LIBRARY
Te Tumu Herenga
THE UNIVERSITY OF AUCKLAND

RESEARCHSPACE@AUCKLAND

**http://researchspace.auckland.ac.nz**

***ResearchSpace@Auckland***

**Copyright Statement**

The digital copy of this thesis is protected by the Copyright Act 1994 (New Zealand).

This thesis may be consulted by you, provided you comply with the provisions of the Act and the following conditions of use:

- Any use you make of these documents or images must be for research or private study purposes only, and you may not make them available to any other person.
- Authors control the copyright of their thesis. You will recognise the author's right to be identified as the author of this thesis, and due acknowledgement will be made to the author where appropriate.
- You will obtain the author's permission before publishing any material from their thesis.

To request permissions please use the Feedback form on our webpage. http://researchspace.auckland.ac.nz/feedback

**General copyright and disclaimer**

In addition to the above conditions, authors give their consent for the digital copy of their work to be used subject to the conditions specified on the Library Thesis Consent Form and Deposit Licence.

**Note : Masters Theses**

The digital copy of a masters thesis is as submitted for examination and contains no corrections. The print copy, usually available in the University Library, may contain corrections made by hand, which have been requested by the supervisor.

# The Effects of Earthquakes on Wastewater Pipelines in New Zealand: Evaluation and Rehabilitation

**Mohammad Reza Zare**

*A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy,*

*Supervised by Dr Suzanne Wilkinson,*

*The University of Auckland,*

*Department of Civil and Environmental Engineering*

*New Zealand*

*November 2012*

# Abstract

*Utility managers are always looking for appropriate tools to estimate seismic damage in wastewater networks located in earthquake prone areas. Fragility curves, as an appropriate tool, are recommended for seismic vulnerability analysis of buried pipelines, including pressurised and unpressurised networks.*

*Fragility curves are developed in pressurised networks mainly for water networks. Fragility curves are also recommended for seismic analysis in unpressurised networks. Applying fragility curves in unpressurised networks affects accuracy of seismic damage estimation. This study shows limitations of these curves in unpressurised networks.*

*Multiple case study analysis was applied to demonstrate the limitations of the application of fragility curves in unpressurised networks in New Zealand. Four wastewater networks within New Zealand were selected as case studies and various fragility curves used for seismic damage estimation. Observed damage in unpressurised networks after the 2007 earthquake in Gisborne and the 2010 earthquake in Christchurch demonstrate the appropriateness of the applied fragility curves to New Zealand wastewater networks.*

*This study shows that the application of fragility curves, which are developed from pressurised networks, cannot be accurately used for seismic damage assessment in unpressurised wastewater networks. This study demonstrated the effects of different parameters on seismic damage vulnerability of unpressurised networks.*

# Acknowledgements

I would like to express my special thanks to my supervisor Dr Suzanne Wilkinson, who has supported me at all stages of my study and has encouraged me to work hard for better research outcomes.

I would also like to thank my co-supervisor Dr Regan Potangarua, whose advice has been invaluable to me in completing this study.

Financial support given to me by the Retrofit Solution project, under the directorship of Dr Jason Ingham during my study at the University of Auckland was greatly appreciated.

All participants who have provided information and recommendations are gratefully acknowledged, including the Hutt City Council, the Gisborne Regional Council, the Marlborough District Council, the Christchurch City Council, the Waimakariri District Council, the Wellington City Council, New Zealand Engineering Lifelines, Capacity Company, GNS science, MWH, BECA, and Water Care. In this study, numerous individuals have provided valuable data and recommendations and deserve acknowledgement; they are Yon Cheong (Capacity Company & Hutt City Council), John Zhao, Jim Cousins and Grant Dellow (GNS science), Bruce Sherlock, John Floyd, Ross Corkin and Lani Fatagi (Hutt City Council), Steve J. Hutchison (MWH), Dave Brunsdon (National Engineering Lifelines), Mark Nelson (the Blenheim City Council), John Rentoul and Dave Hemington (Gisborne District Council), Mark Gordon (AECOM), Gerard Cleary (Wimakariri District Council), Veronika Frank (Christchurch City Council), Keith Adam (University of Auckland), Jaberi, Mortazavi, Hekmatian, Mollabashi and Rostami (Isfahan Water and Sewage Company), Abbas-zadeh and Razi (Isfahan Water and Wastewater Research and Design Consulting Engineers), Hossein Mirmohammad Sadeghi (Isfahan Higher Education and Research Institute) and Saeed Nadi (Esfahan University).

I dedicate the thesis to my wife Leila because of her encouragement, passion and responsibility for our family, and to my son Amir Hossein because I did not share enough time with him and missed him badly during my study.

I also dedicate the thesis to my mother (Sedigheh Hoghoughi Esfahani) and father (Hassan) because of their encouragements and passions.

# Table of Contents

# List of Figures

**List of Tables**

# Chapter 1. Introduction

## 1.1. Wastewater system components

Wastewater systems in modern societies are an integral part of urban facilities and are classified as lifelines; this means that they are essential services for society to function well. The wastewater system is not only a lifeline but it also is one of the most capital intensive municipal investments. Sewer systems cover approximately 40% of the value of a municipality's inventory (Allouche & Parhami, 2003). As freshwater is necessary for human life, transferring and treating the wastewater are equally essential. Wastewater systems receive industrial and residential wastewater as well as storm water. Storm water usually mixes with the collected wastewater, especially during rainy seasons. Wastewater systems are sometimes designed to collect wastewater and storm water simultaneously.

A wastewater network is the main component of a wastewater system which comprises of three main parts; the wastewater network, the pumping stations and the treatment plants. Wastewater networks are designed to collect and transfer the received wastewater. This is collected in small diameter pipes, and then transferred in a medium to large diameter pipes either by gravity or pumping stations to a wastewater treatment plant. Wastewater pipelines are not only the main part of the collecting system but also an essential component of the transferring and discharging network.

Gravity is commonly used to transfer fluids, particularly wastewater; however, the region's topography is the main factor which hinders application of gravity especially in flat areas. Consequently, these networks are usually designed to carry the collected wastewater by the force of gravity. If the network is designed to carry wastewater by gravity force, pipelines should have a reasonable depth. Increasing the burial depth requires significant investment. Technical and financial issues are the main barriers in applying gravity force. In order to overcome these difficulties in gravity designed wastewater network, pumping stations are the only technical choice.

Wastewater treatment plants are fed with wastewater to fulfil the required recycling standards. The collected wastewater passes through several treatment processes prior to be reused or released into nature. The volume of treated wastewater is significant especially in metropolitan areas. The treated wastewater can be used as irrigation water in an area with water shortage.

In countries with sufficient water resources such as New Zealand, the treated wastewater is usually released into nature; due to the restricted environmental laws, almost all of the treated wastewater is released back to the rivers or the sea. Moreover, transferring the treated wastewater to discharge points usually requires separate pumping stations and pipelines, thus increasing the required infrastructure.

Figure 1.1 shows the network scheme of a typical wastewater system, where pipelines are the main connectors of the different components of the system.

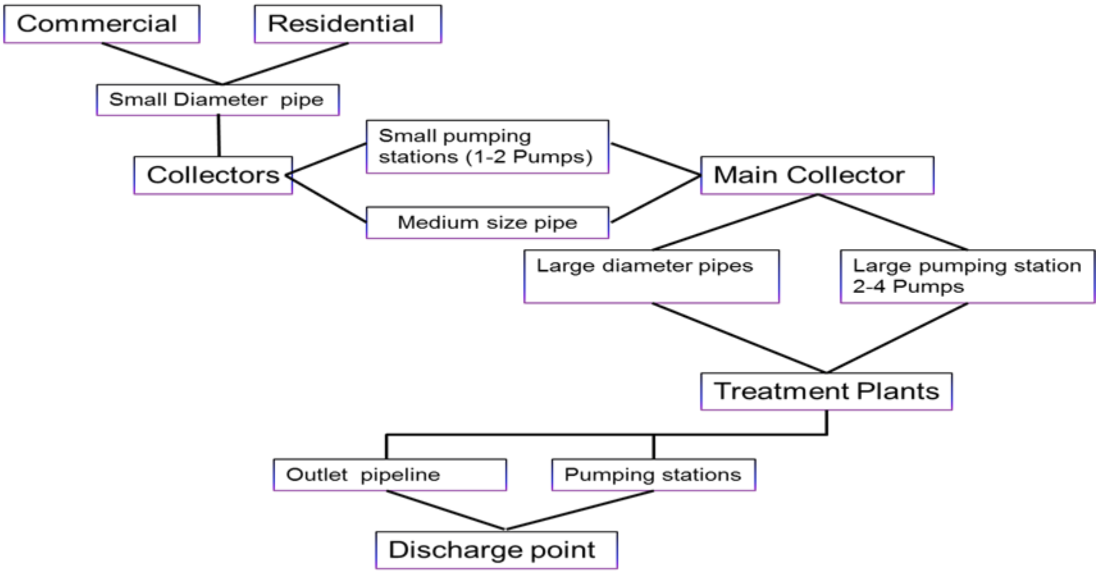


Figure 1.1: Wastewater system components

As illustrated, any failure in wastewater networks can seriously affect the function of two other components, thereby making the whole system fail.

One of the focuses of this thesis is on understanding the vulnerability of the buried wastewater pipelines against seismic shocks in New Zealand.

## 1.2. Background

The wastewater system as a lifeline is one of the fundamental structures in urban areas, and its failure would significantly affect the whole population within the region. The difference between failure of wastewater pipelines and the failure of other lifelines is that there is no alternative but to quickly replace wastewater network. Temporary methods used in an emergency situation are the provision of portable toilets and the digging of deep holes in gardens which are known as “long drops” in New Zealand. Due to high maintenance costs the utilization of portable toilets are expensive especially when used for long periods of time. Regions exposed to seismic shocks can only be supported for a short time with portable toilets before communities become frustrated and the public health becomes affected.

The 2010 and 2011 Christchurch earthquakes in New Zealand showed and continues to show how the failure of wastewater pipelines, can affect a community. These earthquakes proved how costly the repairs and restoration of the wastewater pipelines are.

Seismic effects on wastewater network can be categorised by the type of damage such as direct and indirect damages. Direct damage to wastewater pipelines is breakage and leakage. Environmental pollution, including water and soil pollution, is the indirect damage. Wastewater network can fail immediately after an earthquake due to severe damage to pipelines such as breakage and collapse of pipes. Sometimes wastewater network does not fail immediately after an earthquake. For instance, cracks and leakages caused by an earthquake may cause pipe failure after several days, weeks or months.

An earthquake causes significant damage to wastewater network and as a result causes environment pollution and difficulties for residents of affected urban areas. For instance, the 2010 Chile earthquake, the 2010 and 2011 Christchurch earthquake are some examples of the recent earthquakes which significantly affected wastewater network of the affected zones (EERI, 2010; Shinozuka, 1995). Furthermore, the 1987 Edgecombe New Zealand, the 1989 Loma Prieta, the 1994 Northridge in the US, the 1999 Taiwan and the 2004 Nigata earthquakes in Japan are examples of how badly seismic forces could affect wastewater pipelines (Lund et al., 1998; Pender, 1987; Scawthorn et al., 2006; Schiff et al., 2000).

In the past, the urban planners concentrated on lifelines which provided consumable fluids and energy to the earthquake prone areas, such as potable water, gas and energy. During the past decades, the population in seismic prone regions such as Christchurch has increased, however, the post-earthquake population has decreased (Bascand, 2011). Pollution and environmental issues including the wastewater system failure have become the focus point in the event of an earthquake (Baratz, 2010). As a consequence, the significance of a sustainable wastewater system in seismic prone areas is one of the main concerns of city utility management and policy makers. Not enough research is conducted regarding how the wastewater network in seismic prone regions can be more resilient to seismic shocks.

Improvements to wastewater network can be made to decrease earthquake impact on affected communities. The pre-earthquake and post-earthquake measures can be applied to improve the seismic sustainability of wastewater network in earthquake prone regions. Pre-earthquake measures are designed to reduce earthquake effects and include better prediction, preparedness, awareness, codes and structural specification (Bruke, 1999). Emergency management, recovery and rehabilitation plans are the main post-earthquake measures used in reducing the damage impact on wastewater system when an earthquake occurs (Pan American Health Organization, 1998; Shuji, 2005). Appropriate seismic damage estimation in buried pipelines is the first step of any pre and post-earthquake strategy.

One of the main concerns of this study is to identify the factors which can affect the estimation of seismic damage on unpressurised wastewater networks. Fragility curves are well known tools used for seismic vulnerability assessment of buried pipelines. Regarding the significant effects of seismic forces on buried pipelines, many researchers have developed various fragility curves in order to contribute to expected damage assessments (Toprak, 1998).

## 1.3. Research overarching aim

The overarching aim of this thesis seeks to understand the limitations of the present fragility curves in evaluating seismic vulnerability of unpressurised wastewater pipelines in New Zealand. The limitation of fragility curves to unpressurised networks have not, to date, been established. Therefore, fragility curves are used as the basis for seismic calculation to damaged pipelines. This research challenges the use of fragility curves in unpressurised networks. This aim will be achieved using the case study method.

Some earthquakes have caused severe damage to both the pressurised (gas and water) and the unpressurised pipelines, especially in the recent earthquakes of heavy magnitude. The severe earthquake-induced damage experienced in buried pipelines encouraged researchers to find a way to assess seismic damage in buried pipelines. As a result, they recommended fragility curves to assess seismic damage. The literature review shows that fragility curves are usually applied to estimate seismic damage in buried pipelines including pressurised and unpressurised networks. Consequently, fragility curves are applied in this study to assess seismic vulnerability in unpressurised wastewater networks in New Zealand.

Fragility curves are developed from correlation of the observed damages in pressurised networks to earthquake relevant parameters in some countries, such as in the United States. As a result, assessment of unpressurised wastewater networks in New Zealand is expected to have some limitations. The reason for the expected limitation is that there are no fragility curves developed specifically for the New Zealand situation. The applied fragility curves have been developed from pressurised networks overseas.

Seismic assessment of all the unpressurised wastewater networks in New Zealand is not only time consuming and costly but requires intensive work to collect and analyse the required date. Such a task is beyond the scope of this thesis, consequently, four unpressurised wastewater networks within New Zealand are selected as case studies to achieve the overarching aim of this research.

## 1.4. Research questions

In order to achieve the aim of this study, the following 6 questions have been posed.

Question 1: How vulnerable are unpressurised wastewater networks against seismic shocks in New Zealand and are pre or post-earthquake measures required to decrease seismic damage in these networks?

New Zealand is located in an active seismic-prone region. It has experienced some devastating earthquakes but there is no accurate and comprehensive study regarding damages in buried pipe networks, particularly in unpressurised wastewater networks. On the other hand, there is no pre or post-earthquake plan to decrease seismic damage in unpressurised networks in New Zealand.

Much overseas research has focused on seismic damage in pressurised pipeline networks, but there are only limited studies regarding seismic damage on unpressurised networks. Furthermore, the literature review shows that there is no study to demonstrate seismic vulnerability of unpressurised wastewater networks in New Zealand. As a result, a study is required to cover this gap and to highlight the idea that earthquakes can critically affect unpressurised wastewater networks in New Zealand cities. Consequently, pre or post-earthquake measures are required to decrease intensity of damage on unpressurised networks.

Question 2: Are the parameters which fragility curves are based on sufficient and if not what other parameters should be taken into consideration?

Fragility curves are mainly developed for a particular type of pipe such as cast iron pipes and pipe material is the only factor which is considered for pipe specification. On the other hand, unpressurised networks comprise of various types of pipe in terms of pipe materials. This as well as others factors affect seismic vulnerability of buried pipelines. As a result, application of fragility curves can cause differences in the calculated number of defects and identification of affecting parameters can improve application of fragility curves.

Question 3: How does the application of a range of fragility curves which have been developed from seismic damage on pressurised networks affect seismic damage estimation on unpressurised networks?

Vulnerability in a pipe network can be defined by the number of defects in a particular length of pipeline. Therefore, an appropriate assessment method should be selected to show the vulnerability of wastewater networks. This assessment method should estimate the number of defects which are expected in a selected wastewater network after a particular earthquake.

Reviewing the literature shows fragility curves are recommended as an appropriate tool to calculate the number of defects after an earthquake. Several fragility curves are developed by correlating the observed damage in the pressurised water or gas networks to some earthquake parameters. No fragility curve has yet been developed from seismic damage on unpressurised networks. On the other hand, some references recommend that the same fragility curves be used for seismic vulnerability analysis of unpressurised networks. The concern is that there is no evidence to show the accuracy of this application in unpressurised networks. As a result, application of fragility curves is likely to have some limitations when used to calculate seismic damages on unpressurised networks. Therefore, several fragility curves recommended by different researchers are used in this study to show the limitation in application of the present fragility curves for assessing seismic vulnerability of unpressurised wastewater networks.

Question 4: What is the appropriate application of fragility curves for estimation of seismic damage on unpressurised networks?

As explained, fragility curves are developed from evaluating seismic effects on pressurised networks and the assumption is that notable differences can also be seen when the same fragility curves are applied to unpressurised wastewater networks. On the other hand, fragility curves are recommended tools for seismic assessment of buried pipelines. As a result, theses curves are likely to have a common application when used in unpressurised networks for some purposes such as identifying the most and least vulnerable zones.

Question 5: How does the seismic damage rate in unpressurised pipelines differ when compared to the current fragility curves in pressurised networks?

The available fragility curves are based on the trend which shows an increase in the value of the earthquake relevant variables and this results in an increase in damage rates. The actual observed relationship in unpressurised networks can differ in comparison with that of the fragility curves. Furthermore, application of fragility curves in unpressurised wastewater networks can estimate damage even for a small value of independent variables, especially in networks with long pipe length. Hence, this study seeks to understand how the damage rate for unpressurised pipelines differs when compared to standard fragility curves.

Question 6: What are the current limitations of trenchless technique use for post-earthquake repair and rehabilitation of unpressurised wastewater networks?

It was during the writing of this thesis that an earthquake struck Christchurch on 4 September 2010. The earthquake caused major damage to the wastewater system and provided the opportunity to understand first-hand problems. The earthquake presented the opportunities to extend the study to understand the limitation of trenchless techniques as well as to test the seismic damage estimation theories in this thesis.

Trenchless techniques are known for their abilities and advantages for repairing and rehabilitating buried pipe networks in comparison with ordinary open-cut methods in urban areas. Evaluating types of earthquake induced defects can demonstrate limitations on application of these methods after an earthquake. There has been no study in the literature examining the applicability of these techniques for pipe repair and rehabilitation after an earthquake.

## 1.5. Research objectives

This research has 6 objectives which endeavour to find the appropriate solutions to achieve the overarching aim of this thesis. A particular objective is developed for each question in order to find an appropriate answer for each question of this thesis.

Question 1 of this study asks how vulnerable unpressurised wastewater networks are against seismic shocks in New Zealand and whether pre or post-earthquake measures are required to decrease seismic damage in these networks. Objective 1 is developed to answer this question. This objective is to demonstrate the extent of seismic damage on the unpressurised wastewater networks. in order to show the necessity of pre or post-earthquake measures in the wastewater networks.

Although significant damage is inflicted on buried pipelines through seismic shocks most often, this matter is not of serious concern with the authorities in terms of pre and post-earthquake measure in wastewater networks. Therefore the focus of this thesis is to investigate seismic vulnerability of wastewater networks and to demonstrate the significance of the measures which can be applied before and after earthquakes to reduce seismic vulnerability. This study identifies how vulnerable the wastewater pipelines are in 4 cities in different parts of New Zealand as case study sites.

Question 2 of this study concerns the appropriateness of the parameters which fragility curves are based on and identifies the key parameters that need to be taken into consideration. Therefore, the second objective of this study is to identify the parameters which need to be included in fragility curves in order to achieve more accurate results.

Various parameters such as pipe specification can affect seismic vulnerability of buried pipelines. Amongst others, researchers Katayama et al. (1975), Eguchi et al. (1983) and Isoyama et al. (2000) have worked on seismic vulnerability analysis of buried pipelines and have recommended fragility curves to evaluate earthquake induced damage in buried pipelines. Pipe material and parameters relevant to earthquakes are usually the only independent variables which are considered and applied in developing fragility curves. On the other hand, many other parameters which are not currently considered in fragility curves can have an effect in seismic vulnerability of buried pipelines. In this study these factors are analysed.

“How does the application of a range of fragility curves which have been developed from seismic damage on pressurised networks affect seismic damage estimation on unpressurised networks?” is an area of concern. Consequently, Objective 3 of this project is to compare the estimated defects in unpressurised wastewater pipes by a range of fragility curves.

Fragility curves are developed by correlation of earthquake parameters with earthquake damage to pressurised pipes. As specification of wastewater pipelines differ from pressurised pipes the difference can affect the estimated amount of damage calculated by fragility curves. These factors will be investigated and discussed for four case studies in New Zealand. In this thesis the seismic damage in each case study will be calculated using fragility curves.

Another project objective 4 is to determine the appropriate application of fragility curves for seismic damage estimation of unpressurised networks. This is to identify and recommend the most effective application of fragility curves in damage estimation of unpressurised networks.

Fragility curves are a common tool for estimating seismic damage in buried pipelines. On the other hand application of these curves can overestimate or underestimate expected damage in the unpressurised networks.

Pre and post-earthquake mitigation plans are applied to reduce the impact of seismic damage. This also needs to apply to wastewater networks. Proper application of these plans on buried pipeline rehabilitation is relevant for quick recovery. Accurate estimation of expected damage after any earthquake is an objective of the mitigation plans. How the application of fragility cures can affect accuracy of pre and post-earthquake measures will be discussed in this thesis.

Question 5 asks “how does the seismic damage rate in unpressurised pipelines differ when compared to the current fragility curves in pressurised networks?” Hence objective 5 identifies the relationship between damage rate and earthquake variables in unpressurised wastewater networks

On-site observation of the 2010 Darfield earthquake damage to wastewater pipelines provided an opportunity to identify the trend by comparing observed with that of calculated damage rates. This shows if fragility curve application is a reliable mean of estimating seismic damage in unpressurised wastewater network after an earthquake in New Zealand.

The final objective of this research is to discover the limitations in application of trenchless techniques for post-earthquake repair and rehabilitation in unpressurised wastewater networks. This is designed to answer Question 6 which asks what the current limitations of trenchless technique use are for post-earthquake repair and rehabilitation of unpressurised wastewater networks.

The September 2010 Christchurch earthquake known as the Darfield earthquake not only showed severity of seismic damage on wastewater pipelines but it also showed the advantages and disadvantages of some rehabilitation and repair methods adopted for system restoration as well. Direct observations of the Darfield earthquake damage to wastewater pipelines and the applied repair methods by the researcher showed the advantages and disadvantages of rehabilitation and repair methods used for restoration of damaged wastewater pipelines. However, since the 2010 Darfield earthquake was more severe than the earthquake in February 2011, the damage caused was more extensive to the wastewater system. This research was completed after the 2010 earthquake and does not cover the 2011Christchurch earthquake.

## 1.6. Structure of the thesis

This thesis comprises of 9 chapters. A brief description of the topic including the thesis questions and objectives are explained in Chapter 1 as an introductory chapter. General parameters of wastewater networks in New Zealand and also of analysing methods applied in seismic vulnerability assessment will be discussed in Chapter 2. This chapter will discuss the significance of earthquakes on wastewater networks. In this chapter the effects of past earthquakes on the wastewater systems will be analysed to show the effects of seismic forces on wastewater networks.

In Chapter 2, the different methods adopted for evaluation of seismic vulnerability of buried pipelines will be explained. Here the various earthquake parameters applied in fragility curves be discussed, but also different pipe specifications which affect pipe vulnerability will be taken into account. Factors affecting seismic vulnerability of buried pipelines will be discussed in this chapter as well. Furthermore, this chapter covers the fragility curves during calculation procedures and also reviews earthquake rehabilitation and repair methods.

The research methodology applied in this thesis will be discussed in Chapter 3. Different research methods will be discussed, and the research methods adopted in this study will be explained. The data collection procedure applied in the research and various types of data used will be explained. Case studies of selected seismic zones in New Zealand will be discussed in this chapter in addition to data analysis methods.

In order to provide an appropriate estimation of earthquake vulnerability of wastewater systems in New Zealand, four case studies were selected in the North and South Island. These cases were Hutt City and Gisborne in the North Island, Blenheim and Christchurch in the South Island. The seismic vulnerability of wastewater network in each case study will be evaluated using the available fragility equations.

The seismic vulnerability of the Hutt City wastewater network will be discussed in Chapter 4. In this and the following two chapters, first wastewater network in each case study will be briefly explained and the particular specification of each network will be taken into account. Description, classification and seismic vulnerability assessment of wastewater pipelines in each case study will be discussed in more detail in relation to their potential to damage due to seismic forces.

Characteristics of the Gisborne wastewater network and seismic vulnerability of wastewater network as the second case study will be discussed in Chapter 5. In this chapter, the 2007 Gisborne earthquake and its effects on the wastewater system will be explained.

The third case study is Blenheim as the earthquake prone city in the South Island, the seismic vulnerability of wastewater network will be analysed in Chapter 6. The Blenheim wastewater network will be classified by pipe specification. In the Blenheim case the findings will be calculated and compared to similar cases.

Outcome of the first three case studies will be compared in Chapter 7. Here the wastewater pipelines in each case will be compared to each other, and finally the earthquake damage in each case would be calculated by applying and comparing the 8 fragility curves to one another.

The 2010 earthquake in Christchurch made a practical evaluation of the thesis findings in terms of significant effects of an earthquake on wastewater network and also the evaluation of some significant factors affecting seismic vulnerability of wastewater network possible. Chapter 8 concentrates on the effects of this earthquake on the Christchurch wastewater system. This chapter will show the necessity of seismic vulnerability assessment of wastewater systems in earthquake prone regions. Expected defects in the Christchurch wastewater network will be calculated similar to other three case studies in Chapter 8.

However, the focus of this thesis is on the buried pipelines and Chapter 8 incorporates discussions on pipeline damage with less detailed assessment on pumping stations and treatment plants.

Calculated damage by the fragility curves will be compared to the observed damage after the earthquake. The observed damage rates after the earthquake will be plotted versus the earthquake parameters applied in the fragility curves, and then the observed trend will be compared to the trend of developed fragility curves.

Chapter 8 will also discuss the rehabilitation of wastewater network components as the main key factor to be made not only by utility managers but by the City Council of each city. This part in Chapter 8 will describe rehabilitation processes for the damaged wastewater network in Christchurch. Different rehabilitation methods for wastewater pipelines will be discussed in detail, as well as their advantages, disadvantages and also restrictions on the post-earthquake restoration and rehabilitation method.

How the thesis objectives have been achieved during this study will be discussed in Chapter 9, as well as the thesis findings summary and future research possibilities.

# Chapter 2. Seismic damage and methods of damage estimation in buried pipeline networks

## Introduction

New Zealand population is concentrated in urban areas where adequacy of wastewater systems is a priority with the municipal authorities (Statistics New Zealand, 2009b). The population increase and standard of life in urban areas has increased the need for good public facilities. One of the public facilities with a direct impact on communities is the wastewater system, the malfunction of which affects a large number of people as experienced by earthquakes. For instance, failure of the wastewater network in Christchurch after the Darfield earthquake in 2010 affected a large number of people. Wastewater systems in densely populated areas have been designated as a lifeline which deserves particular attention due to its direct impact on the community health.

Earthquakes can significantly affect wastewater systems. Reviewing earthquake effects on different parts of the wastewater system shows how and to what extent wastewater systems have been affected by past earthquakes. The finding here would help us improve the resilience of wastewater systems to damages. The first part of this chapter is divided into two main sections. In the first section, the various components of wastewater systems and their junctures will be reviewed. While in Section 2 the effects of past earthquake on the wastewater network will be reviewed. More specifically the impact of past earthquake in different regions will be observed. The effects of past earthquakes mainly on wastewater networks will first be tabulated to show how vulnerable the buried pipelines are against earthquakes.

Many researchers like (Katayama et al. (1975), Kuraoka and Rainer (1996) and Miyamoto (2008)) have concentrated on water pipelines rather than wastewater pipelines. This chapter will highlight the differences in both water and wastewater pipelines damage caused by seismic shocks in the past earthquakes. Wastewater pipelines are usually unpressurised and are different from other underground pipeline networks in terms of pipe specification, design and installation lay out procedures.

## 2.1. Wastewater system components

Wastewater systems as lifelines are the main part of urban underground facilities. Freshwater is necessary for human life and transferring and treating wastewater is also essential. Wastewater systems are comprised of three components, wastewater pipeline network, pumping stations and treatment plants. Wastewater systems usually receive industrial and residential wastewater. In New Zealand, the wastewater systems process both types of wastewater except in some industrial regions like Blenheim industrial zone were the industrial waste is pre-treated before being discharged to an urban wastewater system. In rainy seasons the rain water is mixed with wastewater, like that of the Hutt City wastewater network prior to being discharged into the sea with the minimum treatment.

### 2.1.1. Wastewater system

The wastewater system includes three main functions: collecting, transferring and treating. The wastewater network is not only the main part of the collecting system but is also the essential components of the transferring system. Collecting pipelines are usually 100 mm and 150 mm diameter pipes, whereas the diameter of transferring pipes joining to the treatment plant can reach 1.5 meter. High-rise and industrial buildings are connected to the wastewater network with large diameter pipes, depending on the capacity of the generated wastewater. Small diameter pipes connect residential buildings to collector pipes. The diameter of collector pipes varies according to the number of connections and the flow rate of the collected wastewater. Collectors usually collect wastewater from main streets. Diameters of collector pipes vary from 250mm to 1500 mm according to the maximum expected wastewater volume carried by each collector. Collectors are connected to the larger diameter pipes and finally the main wastewater pipelines (the trunks) transfer all collected wastewater to a treatment plant. In the four case studies examined in this thesis, the collecting pipes inside the city ranged from 75 to 200 mm in diameter, and the maximum diameter for the main wastewater pipes was found to be 1600mm.

Wastewater networks are usually designed to carry collected wastewater by the force of gravity, although because of some practical and technical issues, wastewater cannot be transferred by the force of gravity alone. Gravity is the most reliable means used in fluid transfer, particularly wastewater. Topography, as the main factor, can hinder application of the force of gravity especially in flat areas. For instance, if the whole wastewater network is designed to carry wastewater by the force of gravity, the buried depth of wastewater pipelines will significantly increase, particularly in large cities (Bizier, 2007). Technical and financial issues are the main barriers in applying the gravity force in wastewater transferring system (Bizier, 2007).

### 2.1.2. Types of wastewater collection systems

Wastewater collection systems are classified into 4 categories: gravity based, pressurised, vacuum and small diameter gravity systems. A gravity based system is the most convenient and common type of collection system. The wastewater collection systems in Auckland, Wellington, Blenheim, Christchurch, and Gisborne are all gravity based.

### 2.1.3. Description of wastewater systems.

Conventional gravity based systems are based on unpressured wastewater pipelines that collect and transfer residential and commercial wastewater. The collected wastewater, in contrast with vacuum and small diameter sewer systems, is transferred to wastewater treatment plants by gravity. After treatment processes this water is released back into the environment.

Pressurised wastewater pipelines are installed in the pressurised sewer system like those are installed in Far North District and Queenstown Lake District (Western Bay of Plenty District Council, 2011). The system can be installed in areas where the gravity point is higher than the residential areas like hilly mountainous terrains, coastal flats with very high water tables, and city suburbs. The collected wastewater in each sewage basin unit is pumped into the wastewater network through individual pumping stations.

The vacuum system works in such a way that the collected wastewater is vacuumed to the system and is transferred in to the main pumping station like those installed in Kawakawa Bay. This system is suitable for applying in areas with low population density (Harrison Grierson Company, 2010).

The small diameter gravity sewer is based on combination of gravity pipelines and septic tanks which are used in collecting and separating the solids before releasing the liquid back into the network. The small diameter sewer system can be used in a flat region with small diameter pipes, in contrast to the conventional gravity based sewer system (Falvey, 1996).

### 2.1.4. Wastewater pipelines

Wastewater pipelines in each wastewater system can be categorised in different ways. For instance, wastewater pipelines can be classified in terms of flow characteristics or wastewater pipes specifications like material, diameter, joint type, buried depth and durability (Environmental Services, 2006). For instance, wastewater pipelines can be classified into unpressurised or pressurised wastewater pipelines in term of flow type. Gravity is the driving force in unpressurised wastewater pipelines. In pressurised wastewater pipelines the pumping force is the main driving force.

## 2.2. Earthquake effects on the wastewater system

Earthquakes affect wastewater systems both directly and indirectly (ALA, 2004a). Direct damage can be explained as an immediate and onsite effect caused by wave propagation or ground displacement of an earthquake on different parts of wastewater systems. For instance, pipe breakages and leakages are considered as direct effects (Chen et al., 2002; Lund, et al., 1998; Wang et al., 1991).

### 2.2.1. Direct damage of earthquake on wastewater networks

Eidinger (1998) described breakages and leakages in buried pipelines as direct damage of seismic shocks. Breakages in pipelines are defined as longitudinal cracks, split ruptures and complete circumferential failure, whereas leakages refer to partial circumferential failures or corrosion related failures. The percentage of leakages and breakages in damaged pipes is relevant to the pipe diameter (Eidinger, 1998). In relation to past earthquakes, the percentage of breakages in pipes with diameters less than 12” are expected to be 16.67% of the total damage, while in large diameter pipes it is 50% (Eidinger, 1998).

Direct seismic damage of wastewater networks damages pipes and then the direct damage is followed by indirect seismic effects like pavement collapse, road blockage, disrupted traffic flow, water pollution, soil pollution and potential public health problems. It is worth mentioning that earthquake impacts on other services and lifelines have an adverse impact on wastewater systems, and hinder the post-earthquake rehabilitation process. For instance, in 1999 the Taiwan earthquake, some access roads to the damaged wastewater treatment plants were impaired and deformed, and this hindered the restoration process (Schiff, et al., 2000).

### 2.2.2. Indirect damage of earthquake on wastewater networks

Indirect damage of earthquakes on wastewater systems can be caused by Tsunami (Edwards, 2005). For instance, in the 2004 Thailand Tsunami, the treatment plants and pump stations in the affected regions flooded and caused failures in the whole wastewater system (Edwards, 2005). Another example of indirect impact of earthquakes on the wastewater system is the change of oceanographic characteristics close to wastewater treatment plant outlets (Morkoc et al., 2007). The ocean pollution in the Izmit Bay Turkey was an instance of the earthquake indirect impact after the 1999 Turkey earthquake (Morkoc, et al., 2007). The 1931 earthquake in Napier caused floor uplift in the inner harbour and this decreased the tidal current force. Tidal currents caused accumulation of the sewage solids near the harbour entrance, which contaminated shellfish (Napier City Council, 2008). Consuming the shellfish caused an outbreak of typhoid among the population. This harbour floor uplift disrupted the sewers network in the Napier business centre and the flat areas (Napier City Council, 2008).

Table 2.1 shows a number of earthquakes around the world which severely affected buried wastewater pipelines. The pipe specifications are indicated in column 3 of this table.

