity measures with the OOFIMS hazard module, it was decided to use the maps from the 22 February 2011 event, in terms of Peak Ground Velocity (PGV) and displacement due to Permanent Ground Deformation (PGD). The acronym PGD is used in this paper to indicate resulting displacement as well. In particular, in Christchurch both vertical and horizontal ground displacements occurred due to settlement and lateral spreading, respectively, resulting from extensive liquefaction and ground deformation. The observed value ranges were [42,54] cm/s for PGV, and [0.10,0.35] m for total (vectorial composition) PGD.

These two seismic intensity measures were needed for the case study application since buried pipelines are vulnerable to both ground shaking and permanent ground deformation (O'Rourke et al., 2014). In fact, the fragility model is usually given in terms of two Poisson repair rates per kilometre, functions of PGV and PGD, respectively. For this study the fragility functions provided by ALA (2001) for water supply pipes were adopted (PGV in cm/s and PGD in m):

$$\begin{aligned} \lambda_{repair}\,(PGV) &= K_1 \cdot 0.0024 \cdot PGV \\ \lambda_{repair}\,(PGD) &= K_2 \cdot 11.224 \cdot PGD^{0.319} \end{aligned} \tag{2}$$

where K_1 and K_2 are functions of the pipe material, soil, joint type and diameter, and λ_{repair} is returned in km^{-1}. The number of repairs N_L for the generic pipe is randomly generated using the highest repair rate as the mean of the Poisson distribution, λ. No information is provided in the fragility model about the nature of the generic repair, whether it is a leak or a break (e.g. loss of continuity due to joint pull-out or pipe separation after rupture). If $N_L > 0$, a number N_L of standard uniform numbers is sampled and compared with the pipe failure probability, which is a function of λ: in actuality such probability is set to 0.2 or 0.8, depending on whether damage was caused by PGV or PGD, respectively (ALA, 2001). If at least one sampled number is lower than the failure probability, the pipe is broken and removed from the network, while if no breakage occurs, the total leakage area is determined as the number of leaks multiplied by the opening area of one leak. Hwang et al. (1998) empirically derived the likely extent of a leak area as 3% of the total cross section area. In this study, such a percentage was randomly generated for each pipe and each simulation run from a uniform distribution between 1% and 5%. Based on pipe leakage area, the implemented model is also able to compute the groundwater infiltration through leaks, as a function of hydraulic head. The latter depends on groundwater depth, another parameter affected by uncertainty. Van Ballegooy et al. (2014) noted that the depth to groundwater decreased in some areas in Christchurch due to ground settlement from tectonic movement and compaction of liquefied soils. In this study, groundwater depth was considered to be uniformly distributed between 0.5 m and 1.5 m, considering that Christchurch is a gently sloping coastal plain with some areas affected by saturated soils and tidal effects.

Once the physical damage to pipes had been assessed and network topology updated, and the groundwater infiltration through leaks had been added to the runoff-induced inflow, the flow analysis of the pipeline system was performed. In particular, the Manning or Gauckler–Strickler equation was adopted for computing flow Q in a pipe of assigned diameter, slope and roughness, with free surface gravity flow conditions:

$$Q = \frac{1}{n} \cdot \sigma \cdot R^{2/3} \cdot S^{1/2} \tag{3}$$

where σ is the cross-sectional flow area, R is the hydraulic radius, S is the pipe slope and n is the Manning's roughness coefficient. The factor $1/n = K$ is known as the Gauckler–Strickler coefficient. The K value (or equivalent n) is difficult to assess accurately for each pipe as it depends on a number of variables, including pipe material and internal finishing (Chow, 1959). As such, K was randomly generated, for each pipe and each simulation run, from a uniform distribution between 70 and 90 $m^{1/3}s^{-1}$.

For a generic pipe, the total inflow discharge, Q_d, is computed as the sum of three terms: upstream pipe discharge, runoff-induced inflow and (for the damaged condition) groundwater infiltration through leaks. The demand flow Q_d is compared with pipe maximum flow capacity Q_c, computed by Eq. (3) . When inflow exceeds pipe capacity (i.e. $Q_d > Q_c$) the free surface gravity flow condition changes to a pressure flow condition: as such, only a part of the inflow, equal to the maximum capacity, can be drained downstream, while the remainder, ($Q_d - Q_c$), overflows through grilled drains onto the ground surface (Hsu et al., 2000). For a broken pipe, 100% of Q_d is considered to overflow.

After overflow was estimated for all network pipes, the flood height, h_f, was computed through a simplified 2D overland propagation model, summarised as follows. The overflow discharges for all pipes were multiplied by an assumed rain event duration, set to one hour for the case study, thus obtaining the overflow volumes. It has to be remarked that the same duration is assumed for both undamaged and damaged conditions, therefore not influencing the increased flooding vulnerability assessment. A generic pipe k, with a catchment of area A_k, is supposed to exchange (deliver and receive) its overflow volume with its adjacent pipes p_j, having catchments of area $A_{p,j}$, based on the assumption that the respective catchments are adjacent as well. In particular, the volume for pipe k is exchanged with the adjacent pipes according to an area ratio where the catchment area of one pipe (k or p_j) is considered over the total catchment area: $A_k + \sum_j A_{p,j}$. As such, for smaller pipes characterised by smaller catchments, the outgoing volume will be more than the incoming one, resulting in an accurate flooding height. At the end of the iteration over all pipes, the *updated* overflow volumes are divided by the respective catchment areas, giving an estimation of the flood height.