Table 2.1: Earthquake damage on buried pipelines

| Year | Location | Damage on buried wastewater pipes | References |
|---|---|---|---|
| 1971 | San Fernando (US) | Wastewater pipelines suffered intensive damage.<br>22% of the total length of the main sewer line had to be replaced. | (King & Betz, 1972) |
| 1972 | Managua (Japan) | The total length of water network was 581.1.<br>AC, Galvanised and CI were the main types of pipes, with some minor PVC and Iron pipe types.<br>The failure rate in AC and Galvanised pipes was almost the same (1.18), and the failure rate in CI=0.97 defects/km. | (Katayama et al. 1975) |
| 1983 | Nihonkai-Chubu (Japan) | The wastewater pipelines and facilities were severely damaged.<br>There were 120 severely damaged sites in the Akita sewer system (350 km).<br>The cost of repair was1012.7 million yens.<br>Precast reinforced concrete pipes were the predominant sewer pipes.<br>Pulling out and offset at joints were kinds of failure in wastewater network.<br>Joint failure scored1039 points, while point crack, crash and stick-out scored 1117 points. | (Kawashima et al. 1985) |
| 1983 | Coalinga (US) | Most of the damage in the pipelines was caused by wave propagation.<br>The earthquake accelerated the leakage rate.<br>Breakages and leakages were reported immediately after the earthquake were less than those reported after the earthquake during system performance.<br>Damage was reported on the 4”, 6” and 8” CI and AC pipes. | (Cooper, 1984) |
| 1987 | Edgecombe (NZ) | The wastewater pipelines were severely damaged by the earthquake.<br>The 150mm and 200mm asbestos pipes had the most significant failure.<br>Almost every earthenware sewer pipe was damaged severely and replaced. | (Pender, 1987) |
| 1989 | Loma Prieta (US) | The water and wastewater pipelines were significantly affected.<br>The pipes with 4”, 6” and 8” diameter needed the most repairs.<br>9000 feet of damaged pipes needed replacement.<br>Few main sewer breakages were reported in the main sewer pipe line. | (Lund, et al., 1998) |

Table 2.1: Earthquake damage in buried pipelines (continued)

| Date | Location | Damage to water and wastewater buried pipelines | Reference |
|---|---|---|---|
| 1993 | Ormond (New Zealand) | The wastewater network was heavily damaged.<br>The immediate damage was breakages at the sewer line passing under the bridge. | (Read & Sritharan, 1993) |
| 1994 | Northridge (US) | 1100 repairs were done in the water pipelines.<br>93% of pipe damage occurred in the pipes with diameters of less than 24”. | (Jeon and O’Rourke 2005) |
| 1994 | Hokkaido-Toho-Oki (Japan) | PVC pipes were the predominant type used.<br>The pipes were of diameter between 200 and 300 mm<br>The total length of the damaged wastewater line was 56% of the total sewer network. | (Sasaki et al., 1999) |
| 1995 | Kobe (Japan) | The water pipelines suffered severe damage.<br>Approximately 80 % of the water distribution network was DI.<br>Although the joints were earthquake resistant, the failure rate was significant.<br>The earthquake resistance joints suffered much less damage.<br>An immediate and early survey showed the 18.2% of the total damage was on the water pipelines.<br>There was no correlation between the damage rate and the pipe diameter.<br>Displaced out joints were the predominant types of failures.<br>The estimated cost for sewer repairs was 12 billion yens. | (Shinozuka, 1995) |
| 1995 | Hyogo-ken Nanbu (Japan) | There was severe damage in the water pipelines.<br>There were 3300 breakages in the water pipelines. | (Kuraoka & Rainer, 1996) |
| 1999 | Ji-Ji (Taiwan) | There was severe damage in the water pipelines.<br>Maximum damage rate was about 20 breakages per kilometre.<br>Minor damage in the wastewater pipelines was reported. | (Chen, et al., 2002) |
| 2003 | Bam (Iran) | 70-80% of the water pipelines were damaged.<br>The water pipe network age was from 2 months up to 40 years. | (EERI, 2004) |
| 2004 | Niigata Ken Chuetsu (Japan) | There were 900 breakages in the water pipelines.<br>187 pipe breakages and just 22 water invasions occurred.<br>The immediate damage caused by the earthquake was 12% of the total damage in the sewers.<br>The UPVC and steel pipes suffered more damage than the DI and CI pipes.<br>Joint failure was the predominant pipe failure. | (Scawthorn, et al., 2006) |
| 2007 | Gisborne (New Zealand) | There were wastewater pipelines failures under the two bridges.<br>There was water invasion into the sewer failure 3 months after the earthquake. | Gisborne City Council (2009) |
| 2010 | Chile | Some wastewater systems suffered intensive damage in the 2010 Chile earthquake.<br>Wastewater damage extended into the large diameter interceptor pipes and the small diameter connection pipes.<br>The earthquake caused river water pollution and environmental problems due to direct discharge of wastewater in the rivers | (Tang et al., 2010) |

Table 2.1 shows that most researchers such as Katayama et al (1975), Cooper (1984), Jeon and O’Rourke (2005), Shinozuka (1995), Karaoka and Rainer (1998), Chen et al. (2002) and EERI (2004) only focused on pressurised networks and did not consider or mention damage in unpressurised network. On the other hand, other researchers such as King and Betz (1972), Kawashima et al. (1985), Pender (1987), Read and Srithran (1993) broadly explained seismic damage in wastewater pipeline without any particular detail. The table shows that in spite of the significant cost of repair in the damaged wastewater networks most researchers mainly

focused on seismic effects on pressurised networks and only limited researchers broadly worked on wastewater networks damage after an earthquake (Kawashima et al., 1985 & Shinozuka, 1995).

Katayama et al (1975) explained asbestos AC) and cast iron (CI) pipes were severely damaged while PVC pipes had superior performance. Cooper (1984) also mentioned CI and AC pipes were severely damaged but did not explain which pipe can suffer high or less damage. Kawashima et al. (1985) explained precast concrete suffered the maximum damage rate. These studies show AC, CI and precast concrete pipes cannot appropriately withstand seismic shocks and ductile pipes with proper joints can resist seismic shocks. On the other hand, Shinozuka (1995) explained that damage rates in the ductile iron pipes was significant; however, he mentioned pipes with seismic-resistant joints suffered less damage. Their studies showed how complicate seismic damage on buried pipelines is.

According to Table 2.1, asbestos and cast iron pipes are two types of pipes severely damaged in past earthquakes (Cooper, 1984). Damage in buried pipelines was reported mostly on joints and in term of diameters most pipe defects were reported in small diameter pipes like 4”, 6” and 8” pipes (Lund, et al., 1998). Furthermore, in some locations wastewater pipelines within the wastewater network are more susceptible to seismic force damage than the others. For instance, wastewater pipelines passing through bridges were the most critical lines even in areas where earthquakes did not cause significant damage to buried pipelines (Read & Sritharan, 1993).

Application of new types of seismically resistant pipes in water and wastewater network is expected to notably reduce earthquake induced damage, but due to complicated behaviour of soil and pipe interaction, this type of pipes is not guaranteed for much less damages. For instance, the Kobe water network was notably affected after the earthquake even ductile pipes with earthquake resistant joints (Shinozuka, 1995).

Some researchers like Schiff (1995) and Lund et al. (1998) have announced that the number of repairs in wastewater pipelines should be the same as water pipe repairs in the same area, but there is no study to confirm their work (Lund, et al., 1998; Schiff, 1995). Their idea encouraged other researchers such as FEMA (2003) and ALA (2004) to recommend application of the same fragility for damage estimation in unpressurised networks such as wastewater networks. The recommended fragility curves are established from seismic damage on pressurised networks. On the other hand, Scawthorn et al. (2006) showed the damage rate on wastewater network was higher than that of the water pipelines. They also mentioned the visible effect of the earthquake on the wastewater pipelines represented about 2.5% of the total real damage. As a result, restoration of wastewater pipelines after an earthquake require more time for inspection compared to water pipelines (Scawthorn, et al., 2006).

Comparing the work of Scawthorn et al. (2006), Schiff (1995) and Lund et al. (1998) shows inconsistency in terms of seismic shock effects on pressurised and unpressurised networks. The comparison shows seismic shocks can inversely affect unpressurised networks compared to pressurised networks. Hence, the application of the fragility curves developed in pressurised networks may cause inaccurate results when applied in unpressurised networks.

Kawashima et al. (1985) showed how the damage caused by an earthquake can cause substantial repair cost in restoring the wastewater pipelines. Over 3,000 kilometres of wastewater pipelines in the area affected by the 1994 Northridge earthquake were at risk. The actual repair work of the sewers in the affected areas initiated in 1995 was expected to end in 2008in spite of the FEMA deadline in January 1998 (Rick et al., 2001).

The 2003 Bam earthquake in Iran damaged 70 to 80 % of the water pipeline along 49 km of the water distribution network. It should be noted that the installed pipes aged from 2 months up to 40 years (EERI 2004). This fact shows even recently installed pipes could not tolerate earthquake induced forces, that means the installed pipes were not designed to resist earthquake forces or design codes were not accurately observed.

## 2.3. Summary of seismic damage in buried pipelines

This chapter shows that previous research has focus on pressurised networks such as water pipelines in comparison with unpressurised wastewater networks. There is only limited research which is directly focused on unpressurised wastewater networks.

Seismic damage on wastewater networks mostly affects the connection points of different diameter pipes. Some critical points in a wastewater network can be damaged even if the earthquake is not destructive, for instance moderate earthquakes can solely damage wastewater pipelines which pass under bridges. The material used in pipe construction such as asbestos cement and cast iron are not resistant to earthquake in comparison with other pipe materials. Buried pipes with diameters of 4”, 6” and 8” are expected to experience more damage compared to other pipe diameters. Joint failure such as joint pull-out or displacement can be significant factors in pipe network failure.

The literature review in this chapter shows that there is no accurate information regarding earthquake effects on unpressurised wastewater networks compared to that of pressurised. As a result, research is required to show the extent of earthquake damage in unpressurised wastewater networks. This research would help authorities better decide how vulnerable their wastewater networks are.

There are few references in New Zealand which have even superficially reported the effect of seismic shocks on buried and unpressurised wastewater networks. Consequently, a study is required to focus on seismic vulnerability of buried pipelines in New Zealand, wastewater networks in particular. This study should consider the method which is used internationally for seismic damage assessment in buried pipeline networks.

The literature review shows previous research focused on the observed damages in buried pipelines including wastewater networks, but there is no study to show the vulnerability of unpressurised wastewater networks or to show the extent of predicted damage on the networks by future earthquakes. On the other hand, researchers have mainly focused on a few particular types of pipes which are installed in the pressurised networks while other types of pipe are ignored.

## 2.4. Calculating earthquake effects on buried pipelines

The events described in the previous sections show that wastewater pipelines can be significantly affected by earthquakes. Accordingly, seismic vulnerability analysis of buried pipelines will be discussed in the following sections. A high number of defects in buried pipelines after the earthquakes have forced the seismologists and researchers to find methods for estimating earthquake damage on buried pipelines (Katayama et al., 1975). In the following sections, fragility curves as a common estimation tool used for seismic vulnerability analysis in buried pipelines will be explained along with the critical parameters that affect seismic vulnerability of buried pipelines. The last two sections in this chapter will illustrate calculations used to assess seismic damage on buried pipelines, wastewater network rehabilitation and repair methods, respectively.

### 2.4.1. Ground Transient Deformation and Permanent Ground Deformation effects of earthquakes on buried pipelines

Earthquakes effect on buried pipelines are in two categories: the Ground Transient Deformation (GTD) and the Permanent Ground Deformation (PGD) (O'Rourke et al., 2001).Wave propagation of earthquake waves that affect vulnerable areas usually cause GTD, typically shown by the peak ground velocity (PGV), whereas PGD refers to the nonreturnable ground movement which may be caused by fault movement, landslides, uneven settlement and liquefaction (Toprak & Taskin, 2007).

Buried pipelines can be affected by GTD and PGD simultaneously although sometimes GTD is the predominant cause of damage on buried pipelines. The difference between PGD and GTD in terms of damage in buried pipelines is related to the region affected by an earthquake (O'Rourke, et al., 2001). Areas affected by PGD can be relatively small compared to areas affected by GTD, although damage in pipes caused by PGD can be greater than that of the GTD (O'Rourke, et al., 2001).

Ground shaking or GTD occurs in all earthquakes and presents the greatest hazards. Ground shaking may result from many different earthquake sources(ALA, 2005). In this thesis the focus is on ground shaking hazard. Here, the applied fragility curves show the effect of this hazard on wastewater pipelines.

### 2.4.2. Short and long term effects of earthquakes on buried pipelines

Earthquake effects on buried pipelines can be divided into long and short term effects (Scawthorn, et al., 2006). Short term effects are described as immediate effects on pipelines which may cause failure of a particular pipeline or even the whole transmission system. Long term effects refer to the damage which does not instantly cause system failures, but in the long term can affect the systems efficiency (Ellis, 2001). For instance, in terms of short term effects, a pipe collapse in the main wastewater pipelines can immediately affect the function of the whole network system while, in long term effects, leakages are considered as long effects on the wastewater system. Wastewater leakages can contaminate the groundwater and the surrounding soil. Infiltration of the groundwater and debris to sewage pipelines is possible as well. Water infiltration to wastewater pipelines increases the wastewater volume discharged in a treatment plant, and can significantly increase treatment costs. Accumulation of infiltrated debris in wastewater pipes not only decreases a pipe's capacity but can also cause a flow restriction (Feeney et al., 2009).

### 2.4.3. Direct and indirect effects of earthquakes on buried pipelines

Earthquake stricken communities suffer direct and indirect pipeline damage. Direct damages are pipe breakages and pipe leakages (Scawthorn, et al., 2006). Direct damage can interrupt the main function of a system or can even affect other urban facilities. Indirect damage can be defined as the effect of pipeline failure on other facilities or the environment (ECLAC, 1991). Indirect damage in sewers caused by an earthquake can sometimes be much worse than direct damage in pipelines. For instance, the Avon River, Heathcote River, Avon-Heathcote Estuary and the delta area of the sea were contaminated by untreated human sewage after the 2010 and 2011 earthquakes in Christchurch regions to the point that the Canterbury District Health Board issued public health hazard announcement (Environmental Canterbury Regional Council, 2011).

### 2.4.4. Comparison of pressurised and unpressurised networks

There are differences between damage on pressurised and unpressurised networks as shown by some researchers such as Scawthorn et al. (2006), but there is no specific study to demonstrate the differences between two networks when affected by seismic shocks. In this section the differences between two networks are clarified in order to show the parameters which can affect seismic damage in unpressurised networks compared to pressurised ones.

Indirect damage on pressurised pipelines can immediately affect communities due to rapid spreading of pressurised fluid into the environment. For instance, transportation networks can be adversely affected by water pipe eruption, or gas pipeline breakages and leakages can cause fire within a city after an earthquake. Indirect damage in unpressurised pipelines, like wastewater pipes can have long term effects on the environment. Indirect damage in unpressurised pipelines can affect the environment instantly. The wastewater pipeline breakage in the 2007 Gisborne earthquake polluted the river and its banks (Zare, 2008).

When the unpressurised pipelines are shocked by seismic forces, the wastewater system does not fail immediately except for the main wastewater pipes that collapse. For instance, in the Northridge earthquake, 1994, in spite of 1000 water pipeline failures in Los Angeles which needed immediate repairs, 10 failures were assigned immediate administration (ALA, 2004b).

The number of reported defects on wastewater pipelines is usually much lower than the number of defects in pressurised water pipelines after an earthquake. However, the number of expected defects in wastewater pipelines is likely to be the same as the number of defects in water pipelines (Lund, et al., 1998). Failures in wastewater pipelines can usually be detected by a specific survey. Complete network inspection can last for some months or even years, such as CCTV inspection, although a heavily damaged sewer can be visually detected (Duran et al., 2002).

The corrosion rate in wastewater pipelines, especially in improperly designed sewers, is usually higher than those in water pipelines. Moreover, the joint quality in segmented pressurised pipelines is better than that of the unpressurised pipelines which use only sealant to join segments (Rahman, 2004). For instance, steel pipes with butt welded joints tolerate seismic forces very well, although application of steel pipes in sewer networks is restricted due to steel corrosion in wastewater networks (Eguchi, et al. 1983).

In terms of the pipe material, pressurised pipes are usually built of ductile material which can tolerate higher internal forces, in comparison with unpressurised pipelines which are usually made of inflexible material. For instance, tensile strength of Polyvinyl Chloride (PVC) pipes used in unpressurised sewers is 6000 psi compared to 7000 psi in pressurised PVC pipes (Rahman, 2004). Consequently, pressured pipelines are expected to withstand external forces such as earthquake better than unpressurised pipes.

In terms of the pipe diameter, small diameter pipes are more vulnerable to earthquakes compared to large diameter pipes (O'Rourke & Jeon, 1999). The pressurised pipelines have small diameter pipes in comparison to wastewater pipelines. In Hutt City the minimum and maximum water pipe diameters are 20 mm and 525 mm respectively, whereas the minimum and maximum wastewater pipe diameters are 100mm and 1350 mm (Capacity infrastructure services, 2007; Capcity infrastructure services, 2007).

In terms of buried depth, wastewater pipes, due to their design criteria, are usually buried in more depth, in comparison with water pipelines, and sewers are usually installed below other buried pipes. The buried depth has a direct impact on seismic vulnerability of the pipes, and by increasing the depth, the seismic vulnerability of buried pipes decreases (Eidinger &

Avila, 1999); although, buried depth of pipelines in the fault crossing regions has inverse correlation with pipe damage. Decreasing buried depth means the lower overburden and less friction which give the pipe more freedom to move without severe damage (Eidinger, 1998).

In order to calculate earthquake effects on buried pipelines, both empirical and analytical methods can be applied (Eidinger & Avila, 1999). Practical models correlate past earthquake parameters and pipe specifications to the number of expected damage in length unit i.e. number of defects per km. In analytical approaches, the stress, strain and joint movement in pipes are calculated by the applied earthquake load, then the allowable level of stress, strain and joint movement is compared to the calculated stress, strain and movement (Dash & Jain, 2007). Analytical approaches are applied to the design or rehabilitate large diameter pipes. Analytical approaches require more effort and cost than the empirical methods; consequently, analytical approaches are usually not applied to small diameter pipes (Eidinger & Avila, 1999).

Empirical formulae are usually applied to calculate the expected damage in buried pipelines which may be affected by earthquakes (O'Rourke & Deyoe, 2004). Calculating damage in pipelines by empirical formulae for the specific earthquake can be applied for the purpose of emergency response plans, risk management or hazard mitigation plans (Toprak et al., 2008).

The correlation between earthquake parameters and pipeline damage are the main concern of researchers such as Eguchi et al. (1983), Eidinger and Avila (1999) and O’Rourke & Deyoe (2004) in evaluating earthquake effects on buried pipelines (Eguchi et al., 1983; Eidinger & Avila, 1999; O'Rourke & Deyoe, 2004). Various types of earthquake parameters as well as different types of pipe specifications are taken into account in calculating the buried pipelines vulnerability to earthquakes in the available fragility curves.

### 2.4.5. Earthquake parameters used in seismic vulnerability analysis

The number of damages in unit length of buried pipelines after an earthquake can be shown by correlation between earthquake parameters and pipelines specifications. Fragility curves are mathematical equations used to show the correlation between damage rate, pipe specifications and earthquake parameters. Fragility curves are developed by observed damage in pressurised buried pipelines after earthquakes. Theses curves are used to estimate expected damage in buried pipelines after particular earthquakes according to pipelines specifications and seismic characteristics of region where pipeline network is installed. For instance, by calculating PGA (Peak Ground acceleration), as a relevant parameter, of a specific earthquake prone region and classifying pipe types and length, the rate of expected defects in a length of pipeline can be calculated by fragility curve. Earthquake relevant parameters and pipe relevant parameters can vary in each fragility curve (Eidinger & Avila, 1999; Toprak, 1998). Some earthquake relevant parameters that will be used later are explained in this section.

In order to calculate earthquake effects on buried pipelines, various types of earthquake parameters are applied to correlate seismic forces with expected damage in pressurised pipelines (Katayama, et al., 1975). For instance, PGV, PGD, PGA, MMI (Modified Mercalli Intensity) and SA (Spectral Acceleration) are common types of earthquake parameters, and are usually applied in fragility curves to calculate earthquake induced pipe damage. PGV and PGD are two common parameters which are used in recent formulae to calculate earthquake effects on pipelines (ALA, 2004a; FEMA, 2003).

PGA is defined as the maximum amplitude of recorded acceleration (USGS, 2010). PGV is the maximum respective amplitude of velocity and usually is measured by kine (cm/sec). PGD is the maximum respective displacement after an earthquake. PGD is shown by cm or inch (USGS, 2010). SA is the maximum acceleration in an earthquake on an object. MMI is an earthquake intensity measured on qualitative scale (Chen & Scawthorn, 2003).

The difference between PGA and SA refers to earthquake effects on ground particles or on buildings, respectively. Acceleration of particles on the ground is defined by PGA, whereas SA is defined as an approximation of what buildings experience (USGS 2009). SA is used as an indicator of the earthquake magnitude or as an estimator for PGV (Chen, et al., 2002; FEMA, 2003; Jeon & O'Rourke, 2005). Katayama et al. (1975) correlated PGA to the damage rate in pipelines according to earthquake effects on buried pipelines in different cities in Japan (Katayama, et al., 1975). Wang 1991 recommended formulae for different soil types by applying PGA and MMI, based on historical data from seven past earthquakes (Wang, et al., 1991).

PGA and MMI provide the basis for the preliminary damage formulae (Eguchi, et al., 1983; Katayama, et al., 1975). In some formulae, the differences between earthquake damage caused by wave propagation and permanent ground deformation have not been considered although after Eguchi et al. (1983) both of these have been considered by other researchers (Eguchi, et al. 1983). PGA and MMI as the two well-known parameters are used in some fragility curves in order to illustrate a number of defects in buried pipelines after an earthquake. For instance, PGA multiplied by some correction factors, is applied in order to show the pipe damage (Isoyama et al., 2000). When analysing the Ji-Ji Taiwan earthquake damage on buried pipelines, Chen et al. 2002 emphasised that PGA, in comparison with SA and PGV, is the most appropriate earthquake indicator in calculating the number of defects after an earthquake (Chen, et al., 2002). MMI is the most comprehensible earthquake indicator which is still being used to calculate the number of defects in buried pipelines (Zhao et al., 2008).

Earthquake magnitude has the most impact on buried pipelines. Various types of parameters are applied in the available fragility curves as earthquake magnitude indicators. The most common earthquake parameters used in fragility curves are PGA (Isoyama et al., 2000; Katayama et al., 1975; Wang et al., 1991), MMI (Eguchi, et al. 1983; Wang et al. 1991 and Zhao et al., 2008) and Spectrum Intensity (SI) (Chen, et al., 2002). PGV and PGD are the most popular parameters used for calculating wave propagation and ground deformation effects of earthquakes on buried pipelines (Eguchi, et al., 1983; Toprak & Taskin, 2007). It should be mentioned that other factors are considered in showing earthquake magnitude such as Arias Intensity (AI), VMPGV and scaled PGV (Jeon & O'Rourke, 2005; Toprak, 1998). King and Betz (1972) showed closeness to fault zones can affect damage rate on buried pipelines (King & Betz, 1972).

This section shows each researcher applied different earthquake parameters for calculating seismic damage in buried pipelines and there is not consistency between applied parameters in fragility curves. For instance, Katayama et al. (1975), Toprak (1998) and Chen (2002) applied PGA, Eguchi (1983), Wang (1991) and Zhao (2008) used MMI. This fact shows there is no constant earthquake parameter to use for seismic vulnerability analysis. However, most recent researchers recommend PGV for wave propagation effect and PGD for permanent ground deformation.

In terms of earthquake parameters some researchers show a particular type of earthquake relevant parameter can more adequately show the relationship between damage rate and seismic intensity in comparison with other researchers' works in different areas. For instance, Chen et al. (2002) showed that PGA more accurately shows seismic damage in buried pipelines compared to other parameters.

These fragility curves are established to estimate the pipe damage caused by an earthquake (Chen, et al., 2002; Eguchi, et al., 1983; Katayama, et al. 1975). Table 2.2 shows the significant differences between the available fragility curves in terms of different earthquake parameters used in each fragility curve. In Table 2.2 NA stands for not applicable and is used to show which parameter did not apply in recommended fragility curve.

Table 2.2: Earthquake parameters used in different fragility curves

| **Author (s)** | **PGA** | **PGV** | **PGD** | **SA** | **MMI** |
|---|---|---|---|---|---|
| **Katayama et al. (1975)** | * | NA | NA | NA | NA |
| **Eguchi et al. (1983,1991)** | NA | NA | * | NA | * |
| **Barenberg (1988)** | NA | *( Ground Strain) | ** | NA | NA |
| **Wang et al. (1991)** | * | NA | NA | NA | * |
| **O’Rourke & Ayala (1993)** | NA | * | NA | NA | NA |
| **Eidinger (1998)** | NA | * | * | NA | NA |
| **Toprak (1998)** | * | * | * | * | * |
| **Eidinger & Avila (1999)** | NA | * | * | NA | NA |
| **O’Rourke and Jeon (1999)** | NA | * | NA | NA | NA |
| **ALA(2004)** | NA | * | * | NA | NA |
| **Chen et al. (2002)** | * | * | NA | * | NA |
| **FEMA (2003)** | NA | * | * | * | NA |
| **Zhao et al. (2008)** | NA | NA | NA | NA | * |
| **Maruyama and Yamazaki (2010)** | NA | * | NA | NA | NA |

As table 2.2 shows researchers use different earthquake parameters to demonstrate seismic damage on buried pipelines. PGA and MMI are the initial earthquake parameters used to correlate seismic damage on buried pipelines. PGA is the most commonly used index for engineering purposes. After Eguchi et al. (1983), researchers did not use MMI as a qualitative earthquake intensity factor in fragility curves due to substantial uncertainty involve in fragility curve, however, Zhao et al. (2008) applied MMI to calculate expected damage in buried pipelines. Toprak (2007) added that considerable uncertainty is involved in developing correlation between the damage rare and MMI (Toprak & Taskin, 2007).

Barenberg (1988) recommended that the velocity and velocity induced ground strain are proper earthquake parameters which can be used to develop fragility curves. He is the first person who introduced and recommended velocity induced strain as an earthquake relevant parameter in fragility curves. However, researchers have not considered this as a critical parameter after his study. Barenberg (1988) was the first who recommended ground velocity as a proper estimation parameter to use in fragility curves for wave propagation damage in buried pipelines. Following Barenberg (1988) other researchers such as Toprak (1998) and O’Rourke (2001) confirmed that PGV can be an appropriate parameter for calculating wave propagation effects on buried pipelines (O'Rourke, et al., 2001; Toprak, 1998).

For instance, Katayama et al. (1975) as the pioneer in this field used PGA; however, PGV and PGD are mostly applied in fragility curves to calculate seismic damage recently. Application of different earthquake parameters by researchers shows that there are no constant factors to be applied in predicting the seismic damages on buried pipelines. Consequently, in order to show the differences among available fragility curves in term of parameters used, first the fragility curves are tabulated to show which parameters are used in each fragility curve and then a brief explanation for each fragility curve will be presented.

The table below shows which parameters are used in calculating the rate of damage to pipelines in the event of an earthquake. Several parameters such as pipe type, diameter, joint type and the age of pipe can affect the damage rate in buried pipelines after an earthquake (Lau et al., 1995). Some parameters in Table 2.3 are shown by NA (not applicable) that stands for the parameters not considered in developed fragility curves. Some researchers like Lau et al. (1995) show some of the affecting parameters in seismic vulnerability of buried pipelines but they do not use all parameters in their developed fragility curves (Lau, et al., 1995). In Table 2.3 the unused parameters are shown by “mentioned but NA”. Other abbreviations used in Table 2.3 show type of pipes such as CI, UPVC, AC and PCCP which stand for Cast Iron, Un-plasticised Polyvinyl Chloride pipes, Asbestos Cement and Pre-stressed Concrete Cylinder Pipe, respectively.

This table shows that there is no consistency between the pipe-relevant parameters which can affect seismic vulnerability of buried pipelines. Some researcher only used few pipe-relevant parameters in their recommended fragility curves in spite of the fact that some other parameters can affect seismic vulnerability in buried pipelines. This fact shows application of fragility curves in other regions would give different estimations because some parameters are ignored. Furthermore, formulating fragility curves requires more studies and more parameters should be used in order to achieve a better result.

Table 2.3: Pipe characteristics applied in available fragility curves (part 1)

| Author | Pipe Type | Diameter | Joint Type | Corrosion | Pipe age |
|---|---|---|---|---|---|
| Katayama et al. (1975) | CI | Mentioned but NA | NA | NA | NA |
| Eguchi et al. (1983,1991) | 7 pipe types | NA | Mentioned but NA | Mentioned but NA | NA |
| Wang et al. (1991) | CI | NA | * | NA | Simply entered |
| Barenberg (1988) | CI | Mentioned but NA | NA | NA | NA |
| O'Rourke &Ayala (1993) | CI, AC and CP | Mentioned but NA | NA | Mentioned but NA | NA |
| (Lau, et al., 1995)(water & gas) | Mentioned but NA | Mentioned but NA | Damage in joint are predominant NA | Mentioned but NA | Mentioned but NA |
| Eidinger (1998) | 6 types | *** | *** | *** | Mentioned but NA |
| Toprak (1998) | Mainly CI | Divided by two groups | NA | Na | NA |
| O'Rourke & Jeon (1999) | CI pipes | Mentioned and consider | NA | NA | NA |
| Eidinger & Avila (1999) | 7 types of pipe | Mentioned and simply considered | Considered in formula | Mentioned but NA | Mentioned but NA |
| Isoyama et al. (2000) | 5 pipe types | Mentioned and considered | Mentioned and considered for steel pipes | NA | NA |
| Chen et al. (2002) | Mentioned but NA | Mentioned and indirect use | NA | NA | NA |
| Scawthorn el al (2006) | UPVC (pull out) & steel (screw joint) | NA | NA | NA | NA |
| Zhao et al. (2008) | 3 types | NA | NA | NA | NA |

Katayama et al. (1975) were the first to correlate earthquake parameters with the pipe damage in water and gas pipelines in different cities of Japan. They showed how buried pipelines, especially pressurised pipes, were affected in Tokyo and other Japan cities, and also correlated the damage rate with peak ground acceleration. They recommended PGA to calculate wave propagation and permanent ground deformation effects of seismic shocks contradicting the other researchers and they also explained AC and CI pipes can experience more damage in pressurised networks. Eguchi et al. (1983) were the first to divide buried pipe damage into two types, damage caused by ground shaking and damage caused by permanent ground deformation. Data collected from 25 earthquakes was used in their work to find the correlation between earthquake parameters and the damage rate in buried pipelines (Eguchi, et al. 1983). Their work showed that the damage rates in pipelines increased by rising ground displacement and decreased by increasing distance to fault lines. Eguchi et al. (1983) showed that AC pipes are the most vulnerable types, and that arc welded steel are the least vulnerable pipes in the earthquake affected regions. The type of failure in steel pipes is joint failure due to welding of inadequate joint types whereas in CI and AC pipes, the pipe body failure is the common type. Cement caulked joints in CI pipes tend to shatter but the type of failure in CI pipes with rubber-gasket joints are separation in joints (pulling apart) (Eguchi, et al. 1983). They showed seismic damage in unpressurised pipe compared to the previous researchers more accurately; however, they mostly considered type of pipes which are used in pressurised water and gas networks. They assumed that the same damage rate for CI, AC and WSCJ (welded steel caulked joint) pipes for each MMI (Eguchi, et al., 1983).

Eguchi renewed and extended his graphs in 1991, and they were republished in 1993 by O’Rourke and Ayala (Eguchi, 1991; O'Rourke & Ayala, 1993). Application of their graphs provides insight into what happens after earthquakes of different intensity, and allowed estimation for earthquake vulnerability of different pipe types. The advantage of their graphs is calculation of earthquake damage on different pipe types. According the graphs, clay and earthenware pipes are more vulnerable to earthquakes than concrete and asbestos cement pipes. Comparison of their study to other researchers’ work such as Katayama et al. (1975) shows that the same type of pipe experienced various damage rates in different regions. This fact shows fragility curves which are only based on pipe material cannot accurately be applied for calculating seismic damage in different areas.

Application of MMI in fragility curves for estimating earthquake damage on buried pipelines especially after Barenberg (1988) has decreased because it can cause uncertainty in fragility curves (Toprak & Taskin, 2007). Most of the fragility curves established after Barenberg (1988) apply PGV and PGD in order to show ground shaking and permanent ground deformation effects of earthquakes, such as fragility curves established by Toprak (1998) and Eidinger and Avila (1999).

Barenberg (1988) was the first who recommended the application of the ground strain to correlate the pipe damage to earthquakes. His work showed that the damage on buried pipelines is better characterised by transient ground motion parameter like that of the peak ground strain or peak horizontal particle velocity in regions with low intensity shaking (Barenberg, 1988). Seismic damage on buried pipelines should be characterised by ground displacement parameter in regions with high intensity shaking (Barenberg, 1988).

Barenberg’s work (1988) indicates that two types of equations would be used to calculate seismic damage in low intensity and high intensity shaking. On the other hand, according to FEMA (2003) total damage is a summation of damage caused by wave propagation and that caused by permanent ground deformation (FEMA, 2003). This fact shows that not only there is an inconsistency in types of fragility curves but there is also no consistency in calculating total damage after an earthquake which can be caused by wave propagation and ground deformation.

Upon observing the number of defects in buried pipelines from 8 earthquakes, Wang et al. (1991) suggested 3 basic equations for three different soil types with different shear wave velocity. Wang et al. (1991) classified cast iron, concrete, brick and stone duct, vitrified clay and asbestos pipes as brittle pipes, and ductile iron, steel, reinforced concrete, pre stressed concrete and polyvinyl chloride pipes as ductile pipes. Wang et al. (1991) used logarithmic equations for each type of soil to show damage points per kilometre of a segmented 12” cast iron pipes to PGA and MMI (Wang, et al., 1991). Application of their recommended fragility curves is problematic compared to other fragility curves, because of their soil type classification and application of different fragility curves for each type of soil.

O’Rourke and Ayala (1993) extended Barenberg’s work (1988) in low density earthquake shaking regions for more pipe types from 11 earthquakes (O'Rourke & Ayala, 1993). They applied more earthquake data compared to other researchers but, they only concentrated on the wave propagation effect. Their work shows some differences with that of the Barenberg’s work (1988). O’Rourke and Ayala (1993) explained how corrosion and subsoil condition in earthquakes contributed to differences between their work and Barenberg’s work (1988). They showed that damage rates in asbestos cement, concrete and pre-stressed concrete pipes follow the same general trend as in cast iron pipes (Barenberg, 1988; O'Rourke & Ayala, 1993).

Eidinger (1998) modified the available fragility equations by considering PGV and PGD as well as 3 pipe characteristics on the 1989 Loma Prieta earthquake (Eidinger, 1998). Eidinger (1998) thus assumed that MMI=VII is equivalent of PGV=20 inch/s and MMI=IX to PGV=25 inch/s in order to compare previous fragility equations with his own. Eidinger’s work suggested that if a fragility curve gave a damage prediction within 20-40% difference from the actual damage rate, the applied fragility equation is acceptable. Eidinger (1998) also classified cast iron and asbestos cement pipes as being brittle (Eidinger, 1998). He tried to transfer qualitative data into quantitative data which is not approved nor used by other researchers. He is the first researcher to state that appropriate fragility curves can cause up to 40% difference in the estimated damage in pressurised networks compared with the observed damage rate. He recommended the use of fragility curves in spite of his concern about their inaccuracy.

Toprak (1998) analysed the correlation between the different earthquake parameters and damage rates on the water network after the 1994 Northridge earthquake. He identified damage rates versus MMI, PGA, PGV, PGD, SA and two others parameters. Toprak (1998) found that PGV has the highest correlation with damage rate. The Northridge earthquake in 1994 provided more accurate data in terms of seismic relevant and pipe relevant information in comparison with previous earthquakes.

The damage data after the Kobe earthquake allowed Isoyama et al. (2000) to establish a new fragility curve for pressurised water pipelines (Isoyama, et al., 2000). Their fragility curves are based on more accurate data compared with the previous research in Japan. They proposed the standard fragility curve which consider pipe material, diameter, ground condition and liquefaction for seismic vulnerability analysis (Isoyama, et al., 2000). They recommended a different method compared to other fragility curves. As a result, application and comparison of their method with other methods is limited particularly outside Japan.

O’Rourke and Jeon (1999) similar to Barenberg (1988) and Toprak (1998) confirmed that PGV is the most relevant parameter for calculating the damage rate caused by the wave propagation effect of earthquakes (O'Rourke & Jeon, 1999). Their work showed PGV is an appropriate parameter to use in fragility curves in spite of some researchers such as Chen et al. (2002) that recommended other parameters such as PGA. They applied the pipe type and the pipe diameter in order to develop a fragility curve in contrast with other researchers that only considered pipe type. Their work showed that in areas with the PGV less than 10 cm/sec, no pipe damage was reported and PGV greater than 70 cm/sec caused a permanent ground deformation type of damage (O’Rourke & Jeon, 1999).

The 1999 Taiwan earthquake effects on the gas and water pipelines in Taichung city was analysed by Chen et al. in 2002. The damage was correlated to three earthquake parameters (PGV, PGA and SA). They claimed that PGA based formulae have the best correlation with damage rates in earthquakes compared to PGV and SA in contrast with the previous researchers’ works (Chen, et al., 2002).

Federal Emergency Management Agency (2003) suggested two formulae for calculating seismic effects on buried pipelines (FEMA, 2003). Wave propagation and permanent ground deformation damage on buried pipelines are estimated by PGV and PGD base formulae separately, in contrast with that presented by Barenberg (1988). Type of pipe is only applied in the recommended fragility curves and other affecting parameters introduced by the previous researchers are ignored (FEMA, 2003). This fact shows inconsistency between researchers works regarding seismic effects on buried pipelines. FEMA (2003) asserts that the number of defects in ductile pipes is only 30% of the total defects calculated by the base formula for brittle pipes.

In FEMA equations (2003) Steel (ST), Ductile Iron (DI), plastic and PVC pipes are classified as ductile material, while CI, AC, Reinforced Concrete Cylinder (RCC), Clay and Pre-stressed Concrete Cylinder Pipe (PCCP) are classified as brittle (FEMA, 2003). FEMA (2003) recommended the same fragility curves for calculating seismic damage in unpressurised wastewater pipelines without considering the intrinsic differences between pressurised and unpressurised network. The FEMA equations are based on the previously established fragility curves. For instance, the wave propagation equation in FEMA (2003) is based on the fragility curve established by O’Rourke and Ayala (1993). This fact shows that there has not been any notable progress in fragility curves between 1993 and 2003, in spite of some established fragility curves after 1993.

American Lifelines Alliance (2004) similar to FEMA (2003) provided two formulae to estimate wave propagation and ground deformation effects of earthquake on buried pipelines. These equations are recommended to apply for both pressurised (water) and non-pressurised (wastewater) networks in spite of the significant differences exist between two networks, the same as FEMA’s fragility curves (2003). The equations recommended by ALA (2004) are based on more pipe specifications such as joint types and more type of pipes compared to the FEMA equations (2003) (ALA, 2004b).

Zhao et al. (2008) applied fragility curves in order to calculate earthquake effects caused by Wellington Fault on the Hutt City water distribution. The applied fragility curve used was a function of MMI, pipe types and soil conditions. Three types of pipe classifications were applied including brittle, average and ductile pipes in contrast with ductile and brittle pipes in FEMA’s equation. Zhao et al. (2008) classified asbestos and reinforced concrete as brittle pipes, and cast iron, high density and low density polyethylene pipes and steel pipes as ductile pipes. The average type of pipe covers polyethylene, polyvinyl chloride, galvanised iron and concrete line steel pipes. The pipe classification of Zhao et al. (2008) is different compared to FEMA's pipe classification (2003). Zhao el al. 2008 recommended three different types of pipes compared to two types of pipe in FEMA (2003). Furthermore, cast iron pipes in FEMA (2003) were classified as brittle pipes, but Zhao et al. (2008) classified cast iron pipe as ductile pipes.

Maruyama (2010) extended the work of Isoyama et al. (2000) and explained that, to cause pipe damage in CI pipes, minimum PGV should be 20 cm/sec in contrast with 15 cm/sec in the work of Isoyama et al. (2000), while in ductile iron pipes, it should be 30 cm/sec due to the change in the dominant pipe type in water network (Maruyama & Yamazaki, 2010).

The above literature review shows there is an inconsistency between researchers and they applied altered earthquake parameters in the developed fragility curves. Katayama et al. (1975), Eguchi et al. (1983) and Barenberg (1988) recommended different earthquake parameters to correlate damage rate in buried pipelines to seismic forces (Barenberg 1988, Eguchi, et al., 1983; Katayama, et al. 1975). However, some earthquake relevant parameters like PGA, PGV and PGD are used more often.

Most researchers such as Toprak (1998), Isoyama et al. (2000) and Chen et al. (2002), only concentrate on earthquake effects on pressurised pipelines. They developed the relevant fragility curves for pressurised networks due to instant and visible defects after an earthquake in these networks compared to less visible seismic damage in unpressurised networks (Chen, et al., 2002; Isoyama, et al., 2000; Toprak, 1998).

The fragility curves which are developed by correlation of seismic damage to earthquake parameters in pressurised pipes are applied in estimating seismic damage in unpressurised pipes like wastewater pipes (FEMA, 2003 & ALA, 2004). The pipe material used in both water and wastewater network is the same, but the reported damage is often less in unpressurised networks. Furthermore, leakage in water pipes can be detected easily and affect the water network; however, in case the damages in wastewater pipes do not obstruct the flow, wastewater will continue to flow and the damage will not appear until it is detected by intensive inspection (Schiff, 1995). As a result, the number of instant defects in unpressurised pipelines can be underestimated if fragility curves developed for pressurised pipes are used in estimation of the damage caused by seismic shocks in wastewater pipelines. Furthermore, there is an inconsistency between different researchers including Lund et al. (1998), Schiff (1995) and Scawthorn et al. (2006) regarding similarity between volume of damage in water and wastewater networks.

There are many fragility curves for calculating the seismic damage rate on pipelines, but there are differences between calculation methods, the applied earthquake parameters and the estimated damage (Jeon & O’Rourke, 2005).

## 2.5. Critical parameters affecting the pipeline damage rate

Reviewing the effects of the past earthquakes on buried pipelines shows earthquakes can affect different types of buried pipelines in various ways. Buried pipelines can be divided into two main groups; continuous and segmented pipelines. Continuous pipelines are usually the main water, gas or oil pipelines which are typically constructed by steel pipes. Steel pipes segments in continuous pipelines are connected by butt welded joints which make steel pipes function as continuous pipelines. Application of continuous steel pipes in wastewater network is restricted because of the high corrosion rate in steel pipes and high construction costs, in comparison with other types of pipes. Continuous pipelines are generally not used in wastewater network except High Density Poly Ethylene (HDPE) pipe (Environmental Services, 2006). Segmented pipes are widely used in wastewater networks. Segmented pipes refer to various types of pipe materials and joint types including AC, PVC, RC, CI and EW. Each segmented pipe is connected to the next pipe with detachable joints. For instance, CI pipes can be connected to each other by cement joints, lead caulk joints or bell-spigot joints. Segmented pipelines are usually used to transfer low pressure fluids such as water and wastewater. Recently, new methods of pipelines construction have changed the type of pipe which may be used for pipe renewal or installation (Najafi 2005).

Failures in continuous pipelines can be both tensile and compression failures. Ground displacement caused by seismic loading can induce a high tensile strain in the pipe body and finally the tensile strain can cause a pipe rupture, especially in old steel pipelines (Dash and Jain 2007). For instance, the tensile strain was the predominant failure in the Alaska gas pipelines during the 1964 Alaska earthquake (Dash and Jain 2007). Axial compression can cause a buckling compression failure which leads to either local buckling or beam buckling (Schiff 1998). Local buckling is a common failure mode in steel pipelines (Dash and Jain 2007).

Critical parameters affecting seismic vulnerability of buried pipelines are categorised into three groups; earthquake, soil and pipe relevant parameters. In terms of earthquake relevant parameters, the three main parameters of earthquake magnitude, closeness to epicentre and depth of earthquake, can affect seismic vulnerability of buried pipelines. Pipe direction to the fault line also has an impact on the damage rate. Pipelines which are laid vertically or approximately vertical to the fault line are the most vulnerable as the damage rates are higher in a vertical direction than in other directions (Takada et al., 2002). Ground shaking duration in an earthquake prone region can affect the damage rate in pipelines. The long duration of shaking results in more damage in buried pipelines (Eidinger, 1998).

The pipe material is the main factor which influences pipe vulnerability to earthquake shaking or ground movement. Different types of material affect pipe flexibility. Typically, pipes used in the early stage of each network in urban areas are brittle pipes. These pipes have low flexibility and high earthquake vulnerability such as earthenware and asbestos cement pipes. For instance, some urban regions in NZ have wastewater pipes which are approximately a century old.

During the development of wastewater networks, new pipes were used for development or replacement of old pipes. For example, cast iron pipes in the Tokyo and Kobe water network were replaced with ductile iron pipes with more flexibility as well as with earthquake resistant joints. New pipe types are more flexible than the previously installed pipe types. Consequently, the new installed pipes are more earthquake resistant (Butcher (1998), Avila (1999) and Isoyama et al. (2000)).

The pipe age has an impact on the number of failures during an earthquake. Usually older pipes suffer corrosion more than newly installed pipes. The pipe damage in the Los Angeles area due to the 1987 Whittier Narrow earthquake revealed the increasing trend of the number of pipe breakages with respect to pipe age (Allouche & Bowman, 2006; Eidinger, 1998; Lund, et al., 1998; Wang, et al., 1991). Sometimes aged sewer pipes remain in better condition and as a result are expected to behave better than the pipes installed later. This is because the pipe and construction quality of more recently installed pipes were of lower quality than the old installed pipes (Capacity infrastructure services, 2007).

Corrosion decreases the pipe wall thickness leading to stress concentration in the corroded areas. For instance, steel pipes with threaded joints are more vulnerable to corrosion than those with other types of joints because of high corrosion rates in threaded joints. Corrosion also affects CI pipes and makes them vulnerable to earthquake shaking (Isenberg & Taylor, 1984; Lund, et al., 1998; Schiff, et al., 2000). Eidinger’s (1998) study shows that the damage rate in steel pipes is higher than CI pipes in highly corrosive soil types. Corrosion inside of pipes can also affect the reinforcement bar used in reinforced concrete pipes, and can decrease the strength of concrete pipes. It should be emphasised that corrosion inside of sewers can especially affect concrete pipes installed in the wastewater network. Microorganisms present in municipal wastewater pipelines can convert hydrogen sulphide to sulphuric acid. As a result, sulphuric acid can corrode concrete and steel bars inside the concrete sewers (Wells et al., 2010).

Pipe joints generally have a direct impact on pipe vulnerabilities in earthquakes, as recorded by O’Rourke and Ayala (1993), Lund et al. (1998) and Scawthorn et al. (2006). The pipe joint affects earthquake vulnerability of pipes, especially in segment buried pipelines. The joint pull-out due to axial extension, and bell crushing caused by joint extraction are the two main joint failures (El Hmadi & O'Rourke, 1990). Usually steel pipelines with the butt welded joint can tolerate earthquake shaking better than other types of pipes.

Segmented pipes, typically those with relatively flexible or weak connections such as bell-and-spigot, can fail in several ways (Eguchi, et al., 1983). Extra bending deformation of the pipe barrel, joint excessive rotation and the joint pull-out are forms of damage which may occur in the segmented pipe (Eidinger, 1998). Segmented pipes with rigid caulking (lead or Portland cement) are more vulnerable to ground shaking than those with flexible rubber gasket joints, which can tolerate more ground movement (Allouche and Bowman 2006). For instance, the PVC and AC pipes in the Loma Prieta earthquake showed the best performance due to their flexible rubber gasket joints whereas CI and steel pipes showed the highest damage rate (Allouche & Bowman, 2006). Their finding shows inconsistency between researchers’ beliefs regarding seismic vulnerability of different types of pipes. Some researchers mentioned that AC and CI pipes are both the most vulnerable pipes and even AC pipes resist seismic shocks less than CI pipes (Katayama et al, 1975, Cooper, 1984 and Pender, 1987). In the 1983 Nihonkai-Chubu earthquake (Japan) off set and pulling out at joints were the most typical failures in the Noshiro city wastewater network (Kawashima et al., 1985).

The pipe diameter is another factor which affects the damage rate in buried pipes during an earthquake. The repair rate in small diameter pipes (normally less than 12 inches) is higher than in large diameter pipes (Katayama, et al., 1975). Small diameter pipes usually have more bends, branches and service laterals, which create virtual anchor points and resist pipe movement during an earthquake. Usually small diameter pipes have less thickness than large diameter pipes therefore, corrosion can easily decrease the pipe strength (Kuraoka & Rainer, 1996). For instance, Pender (1987) reported the higher damage occurred in 150 and 200 mm asbestos pipes in the 1987 Edgecombe earthquake in New Zealand (Pender, 1987). The only available formula in which the pipe diameter has been entered directly in fragility equations belongs to O'Rourke and Jeon (1999). Pipe body failure is the common failure in wastewater pipes with the diameter less than 600 mm, whereas the joint failure is significant in these pipes with the diameter greater than 600 mm (Kawashima, et al., 1985). In contrast, King and Betz (1972) believe increasing pipe diameter can increase pipe vulnerability to earthquakes especially in wastewater pipeline.

Pipe diameter can affect other practical and design issues which can be used for pipe rehabilitation and repair. These operations are directly affected by the pipe diameter. Installation and repair time as well as installation costs of large diameter pipes are usually greater than of small diameter pipes (Najafi, 2005). The pipe diameter is also the main design factor in calculating flow rates in gravity pipelines such as wastewater pipes. Small diameter pipes which are usually used as connector pipes, transfer wastewater from individual users to the wastewater network (Environmental Services, 2006).

The buried depth of pipelines usually varies according to the pipe diameter, and usually larger pipe diameters require greater burial depth. The buried depth can influence damage rate, whereas an increase in the burial depth decreases the amount of expected damage (Eidinger & Avila, 1999). On the other hand, decreasing the buried depth means the lower overburden and less friction, which gives the pipe more freedom to move without severe damage (Eidinger, 1998). These studies show that there is no uniformity between researchers regarding effect of burial depth on damage rate in buried pipelines.

The damage rate in the section of pipelines which have less connections, fittings and irregularity is much lower than the damage rate in pipelines with higher irregularity or connections (Dash & Jain, 2007). The failure rate in continuous large diameter pipelines is lower than the failure rate in distribution networks (Dash & Jain, 2007; Eidinger, 1998). For instance, the pipe damage tends to be concentrated in the pipe elbows, tees, in-line valves and service connections. These features create rigid point locations that promote stress in connections (Eidinger, 1998).

The quality of installed pipes and pipeline construction can also affect pipeline vulnerability. For example, in the 2003 Bam earthquake in Iran, 70 to 80% of 490 km of the water distribution system was destroyed and damaged in the earthquake. The age of the Bam water pipeline distributions varied between two months and 40 years (EERI, 2004). The quality of connection joints in steel pipes usually affects the damage rate in steel pipes in earthquakes. For instance evidence shows that, the damage rate in poor quality arc welded joints in steel pipes is higher than in good quality arc welded joints (Eguchi, et al., 1983; Eidinger, 1998).

Sasaki et al. (1999) note the liquefaction of the backfill soil affecting the damage intensity in earthquake affected areas or liquefiable backfill soil increases the damage rate. Backfill liquefaction causes more damage in buried pipelines in areas with impermeable peat or clay as well as of a liquefiable layers in the original soil affects (Sasaki, et al., 1999). Increasing the thickness of the liquefiable soil below the buried pipe can increase the damage rate in buried pipelines. Liquefaction played a key role in earthquake pipe damage in the 2010 Darfield earthquake (Orense, 2010; Yetton et al., 2011). The distance between the water table and the pipe can also affect the damage rate. For instance, Sasaki's (1999) work shows that in damaged pipes, the water table distance to the pipe centre is greater than one meter.

Heubeh (2002) classifies vulnerability of different types of pipes, which can be seen in Table 2.4. According to this table, asbestos pipes, cast iron pipes, concrete pipes have the highest vulnerabilities to earthquakes and poly ethylene, ductile iron and steel pipes have the lowest vulnerability to earthquakes.

This section reveals that various types of fragility curves have been developed in various regions. Fragility curves are developed by correlating the observed seismic damage in some pressurised networks to few earthquake reverent parameters. These curves are developed in some particular regions and for a few types of pipes mainly installed in pressurised networks.

Table 2.4 : Vulnerability of various types of joints to an earthquake (Heubach, 2002)

| Material | Joint Type | Vulnerability | | | | |
|---|---|---|---|---|---|---|
| | | Low | Low to Moderate | Moderate | Moderate to High | High |
| PE | Fused | **** | | | | |
| DI, Steel | Bell & Spigot with rubber gasket – restrained | **** | | | | |
| DI, | Bell & Spigot with rubber gasket – unrestrained | | ***** | | | |
| Steel | Arc Welded | **** | | | | |
| Steel | Riveted | **** | | | | |
| Steel | Bell & Spigot with rubber gasket – unrestrained | | | ***** | | |
| Steel | Gas welded | | | | | **** |
| PVC | Bell and Spigot – restrained | | ***** | | | |
| PVC | Bell and Spigot – unrestrained | | | ***** | | |
| Concrete cylinder | Bell and Spigot – restrained | | ***** | | | |
| Concrete cylinder | Bell and Spigot – unrestrained | | | | | **** |
| Cast Iron | Bell and Spigot with rubber gasket | | | | ***** | |
| Cast Iron | Bell and Spigot – unrestrained | | | | | **** |
| Vitrified clay | Bell and Spigot with rubber gasket | | | | ***** | |
| Vitrified clay | Rigid joint | | | | | **** |
| Asbestos cement | Coupled | | | | | **** |

### 2.5.1. Classification of buried pipelines used in fragility curves

Categorizing the buried pipes into brittle and ductile can provide a general estimation on wastewater pipes in each case study. According to the available data in terms of pipe types used in wastewater network in each case study the following are applied to classified pipes.

CI, AC, Reinforced Concrete Cylinder (RCC), Clay and Pre-stressed Concrete Cylinder Pipe (PCCP) are classified as brittle pipes by FEMA (2003). In FEMA equations (2003) ST, DI (DI), plastic and PVC pipes are classified as ductile material.

In order to classify pipe types into brittle and ductile, first all types of pipes with the same base material should be classified as follows:

_Poly ethylene pipe type are PE80, PE100, PE 80B, MDPE, HDPE and PE pipes.

_Asbestos and FIB pipes are both considered as asbestos pipes.

_UPVC, MPVC and PVC are classified as polyvinyl chloride pipes.

_CLS, ST-CC, ST-SW and ST-SS are considered as steel pipes.

_Pipes with concrete base material are classified as concrete pipes.

PE, PE80 and MDPE are symbolised general poly ethylene pipes. PE100, HDPE and PE80B represent high density poly ethylene pipes. UPVC, MPVC and PVC are abbreviations used for unplasticised polyvinyl chloride, modified polyvinyl chloride and polyvinyl chloride. Both CLS and ST-CC stand for concrete lines steel. While ST-SW and ST-SS stand for spiral welded and stainless steel pipes, respectively.

The concrete pipes are classified into CC-RF, CC-SRS, CC-SRX, CC-SRY, CC-SRZ, CC-SR and CC-SU, which stands for reinforced concrete, spun reinforced concrete class S, spun reinforced concrete class X, spun reinforced concrete class Y, spun reinforced concrete class Z, spun reinforced concrete and spun unreinforced concrete pipe respectively (Marlborough District Council, 2009).

According to the incomplete information on the wastewater network; in the case study areas, a percentage of the pipes in each city is expressed as UNKNOWN or NOT MENTIONED. These types of pipes are considered as the primary installed pipes which lack as built maps. As a result, the UNKNOWN or NOT MENTIONED pipes are considered to be the most common pipe types in each case study such as AC and CI pipes.

Types of pipe which each fragility curve is developed from do not cover all types of pipe installed in the wastewater network. Because of this fact, the fragility curves established for CI pipes are used to calculate damage in brittle pipelines, and the fragility curves developed for DI pipes are used to calculate earthquake effects on ductile pipes (Toprak & Taskin, 2007). On the other hand, some fragility curves which are used in this study are only developed for one type of pipe which is mainly the CI type. Therefore, in this case, according to FEMA (2003), 30% of the calculated damage rate for brittle pipes is considered as the damage rate for ductile pipes.

Jeon and O’Rourke (2005) offer three fragility curves for asbestos, cast iron and ductile iron pipes. In order to apply 3 fragility curves for all these three types of pipes, the length of EW and UNKNOWN pipes are added to the length of AC pipes. Because of flexibility of poly ethylene, polyvinyl chloride and steel pipes, the length of these three types is added to DI pipes. Furthermore, the length of concrete and vitrified clay pipes is added to CI pipes.

This section reviews the damage inflicted on different pipe types and the parameters which can affect their vulnerability. This section showed that the vulnerability of buried pipelines can be affected by a variety of parameters such as pipe material, joint type, pipe diameter, pipe material corrosivity, buried depth, construction quality, closeness to fault line and backfill soil type.

## 2.6. Type of defects in wastewater pipelines

Wastewater pipelines have particular specifications which can affect their seismic vulnerabilities. Most wastewater pipelines are usually categorised as segment pipelines (Bizier, 2007). The tensile and compression stress caused by earthquake can damage segmented pipelines in a similar manner to that of continuous pipelines. Typically, in segmented pipes the pipe body is more resistant to tension than the joint. Consequently, the tensile force in segmented pipes, caused by an earthquake, causes the pipe to pull out from its joints (Scawthorn, et al., 2006). Crushed joints in compression failure, especially in Bell and Spigot joints are a common type of failure in segmented pipes. When bending occurs, if joints cannot tolerate rotation, a joint failure is inevitable in the segmented pipes (Kim et al., 2010).

Kawashima et al. (1985) reported that damage on the body of sewer pipes was much less than that of the joints in the 1983 Nihonkai-Chubu earthquake (Kawashima, et al., 1985). They emphasised that offsetting and pulling out at joints were the predominant failures. Chiba et al. (1996) noticed that cracks, breakages, displacement, opening and breakage at joints, as well as pipe axes displacement, were the most common types of failures in wastewater pipelines. He also recommended the use of flexible pipes and joints, especially between pipelines and manholes in order to reduce seismic effects on wastewater networks.

Areas affected by PGD are usually small, compared to areas affected by wave propagation, although the potential for pipeline damage is higher in PGD affected areas than in others(El Hmadi & O'Rourke, 1990). In the 1906 San Francisco earthquake, the seismic damage in buried pipes was divided evenly according to PGD and wave propagation. They showed that the wave propagation damage was the predominant type of damage causing factor in earthquakes such as the 1964 Puget Sound, 1969 Santa rose, 1983 Coalinga and 1985 Michoacan (El Hmadi & O'Rourke, 1990).

Connection and joints are usually the most vulnerable parts of particularly segmented buried pipelines. For instance, in the Mexico City water network, the damage rate caused by the 1985 Michoacan earthquake was 0.45 leaks/km, and two thirds of the leakages occurred at the junctions or near them at the tees, elbows and valve boxes (El Hmadi & O'Rourke, 1990). The joint pull out and bell crushing were reported as the predominant failure modes in connections and joints (El Hmadi & O'Rourke, 1990). Failures in joints of segmented pipes included excessive bending deformation, excessive rotation and a pull out or a crush (Eidinger, 1998). The lead caulked CI pipe leakages were caused by axial extension (O'Rourke & Ayala, 1993). In the 1985 Michoacan and the 1989 Mexico earthquakes, the damage types in the pre-stressed concrete pipelines were axial crushing of Bell and Spigot (El Hmadi & O'Rourke, 1990).

So far this chapter has shown that wastewater pipelines are vulnerable to earthquakes and there are many parameters which affect the seismic vulnerability of buried pipelines. In order to calculate damages on wastewater networks in New Zealand the following recommendations are made.

### 2.6.1. Calculation of Peak Ground Velocity in New Zealand

Buried pipelines damage in earthquake affected areas can be caused by wave propagation and permanent ground deformation. The significant damage in buried pipes caused by seismic forces is relevant to wave propagation effects of any earthquake; consequently, most researchers have so far concentrated on wave propagation effects of an earthquake. Wave propagation effects can affect the entire area, but ground deformation can only cause damage to particular regions with a high probability of landslide or liquefaction. The calculation of wave propagation effects of an earthquake on buried pipelines makes the bases for seismic damage calculation in earthquake prone zones. Regardless of the distinction between wave propagation and ground deformation effects of earthquake, some researchers still apply a single fragility curve for both types of seismic effects due to significant uncertainty in fragility curves and various affecting parameters.

PGV is the basic earthquake parameter applied in calculating ground shaking effects of earthquakes on buried pipelines. A good estimation of PGV is necessary to achieve a good result. There are two methods for estimating PGV in earthquake prone areas; applying the attenuation model and the empirical method (Zhao, 2009). The preliminary version of PGV attenuation model in New Zealand is derived from domestic and nearby regional data. The preliminary available attenuation equation in New Zealand is based on the domestic earthquakes and their moment magnitude, depth of fault and distance to the rupture plane Zhao, 2009a).

There is no accurate PGV attenuation model in New Zealand therefore, the empirical equation, such as FEMA (2003), is recommended for calculating the PGV in New Zealand (Zhao, 2009). FEMA (2003) recommends the equation to predict PGV in regions where there is no accurate model or appropriate earthquake data.

Spectral acceleration (SA) should be on hand for each region, and thus PGV for each region can be calculated. The first step is to obtain an estimation on spectral acceleration. There are two ways to calculate spectral acceleration for any particular region in New Zealand. The first way is applying the SA maps developed for New Zealand and the second method is calculating SA recommended by New Zealand Standard. Stirling et al. (2002) present spectral attenuation maps for soil type B in New Zealand in relation to active faults characteristics and earthquake return periods (Stirling et al., 2002).

Correction factors should be applied to correlate PGV in soil type B with other soil types. According to FEMA (2003), coefficients greater than 1 are applied to modify PGV from soil type B to soil types C, D and E and the coefficient of 0.8 is used to convert the PGV from soil type B to soil type A. Further, $F_A$ and $F_V$ are the two correction factors which can be applied to convert PGA and PGV from soil type B to other soil types, respectively. Table 2.5 shows that correction factors ($F_A$ and $F_V$) can be used to modify the calculated PGV by different soil types (FEMA, 2003).

The first step to calculate pipe damage is estimation of PGV in regions where ground shaking can inflict damage on buried pipelines. The FEMA (2003) equation is recommended to calculate PGV in New Zealand from spectral acceleration at 1 second spectral period (Zhao, 2009). In order to calculate spectral acceleration at the spectral period of 1.0 s, the New Zealand standard 1170.5 is applied to calculate spectral acceleration (NZS 1170.5, 2004).

$$SA(T)= C_h(T)*Z*R*N(T,D) \qquad \text{(Equation (2.1))}$$

Table 2.5: PGA and PGV correction factors for different soil types

| **Site Class B** | **Site Class** | | | | |
|---|---|---|---|---|---|
| **Spectral Acceleration** | **A** | **B** | **C** | **D** | **E** |
| **Short-Period, SAS (g)** | **Short-Period Amplification Factor, FA** | | | | |
| ≤ 0.25 | 0.8 | 1.0 | 1.2 | 1.6 | 2.5 |
| 0.50 | 0.8 | 1.0 | 1.2 | 1.4 | 1.7 |
| 0.75 | 0.8 | 1.0 | 1.1 | 1.2 | 1.2 |
| 1.0 | 0.8 | 1.0 | 1.0 | 1.1 | 0.9 |
| ≥ 1.25 | 0.8 | 1.0 | 1.0 | 1.0 | 0.8* |
| **1-Second Period, SA1 (g)** | **1.0-Second Period Amplification Factor, FV** | | | | |
| ≤ 0.1 | 0.8 | 1.0 | 1.7 | 2.4 | 3.5 |
| 0.2 | 0.8 | 1.0 | 1.6 | 2.0 | 3.2 |
| 0.3 | 0.8 | 1.0 | 1.5 | 1.8 | 2.8 |
| 0.4 | 0.8 | 1.0 | 1.4 | 1.6 | 2.4 |
| ≥ 0.5 | 0.8 | 1.0 | 1.3 | 1.5 | 2.0* |

In equation 2.1 $C_h(T)$, Z, R and N(T,D) stand for the spectral shape factor, the hazard factor, the return period factor and the near fault factor, respectively. For each region soil type, the period in second $C_h(T)$ can be estimated from Table 1 in Appendix 2 (NZS 1170.5), therefore, the soil type should be determined before calculation of the spectral shape factor begins. Each soil type in the New Zealand Standard (NZS 1170.5) has a special definition and a characteristic which can be used to classify the soil type in any region. Geological specification of each region can be applied to classify the soil type in any region (Dellow, 2009b). For instance, the inflicted city, has a particular guideline recommended by GNS (2009), which is used to evaluate the soil type in each region (Dellow, 2009a). The correlation between geological characteristics of each region and soil types can be extracted from Appendix 4 for Hutt City, Gisborne and Blenheim (Dellow, 2009a, 2009b).

Hazard factor (Z) in Equation (2.1) refers to a particular number assigned to each city, and shows seismic vulnerability of each region. Concerning the required annual probability of exceedance, the return period factor (R) can be estimated from Table 2.6. This table shows that the return period factors for annual probability less than 1/500 are greater than 1, and for annual probability greater than 1/500 are less than 1.

Table 2.6: Return period exceedance

| Required annual probability of exceedance | R |
|---|---|
| **1/2500** | 1.8 |
| **1/2000** | 1.7 |
| **1/1000** | 1.3 |
| **1/500** | 1 |
| **1/250** | 0.075 |
| **1/100** | 0.5 |
| **1/50** | 0.35 |
| **1/25** | 0.25 |
| **1/20** | 0.2 |

There are 11 major faults in New Zealand named: Alpine, Awatere, Clarence, Hope, Kakapo, Kekerengu, Kelly, Mohaka, Wairarapa, Wairau and Wellington. The near fault factor should be calculated in places with less than 20 km distance from the faults lines. According to the distance of each region from faults, two formulae can be employed to calculate N(D,T). If the annual probability of exceedance is less than 1/250, then N(D,T) equals 1; otherwise, the following formulae shall be applied to determine the near fault factor.

If D is less than 2km $N(D,T) = N_{Max}(T)$ (Equation (2.2)

For D between 2 and 20 km $N(D,T) = 1+(N_{Max}(T)-1)*(20-D)/18$ (Equation (2.3)

If D is greater than 20 km $N(D,T) = 1$ (Equation (2.4)

Table 2.7: Maximum near fault factor

| T(s) | $N_{Max}(T)$ |
|---|---|
| **Less or equal 1.5** | 1 |
| **2** | 1.12 |
| **3** | 1.36 |
| **4** | 1.6 |
| **Greater or equal 5** | 1.72 |

Table 2.7 shows the maximum near fault factor. Calculation of PGV requires spectral acceleration in T=1; hence, N(D,T) equals 1 for each case study. The required spectral acceleration for each municipal region in each case study is calculated by multiplying all estimated factors together including $C_h(T)$, Z, R and N(T,D). As noted above, spectral

acceleration for T=1 for earthquakes with the return period of 475 and 1000 years can directly be extracted from the maps for soil type B (Stirling, et al., 2002).

## 2.7. Summary

This chapter shows that many researchers have worked on seismic vulnerability of pressurised buried pipelines and they have developed fragility curves as an estimating tool. Fragility curves are developed from the correlation of some earthquake parameters to the observed damage rates in the pressurised networks. They have recommended fragility curves to assess seismic damage in buried pipelines.

Some sources such as FEMA (2003) and ALA (2004) recommend that the same fragility curves which are applied to calculate seismic vulnerability in pressurised networks can be applied in unpressurised wastewater networks, in spite of inherent differences in pressurised and unpressurised networks in terms of flow, type of joints, material, diameter and installation method.

The above two facts show that the application of fragility curves which were developed from pressurised networks may well be an inadequate tool for estimating seismic damage in unpressurised networks like wastewater pipelines. As a result a study is required to show how the application of fragility curves in wastewater networks can affect damage estimation in such networks.

The available fragility curves are developed overseas and application of these curves in New Zealand probably shows different results. There is no study in this subject to show the application of theses curves for seismic vulnerability analysis of buried pipelines especially unpressurised wastewater networks in New Zealand.

The fundamental of equations which fragility curves are based on is developed from correlation between damage rate and some earthquake parameters. These equations show that an increase in the volume of the applied seismic parameter cause higher damage rate. As a result, each fragility curve can estimate some defects even in areas with low seismic vulnerability. The number of calculated defects can be notable in areas with immense network of unpressurised pipelines. As a result, the accuracy of correlation as well as trigger points for which damages can be observed and fragility curves applied should be studied and taken into account in unpressurised networks.

Most fragility curves are developed for one or two particular types of pipes. On the other hand, various types of pipe are installed in each pressurised or unpressurised pipe network. Consequently, seismic damage on various types of pipe in a network should be estimated by the same equation that can underestimate or overestimate the real expected damage in a particular type of pipe. For instance, all types of polyvinyl chloride pipe are considered as PVC pipes in seismic vulnerability analysis with fragility curves, but each type of polyvinyl chloride pipe can behave differently after an earthquake which is ignored in the literature review. Consequently, a study is required to demonstrate different seismic resistance of various types of pipes which are used under a broad type of pipe in fragility curves.