3. RESULTS

One Monte Carlo simulation (MCS) with 1,000 runs was carried out, which yielded stable estimates of all considered performance metrics. The simulation results are presented below.

Figure 2a) shows a map of the expected maximum number of repairs for all pipes, given that pipes are not broken: such repairs must thus be considered for leaks, which are likely to be caused by PGV (the contours of which are superimposed on the map), since breakage is not accounted for. On the other hand, a contour map of pipe breakage probability is shown in Figure 2b). These metrics indicate the damage scenario one could expect from any future earthquakes similar to the February 2011 event. Infrastructure owners may use such information for retrofit planning purposes.

Figure 3a) shows a contour map of expected overflow discharges, concentrated at pipe centroids for simplicity. It is to be noted that the reported values are averaged over the simulation, hence they are referred to the seismic condition with pipe damage and leak infiltration taken into account. The main flow direction is from NW to SE, but outlet pipes are also located in the southern and northern portion of the network: this explains the positions of the highest discharges in the map. Since no overflow is predicted in the reference, undamaged condition, this map is a first indicator of the reduced drainage capacity of the storm water network, due to earthquake-induced pipe damage.

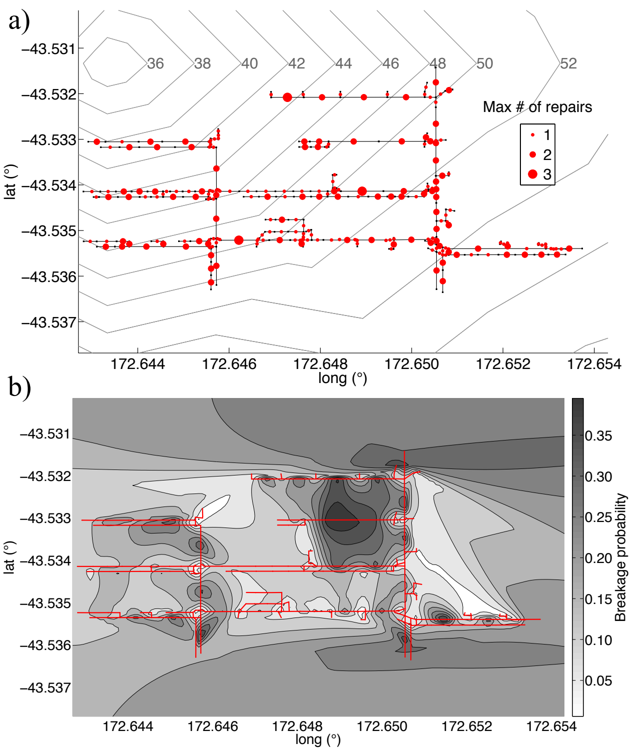

Figure 2: a) Expected maximum number of repairs given that pipes are not broken, with PGV isolines in cm/s superimposed. b) Breakage probability contour map, with the case study network portion sketched in red.

It should be noted that the overflow probability contour map (Figure 3b) is very similar to that related to breakage probability (Figure 2b). This is consistent with the model assumption that for broken pipes, 100% of inflow discharge becomes overflow. The similarity is also due to the presence of geotechnical hazards, in particular the permanent ground deformation caused by liquefaction. Indeed, many studies suggest that inside zones of observed liquefaction, PGD is likely to be the primary cause of damage (O'Rourke et al., 2014). Also, as highlighted earlier, the employed fragility model assigns repairs caused by PGD an 80% probability to be breaks (ALA, 2001). Hence, it is possible to conclude that for the case study network portion, assumed rainfall intensity and observed ground shaking and deformation, the overflow occurrence is driven by pipe breakage (i.e. no overflow is expected to occur when pipes are intact, possibly with some leaks).