Many studies show the application of trenchless techniques for repair and rehabilitation of damaged pipes in unpressurised wastewater networks caused by none-earthquake related defects. Some researchers also work on explaining advantages and disadvantages of the available trenchless techniques. However, there is no study to demonstrate pros and cons of these techniques when used for post-earthquake repair and rehabilitation. Earthquake effects on buried pipelines differ from common types of repair and rehabilitation methods. As a result, a study is required to demonstrate the limitations of trenchless techniques when they are applied to post-earthquake repair and rehabilitation.

# Chapter 3. Methodology

## 3.1. Introduction

The main focus of this chapter is based on the methodology used in this thesis to achieve the research objectives. Firstly some methods of research classification will be briefly discussed and general procedures for conducting research will be explained. In this chapter research philosophy, case study method and application of this method in this study will be discussed.

The purposeful research can be classified as the following 6 classes: exploratory, descriptive, causal, correlative, interpretive and critical. Concepts and problems can be clarified by exploratory research, and a method can be applied in order to show the project feasibility or test the preliminary idea. Descriptive research intends to accurately describe the research topic. When researchers are concerned with explanation rather than description, a causal research method is applied. The purpose of correlative research is to correlate regularities for future prediction. Human intention and the meaning of events to those involved can be extracted by interpretive research, while critical research is applied to demystify social behaviours (Tan, 2004).

In terms of purposeful research this study can be classified as exploratory and causal. In showing the seismic vulnerability of unpressurised wastewater networks in earthquake prone cities in New Zealand is exploratory and is the first purpose of this research. The other two purposes of this research are to find factors which can affect seismic vulnerability of wastewater pipeline and to compare different fragility curves in the case studies, thus this research can be classified as causal.

Research is also classified into two broad categories, quantitative and qualitative (Dawson, 2002). However, Dooley (2002) believes that the qualitative and quantitative description refer to two types of data and cannot be used to classify research types (Dawson, 2002).

According to Creswell (2002), research can be classified into qualitative and quantitative with the same research process. A typical research work comprises of the following 6 stages, i.e. identifying the problems, reviewing the existing literature, selecting participants or samples, collecting data, analysing and interpreting the data and finally reporting and evaluating the research findings (Creswell, 2002).

Yin (2003) classifies research strategies into the following 5 categories; an experiment, a survey, an archival analysis, history and case studies. Table 3.1 presents a comparison between different types of research methods.

Table 3.1: Different research strategies (Yin 2003)

| Research Strategy | Research questions? | Requires control of behavioural events? | Focus on contemporary events? |
|---|---|---|---|
| Experiment | How, why? | Yes | Yes |
| Survey | Who, what, where, how many, how much? | No | Yes |
| Archival Analysis | Who, what, where, how many, how much? | No | Yes/no |
| History | How, why | No | No |
| Case Study | How, why | No | Yes |

## 3.2. The research philosophy

Research philosophy guides researchers to choose an appropriate method and shows which data should be collected, analysed and used (Davison, 1998). The qualitative/quantitative research classification lacks coherent definition and focuses on methods rather than exploring underlying research philosophy. Therefore, research methods described and classified by a philosophical approach is a basic research classification (Clarke, 1998).

Research philosophy can be divided into two broad methods: positivist and post-positivist (Crossan, 2003). In positivist research, prediction is based on the foundation of the previously observed and explained realities and their inter-relations. This method is based on the idea that reality is stable and can be observed and described from an objective viewpoint (Davison, 1998). In this method the researcher is objective and acts independently. The positivist researcher does not influence research results and data is achieved by observation or experimentation (Tobin, 2006). From the positivist point of view, reality is objectively given and is measurable by properties which are independent of the researcher. In this method, quantitative data is used to describe the parameters and their interrelationships (Tobin, 2006).

Nonetheless, the positivist approach provides useful but limited data that only provide superficial view of the phenomenon which is investigated (Crossan, 2003). Some researchers such as Jacob Bronowski (1956) and Karl Popper (1959) believe positivist philosophy provides elementary justification and a new method is required to overcome this deficiency (Crossan, 2003). Post-positivism provides an alternative to the traditions and foundations of positivism for conducting disciplined inquiry. For the post-positivist researcher reality is not a rigid construct, instead it is a creation of those individuals involved in the research. From this perspective, reality does not exist within a vacuum, its composition is influenced by its context, and many constructions of reality are therefore possible (Ramsay, 1998).

Post-positivism is used to prove the existence of a phenomenon in contrast to positivism which makes claims to absolute truth through the establishment of generalisation and laws. According to Doyal (1993) the apparent certainty and high probability in knowledge achieved by a positivist view is an illusion because it cannot be proved to apply in all situations. Similarly, some issues in research observation and experiment which were thought to be true previously have been discovered to be false (Doyal, 1993).

While quantitative research is used to investigate phenomena in the positivist approach, in post–positivism a qualitative perspective is used to describe and explore phenomena in depth, however, both qualitative and quantitative data can be applied in post-positivist methods (Ford-Gilboe et al., 1995). In spite of opposing and polarised methodologies in qualitative and quantitative research, these are frequently used together and complement each other. Furthermore, the distinction between the philosophies is overstated and triangulation of methods is common (Webb, 1989).

From a positivist perspective, theory verification is the main goal, whereas, theory falsification is the key purpose of post-positivist research. Nonetheless, both methods are applied for explanation which leads to prediction and control of phenomena (Ponterotto, 2005)

Research can also be categorised as empirical and non-empirical. Previous research and the pre-existing body of knowledge are essential for conducting research in a particular field of study. When research depends entirely upon on the previous research it is classified as non-empirical. Alternatively, when research is based on observation and experiment it is categorised as empirical research (Tobin, 2006).

Inductive and deductive reasoning are the two methods of reasoning used in research. In the deductive method researchers work from a general premise towards a more specific conclusion. Deductive reasoning moves from the general to the particular. It takes a general premise and deduces particular conclusions (Tobin, 2006). On the other hand, inductive reasoning moves form the particular to the general or moves from specific reasoning to a general conclusion. In this method, collected observation is used to form a series of premises and these premises are used to achieve a general conclusion.

### 3.2.1. Research philosophy of this thesis

New Zealand has experienced some devastating earthquakes but there is no accurate and comprehensive report regarding damages in buried pipe networks, particularly in unpressurised wastewater networks. The literature review shows that there is no study demonstrating seismic vulnerability of unpressurised wastewater networks in New Zealand. On the other hand, much overseas research has focused on seismic damage in pressurised pipeline networks, but there are only limited studies regarding seismic damage on unpressurised pipeline networks. As a result, an in depth study is required to cover this gap and to investigate seismic vulnerability in unpressurised networks.

Analysing unpressurised networks in all the cities of New Zealand not only is a time consuming process but it also requires intensive investment for collecting and generating the required data for the analysis. Therefore, in order to achieve the overarching aim of this study, a case study strategy is chosen to achieve the overarching aim of this study. Several wastewater networks within New Zealand cities are selected as case studies to generalise the results of this study.

An appropriate assessment method should be selected to show the vulnerability of the wastewater networks. Vulnerability in a pipe network can be defined by the number of defects which can affect the function of system. Reviewing the literature shows fragility curves can be an appropriate tool to calculate the number of defects after an earthquake. Therefore, fragility curves are used in this study to show seismic vulnerability of unpressurised wastewater networks. Consequently, quantitative data is required to calculate seismic damage by fragility curves.

According to the above research philosophies and the research aim, a combination of positivist and post-positivist methods are applied in this study, although due to the presence of falsification the method used in this thesis is closer to the post-positivist view.

From the positivist point of view quantitative data is collected from indirect observations. During the data collection procedure, the author was not an active participant so did not influence the data collecting. On the other hand, mathematical methods such as statistical models are not used in this study to generalise results as they would be in positivist research methods. Similar to post-positivist view which requires in-depth study, some case studies are selected in this study in order to conduct an in depth research

In terms of positivist point of view, damage rate is considered as the dependent variable and the following parameters are considered as independent variable: pipe material; peak ground acceleration, peak ground velocity and permanent ground deformation. In fragility curves, there are some other independent variables which can affect seismic vulnerability of buried pipelines but are ignored. Some of these variables are pipe diameter, buried depth, pipes age and other parameters explained in Chapter 2. In this study mathematical models such as statistical analysis which is used to correlate dependent and independent variables are not used in contrast with positivist methods which are purely based on quantitative analysis. However, quantitative data is used to show the differences between calculated damage and the observed damage in unpressurised wastewater networks.

While this research does not meet the full criteria for positivist research due to the breadth and complexity of the topic this is a preliminary research for future work. To confirm the findings of this research a larger sample of data is required to fulfil positivist research requirements.

## 3.3. Application of the case study research strategy

This section explains the advantage and disadvantages of case study method. It also further shows why case study method is used in this study to achieve the thesis overarching aim and the thesis objectives.

An adequate research strategy for each particular research can be selected by considering the form of research questions, which requires control of behavioural events and concentrates on contemporary events. As presented in Table 3.1, the case study method is similar to experimental methodology in terms of research questions. Case study is a preferred strategy, where the ‘how’ or ‘why’ questions are the main types of research questions. Case study methods, in contrast to the experimental ones, have little control on the events observed. The case study methodology focuses on a contemporary phenomenon within a real-life context in contrast to the history methodology which focuses only on the past events.

This research is designed to find an appropriate answer to satisfy the following objectives:

Objective 1: Demonstrate the extent of seismic damage on the unpressurised wastewater networks in New Zealand in order to show the necessity of pre or post-earthquake measures in wastewater networks.

Objective 2: Identify the parameters which need to be included in fragility curves to achieve accurate results.

Objective 3: Compare the estimated defects in unpressurised wastewater pipes by a range of fragility curves

Objective 4: Recommend the most effective application of fragility curves in damage estimation of unpressurised networks.

Objective 5: Identify the relationship between damage rate and earthquake variables in unpressurised wastewater networks.

Objective 6: Discover limitations in application of trenchless techniques for post-earthquake repair and rehabilitation in unpressurised wastewater networks

In order to show the significant characteristics of the case study strategy the research method will be described, and then the advantages and disadvantages of the case study research will be accounted for. A case study can be defined as ”an empirical inquiry that investigates a contemporary phenomenon within its real–life context especially when the boundaries between the phenomenon and the context are not clearly evident” (Yin, 2003). Case study research can provide opportunities to understand complex issue (Dolley 2002).

Case studies are one of the several methods of conducting research, and are more and more applied as a research tools in many disciplines of both natural and social sciences (Dolley 2002). One of the misunderstandings about the case study research strategy is that, case studies are appropriate only for the exploratory phase of investigation (Shavelson & Townes, 2002). The truth is that case studies can be applied in all the three types of research: exploratory, descriptive and explanatory (Shavelson & Townes, 2002). As a result, case study methods can be categorised into three types; exploratory, descriptive and explanatory (Yin 1984).

Researchers claim that there are three disadvantages associated the application of the case study research strategy. The first and probably the greatest concern about application of case study research is that some researchers believe that conducting a case study research is not challenging. Lack of rigor in a case study research occurs when investigators do not follow a systematic procedure or allow biased views or equivocal evidence to influence their findings and conclusions. This lack of challenge is less evident in other strategies, probably due to the existence of numerous methodological texts with a particular procedure which are absent in the case study method. Bias is not restricted to case study research and can be experienced in other research strategies like experiment, survey and historical research methods (Sudman & Bradburn, 1982).

The second concern about the case study research method is the scientific generalization from a single case study. This concern is similar to the concern about generalization made from a single experience, although scientific facts are based on several experiments that are replicated in relation to the same phenomenon under different conditions. Similarly to experiments, case studies can be generalised in theoretical propositions, not in universal truths. To overcome this phenomenon, multiple case studies can be applied to generalise theories without statistical generalization in order to finally generalise scientific facts in the same as experiments.

The third concern about case study research is that it is a time consuming processes that comes with voluminous and unreadable documents. The fourth concern is that what was probably appropriate in the past case study research is not necessary appropriate at present or in the future (Feagin et al., 1991). Not all case study research is time consuming, except for case studies with a specific method of data collection like ethnography.

Flyvbjerg (2006) also explained how case study research is necessary and sufficient for certain imperative research tasks. He discussed five common misunderstandings regarding case study research which may restrict application of case study research. Public misunderstanding about application of case study research are based on the following beliefs (Flyvbjerg, 2006).

1- Practical knowledge in case study research is less valuable compared to theoretical knowledge.

2- The single case study cannot contribute to scientific development.

3- Generating hypothesis is the best outcome of case study research.

4- Verification of case study research method can cause bias.

5- Summarizing case study research is difficult compared to other method.

The case study by itself is not a research method; it consist of a wide range of diverse methods of data collection and analysis methods employed in the case study research (Willing, 2001). A study of one or more cases can be in depth and detailed in with respect to the examined cases (Hossain, 2011).

Table 3.2: The method used to answer the thesis questions

| Research question | Research objective | Research method |
|---|---|---|
| Q1. How vulnerable are unpressurised wastewater networks against seismic shocks in New Zealand and are pre or post-earthquake measures required to decrease seismic damage in these networks? | O1. Demonstrate the extent of seismic damage on the unpressurised wastewater networks in New Zealand in order to show the necessity of pre or post-earthquake measures in the wastewater networks | Literature review<br>Application of archival data in four case studies and calculation of seismic damage by fragility curves. |
| Q2. Are the parameters which fragility curves are based on sufficient and if not what other parameters should be taken into consideration? | O2. Identify the parameters which need to be included in fragility curves to achieve better results | Literature review<br>Comparison of estimated defects in 4 case studies<br>Analysis of the earthquake damage in Christchurch |
| Q3. How does the application of a range of fragility curves which have been developed from seismic damage on pressurised networks affect seismic damage estimation on unpressurised networks | O3. Compare the estimated defects in unpressurised wastewater pipes by a range of fragility curves | Calculation of seismic damage by fragility curves on each case study and comparison of the results. |
| Q4. What is the appropriate application of fragility curves for estimation of seismic damage on unpressurised networks? | O4. Recommend the most effective application of fragility curves in damage estimation of unpressurised networks | Comparison of calculated results by fragility curves in each case study.<br>Comparison of estimated defects with the observed damage in Christchurch |
| Q5. How does the seismic damage rate in unpressurised pipelines differ when compared to the current fragility curves in pressurised networks | O5. Identify the relationship between damage rate and earthquake variables in unpressurised wastewater networks. | Analysis of archival data after the earthquake in Christchurch |
| Q6. What are the current limitations of trenchless technique use for post-earthquake repair and rehabilitation of unpressurised wastewater networks | O6. Discover limitations in application of trenchless techniques for post-earthquake repair and rehabilitation in unpressurised wastewater networks | On-site observation after the 2010 earthquake in Christchurch and in the Waimakariri District |

According to complexity of earthquake effects on buried pipelines and effect of many parameters on seismic vulnerability of unpressurised wastewater networks, an in-depth study is required to achieve the thesis objectives. Furthermore, there is no study in this topic in New Zealand and available data is limited. Therefore, a case study method is used in this thesis to do an in depth study on seismic vulnerability analysis of unpressurised networks. Furthermore, multiple case studies are selected in order to develop and generalise the research findings.

Table 3.2 shows the research questions and the relevant objective to each question. This table shows the method used for each research objective in order to answer the research questions and achieve the research objectives. The attempt made in answering the questions would in a sense satisfy the major questions addressed in this thesis.

## 3.4. Research design

What helps an investigator during the research procedures, such as data collection, data analysis and interpreting research findings, is described as the research plan (Frankfort & Nachmias, 2000). The research design can be defined as a tool used for correlating empirical data to research questions and finally to conclusions, in a logical order (Yin 2003). According to each research objective and scope, various procedures should be considered in order to achieve the research objectives; therefore, in order to achieve the research objectives, the researcher should design and apply appropriate procedure.

### 3.4.1. Component of research design in this study

A research design can be defined as a blueprint of research which should solve at least four problems. These are; what questions need to be studied, what data is relevant to the research, what kind of data should be collected and how would the research results be analysed (Philliber et al., 1980). According to Yin (2003), each research design comprises of the following five components in each case study; research questions, research propositions, unit of analysis, logical linking of the data to the proposition, and criteria for interpreting the research findings. The research questions are discussed in Chapter 1 and the other components of research design are explained in the following sections.

#### 3.4.1.1. Research propositions

The first proposition here is that the wastewater pipelines can experience intensive damage after an earthquake. Many studies have been conducted on seismic damage on pressurised pipes but very few on wastewater pipeline damage. The different specifications can cause differences in expected damage between pressurised and non-pressurised pipes. These specifications are flow type, carrying fluid, pipe material and others.

The application of fragility curves developed for pressurised pipes is not an appropriate tool to accurately investigate the expected damage in wastewater pipelines. It is possible to develop similar fragility curves appropriate to New Zealand but this would be limited by various factors affecting seismic damage.

A further proposition of this thesis is that the application of trenchless techniques for post-earthquake repair and rehabilitation are likely to have some limitations in usage in spite of their advantages in normal usage. On the other hand, it is probable that the traditional open-cut method application is more appropriate than trenchless techniques after an earthquake.

#### 3.4.1.2. Unit of analysis

To fulfil the scope of this research some unpressurised wastewater networks in New Zealand were used as relevant case study within New Zealand. Unit of analysis in this study is unpressurised wastewater network. An unpressurised wastewater system comprises of components like wastewater pipelines, pumping stations, manholes and wastewater treatment plants. The focus of this thesis is on unpressurised wastewater pipelines. In this study the length, location, type of joints, materials, buried depth, date of construction of wastewater pipes constitute the network specifications. The type and nature of damage in wastewater pipelines after an earthquake were also used and studied in this research.

#### 3.4.1.3. Logical linking of the data to the proposition

In order to link the thesis proposition to the data in a rational manner specific type of data should be at hand. Quantitative data from different resources is used for data analysis in this study. Seismic vulnerability in the unit of analysis (wastewater network) in this study is shown by quantitative data such as damage rates and number of expected defects caused by seismic shocks.

The literature review showed that many parameters can affect seismic vulnerability analysis but a few were used in fragility curves. As a result, to calculate seismic damage by fragility curves the required parameters should be considered and relevant data should be collected.

#### 3.4.1.4. Criteria for interpreting the research findings

The criteria for interpreting the research findings of this thesis is dependent on the similarity between the case study specifications in this study and the specifications of the sites to which it would be compared. For instance, the graphs in Chapter 8 which demonstrate the trend between damage rates and PGV or PGD can only be applied in areas with similar soil specification as Christchurch. The graphs demonstrate the trend in unpressurised wastewater networks in an area with a type of soil which is susceptible to liquefaction. Consequently, this graph cannot be used for explaining the trend in areas with different specifications than those of Christchurch.

This thesis concentrates on wave propagation effects of seismic shocks on unpressurised wastewater networks in New Zealand. As a result, the differences observed in the calculated defects by fragility curves will vary when applied in pressurised networks or when used for a network with different specifications.

### 3.4.2. Case study research activities

A research design comprises the following 5 stages in order to achieve the research objectives: Literature Review, Case Studies, Data Collection, Data Analysis and Validation which are discussed in the following sections.

#### 3.4.2.1. Literature Review

Literature review is typically one of the initial parts of every research. This study consists of four main phases of literature review. The initial phase concentrates on seismic effects of the past earthquakes on water and wastewater lifelines, and specifically on wastewater pipelines. The purpose here is to find out how vulnerable the buried pipelines, especially wastewater pipelines, are against an earthquake in different regions. However, there is no accurate and notable document relevant to the effects of the past earthquakes on the wastewater networks in New Zealand except for the two last devastating earthquakes in 2010 and 2011.

The second phase concentrates on earthquake vulnerability assessment of buried pipelines including wastewater pipeline. This phase focuses on the available fragility curves which can be applied in seismic vulnerability analysis of buried pipelines particularly for wave propagation effects of earthquakes on buried pipelines. Application of the available fragility curves for assessing seismic vulnerability of wastewater pipelines in New Zealand requires another literature review to determine the required data. The third phase mostly concentrates on the domestic based publication in order to extract required geological and seismological information for each case study.

The data analysis process reveals how vulnerable wastewater pipelines in New Zealand are. The 2010 and 2011 Christchurch earthquakes showed how the rehabilitation and repair of the damaged wastewater pipelines can be significant in post-earthquake restoration. In order to recommend solutions to increase sustainability of the wastewater system in regions located in earthquake prone areas after earthquakes, the fourth and the final phase of literature review is done. In this phase, advantages and disadvantages of different rehabilitation and renovation methods are investigated in order to show the restriction on application of trenchless techniques after an earthquake

In order to cover all aspects of the thesis objectives, information was sourced from related books, journal articles, technical document, standards, conference proceedings, domestic news, national and international technical web pages as well as some confidential technical reports.

#### 3.4.2.2. Case study sites

In this thesis the following 5 case study sites are selected: Hutt City, Gisborne, Blenheim, Christchurch and Waimakariri. Hutt City, Gisborne, Blenheim, Christchurch cover the first four of the thesis questions. Christchurch City and Waimakarairi District in Canterbury region cover the scopes of the last two thesis questions.

New Zealand is apportioned in 16 main urban areas: Whangarei, Auckland, Dunedin, Hamilton, Invercargill, New Plymouth, Tauranga, Christchurch, Rotorua, Wanganui, Wellington, Nelson, Napier and Hasting, Palmerston North, Gisborne and Blenheim. The main topic in this study is the earthquake effects on wastewater buried pipelines. As a result, selection of case study should cover the vulnerable urban areas in New Zealand.

Hazard factor is the main factor affecting seismic vulnerability of each area in New Zealand. Comparison of hazard factors in each main urban area to the hazard factors of the case study sites in this study shows that the selected case studies are all vulnerable to earthquakes with a high and medium seismic vulnerability to earthquakes (NZS 1170.5, 2004). Figure 3.1 shows hazard factor of the main urban areas in New Zealand. It illustrates that Whangarei, Auckland and Dunedin are the least vulnerable regions and are occupied with 51.15% of the total population.

The urban population exposed to a moderate to high risk of seismic forces constitute 48.85% of the total population. Comparison of the case study areas' populations shows the selected case study areas cover 31.81% of the total population in areas with medium to high risk of earthquakes (Statistics New Zealand, 2011).

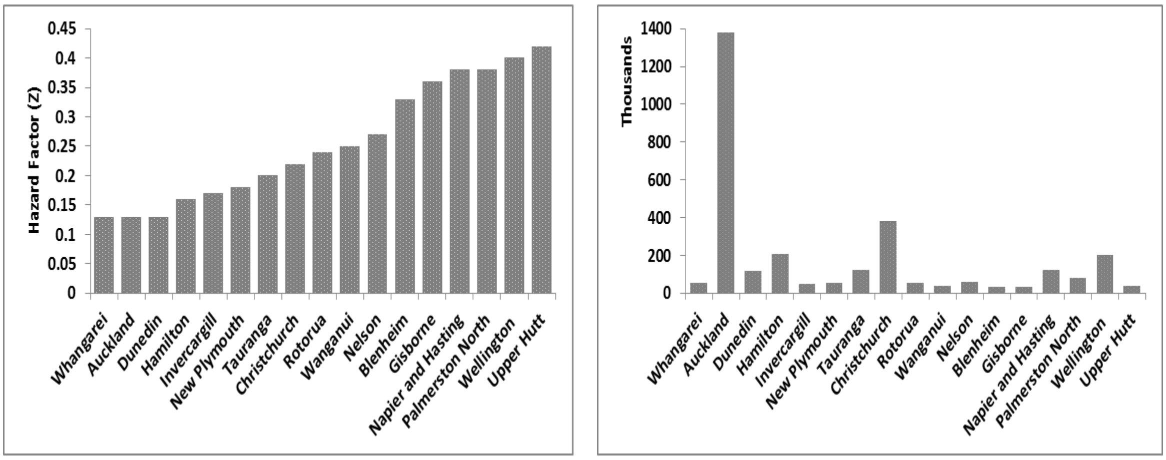


Figure 3.1: The Hazard Factor and population rates in the main urban areas of New Zealand

One of the disadvantages of the case study method is its concentration on a specific case which may be an appropriate representative for the research findings. In order to overcome this disadvantage, four earthquake prone cities are selected. All the selected case studies are located in the earthquake prone regions, where they can be subject to different active faults effects and have different earthquake vulnerability. The North Island and the South Island are two main territories where New Zealand cities are distributed in. Consequently, the case study areas are selected from both Islands.

Gisborne and Hutt City are two case study areas in the North Island, and the Blenheim, Christchurch and Waimakariri are the three in the South Island. Gisborne is selected because in 2007 it experienced earthquake. Consequently, its wastewater system damage can be a good sample for earthquake prone cities. For instance, the Gisborne earthquake in 2007 demonstrated that the earthquake can pollute streams and endanger the environment significantly even if it is not devastating. The Gisborne earthquake also showed that particular points in wastewater networks can be damaged severely and cause severe damage in spite of their very small length.

Hutt City is selected as the second case study area because it is located on the same active fault belt as the capital city. Thus, the findings in terms of seismic vulnerability of wastewater networks can be applied for the capital city as well. The Hutt City wastewater network is selected as the pilot among the first three case study areas because it has more data associated with it. Gisborne and Blenheim have less data in terms of wastewater network, geological and seismological features. The wastewater network layout in Hutt City is different from other wastewater networks in New Zealand. Comparison of this network with other wastewater networks reveals other types of seismic vulnerability in wastewater network which can affect wastewater network more severely. For instance, the Hutt City wastewater network review shows that the pressurised wastewater pipelines used in transferring the treated wastewater to the discharge points can significantly increase seismic vulnerability of the network compared to networks without this component.

The city of Blenheim is located in the northeast of the South Island as the largest city in Marlborough region in vicinity of some active faults. The Blenheim wastewater system is selected as an example of municipal regions in the South Island. The significant difference between the Blenheim and Hutt City and Gisborne is discharge of a significant volume of industry wastewater to the Blenheim wastewater network. As a consequence, wastewater system failure which may be caused by an earthquake can, in turn, not only causes notable effects on the residential areas but can adversely affect the industrial regions.

The Darfield earthquake, in September 2010, was one of the most destructive and costly earthquakes in the history of New Zealand, that caused severe damage in lifelines of the affected regions. Christchurch City and Waimakariri District, as the most affected regions after the 2010 Darfield earthquake are selected for demonstrating the seismic shocks damage on the wastewater network. Comparison of the research finding with the earthquake effects in Christchurch demonstrates the wastewater system vulnerability, significant affecting parameters and applicability of the available fragility curves in New Zealand. It is also used to identify the advantages and disadvantages of the post-earthquake rehabilitation methods.

### 3.4.2.3. Data collection procedure

Case study research requires evidence to conduct analysis, and these evidences are extracted from the following six sources: documents, archival records, interviews, direct observations, indirect observation and physical evidence (Yin 2003). Each source has some advantages and disadvantages compared to other sources. Here, first the advantages and disadvantages of each method will be briefly discussed and then different types of evidence used in the thesis as the data collection procedure will be explained.

### 3.4.2.4. Documents

Documents are a common type of evidence for all case study type research. Different sources of documents can be used in case study research, including communication, written reports, formal studies and other articles. As for different types of data required for the study, various types of documents are used in the data collection procedure.

Documentation has some advantages and disadvantages compared to other types of evidence. For instance, documentation can be reviewed repeatedly, contain exact details and usually have a broad coverage in terms of chronology in relation to many events and many relevant subjects to the research topic. Documentation also has some weaknesses like bias, which may be caused by incomplete data collection or by a document’s author in terms of reporting: while sometimes access to documents may be restricted by the relevant authorities.

Technical reports prepared by consulting engineering companies for their clients (city councils) are used in collecting data about the wastewater network and its components in the case study areas. For instance, data on the Hutt City wastewater network is obtained from Capacity Company, the Hutt City water and wastewater networks’ main contractor, and a consulting company, that designed some parts of the Hutt City’s wastewater system. In each case study, the required and relevant general documents about the wastewater system are provided by the city councils, or directly extracted from the city councils’ web pages.

The geological information about the case study areas used for analysing is extracted from available scientific articles or technical reports provided by geologists. For instance, soil classification and some geological specification of each case study area is extracted from technical reports provided by GNS science researchers.

Various fragility curves applied in this thesis for comparison of earthquake effects on buried pipelines in the case study areas are extracted from different documents mostly journal articles (presented in the literature review chapter).

### 3.4.2.5. Archival records

Service records, organization records, maps, charts, lists of relevant items and personal records are archival records in addition to computer files. Archival records are similar to documentation in terms of advantages and disadvantages. The significant advantage of archival records over documentation is that they are precise and quantitative; while hard accessibility to these records is the notable disadvantage.

City councils are responsible for provision of an appropriate access for city residents to the wastewater system, as well as for operating, repairing and rehabilitating the wastewater system. In each case study, the city council has provided and updated the required information about the wastewater system. Consequently, each city council is the main authorised source of the data; however sometimes operating contactors can provide more detailed data in a specific field.

Archival records for each case study are collected and used for the data analysis as the most accurate and available data in this thesis. Wastewater pipelines connect each outlet to the final destination of the collected wastewater which is the wastewater treatment plant. Consequently, a wastewater network usually comprises of various types of pipes in terms of all the aforementioned pipe specifications, like the pipe type and the diameter, which cannot be easily managed without adequate tools. As a result, Geographic Information System (GIS) is typically applied as a powerful tool by city councils to manage wastewater pipelines data. For instance, GIS information about wastewater pipelines and wastewater pumping stations in Hutt City, Blenheim and Christchurch are collected for each case study area as GIS files.

Wastewater pipeline specifications such as the length, type, diameter, durability, buried depth and joint type of pipes are obtained from GIS data base of each city. However, some pipe specifications like the buried depth, durability and the joint type are not available for notable portions of buried pipelines in each network. Information about wastewater pipeline network in Gisborne is available in AutoCAD files provided by the Gisborne district council.

In this study not only the relevant information about each case study area's wastewater network is important but the seismological characteristics of each case study area should be extracted. Geological characteristics of each case study area are available from various published scientific works and from technical reports archived by research institutes. For instance, technical reports of GNS science are used in each case study to classify soil types. All the case study documents and references used here are referenced in the case study chapters (4, 5, 6 and 8).

#### 3.4.2.6. Direct observation

Direct observation covers the events in real time as well as the context of the event. These are the two significant advantages of direct observation. Extra time and costs allocated to direct observation are two disadvantages of this type of data collection. Bias can also be entered into direct observation because of selectivity of observations and reflexivity (an event may proceed differently when it is being observed). Direct observation is also applied as one of the main sources of data collection in the rehabilitation part of the thesis.

In Christchurch the damaged parts of the main wastewater treatment plants, damaged wastewater pump stations and also many wastewater pipelines repair sites were visited and required data was collected from repair groups which mostly were the contactors.

The most significant damaged components of the Waimakariri wastewater system was the wastewater pipelines network. Here, repair sites of the damaged pump stations and wastewater pipelines were visited similar to what was done in Christchurch. The personal observation accompanying professional engineers, facility managers, professional repair teams, contactors, facility operators and residents of the regions provide an opportunity to evaluate post-earthquake repair and rehabilitation processes.

#### 3.4.2.7. Consultation

Many professional consulting engineers and contractors in New Zealand and Iran were asked about application of new methods of repairs and of trenchless techniques. In this thesis professional contactors, especially those involved in the trenchless repairs and inspection in construction sites located in the Christchurch and Waimakariri district, were consulted.

Table 3.3: Data collection techniques

| Types of Data Collection Methods | Applied Sources | Application |
|---|---|---|
| Document analysis | Technical reports<br>Consultant reports<br>Books, journals, papers<br>Standards<br>Web pages | Literature review<br>Selecting fragility curves<br>Calculating required data applicable in fragility curves<br>Correlating data<br>WWS specification and WWS components characteristics of each case study |
| Archival records | GIS data base (provided by the city council)<br>Maps (provided by the city council and extracted from documents) | Extracting data relevant to the wastewater network of each case study (except for Gisborne) including pipelines and pumping stations<br>Extracting data relevant to the wastewater network Gisborne)<br>Collecting data about seismic vulnerability and geological characteristics of each case study city |
| Direct observation and consultation | City council ( Gisborne, Blenheim Hutt City, Christchurch, and Waimakariri)<br>Consultants and Contractors (Christchurch and Wellington) | Real condition of different components of wastewater system especially WWPSs and WWTP<br>Post-earthquake rehabilitation problems<br>Applied repair techniques after the earthquake |

The author also meet with academic staff in Iran including Isfahan University of Technology and the Isfahan Higher Education and Research Institute. The three methods of data collection techniques, the document analysis, archival record and observation and consultation are used in this study. Table 3.3 also shows the application of each data collection method used in this thesis.

As this study progressed, the author had consultation meetings with some researchers and senior engineers from GNS science, Hutt City Council, Gisborne City Council, Christchurch City Council, Blenheim City Council, Waimakariri District Council, the Esfahan Water and Wastewater Organization, Mahab Ghods consulting Engineers, and Esfahan Regional Water Organization. Table 3.4 shows the list of the organizations which have participated in different stages of the research.

Table 3.4: The list of the organizations which have participated in this study

| Organization | Profession | Case study |
|---|---|---|
| GNS | Risk researcher | General |
| GNS | Geologists | General |
| GNS | Seismologists | General |
| Kestrel Group - Risk, Continuity and Emergency Management | Director | General |
| Marlborough district Council | Manager | Blenheim |
| Gisborne District Council | Manager | Gisborne |
| Gisborne District Council | Draughtsman | Gisborne |
| Capacity Infrastructure Services Ltd. | Asset Planner | Hutt City |
| MWH | Consultant | Hutt City |
| Hutt City Council | GIS technician | Hutt City |
| Christchurch City Council | GIS technician | Christchurch |
| BECA | Manager | Christchurch |
| AECOM | Manager | Christchurch |
| Waimakariri District Council | Manager | Waimakariri |
| Pipework Ltd. | CIPP Repair Group | Waimakariri |
| Transpacific Industrial Solutions Ltd. | Site Manager | Christchurch |
| Waimakariri District Council | Utilities Officer | Waimakariri |
| Transpacific Industrial Solutions Ltd. | CCTV Inspector | Christchurch |

### 3.4.3. Data analysis process

Data analysis can be defined as breaking up, separating or disassembling of research materials into manageable pieces to look for classes, sequences, processes or patterns. Meaningful and comprehensive data is a result of a proper data analysis process (Jorgensen, 1989).This study is based on case study research where quantitative data can be used and data analysis process of a case study research is accounted for.

According to Yin (2003) the data analysis process is involved with examining, categorizing, tabulating and testing the case study evidence. Similarly, the data analysis process is a combination of quantitative and qualitative evidence employed to address the propositions of a study. Yin (2003) noted that every case study should have an analysing strategy for defining priorities for what to analyse and why.

Yin (2003) recommended 5 different techniques which can be used in each strategy for analysing the case study evidence: pattern matching, explanation building, time series analysis, logic models and cross-case synthesis.

Pattern matching is one of the most appropriate data analysis process, which compares an empirically based pattern with a predicted pattern or patterns that ,in explanatory case studies, may be related to different variables observed in the study (Trochim, 1985). Explanation building is a specific pattern matching technique, but this is used to build an explanation about the case study in order to analyse the case study data. Explanation building has been extensively used in narrative form in most available case studies.

Time series analysis as the third analysing technique is used in case studies, similar to time series analysis used in experiments or quasi-experiments. Time series analysis can be divided into three main types: simple time series analysis, complex time series analysis and chronologies. The logic model is another form of pattern matching technique with sequential stages. The logic model consciously stipulates a complex chain of events over time (Eisenhardt, 1989). The last technique as cross-case synthesis is used for analysing multiple cases. The cross–case synthesis, like cross experiment interpretations, has no numeric properties in case only few experiments are available, and only argues about what can be supported by the data.

The data collected in this thesis is not chorological or restricted in a time period; hence, pattern matching techniques are used as the analytical technique. Being a multi-case study, the cross-case synthesis technique is also employed.

#### 3.4.3.1. Data analysis strategy used in the thesis

One of the objectives of the thesis is to show the seismic vulnerability of wastewater pipelines in New Zealand. With reference to the literature review, earthquake effects on buried pipelines can be calculated by two groups of empirical equations. The first group calculates the damage on buried pipelines caused by the overall effects of an earthquake including wave propagation and permanent ground deformation. In the first group, there is no distinction between different effects of earthquakes. As indicated in Chapter 2, seismic damage in buried pipelines can be caused by two distinct phenomena; wave propagation and ground deformation. The wave propagation effect of earthquakes is selected as the most significant factor which can affect earthquake struck regions.

In the context of the classification explained above, the data analysis procedure in this thesis is divided into two phases. The first phase comprises of, case study selection, required data extraction, zoning each case study for better and accurate analysis, calculating damage inflicted by seismic forces and finally comparing the calculated results for better conclusion. Figure 3.2 shows the flowchart for the first phase of data analysis strategy.

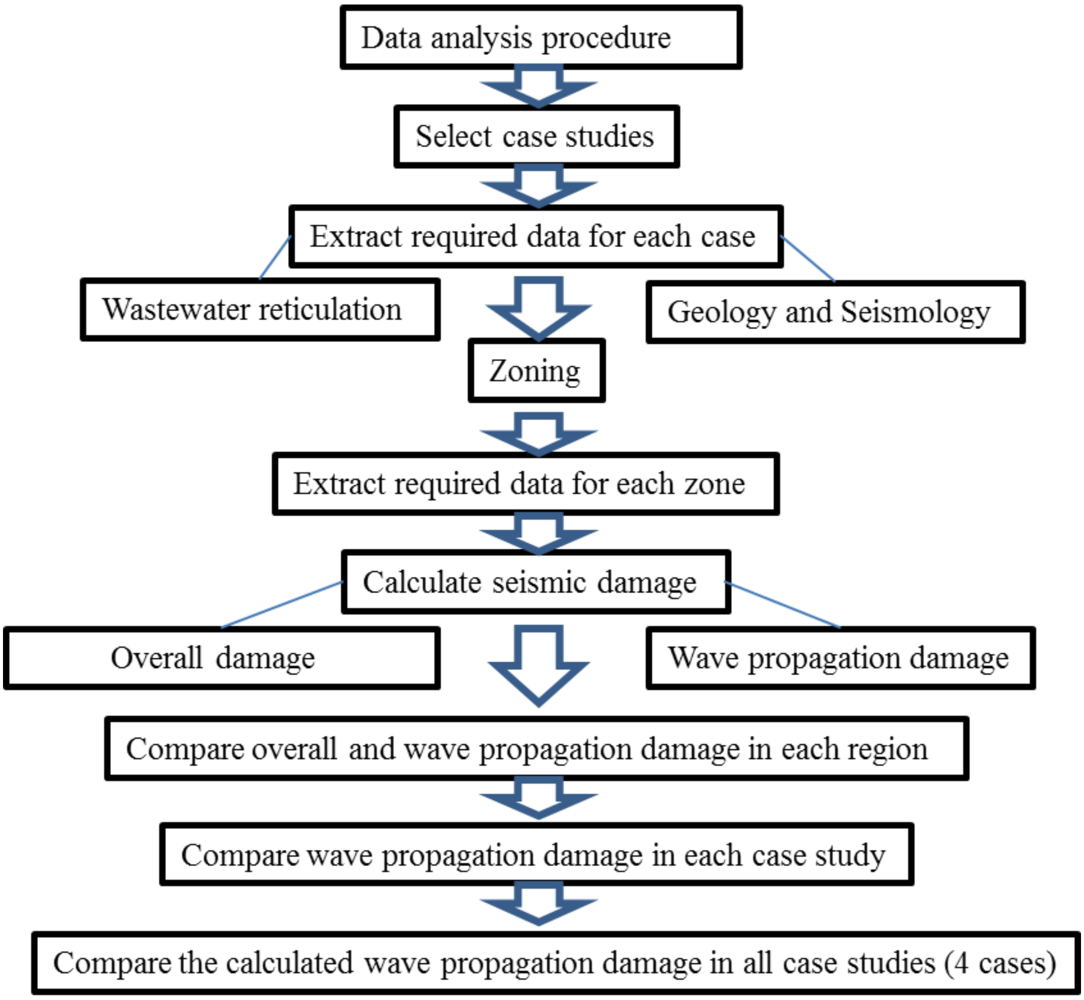


Figure 3.2: The first phase of data analysis strategy

Earthquake damage in each case study is calculated by available fragility curves. The overall and wave propagation effects of earthquakes are calculated in each study. Each case study is divided into some smaller regions in order to calculate more accurate results. Main streets, rivers and city boundaries are main factors used to divide each city into some regions with more similar characteristics.

Some criteria are employed to select fragility curves which are used to calculate earthquake damage in each case study. The main criteria for selecting fragility curves are the earthquake parameters used in each fragility curve which should be easily extractable and applicable from available resources in New Zealand. The fragility curves are selected from well-known resources which have already been used here as estimation tools to assess seismic damage, including fragility curves recommended by FEMA (2003) and ALA (2004).

Application of the available fragility curves in the thesis requires not only adequate earthquake parameters but also proper and relevant pipelines data. In order to achieve better estimation of seismic damage, and extract more accurate data in terms of earthquake parameters and pipelines specifications, each case study area is divided into at least 4 regions and then data is extracted for each region.

The first step in calculating expected seismic damage in wastewater network of each case study is extracting the required seismic parameters. To analyse each case study, the required earthquake parameters for each city are first extracted from available resources and then the parameters are amended according to soil classification for each municipal region inside each case study.

Each fragility curve used in the thesis is developed for a special type of pipe. Accordingly, in order to calculate seismic damage on wastewater pipelines in each case study, pipelines data should be extracted and classified for each case study. For instance, wastewater network in each case study comprises of different types of pipes, and each fragility curve is applicable only for a few types of pipes; consequently, pipes should be classified in the most appropriate way to be applicable to any particular fragility curves.

Calculated damage in each case study is compared to one another in each zone and each case study. Calculated damage rates are also compared in case studies where the intrinsic difference which can be caused by application of each fragility curve is revealed.

The second phase of data analysis strategy is correlating the observed defects given a real earthquake in New Zealand to the observed seismic characteristics after the earthquake. In this phase the earthquake characteristics used in the applied fragility curves are extracted and correlated to the obtained damage rate.

The damage distribution calculated by the fragility curves is compared to the distribution of observed damages in the last case study. Figure 3.3 shows the flowchart for the second phase of data analysis strategy in this study.

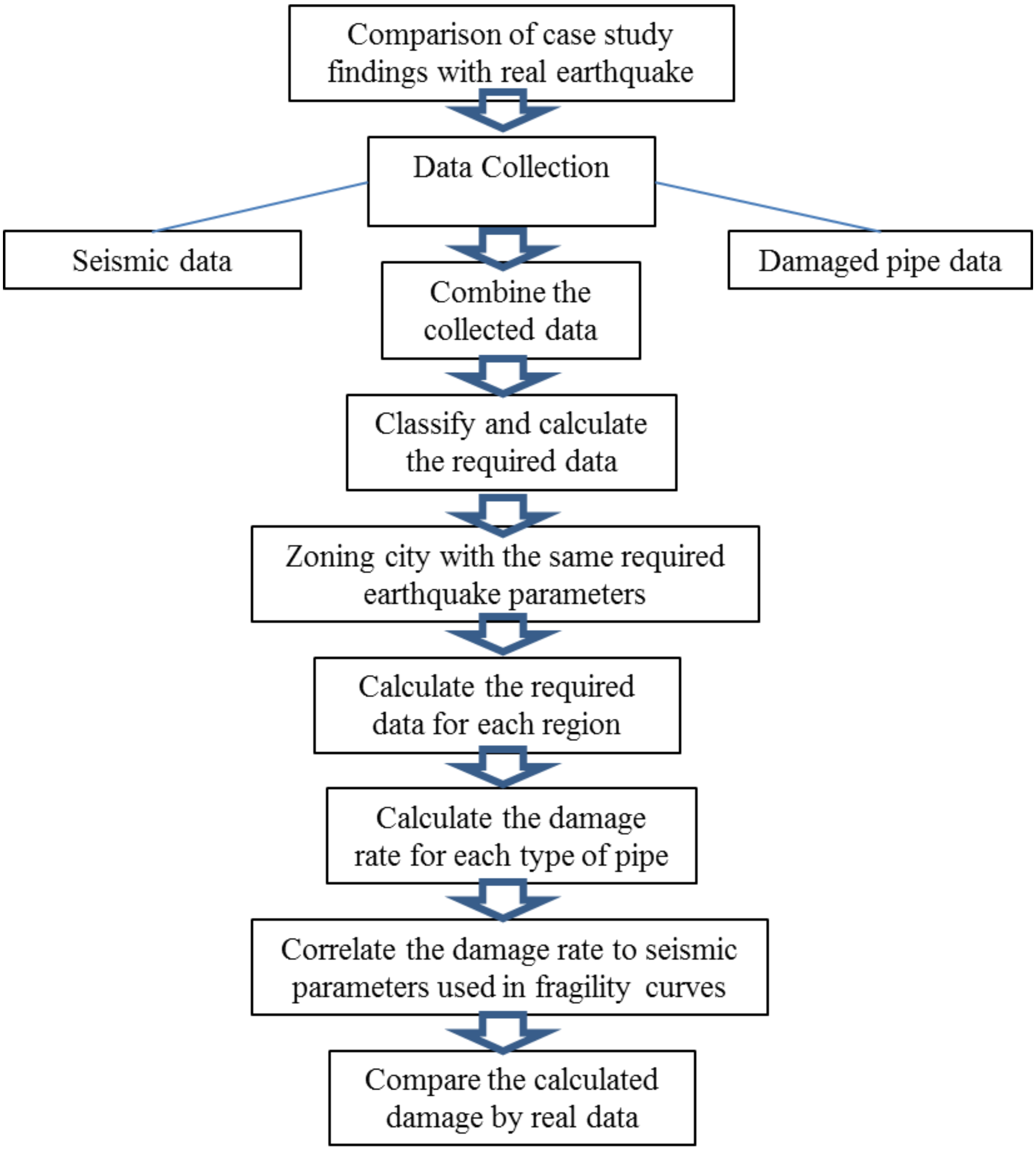


Figure 3.3: The second phase of data analysis strategy

### 3.4.4. Reliability and validity

Each research design should represent logical statements which can be judged by logical tests. Four tests are usually used to show the quality of each case study research, and each test is comprised of tactics to be dealt with. Table 3.5 shows different types of research design tests and tactics which can be used in each test.

Table 3.5: Research design test

| Test | Case Study Tactic | Phase of Research |
|---|---|---|
| Construct validity | Using multiple source of evidence<br>Establishing a chain of evidence<br>Having key informants<br>Reviewing the draft of case study report | Data collection<br>Data collection<br>Data collection<br>Composition |
| Internal validity | Doing pattern matching<br>Doing explanation-building<br>Addressing rival explanation<br>Using the logic model | Data analysis<br>Data analysis<br>Data analysis<br>Data analysis |
| External validity | Using a theory in single-case studies<br>Using replication logic in multiple case studies | Research design<br>Research design |
| Reliability | Using case study protocol<br>Developing case study database | Data collection<br>Data collection |

### 3.4.4.1. Construct validity

The literature review in this thesis provides an opportunity to follow multiple sources of evidence from various researchers and in different earthquakes in order to develop and achieve the objectives. For instance, available evidence reported by many researchers in different locations is used to explain parameters affecting seismic vulnerability in buried pipelines. Furthermore, multiple sources of evidence to achieve construct validity is reflected through the use of four separate case studies.

Different sources such as scientific publications and reports are used in this study to collect information in order to develop a chain of evidence. The chain of evidence is another factor used to achieve construct validity.

Another way to achieve construct validity is through the collection of the most accurate data from key information sources for each case study. For instance, a GIS database for each case study area is collected as the most comprehensive and accurate available data set. The database is collected from the city council of each case study area as a key informant.

The author reviews the data analysis reports of a previous case study or case studies before replicating analysis of any new fragility curve or new case study. The same procedure is used for each study. Finally, all collected reports are reviewed together in a draft form prior to writing the final report of each case study. Continuous reviewing of each case study reports and comparison between reports provide opportunity to reduce errors in data analysis procedure which is another step taken to achieve construct validity.

#### 3.4.4.2. Internal validity

In this study pattern matching is used to compare calculated results by different fragility curves to achieve internal validity. In this thesis, first a fragility curve is used for seismic vulnerability analysis of the first case study. The analysis is repeated for other fragility curves within the same case study and pattern matching is used to compare and control the calculated defects. The same analysis is conducted for the other three case studies and pattern matching is used to control errors and to develop a result comparison in order to achieve the thesis objectives. This process contributes to the internal validity of this case study method.

Observed damage in the unpressurised wastewater network after the Christchurch earthquake in 2010 provides an opportunity for pattern matching. This pattern matching is used to compare the observed correlation of damage rates versus both PGV and PGD for various types of pipe in Christchurch with the equations recommended by other researchers.

There is a rival explanation expressed in the literature review that fragility curves can be used for effective seismic vulnerability analysis of unpressurised networks. The author disagrees with this opinion and addresses this issue in chapters 4, 5, 6, and 8. For example, observed seismic damage in the Christchurch wastewater network after the 2010 earthquake in Christchurch indicates some limitations for application of fragility curves.

A logic model is used in this study and is initially based on the literature review. Then, a recommended fragility curve is applied to analyse seismic vulnerability of an unpressurised network in the first case study. Next, other fragility curves developed from unpressurised networks are used to show the differences within calculated damages.

Subsequently, the same analysis procedure is used for the other three case study areas. Finally, the observed seismic damage in a unpressurised network is compared with the calculated damage. The observed damage in the Gisborne wastewater network after the 2007 earthquake is compared to the calculated damage from fragility curves. The same procedure is replicated for the Christchurch wastewater networks. These are the logical steps taken for internal validity. Figures 4.2 and 4.3 also show the visual representation of the logical sequence.

### 3.4.4.3. External validity

A comparison of the defects calculated by fragility curves and the recorded post-earthquake defects in Gisborne is used to test the theory that application of fragility curves has some limitations when applied to unpressurised networks. The same procedure is then applied to the Christchurch unpressurised wastewater network to achieve external validity.

A single case study is sometimes criticised for offering an inadequate basis for generalization. External validity is therefore hard to achieve when doing a single case study. In this thesis, four case studies are used to replicate the research procedure. The results are compared to each other and the research findings can be generalised in order to confirm external validity.

Furthermore, the theory that application of trenchless rehabilitation and repair methods can have some limitations for post-earthquake repair and rehabilitations is analysed in the Waimakariri district wastewater network and is then replicated in the Christchurch wastewater network. This process of replication is a further step to confirm external validity.

### 3.4.4.4. Reliability test

The objective of the reliability test is to enable other investigators to conduct a similar case study with the same procedures in the future, and obtain similar result as the previous researcher did. The objective of reliability is to minimise errors and bias in a study. Case study protocol is a tactic which contributes to the reliability of case study research. The protocol is a set of guidelines which guarantee uniformity in data collection and analysis in a case study research (Pervan & Maimbo, 2005). Case study protocol includes the following four steps: overview of the case study project e.g. topics being investigated, field procedures e.g. sources of information, case study questions considered during data collection and finally a guide for case study report (Yin, 1994).

As explained above, a uniform method is used for data collection and data analysis for each case study. The author follows the same procedure required for data collection and data analysis. For instance, Figure 4.2 shows the case study protocol which is applied in this study for each case study.

As explained in Chapter 2, seismic vulnerability analysis in buried pipelines needs specific data for each part of analysis. Required data is collected from different resources to develop a database for each case study. The required data is categorised into three groups in order to develop a case study database: wastewater network specification, soil type and seismic vulnerability.

Required data for each case study wastewater network is collected from the city council as the best source of information. Essential data needed for soil type and seismic vulnerability of each case study is also collected from trustworthy sources. Publications of the New Zealand Standard are the principal sources of data required for soil type and seismic vulnerability. Other trustful sources of information are taken into account for developing each case study database such as journal papers, reports and websites from well-known professional societies in New Zealand such as GNS science. The comprehensive database confirms reliability in this case study research

# Chapter 4. Hutt City case study

## Introduction:

In this first case study, the wastewater network in Hutt City will be discussed in more detail compared to other case studies. In this study, 3 main case studies are employed in order to follow the research objectives, although one more case study will be discussed later to evaluate the thesis findings. This chapter concentrates on the seismic vulnerability of the Hutt City wastewater network. Several fragility curves are applied in this chapter to not only investigate seismic vulnerability of this network but to investigate the differences within calculated results through each fragility curve.

## 4.1. Hutt City

Hutt City in the North Island is geologically known as Lower Hutt City and is located at the southern part of the North Island in New Zealand. Hutt City is the second major city in the Wellington region, and is located in the Hutt River Valley in the area of 7,988 hectares, between Wellington (capital) in the southwest and Upper Hutt City in the north (Hutt-City-Council 2006).

There are 34,662 households in Hutt City, 20% of which are located in the central Hutt City region (Statistics 2009). The population of Hutt City is 102,100 and spreads in 8 municipal areas, according to the latest population estimation in June 30, 2009 (Statistics New Zealand, 2009a). All urban areas in Hutt City are covered by the same wastewater network which is operated by the Hutt City council.

This case study shows the seismic vulnerability of the Hutt City wastewater pipelines and is a good estimation of seismic vulnerability of Wellington. Both Hutt City and Wellington could be affected by the same fault lines (GNS 2009). In addition, Hutt City and Wellington wastewater networks were constructed and developed around the same time, although some parts of the Wellington wastewater system had been established before the Hutt City wastewater system (Wellington City Council, 2010). Similarly, both wastewater systems are managed and maintained with the same company and the research results can be applicable to Wellington (Wellington City Council 2010).

### 4.1.1. Active faults in Hutt City

The Wellington region including Hutt City is surrounded by the four main faults; Wairarapa, Ohariu, Wellington and sub-duction interface faults (Stirling, et al., 2002; Van-Dissen et al., 1992). The Wellington fault directly passes through the Hutt Valley region, whereas the two other faults; Wairarapa and Ohariu pass through the vicinity of the Hutt Valley. The Wairarapa and Ohariu faults are both located to the west of Hutt Valley (Van-Dissen, et al., 1992). Figure 4.1 shows the Wellington region, which comprises Wellington (capital) and the Hutt Valley region including Hutt City and Upper Hutt.

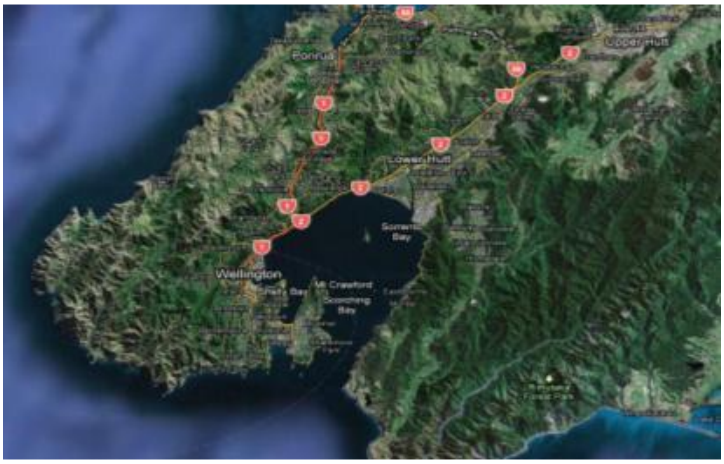


Figure 4.1: Wellington region including Wellington, Hutt City and Lower Hutt (Source: Google Maps 2010)

Table 4.1 shows the active faults near Hutt City, that makes Hutt City vulnerable against seismic shocks (GNS science, 2009). As shown in this table, the Wairarapa, Ohariu and Wellington faults have a recurrence interval of almost 2,000 years, whereas the recurrence interval of the Whitemans and Akatarawa faults is at least 5000 years (GNS 2009).

Table 4.1: Active faults in the Hutt City region (Source : GNS science, 2009)

| **Fault Name** | **Fault type** | **Recurrence interval** | **GNMS-ID** |
|---|---|---|---|
| **Wellington** | Dextral | 2000 | 21463 |
| **Ohario** | Dextral | 2000-3500 | 21455 |
| **Wairarapa** | Dextral | 2000 | 21475 |
| **Whitemans** | Reverse | 10000-20000 | 21473 |
| **Akatarawa** | Dextral | 5000-10000 | 21461 |

Figure 4.2 shows the location of active faults in the south west of the North Island, the Wellington fault passing through the west side of the Hutt City. The other faults are located in the vicinity of the Hutt Valley region.

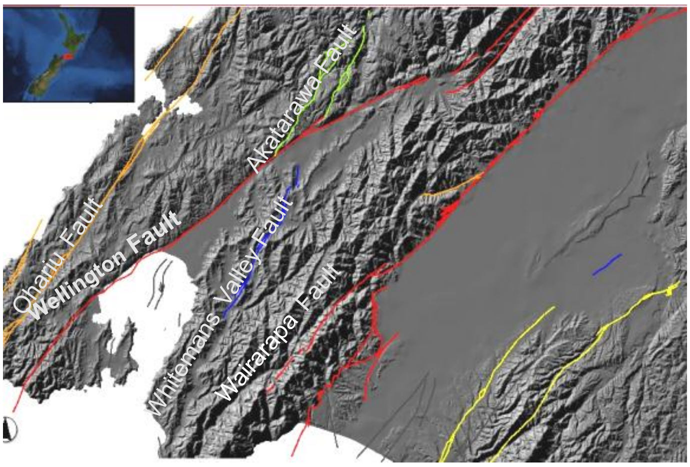


Figure 4.2: Active faults in the southwest region of the North Island (Source: (GNS 2009))

According to the geotechnical characteristics of the Wellington region, Stirling et al. (2002) estimated different PGAs for soil type B in the Hutt Valley region. He also calculated different ground accelerations for earthquakes with 150, 475 and 1000 year return periods. According to their study, PGAs of 0.3g, 0.5g and 0.55g are expected in Hutt City in earthquakes with return periods of 150 years, 475 years and 1000 years, respectively. The geotechnical characteristics of Hutt City indicate that an increase in return period means strong earthquake (Stirling, et al., 2002).

The other earthquake parameter used for calculating earthquake vulnerability of buried pipelines is the spectral acceleration of one second period (SA). Stirling et al. (2002) estimate SA (t=1) of 0.4 and 0.55 for earthquakes with 475 and 1,000 years return periods in Hutt City.

#### 4.1.1.1. Hutt City Earthquake vulnerability

Geological and geotechnical characteristics of each city have the greatest impact on the earthquake vulnerability of lifelines, especially on buried pipelines. The Wellington region is surrounded by the four main faults, Wellington fault passes through the Hutt Valley region and the other three pass in the vicinity of Hutt Valley. According to the geotechnical characteristics of the Wellington region, Stirling et al. (2002) have estimated different PGAs for soil type B in the Hutt Valley region. They have also calculated different ground accelerations for earthquakes with 150, 475 and 1000 year return periods. According to their study, PGAs of 0.3g, 0.5g and 0.55g are expected in Hutt City in earthquakes with return periods of 150, 475 and 1000 years, respectively.

The other earthquake parameter used for calculating earthquake vulnerability of buried pipelines is the spectral acceleration of one second period (SA). Stirling et al. (2002) estimate SA (t=1) of 0.4 and 0.55 for earthquakes with 475 and 1,000 years return periods respectively in Hutt City. Figure 4.3 shows seismic classification of Hutt City by Van Dissen (1992).

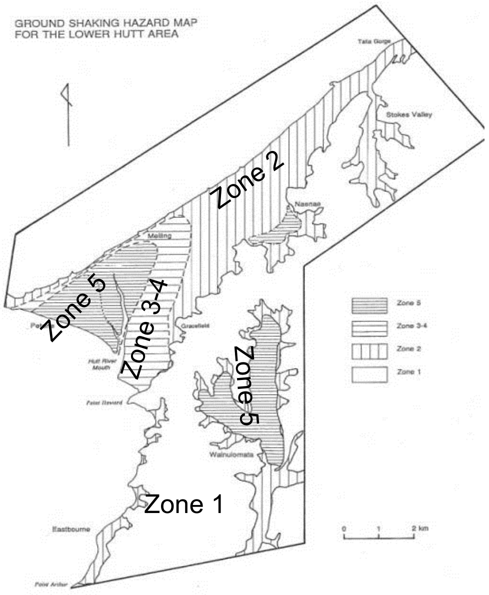


Figure 4.3: Hutt City Hazard map (source: (Van Dissen 1992))

### 4.1.1.2. Soil classification in Hutt City

Hutt City is geologically known as Lower Hutt city and is located in the southern part of the North Island. Dellow et al. (1992) collected and classified the geological characteristics of the Hutt Valley region including sediment types and near surface soil types (Dellow et al., 1992). According to Dellow et al. (1992) study, Hutt City lies on the variable Quaternary-age sediments (Dellow, et al., 1992). The sediments in Hutt City can be classified by their strength characteristics into soft sediments and loose to compact coarser-grained material sediments. Consolidated and fine-grained substances (clay, silt and sand) are normally the main constituents of soft sediments. Sand and gravel are the main materials in loose to compact sediments (Dellow, et al., 1992). The near-surface soft sediments with thickness greater than 10m are the predominant soil type in the Lower Hutt valley which covers Petone, Lower Hutt urban and the centre of Hutt City. The total depth of the Quaternary-age sediments in the Lower Hutt valley is 300 meters including near surface soft sediments. Soft sediment thickness decreases from the sea level in Petone onward to the centre of Hutt City, and varies from 27m in Petone to 10m in the city centre (Dellow, et al., 1992).

Hutt Valley, where Hutt City is located comprises of grained alluvial materials of different sizes which vary from fine to coarse. In the south of Hutt City (Petone), alluvial deposits are combined with marine beach deposits. Hutt Valley is underlain with 30 meters of the surface soil which varies from coarse gravel to fine grained silt and clayey silt (Dellow, et al., 1992).

Van Dissen et al. (1992) divide Lower Hutt into 5 zones for two earthquake scenarios according to the geological and geotechnical characteristics of Hutt Valley (Van-Dissen, et al., 1992). Two different scenarios were taken into consideration to delineate an earthquake ground shaking hazard in the Hutt City regions. Moderate to high, shallow and distant earthquakes which cause shaking on the bedrock with MMI V-VI are classified as scenario one, whereas scenario two involves great local Wellington fault earthquakes.

Van Dissen et al. (1992) note that the ground shaking hazard in Hutt City varies from zone 1 (bedrock) to the worst hazard case in Zone 5 (flexible sediments). According to the earthquake hazard classification, Petone, in the southern part of Hutt City central, and Wainuimata are all located in zone 5. In Scenario one, the MMI varies from V-VI in zone 1 to VIII-IX in zone 5. PGA in Scenario one can reach the highest value of 0.3g in Zone 5 to 0.01g in Zone 1. Amplification of ground motion which results from direct impact of soil types varies from 1-3 times in Zone 1 to 10-20 times in Zone 5 in scenario one. The earthquake hazard caused by the Wellington fault significantly affects the MMI and PGA of different zones in Hutt City. For instance, MMI varies from IX in zone 1 to XI in zone 5 and PGA varies from 0.5 to 0.8g respectively (Van-Dissen, et al., 1992).

Soil type in each municipal region should be estimated and taken into account as one of the main factors which can affect seismic vulnerability of buried pipes. According to the 1170.5 soil classification, soil type C is predominant in Seaview Gracefield (Zone 6). According to the depth of soft soil layer which is greater than 20 thick meters in Petone (Zone 4), soil type D is prevalent in this region. The depth of soil in Hutt City central (Zone 5) varies from greater than 20m to less than 10m. Thus the soil types in this region can be C and D. The south of Zone 5 is of alluvial gravel nature which can be classified as soil type D (Pickett & Laverty, 1998). Finally, Zone 5 can be classified as soil types C and D which are distributed evenly in the central parts of Hutt City.

In the Wainuimata region or Zone 8, the depth of soft sediment soil varies from more than 30 meters in the middle to less than 10 meters in the boundaries. Zone 8 is similar to Zone 5 as it contains alluvial gravel. Consequently, evenly distributed soil types D and C can contribute to the soil type evaluation in Zone 8. Taita-Naenae (Zone 2) and Stoke Valley (Zone 1) are underlain by fan sediments with the maximum depth of 20m in Zone 2. Regarding the soil type and the soft sediment depth, soil type C is the predominant soil type in Zone 1 and 2. Regarding the geological information even distribution of soil type B and C are assumed in Western Hill (Zone 3) in the east of Hutt valley.

### 4.1.2. Hutt City wastewater network

On behalf of the Hutt City Council, Capacity Company is in charge of running and maintaining the Hutt City wastewater network. According to the Capacity Company’s 2007 report, the network consists of two components: a wastewater network and a trunk wastewater network. This network has 672 kilometres of wastewater pipelines, 84.5 % of which is of the local wastewater network and the remaining 15.5% is a part of the trunk wastewater network (Capacity infrastructure services, 2007).

The wastewater collected from the Upper Hutt is added to the Hutt City wastewater collection system. The collected wastewater is transferred through the trunk lines to the treatment plant located at Sea View (Capacity Company & Hutt City Council, 2008). The treated effluent in the Hutt City WWTP is transferred through 18 km of the 1350mm pressurised pipeline to the discharge point at Pencarrow at the eastern entrance of Wellington harbour (Capacity infrastructure services, 2007).

### 4.1.3. Hutt City Local wastewater network

The local wastewater network with 568 km of wastewater pipelines services 39,336 properties in Hutt City. The collected wastewater is transferred by 22 pump stations. Network inspection and network maintenance are accomplished by using 13,863 manholes or other nodes on the entire the network (Capacity infrastructure services, 2007). The replacement value of the Hutt City local wastewater is NZ $ 185,510,748.00 estimated on 30 June, 2006

(Capacity infrastructure services, 2007). Figure 4.4 shows the value of each portion of the Hutt City wastewater system. Furthermore, the wastewater pipelines are the most valuable asset in the Hutt City wastewater system.

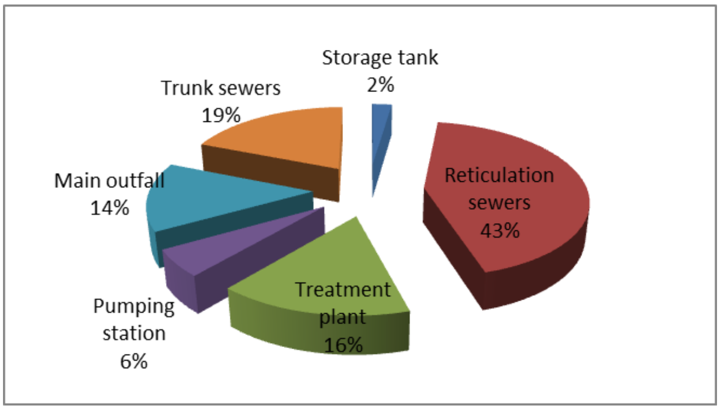


Figure 4.4: Asset value of the Hutt City wastewater system (Capacity infrastructure services, 2007)

## 4.2. Hutt City wastewater pipeline

Gravity is the main way of transferring wastewater within the Hutt City wastewater network. Consequently, 92% of the network is unpressurised, and the remainder is pressurised (Hutt City Council, 2008b).

The Capacity Company and Hutt City Council have divided Hutt City into the following 8 zones: Stokes Valley (Zone 1), Taita and Naenae (Zone 2), Western Hills (Zone 3), Petone (Zone 4), Hutt Central (Zone 5), Seaview (Zone 6), Eastbourne (Zone 7) and Wainuiomata (Zone 8) (Capacity infrastructure services, 2007). In this case study analysis to investigate the restriction on the use of available fragility curves was conducted. As pointed out in Chapter 2, pipe type, length and diameter are the three main factors which affect seismic vulnerability of buried pipelines. Wastewater pipelines in each zone are hence classified in terms of their type, length and diameter. The classification involves the Hutt City's 568 km of wastewater pipelines (Figure 4.5).

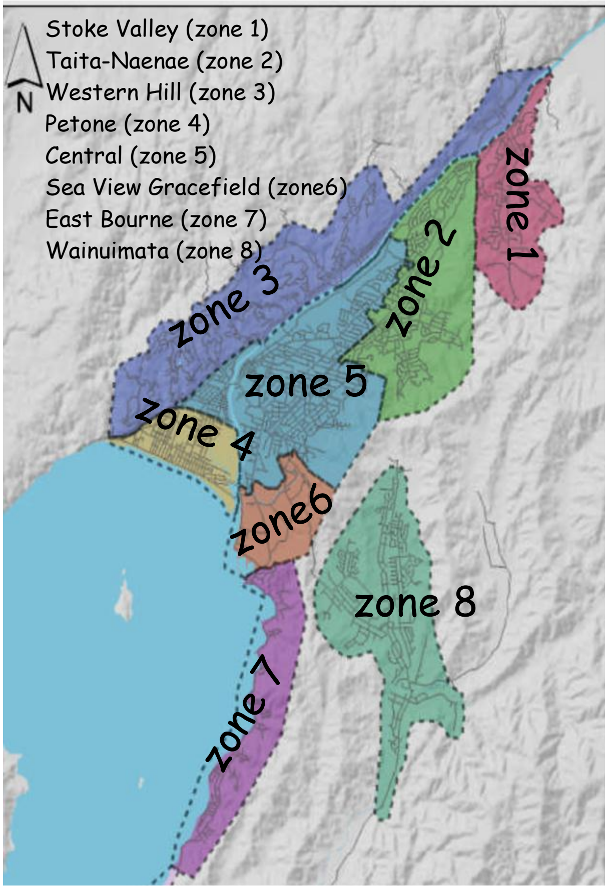


Figure 4.5: Hutt City wastewater network zones (source: Capacity Company and Hutt City Council 2007)

Pipe material is one of the indicators of the vulnerability against seismic shocks and shows the pipe resistance and behaviour (Bizier 2007). The Hutt City sewer network has had different types of pipes installed during the past century. Concerning their characteristics, the wastewater pipelines in Hutt City can be grouped into several categories: plastic base pipes and non-plastic pipes. Plastic pipes generally comprise of PVC and PE pipes. The PVC can have the Modified Polyvinyl Chloride (MPVC) and Un-plasticised Polyvinyl Chloride pipes (UPVC) as subclasses. The subclasses of poly ethylene pipes are Medium Density Poly Ethylene pipes (MDPE) and High Density Poly Ethylene pipes (HDPE).