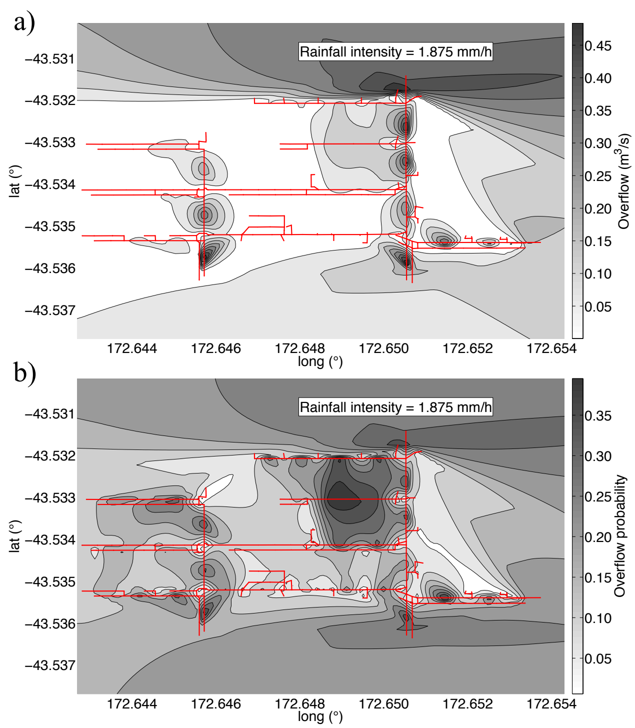

Figure 3: a) Expected pipeline overflow. b) Overflow probability contour maps.

Figure 4 shows a contour map of the expected flood heights or road water depths. This performance metric represents the last step in the IFV assessment. Flood heights for all pipe catchments were estimated following the procedure in Section 2.1, starting from the overflow values (Figure 3a) averaged over the simulation. As expected, the flood height map closely resembles the overflow map, except that the overflow peaks (dark-coloured areas) became smoothed in the computation of flood height, due to the redistribution of overflow volumes between adjacent pipe catchments. Maximum flood height values of around 30 cm are only expected in some localised zones. The case study area would therefore not be expected to suffer severe flooding under the assumed rainfall intensity, even after a significant earthquake. These results are in line with the observed flooding heights in Christchurch after the March-April 2014 events (e.g. Tonkin & Taylor, 2014a). A detailed model validation against observed flooding is outside of the scope of this paper. The flood height map for the undamaged condition is not reported since ‘no overflow’ was predicted.

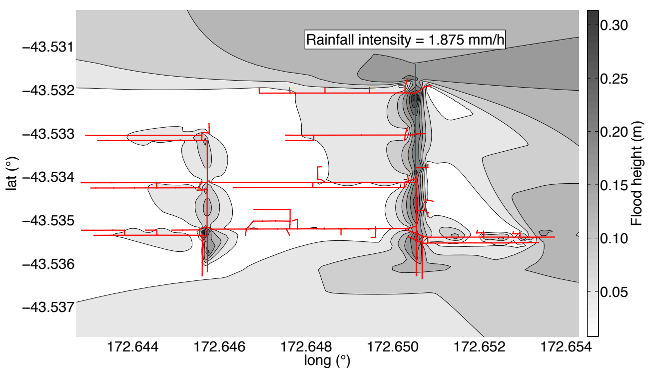

Figure 4: Expected flood height contour map.

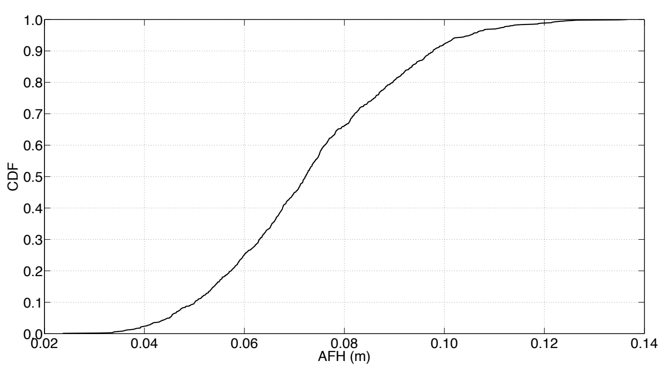

Figure 5: Cumulative Distribution Function (CDF) of average flood height.

Given the h_f map for a generic run, the average flood height (AFH), defined as the average of h_f over the network pipes, can be computed. Figure 5 shows the Cumulative Distribution Function (CDF) of AFH. The highest values are around AFH = 0.14 m, corresponding to simulated scenarios where the network suffers from severe damage to pipes and is subjected to high inflow discharge due to both runoff and groundwater infiltrating through leaks.

4. CONCLUSIONS

This paper presents a methodology for the probabilistic assessment of increased urban flooding vulnerability induced by damage from major earthquakes. This scenario occurred in Christchurch city, New Zealand, after the 2010-2011 earthquake sequence. IFV was observed during the subsequent high-intensity rainfall events of March-April 2014. A recently-developed simulation framework for vulnerability and risk assessment (OOFIMS, 2010-2014) was extended to simulate earthquake-induced physical damage to a piped storm water network, plus its reduced drainage capacity when subjected to intense rainfall. A Monte Carlo simulation was employed to carry out a probabilistic analysis including the treatment of several uncertainties in the problem. The model was able to capture the water overflow through grilled drains in case of pressure flow conditions and to estimate the corresponding road flood heights, an impact metric of great interest for infrastructure owners and emergency managers. Final results, presented in terms of expected values and distributions of flood heights, are in line with observations in the city from recent rainfall events. Future work aims to include a rainfall event model as well as the complete piped and un-piped drainage system in the simulation tool. This should allow us to produce more complex and detailed results, and to conduct a thorough model validation.

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