Pipes without plastic base material can also be divided into two types; the mortar based pipes and metal based pipes. The mortar base pipes can be subdivided into cement based pipes and clay based pipes. The AC, Fibrolite pipes (FIB), concrete pipes and RC are cement based pipes, while Earthenware and Ceramic pipes (EW and CE) are clay based pipes. CI pipes, ST pipes and Concrete Lined Steel pipes (CLS) are different types of metal based pipes (Hutt-City-Council 2008). Figure 4.6 shows the distribution of wastewater pipes in each municipal zone of Hutt City

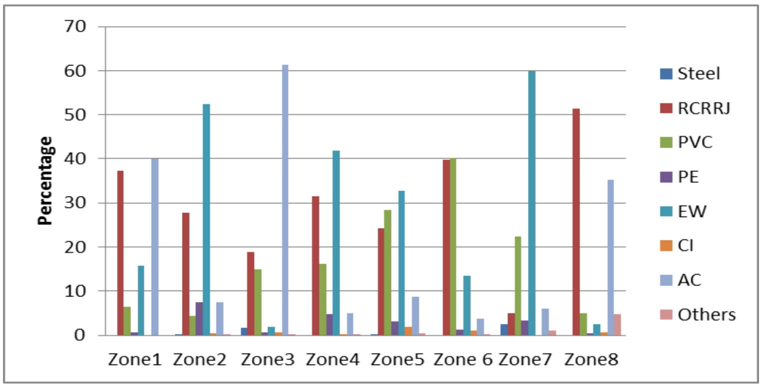


Figure 4.6: Distribution of pipes in the Hutt City wastewater network (source: (Capacity infrastructure services, 2007)

The GIS database in Hutt City established by the Hutt City Council was used to categorise all the wastewater pipelines in Hutt City including the regional and trunk wastewater networks. Reinforced concrete pipes, asbestos cement pipes, earthenware pipes and polyvinyl chloride pipes are the main types of pipes used in the network. (Hutt City Council, 2008a). This study analysed, tabulated and classified the wastewater network in Hutt City by applying the Hutt City GIS data base. Table 4.2 shows the length and percentage of each predominant pipe type used in the network. The analysis shows the concrete pipes are the most popular types of pipes in this network, with almost one third coverage of the total length. After concrete pipes AC, CE and PVC pipes are the predominant pipe types and cover 21.6 %, 16.6 % and 13.1% of the total length, respectively. It should be noted that there is no accurate data on 12% of the Hutt City wastewater network in term of pipe types in the GIS database.

Various types of pipes were used to develop or rehabilitate the Hutt City wastewater network during its development. The analysis of pipe materials in the network in Figure 4.7 shows that besides the four main types of pipes, there are at least 13 other types of pipes in this network. In spite of the variety of different types of pipes, only about 6.8% of the network is covered by other pipe types. The less used types of pipes are compared to each other in Figure 4.7.

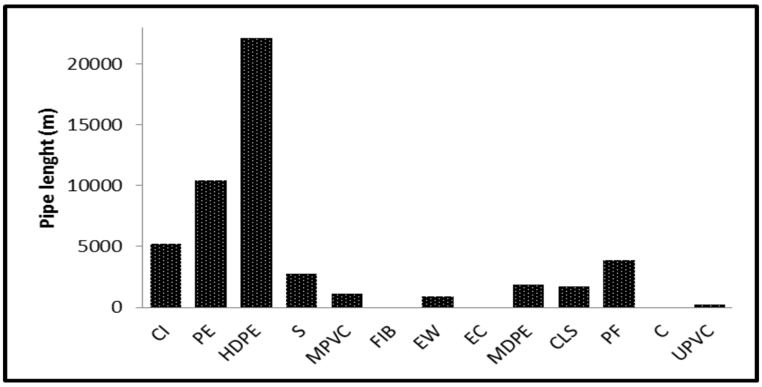


Figure 4.7: Hutt City wastewater pipes classified by material types (non-major types)

If all poly ethylene based pipes are considered as PE pipes in the analysis, then the PE pipes stand as the 6th most common pipe type used in the Hutt City wastewater network. The percentage of each type of wastewater pipe after analysis is shown in Table 4.2 after analysis.

Table 4.2: Most common types of wastewater pipes in Hutt City

| Material | RC | AC | CE | PVC | NM | PE | CI | Steel | Miscellaneous |
|---|---|---|---|---|---|---|---|---|---|
| Length | **197613** | **140703** | **108561** | **85170** | **75951** | **34344** | **5205** | **4479** | **112** |
| % of Total | **30.3** | **21.6** | **16.6** | **13.1** | **11.6** | **5.3** | **0.8** | **0.7** | **0.02** |

As established in Chapter 2, pipe type is the main parameter which affects pipe vulnerability against earthquake. Wastewater buried pipes can be classified into brittle and ductile pipes and this classification not only provides a general evaluation for pipe types in each network but can also be used in direct estimating of the expected damage in some fragility curves such as FEMA (FEMA, 2003). As discussed in Chapter 2, the available fragility curves are usually developed for particular types of buried pipes. In the US, for instance, some fragility curves are established only for CI pipes.

In Hutt City, 81 % of the wastewater pipelines are brittle pipes. It also should be added that in each category there are some differences amongst each type of pipe in term of flexibility. For instance, UPVC pipes compared to PVC pipes are less flexible and can tolerate seismic force less than PVC pipes can.

### 4.2.1. Hutt City wastewater pipelines classified by pipe diameter

Reviewing the literature showed that pipe diameter can also affect seismic vulnerability just as the material does. However, pipe diameter can have its effect on the buried depth and type of repair. The analysis in this study indicates that 150mm pipes are the predominant type of pipes in the Hutt City wastewater network. The pipes with 150mm diameter cover 69.3% of the whole network. In terms of the pipe diameter, the most common diameters, beside the pipes with 150mm diameter, are the pipes with 225 mm, 300 mm, 1350mm, 100mm, and 375 mm diameters which cover 9%, 5%, 2.9%, 2.4 % and 2.3 % of the wastewater network, respectively. Table 4.3 shows the percentage of various types of wastewater pipes in the Hutt City wastewater network in terms of the pipe diameter.

Table 4.3: Main pipe diameter in Hutt City Sewer pipeline

| Diameter (mm) | 150 | 225 | 300 | 1350 | 100 | 375 |
|---|---|---|---|---|---|---|
| Length (m) | **451875** | **58779** | **32441** | **18940** | **15515** | **14865** |
| % of Total | **69.3** | **9** | **5** | **2.9** | **2.4** | **2.3** |

Twenty three types of pipes in terms of pipe diameter are used in the Hutt City wastewater network. In this network, only 5% of the wastewater pipes have the diameters more than 750mm. Table 4.4 shows the result of pipe diameter analysis in the Hutt City wastewater network. As this table shows, 86.5% of the network has diameters less than 300mm.

Table 4.4: Distribution of sewers in Hutt City in term of diameters

| Diameter (mm) | $\leq 150$ | $150 < D \leq 300$ | $300 < D \leq 750$ | $750 < D \leq 975$ | $975 < D \leq 1800$ |
|---|---|---|---|---|---|
| Length (m) | **467716** | **96473** | **55365** | **10654** | **21930** |
| % OF Total length | **71.7** | **14.8** | **8.5** | **1.6** | **3.4** |

The distribution of the Hutt City wastewater network in terms of pipe diameter, except 150mm pipes, is shown in Figure 4.8. Here the pipes with the diameter of 345, 110mm and pipes with diameters less than 100 mm are the least popular type.

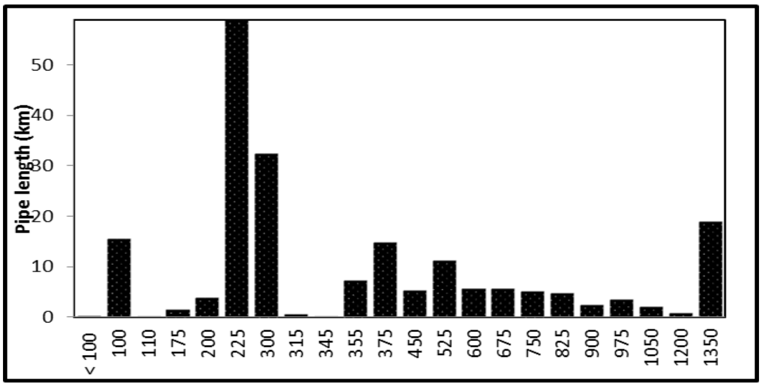


Figure 4.8: Hutt City wastewater pipes distributed by pipe diameters mm (except 150mm)

### 4.2.2. Hutt City 150mm wastewater pipes

The 150mm diameter wastewater pipes are the most common and cover 69.3 % of the total length of the Hutt City wastewater network. If the 150mm diameter pipes are analysed and classified by pipe types, the asbestos cement pipes comprise of 26.5 % of the total length. Reinforced concrete pipes, ceramic pipes and polyvinyl chloride pipes are the three main types of 150mm pipes which make up 22.63, 20.9 and 15.7 % of the total length, respectively. Table 4.5 is extracted by analysing the Hutt City wastewater network and shows the distribution of 150mm wastewater pipes in the network in terms of pipe materials.

Table 4.5: Pipe material in Hutt city sewer with 150 mm diameter

| Pipe material | AC | RC | CE | PVC |
|---|---|---|---|---|
| Length (m) | **119960** | **102273** | **94598** | **71020** |
| % of 150 mm pipes | **26.5** | **22.63** | **20.9** | **15.7** |
| % of total Length | **18.4** | **15.7** | **14.5** | **10.9** |

Figure 4.9 shows the various 150 mm diameter pipes applied in the Hutt City wastewater network with the four major types.

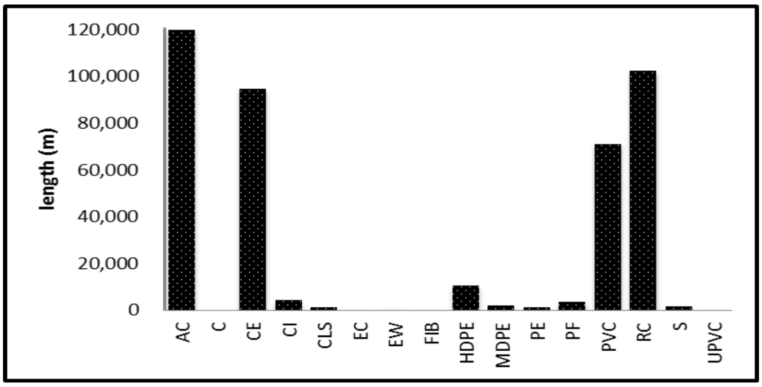


Figure 4.9: 150 mm diameter pipe types installed in the Hutt City wastewater network

If the 150mm diameter wastewater pipes in Hutt City are classified as brittle and ductile, 78 % of the 150mm pipes are brittle and 22 % are ductile pipes.

### 4.2.3. Seismic effect on the Hutt City wastewater pipelines

Seismic characteristics of each earthquake prone city are one of the main factors that affect seismic vulnerability of buried pipelines. Consequently, before seismic vulnerability analysis of the Hutt City wastewater pipelines, seismic vulnerability of Hutt City should be investigated in relation to active faults. In this part, the earthquake vulnerability of Hutt City and the required data used in calculating the earthquake induced damage on wastewater pipelines will be discussed. After the total expected damage caused by earthquakes on the wastewater network is calculated, the wave propagation effects of earthquakes on the Hutt City wastewater pipelines will be assessed. Toprak’s (1998) equation will be used to calculate the total expected damage in the Hutt City wastewater network. This equation is based on one of the most comprehensive and accurate data source. Toprak (1998) correlated buried pipe damage caused by the 1994 Northridge earthquake to the earthquake parameters (Toprak, 1998). Application of the equation provides the holistic expected damage on the wastewater pipeline reticulation that covers both wave propagation and ground displacement effects.

As pointed out in Chapter 2, wave propagation not only can cause damage on buried pipes, but can also affect a much larger area than the permanent ground deformation effect could. Accordingly, the wave propagation effects of earthquakes on the Hutt City wastewater network will be assessed as one of the main causes of damage in buried pipelines.

#### 4.2.3.1. General Earthquake effects on the Hutt City wastewater network

Researchers such as Katayama et al. (1975), Eguchi et al. (1983), O'Rourke and Ayala (1993) and Isoyama et al. (2000) try to correlate earthquake induced damage on buried pipelines with earthquake parameters. They have recommended some equations for estimating the expected damage on buried pipes. These equations correlate the damage rates (number of damage by unit length of pipe) to the earthquake parameters.

Earthquake effects on buried pipelines can be evaluated in two different manners: holistic and individual approaches. In the holistic approaches, usually an earthquake parameter is used to calculate both the wave propagation and permanent ground deformations' effects on buried pipelines. In individual approaches, each damaging factor is calculated separately, and wave propagation effects are always considered as the main cause of earthquake induced damage on an affected zone. For evaluating seismic vulnerability of wastewater pipelines in each case study in this study, first a holistic approach will be applied, and then wave propagation effects of specific earthquakes will be assessed.

The 475 and 1000 year return periods are two common types of periods used for seismic risk analysis and the 475-year return period is the most common standards used for seismic risk analysis (Gould, 2003). As a result, earthquakes with two return periods (475 and 1000 year) are selected to show the seismic vulnerability of the Hutt City wastewater network.

PGA should be estimated as an earthquake parameter which is used in the holistic approach fragility curve. To estimate earthquake damage in each case study, the PGAs are extracted from the available PGA maps for earthquakes with the return periods of 475 and 1000 years, as recommended by Stirling et al. (2002). The estimated PGAs should be modified with respect to each municipal zone soil type. Consequently, FEMA (2003) recommendations are used in modifying the expected PGA in each soil type.

Figure 4.10 shows the expected damage on the Hutt City wastewater network that might be caused by the 475 year return and 1000 year return period earthquakes. This figure shows the portion of the network located in the Hutt City central (Zone 5) is the most vulnerable. Zone 5 not only accommodates many residential buildings but also covers the main business centre of Hutt City. Consequently, wastewater failure in this zone can significantly affect the life of the city as a whole. As shown in Figure 4.10, the earthquakes with return period of 1000 years can cause more damage compared to that of the one with 475 years return period.

However, there is no significant difference between amounts of damage in an earthquake with the return period of 475 and 1000 years.

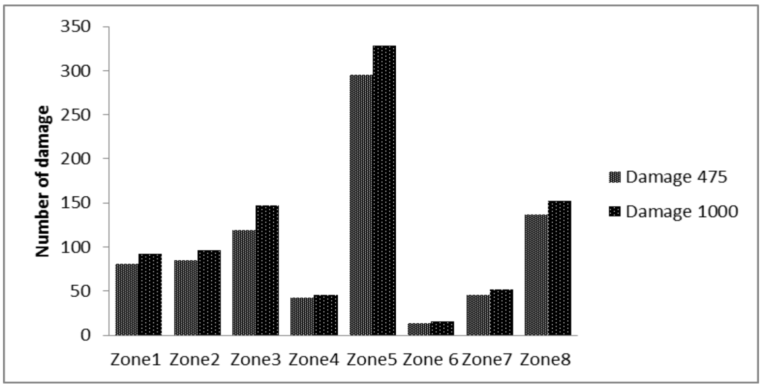


Figure 4.10: Earthquake effects on the Hutt City wastewater network (holistic approach)

## 4.3. Wave propagation damages in the Hutt City wastewater network

As explained in Chapter 2 various empirical formulae are available for calculating wave propagation effects of an earthquake on buried pipelines. According to the data available fragility curves are employed to calculate seismic effects on wastewater pipelines. Selection of fragility curves correlates with the data available on the pipeline of each zone and the available geological and geotechnical characteristics of the same (Jeon & O'Rourke, 2005).

Reviewing the literature shows that fragility curves are adequate tools for estimating the volume of expected defects on buried pipelines, and finding seismic vulnerability of buried pipelines in any particular zone as to locate the most vulnerable places within each city. Proper estimation of the amount of defects in each type of wastewater pipeline can help the utility managers to cope with the network restoration and do repairs quickly and efficiently after the damaging earthquakes. Some well-known and recently established fragility curves e.g. Eidinger and Avila (1999), O'Rourke and Jeon (1999), ALA (2004), FEMA (2003), O'Rourke and Deyoe (2004), Jeon and O'Rourke (2005) are applied in this thesis to calculate ground shaking effects of the earthquake in each case study. The applied fragility curves are recent and are based on more accurate data compared to those ones developed by Katayama et al. (1975), Eguchi et al. (1983) and Chen et al. (2002). The seismic vulnerability analysis of the Hutt City wastewater network is evaluated by 9 fragility curves, and 8 fragility curves are used in relation to the Gisborne, Blenheim and Christchurch wastewater pipelines. Table 4.6 shows the characteristics of the empirical formulae used in calculating the earthquake vulnerability in the Hutt City, Gisborne and Blenheim wastewater networks (Eguchi, et al., 1983).

Table 4.6: Applied formulae used in calculating the earthquake damage in Hutt City

| **Formula** | **Eguchi** | **FEMA (HAZUS)** | **Eidinger and Avila** | **O'Rourke and Deyoe (R waves)** | **Jeon and O'Rourke** | **Toprak** | **O'Rourke and Jeon** | **ALA** | **O'Rourke and Deyoe (S waves)** |
|---|---|---|---|---|---|---|---|---|---|
| **Date of publish** | 1991 | 2003 | 1999 | 2004 | 2005 | 1998 | 1999 | 2004 | 2004 |
| **pipe type restriction** | 10 types | Brittle Ductile | 6 pipe types | Brittle MIX | 4 pipe types | CI | CI&DI | 5 types | Brittle MIX |
| **pipe Diameter restriction** | NA | MIX | large and small | MIX | NA | Dp≤600 mm | Dp≤600 mm | NA | MIX |
| **joint type restriction** | NA | NA | 6 pipe types | NA | NA | NA | NA | 6 types | NA |

The damage calculated by each formula will be compared to other fragility curves' defects estimations for each zone. Updated the model of Eguchi et al. (1983), represented by O'Rourke and Ayala (1993) is the only formula which is applied MMI in order to calculate wave propagation effects of earthquakes on the Hutt City wastewater network. The O'Rourke and Ayalas' model (1993) is only applied in Hutt City to calculate seismic damage, because of specific available MMI in some areas in the Hutt City. PGV is used in the rest of the formulae to calculate the amount of damages caused by wave propagation effects on buried pipelines.

The following section will explain how PGV can be estimated by available data.

### 4.3.1. Calculation of Peak Ground Velocity

As explained in Chapter 2 the required earthquake parameters used in the applied fragility curves in this thesis can be extracted from some recorded sources. For instance, Table 4.7 can be applied to estimate spectral acceleration for soil type B in each case study. Correction factors then should be applied to correlate PGV to soil type B with other soil types.

Table 4.7: PGA and ST (t=1 sec) in class B soil (Stirling et al. 2002)

| | **PGA (g)** | | | **SA (t=1 sec)** | |
|---|---|---|---|---|---|
| **Return period (years)** | 150 | 475 | 1000 | 475 | 1000 |
| **Hutt City** | 0.3-0.4 | 0.5-0.6 | 0.6-0.7 | 0.4-0.5 | 0.5-0.6 |

According to FEMA (2003), coefficients greater than 1 are applied in modifying PGV from soil type B to soil types C, D and E and coefficient less than 1 is only used to convert the extracted PGV from soil type B to soil type A. Further, $F_A$ and $F_V$ are the two correction factors which can be applied to convert PGA and PGV from soil type B to other soil types, respectively.

The correlation between geological characteristics of each zone and soil types can be extracted by Appendix 3 for Hutt City, Gisborne, Blenheim and Christchurch (Dellow, 2009a, 2009b).

### 4.3.2. Calculating the wave propagation effects of earthquakes in the Hutt City wastewater network

Amongst the four case studies in this study in New Zealand, Hutt City is expected to have experienced the strongest earthquakes because of its geological characteristics, that is its closeness to stronger active faults compared to Gisborne, Blenheim and Christchurch cities (Stirling, et al., 2002).

In this part of the thesis, first the earthquake effects on the Hutt City wastewater network will be estimated. O’Rourke and Ayala (1993) and then 8 other fragility curves will be applied to show ground shaking effects of earthquakes in the Hutt City wastewater pipes.

MMI intensity is used in the fragility curve recommended by Eguchi et al. (1983) to calculate ground shaking damage instead of the PGV which is used in the other 8 fragility curves. Thus, each zone is divided into hazard zones as defined by Van Dissen (1992), who recommends that the expected damage in each municipal zone of Hutt City should be calculated for Scenario one and two. Moderate to large, shallow and distant earthquakes which cause shaking on bedrock with MMI of V-VI is classified as scenario one whereas scenario two is for large, local, Wellington fault earthquakes.

All types of the wastewater pipes in Hutt City are classified into 6 main groups which are used in Eguchi et al.'s fragility curve (1983), i.e. ST, AC and EW, PVC, PE and CI. Damage rates are derived from their graphs for each main pipe type. The volume of defects for each zone is calculated by multiplying the damage rate by the length of each pipe type.

Figure 4.11 shows the Wainuimata wastewater network is the most vulnerable zone in Hutt City in case of a moderate earthquake or scenario one. Almost all 107 defects in Zone 8 are expected to occur in the two main types of wastewater pipelines, Concrete Pipes and Asbestos Cement pipes which are shown in AC and EW group. The Wainuimata wastewater network, even in the event of a 5 or 6 MMI earthquake, is susceptible to a high damage rate and an adequate inspection and repair plan should be considered for this zone even after a moderate earthquake. After a moderate earthquake, even if no immediate visible damage is detected, health and environmental issues can be predominant because of wastewater infiltration. It should be taken into account that wastewater pipelines in this zone are in a poor condition, and the number of repairs in this zone may exceed expectations.

The Petone wastewater network can suffer significantly in scenario one, although the volume of defects is less than half of what is expected in Wainuimata. The number of expected defects in a moderate earthquake in Petone is 52, followed by Hutt Central and Tiata-Naenae, which may have 43 and 38 expected defects, respectively. Other zones, Zone 6, 7, 1 and 3, in a moderate earthquake, are predicted to behave well. For instance, the maximum number of expected defects in Zone 3 is only 5 defects calculated for this study. The wastewater network in zone 6 behaves well.

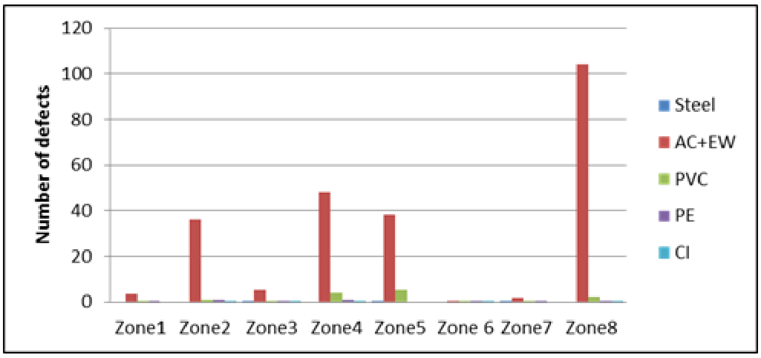


Figure 4.11: Wave propagation effects on Hutt City wastewater network (Scenario 1)

The Hutt City wastewater pipelines would suffer overwhelming damage from an earthquake in Scenario 2, with the volume of expected defects being more than 200 in each of the four zones (2, 3, 5 and 8).

Figure 4.12 shows that Hutt City central region is the most vulnerable zone in large scale earthquakes. The number of wave propagation defects is 365 in the Hutt City central region, and is followed by Zone 8 with 379, Zone 4 with 175 and Zone 3 with 205 defects in a regressive manner. The expected number of defects in Zone 1, 2, 4, 5 and 8 due to some particular conditions such as poor pipe quality and poor construction are anticipated to exceed the estimated number of defects.

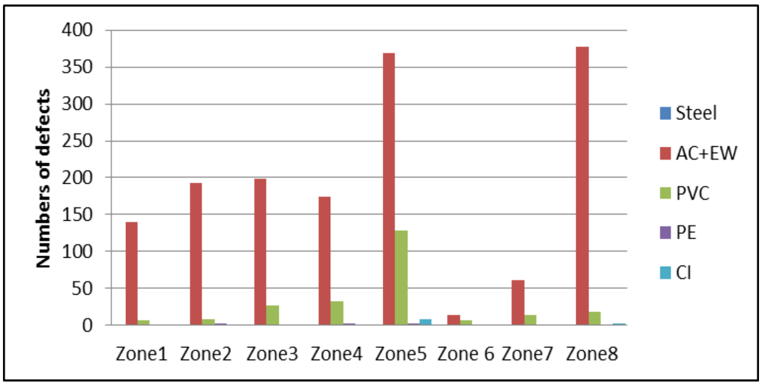


Figure 4.12: Wave propagation effect on Hutt City wastewater network (Scenario 2)

### 4.3.3. Calculate earthquake effects using the PGV based fragility curves

The wastewater network in each case study comprises of at least 7 types of pipes. For instance, the Hutt City wastewater network comprises 16 types of pipes. The wastewater pipes in each wastewater network thus should be reclassified according to the pipe material from which each fragility curve is extracted. In order to apply FEMA fragility curves (2003), wastewater pipes should be classified into brittle and ductile pipes.

PGV based fragility curves stand for empirical equations which are developed by correlation of PGV as an earthquake characteristic to number of damage per unit length of buried pipeline (O'Rourke & Ayala, 1993). Fragility curves are usually established for particular type of pipes (Toprak & Taskin, 2007).

In the Hutt City wastewater network, 8 formulae are applied to show how various fragility curves affect the number of expected failures. In relation to the distribution of wastewater pipes in each zone, as well as a particular classification of pipe types in each fragility curve, the number of failures is calculated. In this part, first the seismic effects of earthquakes with annual probability of exceedance 1/1000 on this network will be calculated, and then seismic effects of earthquakes with annual probability of exceedance 1/500 will be calculated.

Earthquake hazard risk can be defined by return period, which refers to a time frame or annual probability of exceedance (Saunders, 2011). For instance, earthquakes with annual probability of exceedance 1/500 are equivalent to earthquake with return period of 500 years. On the other hand, Earthquakes with return period of 475 years have a probability of 0.99 rather than 0.95 for earthquakes with return period of 500 years in New Zealand (Saunders, 2011). In this study earthquakes with annual probability of exceedance 1/1500 are the same as earthquakes with return period of 475 years

Figure 4.13 shows distribution of the expected failures in each urban area in Hutt City, each of which may occur in earthquakes with annual probability of exceedance 1/1000. This figure illustrates Zone 5 and Zone 8, which may expect more damage in earthquakes compared to the other zones. Zone 6 may suffer the minimum damage in the whole network.

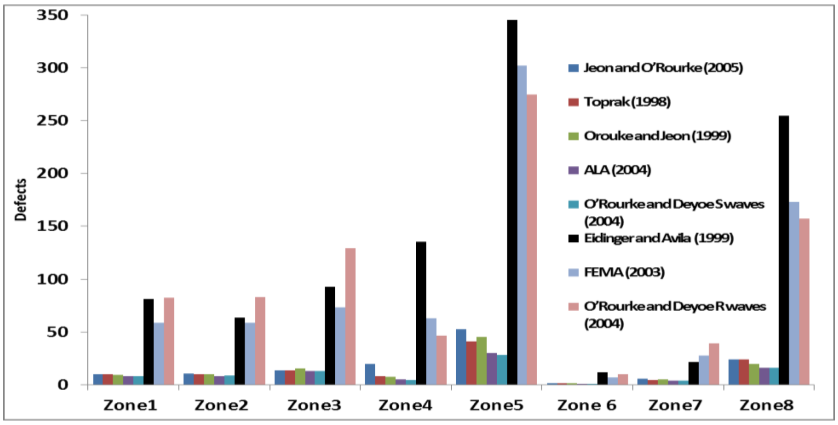


Figure 4.13: Expected failures in the Hutt City wastewater network (annual probability of exceedance 1/1000)

As Figure 4.13 shows, there are significant differences between the expected damage in each zone. Here some fragility curves provide number of defects much more than the defects calculated by others fragility curves. As a result, fragility curves can be classified into Groups 1 and 2 and fragility curves which produce almost similar damage rate are classified in the same group. Group 1 includes damage rates calculated by the fragility curves of FEMA (2003), O'Rourke and Deyoe (2004) (R wave) and Eidinger and Avila (1999), while Group 2 comprises of damage rates calculated by Jeon and O'Rourke (2005), O'Rourke and Jeon

(1999), ALA (2004), O'Rourke and Deyoe (2004) (S wave) and Topraks' (1998) fragility curves.

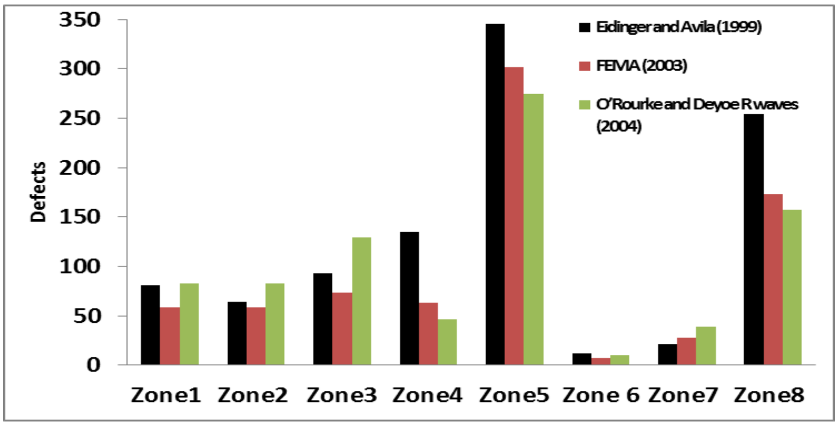


Figure 4.14: Expected failures in the Hutt City wastewater network Group 1 equations (annual probability of exceedance 1/1000)

Figure 4.14 shows that the Eidinger and Avilas' equation (1999) provide the maximum estimated defects for 4 zones out of the 8 zones in Hutt City. The fragility curve developed by O'Rourke and Deyoe R wave (2004) estimates the highest estimated defects in 3 zones and the same estimation as Eidinger and Avilas' equation (1999) for only one zone. On the other hand, the R wave equation of O'Rourke and Deyoe (2004) estimates the minimum number of defects in zones that the maximum expected defects in which are calculated by the Eidinger and Avilas' equation (1999).

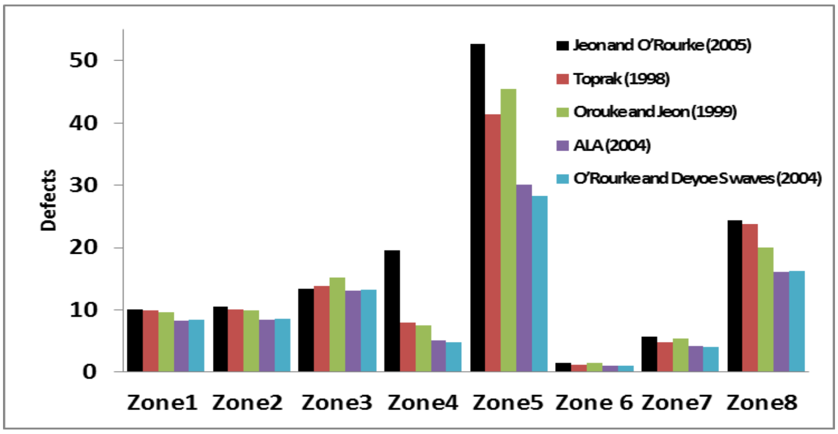


Figure 4.15: Expected failures in the Hutt City wastewater network Group 2 equations (annual probability of exceedance 1/1000)

Comparison of calculated defects by the fragility curves in Group 2 shown in Figure 4.15 indicates a similarity among the zones except zones 5, 8 and 4; zones 5 and 8 with the maximum length of installed pipes. This figure shows the maximum number of defects in 5 zones is calculated by the fragility curve of Jeon and O'Rourke (2005). However, in 3 zones the maximum number of estimated defects are almost the same as calculated defects by the other fragility curves in Group 2, see for example zones 1, 2 and 6.

The annual probability of exceedance 1/500 on the Hutt City wastewater network and the distribution of the expected damage in different zones can be seen in Figure 4.16.

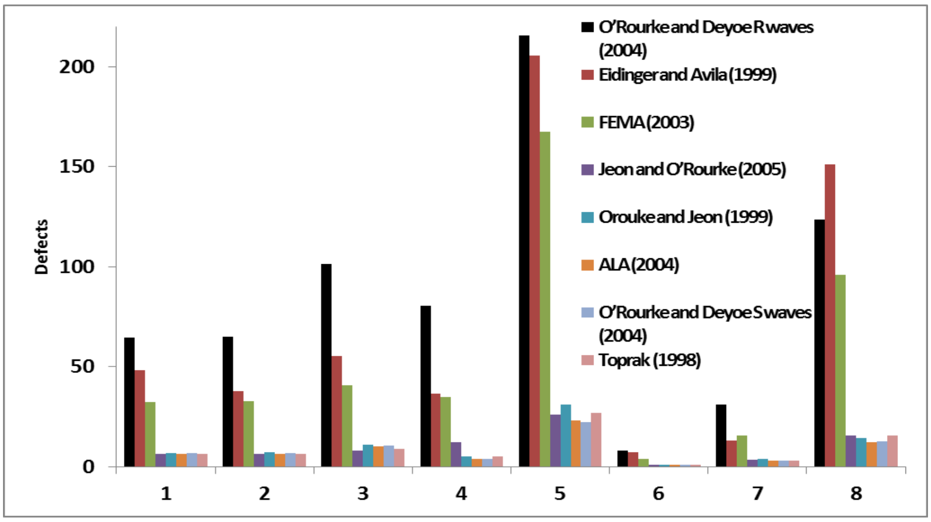


Figure 4.16: Distribution of damages in the Hutt City WW reticulation caused by earthquakes with annual probability of exceedance 1/500

## 4.4. Comparison and outcome

Wastewater pipelines in the Hutt City can be classified into two main categories; brittle and ductile pipes. Earthquake effects on each type of pipe here can be calculated and compared to each other in order to find differences within the fragility curves results and also investigate the parameters which may affect the estimated results.

### 4.4.1. Comparison of results in brittle pipes calculated by Group 1 equations

If the number of defects in brittle pipes of the Hutt City wastewater network with an annual probability of exceedance 1/500 are compared to each other the following results can be seen.

There are maximum number of defects in zones 5 and 8 which are calculated by the O'Rourke & Deyoe's (R-wave) (2004) and Eidinger & Avila (1999) equations. In 6 out of 8 zones, the O'Rourke and Deyoes' (R-wave) equation (2004) provides the maximum number of defects compared to the Eidinger and Avilas' (1999) fragility curve that only provide the maximum defects for two zones ( zones 4 and 8).

If the differences among estimated defects by Group 1 equations are compared, the maximum difference can be seen in zone 7 where the length of brittle pipes is about 23 km. In zone 7 the length of brittle pipes is about 5 % of the total brittle pipe length in all zones and earthenware pipes cover about 83% of total length of brittle pipes in this zone. As a result, by increasing the brittle pipes length the differences within the calculated defects would decrease.

There are minimum number of defects in zones 6 and 7 which are calculated by the fragility curves of FEMA (2003) and Eidinger & Avila (1999) respectively. The FEMA's equation (2003) in almost all zones provides the minimum number of defects except zone 7. The difference in zone 7 can be the result of small lengths of brittle pipes and also a combination of different type of pipes. On the other hand, in spite of small length of brittle pipes in zone 6 (even much less than zone 7) the estimated defects still follow the general trend, however the difference between the maximum and minimum number of defects in this zone is greater than zone 8 and almost same as zones 1 and 2. The length of brittle pipes in zones 1, 2 and 8 are more than zone 6.

There are differences between the maximum and minimum number of calculated defects in each zone. The greatest difference belongs to zones 3 and 4, because of the length and combination of brittle pipe types. For instance, asbestos cement pipes have the highest percentage in Zone 3 compared to with other zones and the percentage of earthenware and concrete pipes in Zone 4 is notable compared to other zones.

The difference within the highest and lowest estimated defects seems to be the same in zones with the same length of pipes; however, there is an exception again. The differences within the highest and lowest estimated defects are the same in zones 1 and 2; both have the same length but different types of brittle pipes. On the other hand, the difference between the maximum and minimum estimated defects in zone 6 and zone 1 or zone 2 is the same but the lengths of brittle pipes are significantly different.

The minimum difference between the maximum and minimum estimated defects belongs to zone 5 which covers the maximum length of brittle pipes. Zone 5 covers approximately 27% of the total length of brittle pipes in the Hutt City wastewater network.

The above mentioned comparisons show the differences within the highest and lowest estimated number of defects calculated by Group 1 equations and show a discrepancy when the total length of brittle pipes in each zone changes. In general the differences between the maximum and minimum calculated defects usually decrease when the length of brittle pipes increase in each zone.

### 4.4.2. Comparison of results in brittle pipes calculated by Group 2 equations

If the comparisons of defects calculated by Group 2 equations are taken into account the following results can be seen in the brittle pipes of the Hutt City wastewater network; the S wave equation of O’Rourke and Deyoe (2004) provides the maximum number of defects in 5 out of 8 zones except zones 4, 5 and 8. On the other hand, the Toprak’s fragility curve (1998) provides the maximum number of defects in zone 5 and 8. The Jeon and O’Rourkes’ equation (2005) gives the maximum in zone 4.

The fragility curves of Jeon and O’Rourke (2005) and ALA (2004) estimate the minimum number of defects in brittle pipes of the Hutt City wastewater network. Jeon and O’Rourkes’ fragility curve (2005) estimates the minimum number of defects in zones 1, 2, 3, 6 and 7 and ALA’s equation (2004) gives the minimum estimation in zone 4, 5 and 8.

It should be mentioned that the difference within calculated defects by Group 2 fragility curves in zones with small length of brittle pipes are similar to each other even when the length of brittle pipes are less than 50 km. For instance, in Zone 1 and 2 the length of brittle pipes is about 53 km almost all fragility curves provide the same number of defects. The maximum difference within the estimated number of defects by Group 2 fragility curves belongs to Zone 5 where the length of brittle pipes is about 124km. This shows that if length of brittle pipes increases the differences within the maximum and minimum expected defects also increases when defects are calculated by the Group 2 fragility curves.

If averages of the estimated defects calculated by each group of fragility curves are compared to each other, Group 1 equations provide approximately 7.6 times the number of defects than Group 2 equations on the brittle pipes of the Hutt City wastewater network. The differences within the calculated defects by each fragility curve in two groups of fragility curves are relevant to two main parts. The first part is relevant to inherent differences in each equation compared to other equations, such as power of PGV in each fragility curve and also coefficients used in each fragility curves. For instance, the PGV power in FEMA’s equation (2003) is 2.25 and for O’Rourke and Deyoe (2004) is 0.92. The second part is type of pipes that each fragility curve is developed form or adjusted for. For instance, two main types of pipes; brittle and ductile pipes are used in the FEMA fragility curve (2003) and CI and DI pipes are applied in the O'Rourke and Deyoe (2004) equations, regardless of many types of pipes available in wastewater network.

### 4.4.3. Comparison of results in ductile pipes calculated by Group 1 equations

Comparison of seismic damage in each zone for ductile pipes of the Hutt City wastewater network shows the differences between the maximum and minimum expected damage. The differences in ductile pipes are greater than that of the brittle pipes using both group 1 and 2 fragility curves; however, the number of calculated defects in ductile pipes of each zone is less than the calculated defects by Group 1 and Group 2 fragility curves.

Comparison of results calculated by Group 1 fragility curves in ductile pipes shows the maximum difference between the maximum and minimum calculated damage belong to Zone 4. The difference between the maximum and minimum in Zone 4 is almost 3 times more than the minimum calculated defect. Length of ductile pipes in Zone 4 is about 10 km and cover 21% of the total length of wastewater pipes in this zone. Poly ethylene and polyvinyl chloride pipes are two main types of ductile pipes in this zone and PVC pipes cover 77% of the total length of ductile pipes in Zone 4.

The minimum difference between the maximum and minimum estimated damage by Group 1 equations belong to Zones 5 and 8.The percentages of ductile pipes in Zones 5 and 8 are different and vary form 5.4% in zone 8 to 32% in Zone 5. Zone 8 has the minimum length of ductile pipes. On the other hand, Zone 5 has the second highest length of ductile pipes after Zone 6 with about 42% of ductile pipes. The highest length of ductile pipes belongs to Zone 5. Due to the small length of ductile pipes in some zones such as zones 1 and 6, the difference between estimated defects is negligible.

The above comparison shows the difference between calculated defects by Group 1 fragility curves decreases when the length of ductile pipes increases. For instance, zone 5 has the minimum differences within the highest and lowest estimated defects calculated by Group 1 fragility curves. Furthermore, comparison of results in Group 2 equations shows the minimum difference between the maximum and minimum estimated defects is 1.7 times more than the minimum number of defects in zone 3.

### 4.4.4. Comparison of results in ductile pipes calculated by Group 2 equations

The maximum difference between the highest and lowest estimated defects in ductile pipes belongs to Zone 4 of the Hutt City wastewater network. The difference between the maximum and minimum calculated defects is about 27 times more than that of the Zone 4. In zone 4 the R-wave equation of O'Rourke and Deyoe (2004), the equations of Jeon and O'Rourke (2005) and FEMA (2003) show close number of defects (3 defects) while the equations suggested by ALA (2004), Toprak (1998) and O'Rourke and Deyoe S-wave (2004) show no defects.

Zone 6 has the maximum percentage of ductile pipes compared to other zones, while a minimum length, 4km ductile pipes. When defects are calculated by Group 2 equations no defects are observed.

Comparison of calculated defects in ductile pipes indicates that there are notable differences between the maximum and minimum calculated defects in both groups of fragility curves. Comparing calculated damage by equations in Group 1 and Group 2 reveals the fragility curves in Group 1 estimate on average approximately 4.4 times more defects than Group 2 equations.

Great differences in calculated defects revealed inaccuracy of results. Here many parameters, expect those mentioned in the applied fragility curves can affect the number of damage after an earthquake was ignored in the fragility curves. The differences can also show complicated interaction between soil and buried pipes. As mentioned in Chapter 2 and 3 the fragility curves were established by correlation of seismic damage to an earthquake parameter in one or more earthquake. As a result, characteristics of pipeline network in each case study in New Zealand can vary from the pipeline network that a fragility curve is developed from.

In Chapter 2 and 3 that fragility curves were established based on pressurised pipes are explained. They have different behaviour in earthquake compared to the unpressurised pipelines. As a result, the differences obtained here can show the differences between pipe specifications which fragility curves are developed from and pipes that fragility curves are applied to. The revealed notable differences make it hard for post-disaster recovery and rehabilitation in terms of estimating resources, time and cost, and the uncertainty in the process would make decision making difficult. These differences can also cause uncertainty to evaluate seismic vulnerability of wastewater network within city zones. The uncertainty to estimate future earthquake effects on pipes network of each zone can affect accuracy and efficiency of rehabilitation plan selected to decrease or eliminate seismic effect on a network of wastewater pipes. A rehabilitation plan that its seismic damage estimation is based on fragility curves can either overestimate or underestimate the expected damage. Inaccurate pre-earthquake rehabilitation plan and post-earthquake recovery plan can have an adverse effect on communities.

## 4.5. Comparison of the calculated defects with real observation

Comparison of the calculated defects with the recorded earthquake induced damages in Hutt City provide further information regarding advantages and disadvantages of fragility curves for calculating seismic-induced damage in unpressurised wastewater network.

As explained in Chapter 2, Hutt City is highly vulnerable to seismic shocks which can be caused by nearby actives faults. Assessment of seismic activities in Hutt City based on the Geonet data base (2012) shows that Hutt City has experienced several weak to moderate earthquakes, but the city has not experienced strong earthquakes which could cause damage in its’ wastewater network.

Reviewing the available sources of information shows that no report exists regarding seismic damage on buried pipelines networks including pressurised and unpressurised networks. Consequently, in this chapter the calculated damages are not compared to actual earthquake induced damage in Hutt City.

## 4.6. Summary

The Hutt City wastewater network with a particular focus on the likely damage of brittle and ductile pipes is studied in this chapter. The 8 fragility curves were analysed and applied to the network to examine what the likely limitations are.

The research shows there are great differences in calculated damages by Group 1 and 2 fragility curves. Furthermore, there are differences in calculated damages by each fragility curves within each group. Combination of different types of pipe in each zone and in addition to the length of each type of pipe can have an effect on the calculated defects.

The maximum and minimum number of expected defects in each zone caused by recurrence frequency of 475 and 1000 in the Hutt City wastewater network are shown in Table 4.9. This table shows the maximum number of defects in brittle pipes is expected in Zone 5. Hutt City Council in Zone 5 should expect 189 defects in earthquake with annual probability of exceedance 1/500. Minimum number of defects in fragile pipes of 8 zones is expected in Zone 6. As a result, Zone 6 should be in the last priority for post-earthquake restoration of wastewater network in Hutt City.

Ductile pipes in each zone are expected to suffer less damage compared to brittle pipes in an earthquake with annual probability of exceedance 1/500. For instance, the maximum number of defects in ductile pipes is expected in Zone 5 which compared to brittle pipe is about 18% of the maximum defects.

Hutt City wastewater network in earthquakes with annual probability of exceedance 1/1000 suffers more damage compared to earthquake with annual probability of exceedance 1/500. For instance, Zone 5 expects the maximum number of defects to be 289 which is about 1.5 times more than the maximum expected defects in this zone in earthquake with annual probability of exceedance 1/500.

Table 4.9: Maximum and minimum number of defects in each zone of the Hutt City wastewater network

| Hutt City zones | Brittle (1/500) | | Ductile (1/500) | | Brittle (1/1000) | | Ductile (1/1000) | |
|---|---|---|---|---|---|---|---|---|
| | Max | Min | Max | Min | Max | Min | Max | Min |
| Zone 1 | 63 | 6 | 1 | 0 | 81 | 8 | 2 | 0 |
| Zone 2 | 63 | 6 | 3 | 0 | 80 | 8 | 3 | 0 |
| Zone 3 | 95 | 7 | 6 | 1 | 121 | 12 | 6 | 1 |
| Zone 4 | 73 | 3 | 8 | 0 | 175 | 7 | 6 | 1 |
| Zone 5 | 189 | 19 | 34 | 3 | 289 | 24 | 57 | 3 |
| Zone 6 | 6 | 1 | 1 | 0 | 10 | 1 | 2 | 0 |
| Zone 7 | 28 | 2 | 3 | 0 | 35 | 3 | 5 | 0 |
| Zone 8 | 149 | 12 | 3 | 0 | 250 | 16 | 5 | 0 |
| Total | 666 | 56 | 72 | 4 | 1041 | 79 | 86 | 5 |

# Chapter 5. Gisborne case study

In this chapter, after a brief description of active faults and seismic vulnerability in Gisborne, the Gisborne wastewater system will be briefly described. The last damaging earthquake in Gisborne and its effect on the Gisborne wastewater system will also be investigated. This work mentioning that before the 2010 earthquake in Christchurch the Gisborne earthquake was the latest remarkable earthquake in New Zealand. Finally, the city will be divided into four zones and seismic vulnerability of the wastewater pipelines in each zone will be calculated and compared.

## 5.1. Active faults and seismic vulnerability in Gisborne,

Gisborne is located at the northeast of the central North Island in New Zealand, with an 8,355 square kilometres area. Most of the territory in the North Island belongs to Gisborne. Gisborne city accommodates 41,922 inhabitants (Gisborne District Council, 2009b).

According to GNS active faults data base (2009), there are three active faults in the Gisborne zone: Otoko-Tontagi, Repongarere and Arakihi. These faults are located next to each other in the northwest of the Gisborne region, (GNS 2009). Table 5.1 shows the types of faults in the Gisborne region.

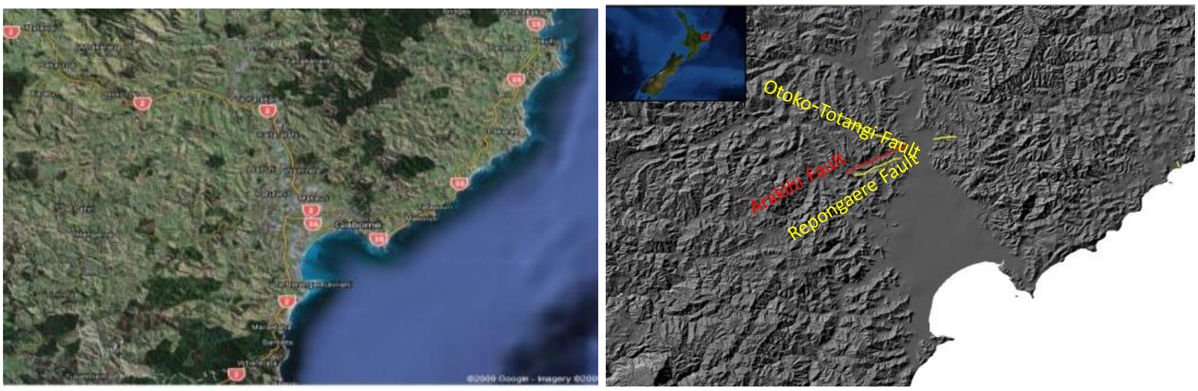
Table 5.1 : Faults in the Gisborne region (Source: (GNS science, 2009))

| Fault Name | Fault type | Recurrence interval | GNMS-ID |
|---|---|---|---|
| Otoko-Totangi | Dextral | -- | 23955 |
| Repongaere | Normal | -- | 23957 |
| Arakihi | Dextral | --- | 39844 |

Gisborne is located on the main seismic region affected by the subduct Pacific Plate and the Australian Plate. The 1993 and 2007 Gisborne earthquakes were the direct cause of the subducting Pacific Plate (François-Holden et al., 2008).

Berryman (2009) defines and explains the impact of the Repongaere fault on the Waipaoa Sedimentary System located in the north-west of Gisborne. According to Berryman's (2009) study, the Repongaere fault is a normal active fault with the dip-slip rate of 0.1 mm/year and with the maximum recurrence interval of 4490-6900 years (Berryman et al., 2009). The Repongaere fault earthquakes have a very low localised impact on Gisborne. It is located below the Waipaoa Sedimentary System (Berryman, et al., 2009).

Figure 5.1 (a) shows the geographical location of Gisborne in the east of the North Island (source (Google Maps 2009)). Figure 5.1 (b) shows the active faults located in the Gisborne region, and all the fault lines laid in the northwest and out of Gisborne city. This figure shows none of the three faults in the Gisborne region passes through the city.



a b

Figure 5.1: Gisborne geographical location (Source: (Google map 2009)) image (a) and the active faults in the Gisborne region (Source: (GNS active faults data base, 2010)) image (b)

### 5.1.1. Gisborne Geology

According to Bevin's study (2000), Gisborne city lies on young, soft and consolidated sediments where the ground shake earthquake intensity varies (Bevin, 2000). Gisborne is located on Holocene sediment which is created by flooding and sea level fluctuation. He noted that the 1993 Ormond earthquake is a good example to illustrate the vulnerability of Gisborne to earthquake. During the Ormond earthquake, moderate to strong shaking was felt over more than 1000km$^2$ (Bevin, 2000).

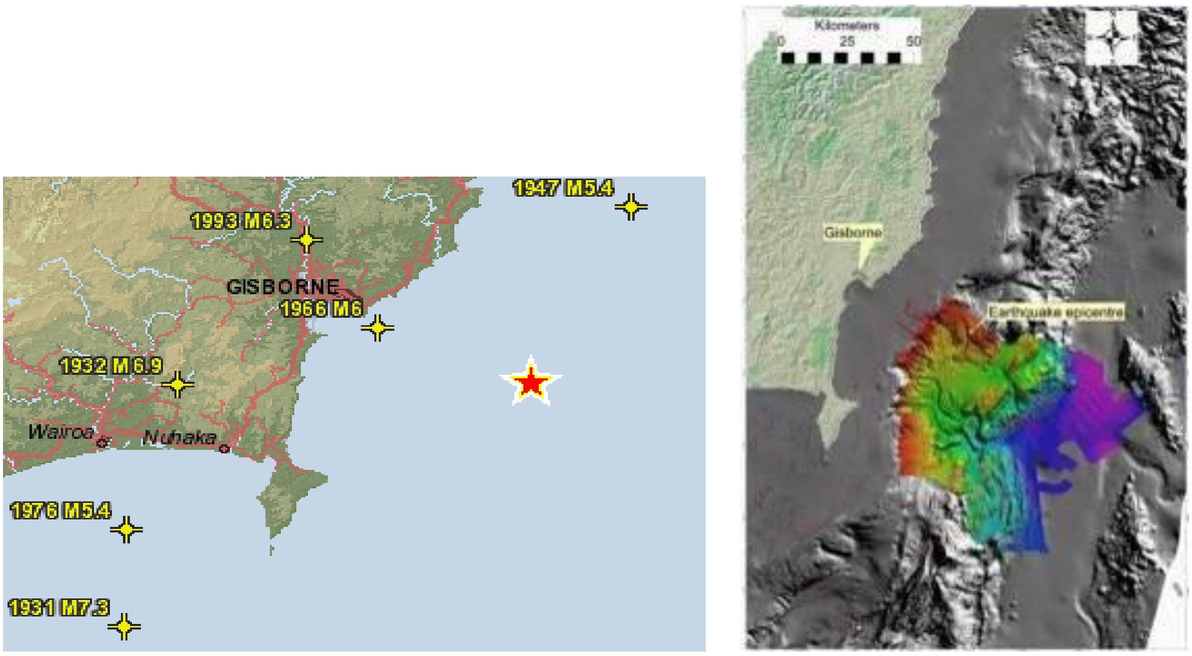
Te Hapara and Elgin are set on sand, silty sand and mixture of sand and soil with various thicknesses. Gisborne central is also underlain by loose to medium dense sand and sand mixture. In relation to the shear wave velocity for all Gisborne suburbs, as suggested by Bevin (2000), none of the regions and suburbs in Gisborne has the soil type A (Strong Rock) or B (Weak Rock). In a further description of soil types in the Gisborne region (see Appendix

3), soil E (Shallow soils) is considerably present only in the river basin, the swamp deposit or the reclamation region.

People in Gisborne expect 13 earthquakes annually with magnitude greater than 4 within 100 km of Gisborne (Webb et al., 1985). Gisborne experiences earthquakes with magnitude greater than MMI=VIII every 200 years, and earthquakes greater than MMI=IX every 1000 years.

## 5.2. The 2007 Gisborne earthquake concurred

Gisborne was struck by a 6.8 magnitude earthquake on December 20, 2007. It was the subduction of the Pacific Plate underneath the Gisborne region. Three buildings collapsed in the business centre and many were damaged. This earthquake caused over $50 million of claim, 29 assigned residential buildings, and 27 assigned commercial buildings (Petty, 2008). Figure 5.2 (a) shows the notable earthquakes which have struck the Gisborne region since 1931. The location of the 2007 Gisborne earthquake is shown in Figure 5.2 (b) (Winkler, 2008).



a

b

Figure 5.2: The 2007 Gisborne earthquake (Source: Winkler 2008)

Past earthquakes in Gisborne caused considerable damage to the lifelines. For instance, main water pipes broke, and about a dozen of the main pipes leaked after the 1966 earthquake in Gisborne. This also caused 40 service leakages in the pump stations (Strachan & Glogau, 1966). The sewer pipe crossing under the bridge on Peel Street broke because of the pipe movement caused by the earthquake shaking (Strachan & Glogau, 1966). The 1993 earthquake in Gisborne caused minor damages, in the main water and wastewater pipelines, except the section under the Peel Street bridge broke (Sirtharan, 1993).

The latest severe earthquake in Gisborne (2007) did not affect the wastewater treatment plant directly, but a power outage immediately after the earthquake affected the function of the treatment plant (Rentoul, 2008). The wastewater pumping stations in Gisborne did not suffer significant damage either, except for leakages in some pump stations. The Gisborne wastewater pump stations, similar the treatment plant, were affected by the power outage after the earthquake although the power outage did not have severe effects on the function of the treatment plant and the wastewater pump stations. The earthquake hit Gisborne at night at about 9 PM. Consequently, the wastewater flow rate at that time was low compared to the rest of the day.

The 2007 Gisborne earthquake caused 2 main breakages in the main wastewater pipelines which pass under the bridge. It took two weeks to repair the sewer breakages. The sewer breakages contaminated the river and the river bank. Figure 5.3 (a), shows the contaminated river due to pipe breakages after the 2007 earthquake in Gisborne. As seen in this photo the solid waste accumulated in the river bank that not only polluted the river and its bank but changed the river appearance. The location of the damaged main sewer under the bridge is shown in Figure 5.3 (b).

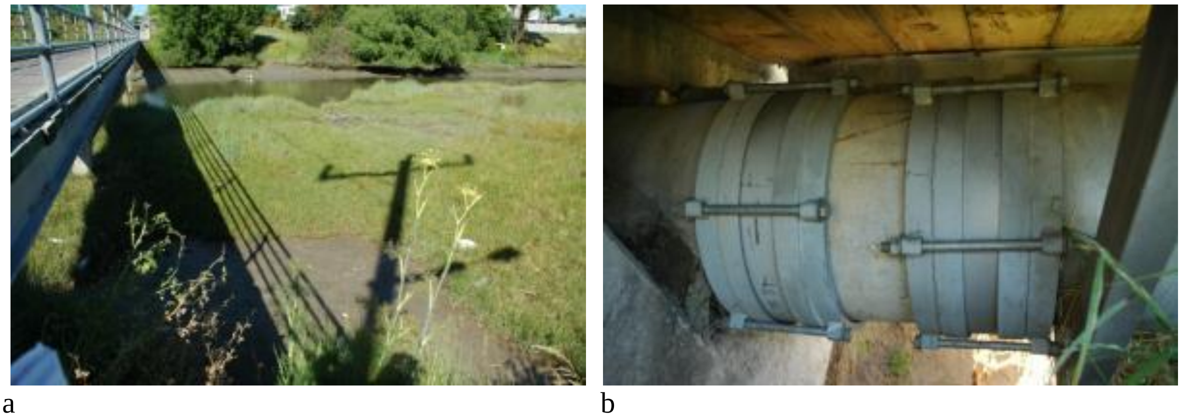

a b

Photo 5.3: The 2007 Gisborne earthquake effects (Source: author own photo)

The earthquakes caused damage in the Gisborne wastewater network from 1966 to 2007 in a few manners., but there is not any accurate data after an earthquake. As a result, in this case study expected damage calculated by some fragility curves are compared to each other to follow the thesis objective.

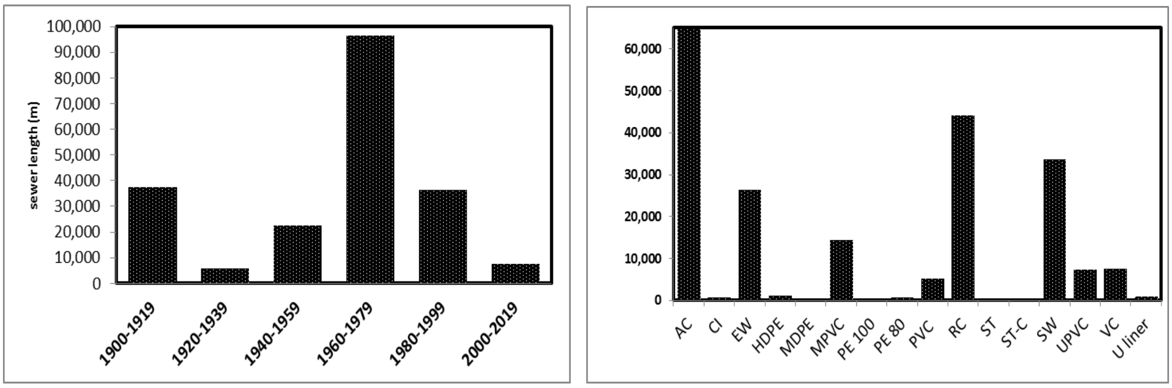
## 5.3. Gisborne wastewater network

The Gisborne wastewater system was installed in 1965. The wastewater system in Gisborne collects and transfers about 5 million cubic meters of wastewater annually (Gisborne District Council, 2009a). The significant difference between the Hutt City and the Gisborne wastewater networks is the combination of wastewater and storm water pipes in the Gisborne wastewater system.

According to the Gisborne City Council report (2009), the Gisborne wastewater network consists 209 km of pipes that covers 71% of the whole Gisborne population (Gisborne District Council, 2009a). The collected wastewater is transferred to the Gisborne's wastewater treatment plant by gravity force and 35 pump stations.

The Gisborne waste water network has 35 wastewater pump stations, 30 pump stations are located in the Gisborne city, and 5 are in the Te Karaka region. Before 1999, the collected wastewater was being discharged directly to the harbour through the last pump station near the sea shore, without any treatment. The Gisborne wastewater treatment plant was established in 1999 to extract solid waste with the diameter greater than 2-3 millimetres from the collected wastewater. The treated wastewater is transferred to Poverty Bay through a 750 mm diameter pre-stressed concrete pipeline of 1,828 meters long (Gisborne District Council, 2009a).

The Gisborne wastewater network was established in the early 1900's. The 18.1 % of the network was installed between 1900 and 1919. The network comprises 209 km of pipes with different pipe specifications with 2,427 manholes (Gisborne District Council, 2010). The initial wastewater pipe types used in this network are earthenware and stoneware pipes. The wastewater network in Gisborne was developed during the last century, although about 47% of the whole reticulation was established between the 1960 and 1979. This fact shows the pipe age in about half of the network is at least three decades old. Figure 5.4 (a) shows the length of installed wastewater pipes from the first sewer installation in 1900 to 2009. Here the age of about 80% of installed pipes is more than 30 years.



a b

Figure 5.4: Development of Gisborne wastewater network (source: Gisborne District Council (2008))

Various types of pipes were used to construct the Gisborne wastewater network in terms of the material and the diameter. The wastewater pipelines in the network are divided into two groups; plastic based pipes and non-plastic based pipes. Plastic pipes used in this network include PVC, MPVC, UPVC, PE (PE 100, PE 80) and HDPE. The non-plastic group consists of AC, CI (generic and concrete encased), RC (generic, pre-stressed, encased and sulphide resistant), earthenware ceramic (generic, glazed or concrete cradle), stoneware, VC and ST (Steel and concrete lined steel). Figure 5.4 (b), produced by the researcher of the thesis, shows the distribution of different types of pipes in the Gisborne wastewater network. This graph shows the major pipe types are AC, RC, SW and EW which cover 31.4 %, 20.4%, 16.3 % and 9.1% of the network, respectively. 51.8% of the network consists of AC and RC pipes.

Here the pipes are divided into brittle and ductile types as usual. The characteristics of CI, AC, RC, VC and EW pipes, classify them as brittle. Due to the higher flexibility to external forces the PE, PVC, MPVC, PE, HSPE, U-lined and ST pipes are classified these as ductile. According to these classifications, 14.24% of the pipes in the Gisborne wastewater network are ductile pipes, and 85.76% are of brittle type.

The pipe diameters in the network vary from 50mm to 750mm maximum. The pipe diameters in the network are 150, 225 and 300mm, which cover 68.5%, 10.9% and 6.6% of the Gisborne wastewater pipes, respectively. Figure 5.5 shows the distribution of pipes in the Gisborne wastewater network in terms of the pipe diameter.

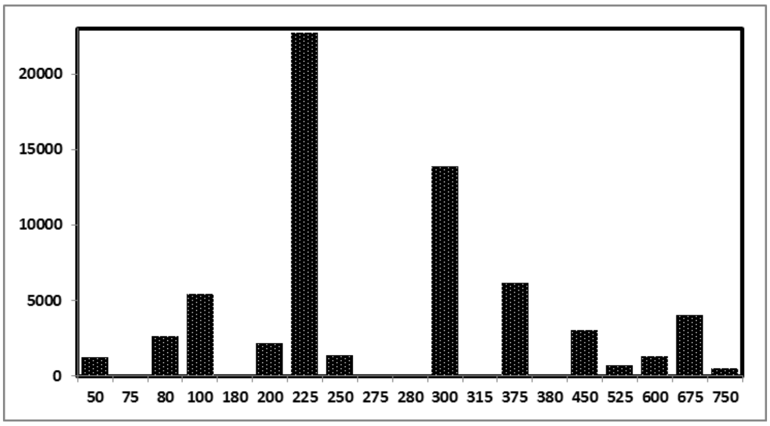


Figure 5.5: Distribution of the Gisborne wastewater pipes in terms of pipe diameter

The wastewater pipes diameters in the Gisborne wastewater network are classified into three different groups. Table 5.2 shows that 73% of the Gisborne wastewater pipes are pipes with the diameter equal to or less than 150mm, and 92.3% are pipes with the diameter less than 300mm.

Table 5.2: Gisborne wastewater pipes classified by pipe diameter

| **Diameter (mm)** | $\leq 150$ | $150 < D \leq 300$ | $300 < D \leq 750$ |
|---|---|---|---|
| **Length (m)** | **152572.9** | **40331.65** | **15949.1** |
| **% OF Total length** | **73.05** | **19.31** | **7.64** |

### 5.3.1. Gisborne wastewater pipes classification in 4 zones

In order to compare earthquake effects on different parts of the Gisborne's wastewater network and to determine severity of damage caused by an earthquake, Gisborne city is divided into 4 zones according to main municipal regions. The zones are divided by main streets, roads and the river crossing the city and the city boundaries. Zone 1 comprises the Gisborne city centre between the Turanganui River in the north and east, Poverty Bay in the south, and Stanley Road in the west (the road divides the right side of the river into two zones with almost the same area). Zone 2 covers the Te Papara and Elgin regions, and is surrounded by Stanley Road in the east, the Turanganui River in the north, and the Gisborne airport in the east. Zone 4 is a long and thin region covering the Mangapapa and Whataupoko regions. Zone 4 is bounded by Zone 1 and 2 in the south, Valley Road in the north, and the Taruheru River in the south. Zone 3 is located between Kaiti and Outer Kaiti. The Zone 3 boundaries are the Taruheru River in the north and west, and the city boundary in the east. Figure 5.6 shows the four zones in Gisborne.

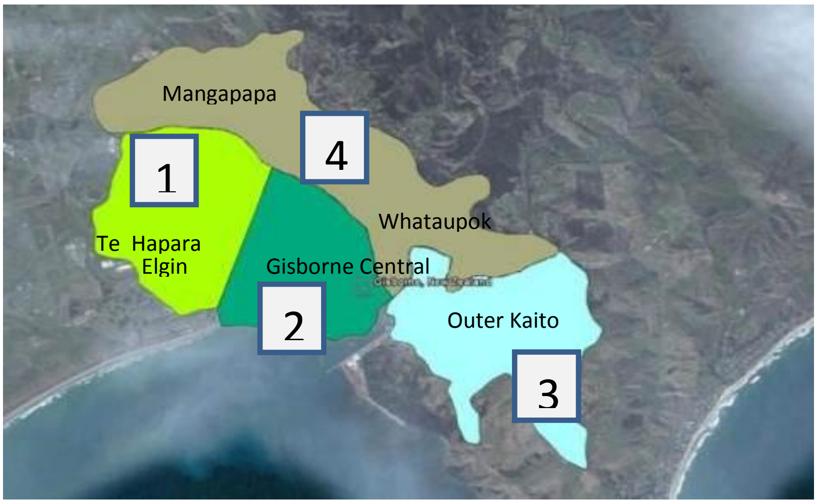
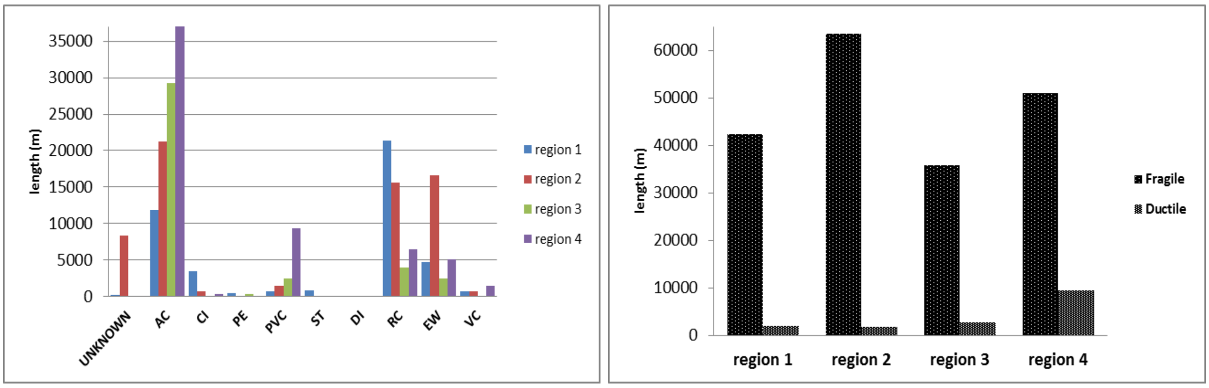


Figure 5.6: The four zones of Gisborne ( Google earth 2009)

The characteristics of wastewater network in each zone can affect seismic vulnerability of that zone to seismic shocks. The type of pipe is a significant factor which directly affects earthquake vulnerability of a pipeline during an earthquake. Consequently, the information about the pipe types of wastewater network in each zone is extracted from available resources and classified (Pineda-Porras & Ordaz, 2007). The Gisborne wastewater network consists of three different pipe types, i.e. wastewater, storm water and connection pipes. The information about the wastewater pipes is available in Auto-cad maps, from Gisborne District Council but there was no adequate information found regarding the storm water pipes and the connection pipes. The total length of the Gisborne wastewater network is about 209 km, 68.9% of which is covered by wastewater pipes, 16.3% by storm water and 14.8% by connection pipes. It should be added that 6% of the pipes here are classified as unknown pipes because there is no information available for these types of pipes. Due to uncertainty about types of pipes in this network, the assumption below is considered in relation to the storm water, connection and unknown sewer pipes in order to calculate earthquake vulnerability of the wastewater pipes in the network.

In relation to the length of the connection pipes, there are 3 main common pipe types (AC, EW and PVC) in the Gisborne wastewater network, each of which is distributed within the length of connection pipes in each zone. It is assumed the wastewater pipelines, installed in the early stages of the city network development are unknown due to inaccurate documentation then. As a result, these unknown pipes are classified as brittle pipes due to the popularity of this type of pipe then.

In order to use pipe types to calculate sewer damage in earthquakes for this city, pipes are classified into the unknown, AC, CI, PE, PVC, ST, DI, RC, EW and VC pipes. For instance, all types of poly ethylene pipes, including PE100, PE80, MDPE and HDPE, are classified as PE pipes. UPVC and MPVC pipes are classified as PVC pipes. This study has analysed wastewater pipes in each zone and results are shown in Figure 5.7 (a). The graph shows the distribution of various wastewater pipes in each zone of Gisborne city.



a b

Figure 5.7: Distribution of pipe types in the Gisborne wastewater network

According to the pipes specifications, Figure 5.7 (b) shows the distribution of brittle and ductile pipes in each zone of the Gisborne wastewater network. The graph shows that the predominant pipes in this network are brittle pipes, while Zone 4 has the maximum percentage of ductile pipes (15.6%).

## 5.4. Overall effects of earthquakes on the Gisborne wastewater network

Earthquakes with two return periods (475 and 1000 year) are selected to show the seismic vulnerability of the network. Toprak's (1998) equation is applied to show overall effects of the earthquakes in the Gisborne reticulation. Calculation of the expected earthquake damage in Gisborne can be divided into 3 main steps, i.e. estimating the expected PGAs, adjusting the estimated PGA in regard to soil characteristics, and applying a fragility curve.

Stirling et al. (2002) provide maps for PGA assessment of each region in New Zealand including Gisborne for different earthquakes. The estimated PGAs are then adjusted regarding soil types, and finally the expected damage in the network is calculated by using the fragility curve.

Figure 5.8 shows the expected damage in different zones of the Gisborne wastewater network caused by the 475 and 1000 year return period earthquakes. Figure 5.8 shows that Zone 4 is the most vulnerable zone with the most vulnerable pipelines and Zone 4, while Zone 3 is the least vulnerable zone in Gisborne. As figure 5.8 shows, earthquakes with 1000 years return period inflict more damage than the 475 year return period earthquakes, although there is no significant difference in the expected damage between the two types of earthquakes, except for Zone 3.

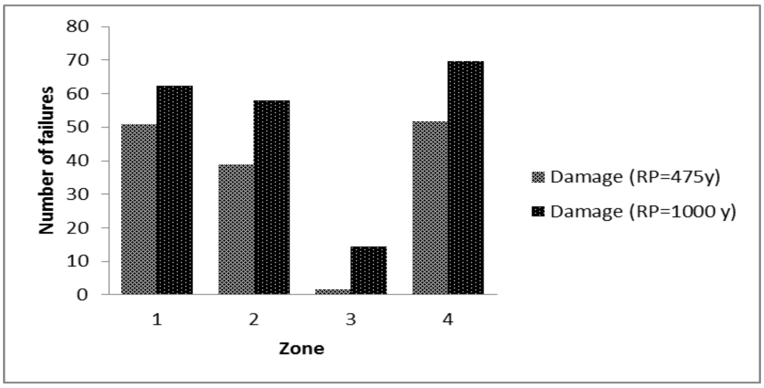


Figure 5.8: Earthquake effects on the Gisborne wastewater network

## 5.5. Earthquake effects (wave propagation) on the Gisborne wastewater network

To estimate earthquake damage rate in the Gisborne wastewater network, the procedure applied here is similar to the one applied in Hutt City and Blenheim. In this part of the thesis, first, seismic effects of earthquakes with annual probability of exceedance 1/1000 on the network will be determined, and then seismic effects of earthquakes with annual probability of exceedance 1/500 will be calculated and discussed. Figure 5.9 shows the differences between the number of calculated damages by different fragility curves in each zone on Gisborne wastewater network affected by earthquakes with annual probability of exceedance 1/1000. Similar to the Hutt City case study, the FEMA (2003), O’Rourke and Deyoe (2004) (R wave) and Eidinger and Avila’s (1999) fragility curves estimate the maximum number of failures in the network.

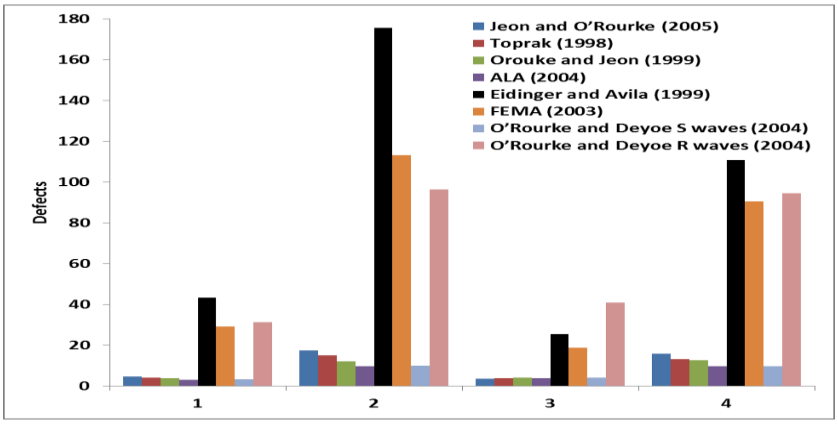
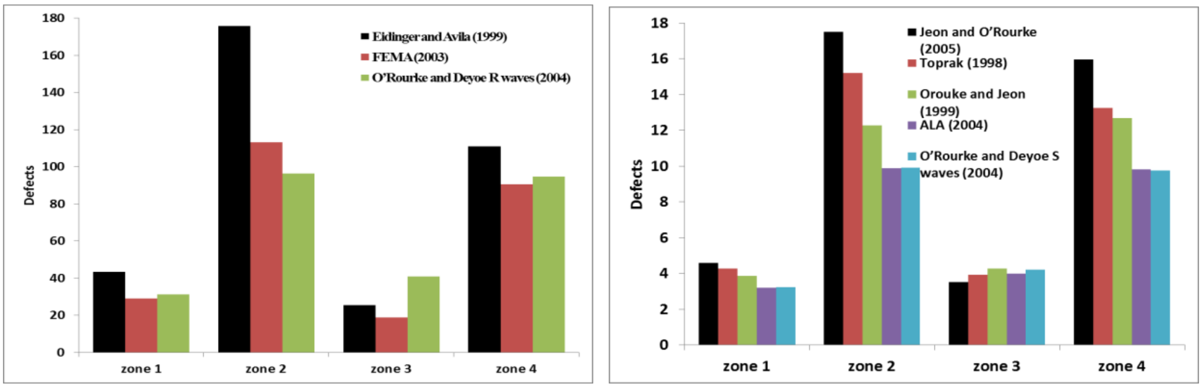


Figure 5.9: Earthquake damage in the Gisborne wastewater network (annual probability of exceedance 1/1000)

Figure 5.10 shows the distribution of calculated damage by Group 1 and Group 2 fragility curves on the Gisborne wastewater network when is affected by earthquakes with annual probability of exceedance 1/1000. Figure 5.10 (a) shows that the Eidinger and Avilas' (1999) fragility curve gives the maximum number of failures in Group 1 and FEMA's equation (2003) gives the minimum number of failures in each zone of the network except zone 2.



a b

Figure 5.10: Earthquake effect on Gisborne wastewater network (Group 1 by graph (a) and Group 2 by graph (b) (annual probability of exceedance 1/1000).

Figure 5.10 (b) shows the impact of earthquakes with annual probability of exceedance 1/1000 on the network by using Group 2 fragility curves. In this group, the Jeon and O'Rourkes' (2005) fragility curve gives the maximum number of damage, compared to other fragility curves in Group 2.

If earthquakes with annual probability of exceedance 1/500 are observed, Figure 5.11 shows the differences between the amount of estimated damage in each zone of the Gisborne wastewater network.

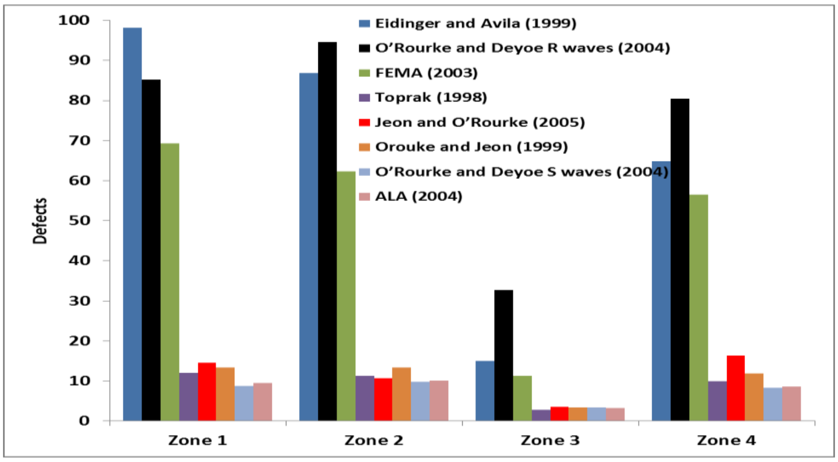


Figure 5.11: Earthquake induced damages in the Gisborne wastewater network (annual probability of exceedance 1/500)

### 5.5.1. Comparison of calculated defects in brittle pipes of the Gisborne WWR in earthquakes with annual probability of exceedance 1/500

Comparison of estimated damage by fragility curves in the Gisborne wastewater network shows there are significant differences between the maximum and minimum expected defects on brittle pipes in each zone in this network. The equation suggested by Eidinger and Avila (1999) as one of the Group 1 fragility curves provides the maximum number of damages. On the other hand, the ALA's fragility curve (2004) as one of the Group 2 equations estimates the minimum number of damages. The Eidinger and Avilas' equation (1999) estimates the number of defects to be about 9.3 times more than that of the ALA's fragility curve (2004).

If the average of calculated defects by Group 2 equations is compared to Group 1 equations, the Group 2 equations provide significantly less number of defects. For instance, if the calculated defects by the Eidinger and Avilas' equation (1999) are compared to the average defects calculated by Group 2 equations, the fragility curve of Eidinger and Avila (1999) provides number of defects 8.4 times more than the average of Group 2 equations. On the other hand, if calculated damages in Group 1 equations are compared to each other, the calculated defects by Eidinger and Avilas' equation (1999) would be 52% more than that of the calculated defects by the FEMA's fragility curve (2003).

The comparison of estimated damage in brittle pipes of each zone in Gisborne shows the fragility curves suggested by Eidinger and Avila (1999) and FEMA (2003) both can estimate the maximum and minimum number of defects. The differences between the maximum and minimum estimation is about 60% in Zone 1 and 2. RC and AC pipes are predominant types of pipe used in zones 1 and 2. For instance, AC pipes cover about 30% of the total length in both Zones.

Zones 3 almost with 82% and 4 with almost 74% contain AC pipes. In these zones in contrast to zones 1 and 2, the R-wave equation of O'Rourke and Deyoe (2004) gives the maximum number of defects and FEMA's fragility curves (2003) provides the minimum number of defects. The difference between the maximum and minimum defect is significant in Zone 3 by 2 times. The significant difference in Zone 3 may be relevant to small length of sewers or combination of different type of pipes in this zone compared to other zones.

### 5.5.2. Comparison of calculated defects in brittle pipes of the Gisborne WWR in earthquakes with annual probability of exceedance 1/500

Gisborne wastewater network comprises of the minimum length of ductile pipes in each zone compared to other case studies. Small length of ductile pipes in each zone in Gisborne can cause bias in results. As a result, comparison of results is not comprehensive in ductile pipes of the Gisborne WWR. If the estimated numbers of defects in ductile pipes are compared to one another, the fragility curves suggested by Eidinger and Avila (1999) gives the maximum number of defects. Jeon and O'Rourkes' equation (2005) provides the minimum damage rate in all zones.

There are significant differences between the maximum and minimum estimated defects in ductile pipes of each zone in the Gisborne WWR. For instance, the number of defects calculated by Eidinger and Avilas' equation (1999) is approximately 19 times more than the defects calculated by Jeon and O'Rourkes' fragility curve (2005). The significant difference between the estimated defects is not only relevant to the inherent differences between each fragility curve but it can be relevant to the short length of ductile pipe in each zone of the Gisborne wastewater network. As mentioned, 82% of each zone total length is comprises of brittle pipes and only 18% of each zone consists of ductile pipes.

## 5.6. Comparison of calculated data with observed data

Comparison of the calculated damages with the observed damage after the Gisborne earthquake in 2007 can assess the accuracy of the applied fragility curve applied for seismic vulnerability assessment of the Gisborne wastewater network.

Inspection of the Gisborne wastewater network after the Gisborne earthquake in 2007 revealed that the earthquake caused the total number of 4 defects; two breakages in Nelson Road Bridge and two leakages one in Glastone Road and another in Oak Street (Evans & Wells, 2008). When the observed damages are placed on the Gisborne zone map in Figure 5.7, the earthquake caused 2 defects in Zone 4 and 2 defects in Zone 2,.

Comparison of peak ground acceleration in the 2007 Gisborne earthquake with that of the expected for earthquakes with 150 years return periods shows that the Gisborne earthquake can be classified as an earthquake with 150 years return period. As a result, if the same procedure which was used for the earthquake with return period of 475 years applies the following results can be seen.

The calculated defects in the Gisborne wastewater network shows that the maximum expected defects in ductile pipes is 3 and is calculated the R-wave fragility curves developed by O'Rourke and Deyoe (2004). The fragility curves recommended by Jeon and O'Rourke (2005), ALA (2004), O'Rourke and Deyoe (S-wave) (2004), O'Rourke and Jeon (1999) and Toprak (1998) estimate no defects in ductile pipes of the network.

Comparison of the total number of the observed defects and the calculated defects in the Gisborne wastewater network shows the fragility curve developed by Jeon and O'Rourke (2005) estimates 5 defects which is the closest number of calculated defects. On the other hand, the comparison shows that the maximum number of 138 defects is estimated by the R-wave fragility curves developed by O'Rourke and Deyoe (2004).

The numbers of 10, 12, 12 and 14 defects are calculated in the Gisborne wastewater network by the fragility curves recommended by Toprak (1998), ALA (2004), O'Rourke and Jeon (1999) and O'Rourke and Deyoe (S-wane) (2004) as Group 1 fragility curves, respectively. The numbers of 66 and 35 defects are calculated by Eidinger and Avila (1999) and FEMA (2003) as Group 1 fragility curves, respectively.

The comparison shows that the fragility curves in Group 2 fragility curves estimate number of defects close to that of the fragility curves in Group 1.

## 5.7. Summary

Gisborne as the second case study is used to show the differences in expected damages in brittle and ductile pipes of each zone in order to identify the similarities and dissimilarities of results compared to Hutt City.

This case study shows there are great differences in calculated damages by each fragility curve within each group. Combinations of different types and lengths of pipes applied in each zone can affect the calculated defects and differences compared to other fragility curves in brittle and ductile pipes.

This chapter as well as Chapter 5 shows that there are significant differences between calculated damage by different fragility curves. However, it is focused in this chapter that in small cities similar Gisborne even if divided into few zones, calculated defects in each zone by some fragility curves can be neglected even for earthquakes with annual probability of exceedance 1/1000. As a result, pre-earthquake rehabilitation in similar cases seems to be an inadequate measure in decreasing seismic vulnerability. However, critical points in a wastewater network should be identified and rehabilitated in advance or in post-earthquake restoration plan with the first priority in the list. For instance, the sections of the network passing through the bridges are recommended to be reinforced and strengthened beforehand.

Table 5.3 predicts the maximum number of defects in brittle pipes is expected in Zone 1, but in contrast with Hutt City the minimum number of defects in brittle pipes are almost the same in all zones for earthquake with annual probability of exceedance 1/500 and 1/1000. On the other hand, the maximum number of defects in ductile pipes affected by earthquakes with annual probability of exceedance 1/500 is 5 and belongs to Zone 4. No defect is expected in ductile pipes of all zones in earthquakes with annual probability of exceedance 1/500 and only 1 defect is expected in Zone 4 in earthquakes with annual probability of exceedance 1/1000.

In earthquakes with annual probability of exceedance 1/1000, the maximum number of 161 defects are expected in Zone 1 and the minimum number of 40 defects are expected in Zone 3 in fragile pipes. Maximum number of defects in ductile pipes when affected by earthquakes with annual probability of exceedance 1/1000 is 5, in Zone 5.

Reviewing the maximum calculated defects in Table 5.3 indicates that the expected calculated defects especially in earthquakes with annual probability of exceedance 1/1000 are notable for a small city such as Gisborne, although, the minimum number of defects in each zone can be neglected. As a result, post-earthquake restoration plan is necessary in case of a strong earthquake in order to increase network efficiency.

Table 5.3: Maximum and minimum expected defects in each zone of the Gisborne wastewater network affected by two types of earthquakes

| Gisborne Zones | Brittle (1/500) | | Ductile (1/500) | | Brittle (1/1000) | | Ductile (1/1000) | |
|---|---|---|---|---|---|---|---|---|
| | Max | Min | Max | Min | Max | Min | Max | Min |
| Zone 1 | 95 | 7 | 1 | 0 | 161 | 9 | 2 | 0 |
| Zone 2 | 85 | 7 | 1 | 0 | 142 | 10 | 1 | 0 |
| Zone 3 | 31 | 6 | 1 | 0 | 40 | 7 | 1 | 0 |
| Zone 4 | 70 | 7 | 5 | 0 | 106 | 9 | 5 | 1 |
| Total | 281 | 27 | 8 | 0 | 449 | 31 | 9 | 1 |

# Chapter 6. Blenheim case study

## Introduction:

The Blenheim wastewater system was installed in the early 1930s. It has been developed and upgraded several times (CH2M Beca Ltd, 2007). Similar to the Hutt City and Gisborne wastewater systems, it consists of three major components, wastewater pipelines network, pump stations and treatment plants. In the wastewater system of Blenheim, domestic and industrial wastewater is collected by about 210 km wastewater pipelines (Marlborough District Council, 2009).

Blenheim is located in a flat area therefore a significant number of pump stations are required to transfer the collected wastewater to the Blenheim wastewater treatment plant. There are 37 WWPSs in operation in this system.

The wastewater collected from the residential and industrial zones is transferred to treatment plant before being released to the Opawa River through 825mm diameter pipes and to the Wairau Estuary by the 375mm diameter pipes (Blenheim City Council, 2004).

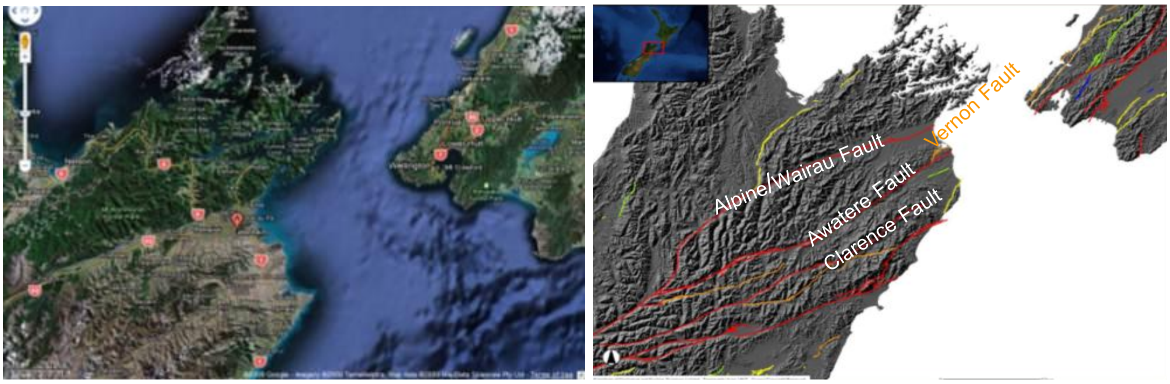
## 6.1. Seismic vulnerability and active faults in Blenheim,

The GNS faults data base (2009) shows three active faults in the Blenheim region; the Alpine/Wairau, Awatere and Vernon Faults. The Alpine Fault passes though the Blenheim valley, and the two other faults pass near the Blenheim region. Blenheim lies about 10km away from the two active faults; the Wairau and Awatere Faults. All the observed faults are capable of producing earthquakes with magnitude of Mw of 6.9-7.9 in the return period range of 28-51 years (Robertson & Smith, 2004). Table 6.1 shows the active faults near the Blenheim region by their recurrence intervals and fault types.

Table 6.1: Faults in the Blenheim region (source: (GNS science, 2009))

| Fault Name | Fault type | Recurrence interval | GNMS-ID |
|---|---|---|---|
| **Alpine/Wairau** | Dextral | 2000 | 22165 |
| **Awatere** | Dextral | 2000 | 22167 |
| **Vernon** | ------------ | 2000-3500 | 22169 |

Figure 6.1 (a) shows the location of Blenheim city in the Sorth Island, while figure 6.1 (b) shows the active fault lines in the northeast of the South Island, which may affect Blenheim.



a b

Figure 6.1: Blenheim region graph (a) (Source (Google earth 2010)) and active faults in the Blenheim region (source: (GNS 2009))

Robertson and Smith (2004) derived the Blenheim earthquake hazards from the two close faults, Wairau and Awatere, and slightly more distant faults, the Marlborough Fault System. The Marlborough Fault system comprises the Clarence, Kekerengu, Elliot, Jordan and Hope Faults (Robertson & Smith, 2004). All these faults have capability to generate earthquakes with magnitude of Mw= 6.9-7.9. The average interval time for earthquakes caused by the Awatere and Wairau Faults varies between 350 and 950 years (Robertson & Smith, 2004). The earthquake with magnitude of Mw=7.5 could create ground acceleration up to 1.4g and MMI intensity from VII to more than IX. The Hope Fault is the most frequently occurring hazard, while the Wairau and Awatere Faults cause the maximum shaking level in Blenheim.

Historical seismicity records in Blenheim show Blenheim can expect earthquakes with intensity of MMI=VII every 58 years, or earthquakes with magnitude of MMI=IX every 210 years (Smith & Berryman, 1983). Table 6.1 shows the average return periods and the maximum earthquake magnitude of the active faults in the Marlborough region. Table 6.2

shows the Wairau, Awatere and Clarence Faults potentially create the maximum earthquake magnitudes, while the Jordan Fault has the maximum return period and the minimum effects in the Marlborough region.

Table 6.2: Marlborough faults (source: (Robertson & Smith, 2004))

| Fault | Average return period (years) | Maximum Magnitude ($M_w$) |
|---|---|---|
| Wairau | 1000-2300 | 7.9 |
| Awatere | 577-1607 | 7.8 |
| Clarence | 900 | 7.5 |
| Kekerengu | 778 | 7.2 |
| Elliot | 1064 | 6.9 |
| Jordon | 1808-3357 | 7.2 |
| Hope NE | 100-300 | 7.3 |
| 1888 | 120 | 7.3 |
| Hope SW | 100-300 | 7.3 |

### 6.1.1. Blenheim soil type

Brown’s (1981) analysis of geology of Wairau Plain, where Blenheim is located shows that Wairau Plain underneath the Wairau River is combined with coastal and lagoon deposits, fluvial and swamp deposits and glacial outwash gravel. Blenheim is underlain with Holocence drained swamps and alluvial silt (Brown, 1981). With reference to the classification of soil types in Blenheim in Appendix 3, soil type D is the predominant soil type. Robertson (2004) suggests that BNM (see Figure 6.2), which is located in Zone 4 (Blenheim central), has the greatest expected earthquake intensity, followed by BBR in Zone 2, and BSS in the north of Zone 4. The correlation between earthquake hazard and soil type, and in order to enter hazard zone classification in Blenheim, 20% and 10% of soil types in Zone 4 and 3 respectively are considered as soil type E. Due to closeness of Zone 2 to BCD and BWF, which are located beside the weak rock, 20% of the soil types in Zone 2 is considered as soil type B.

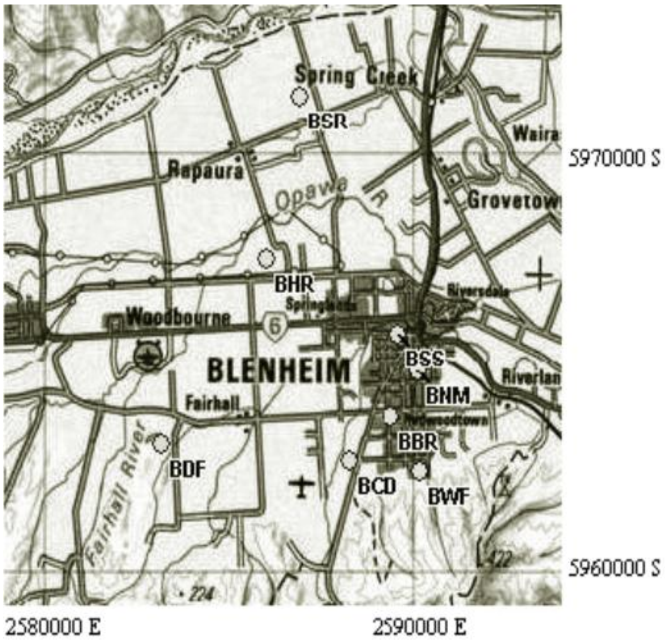


Figure 6.2: Location of test sites in Blenheim and nearby (source: Robertson and Smith (2004))

## 6.2. Wastewater pipelines in Blenheim

Blenheim wastewater system was established in the early 1930’s to serve 6,000 inhabitants in Blenheim (CH2M Beca Ltd, 2007). The Marlborough District Council (2009) data bases show that this system is made of 209.915km of wastewater pipelines.

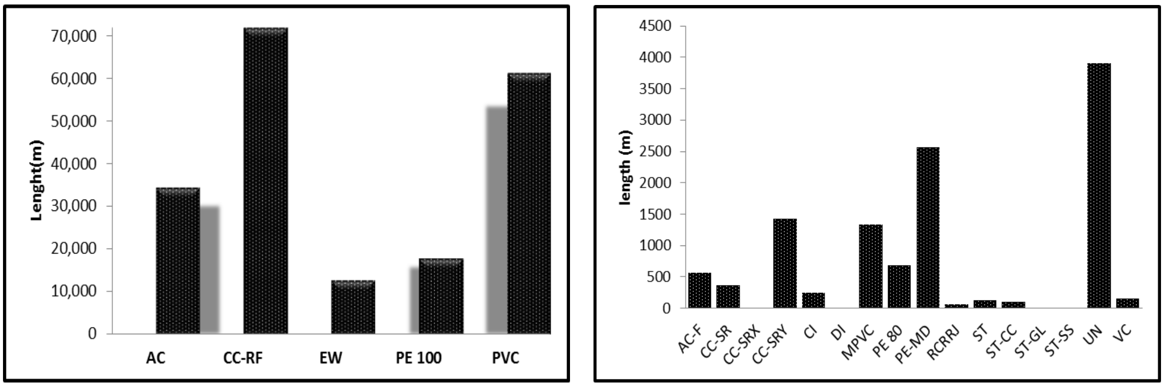
Analysing the GIS dataset established by Marlborough District Council (2009) indicates that the Blenheim wastewater network consists of plastic and non-plastic pipes. Non plastic pipes are the main type of sewers here and are made of asbestos cement pipes (AC-F and AC-I), CI, concrete pipes (CC-RF, CC-SRS, CC-SRX, CC-SRY, CC-SRZ, CC-SR, CC-SU), CE or EW and steel pipes (ST, ST-CC, ST-GL, ST-SW, ST-SS) (Marlborough District Council, 2009).

Analysing the GIS dataset established by Marlborough District Council (2009) indicates that CC-RF, CC-SRS, CC-SRX, CC-SRY, CC-SRZ, CC-SR and CC-SU are different types of concrete pipes used in the Blenheim network. Steel pipes can also be of different types, i.e. ST-CC, ST-SW and ST-SS.

The plastic pipes used in this network are of PVC and PE types (Marlborough District Council, 2009). PVC and MPVC are two types of PVC pipes, and poly ethylene (PE 100, PE 80, PE-LD), medium density poly ethylene (PE-MD, PE 80B) and high density poly ethylene (PE-HD, PE 80C) are different types of poly ethylene pipes in the network.

By analysing the GIS dataset established by Marlborough District Council (2009) the researcher of this thesis found that the reinforced concrete, polyvinyl chloride, asbestos cement, poly ethylene and earthenware pipes are the predominant types of pipes used in the Blenheim wastewater network, and cover 34.25%, 29.2%, 16.4%, 8.5% and 6.1% of the Blenheim sewerage reticulation, respectively, that makes 94.5% of all pipe used. The reinforcement, concrete pipes are the predominant type, and cover about one third of the network.

The GIS dataset analysed by the researcher of this thesis indicates CC-RF and PVC respectively, as the predominant types of wastewater pipes in the Blenheim wastewater network (Figure 6.3 (a)).



a b

Figure 6.3: Predominant pipe types in the Blenheim wastewater network (Source: Marlborough District Council (2009))

There is no information about the types of pipes in 1.3 % of this network (Marlborough District Council, 2009). Figure 6.3 (b) shows the different types of pipes installed in the Blenheim wastewater network in low quantities.

The wastewater pipes in the Blenheim wastewater network are classified into ductile and brittle pipes like elsewhere. Concerning pipe specifications the AC, CI, CC, EW and unknown pipes are classified as brittle pipes, while PVC, MPVC, PE, PE-100, PE-80, MDPE, ST, ST-GL, ST-SS, ST-CC, DI are classified as ductile pipes just similar other study cases. According to the above classification, 60% of the wastewater pipes in this network are of brittle characteristics, and can be affected by external forces such as earthquakes.

### 6.2.1. Blenheim sewers classification (pipe diameter)

The Blenheim wastewater network consists of different types of pipe materials and various types of pipe diameters. The predominant pipe diameter in this network is 150mm, with about 61% coverage.

Figure 6.4 shows the distribution of various pipe types in the Blenheim wastewater network in terms of the diameter, with the exception of 150mm diameter pipes. This figure shows that 225, 600, 300 and 450mm are the predominant pipe diameters in this network next to 150mm.

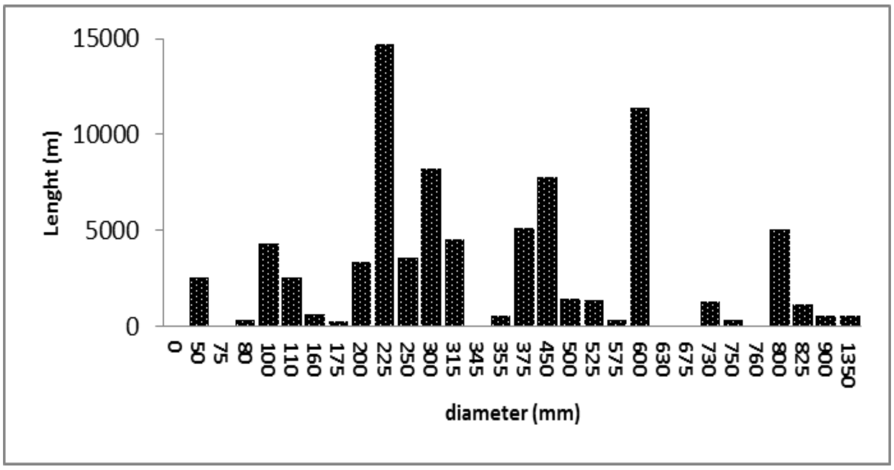


Figure 6.4: Blenheim wastewater pipes distribution diameter (excluding 150 mm)

Table 6.3 shows the major pipe diameters in the Blenheim wastewater network, as well as the significant differences between the percentage of 150 mm diameter pipes and other pipes.

Table 6.3: Main pipe diameters in Blenheim Sewer pipeline (source: Marlborough district Council (2009))

| **Diameter (mm)** | **150** | **225** | **600** | **300** | **450** | **375** |
|---|---|---|---|---|---|---|
| **Length (m)** | 127684.32 | 14696.07 | 11415.53 | 8247.8 | 7805.4 | 5097.27 |
| **% of Total** | 60.83 | 7 | 5.44 | 3.93 | 3.72 | 2.43 |

Table 6.4 shows the pipe diameter distribution among the wastewater pipes in the Blenheim wastewater network. Here more than 80% of the wastewater pipes of less than 300mm, and that the sewers with diameters of 150mm and less cover 65.5 % of the whole reticulation.

Table 6.4: Classification of pipes in the Blenheim sewer network by diameter (source: Marlborough district Council (2009))

| Diameter (mm) | $\leq 150$ | $150 < D \leq 300$ | $300 < D \leq 750$ | $750 < D \leq 975$ | $975 < D \leq 1800$ |
|---|---|---|---|---|---|
| Length (m) | **137421** | **30819.55** | **34352.79** | **6741.05** | **580.74** |
| % OF Total length | **65.47** | **14.68** | **16.37** | **3.21** | **0.28** |

### 6.2.2. Most popular pipes in Blenheim sewer network (150 mm sewers)

The majority of the Blenheim sewers are 150mm sewers, and cover about 128km of the total 210km. Hence, the pipe specifications of the 150mm diameter sewers can significantly influence seismic vulnerability of the whole reticulation system. Table 7.3 shows the predominant types of wastewater pipes in the Blenheim wastewater network. As table 6.5 shows more than one third of the 150mm wastewater pipes are of PVC type followed by CC-RF and AC pipes with 32.1% and 17.6% of the total length, respectively.

Table 6.5: classification of material in the Blenheim 150mm diameter wastewater pipes (source: Marlborough district Council (2009))

| Material | AC | CC.RF | PVC | EW | Miscellaneous |
|---|---|---|---|---|---|
| Length | **22493.77** | **41002.13** | **46040.83** | **12457.97** | **5689.62** |
| % of Total | **17.62** | **32.11** | **36.06** | **9.76** | **4.45** |

If the 150mm diameter sewers in the Blenheim sewer network are classified into brittle and ductile pipes, then 63.3 % of the 150mm sewers can be classified as brittle, and 36.7 % as ductile.

### 6.2.3. Classification of the Blenheim wastewater network in 4 municipal zones

Blenheim city is divided by its main roads and natural features such as rivers into 4 zones in order to analyse earthquake effects on the city wastewater network. Dividing Blenheim into smaller zones and then calculating seismic damage in each zone will identify the most and least vulnerable zones in terms of seismic damage to each zone’s wastewater pipelines. Identifying seismic damage in each zone would help Marlborough District Council to develop pre-earthquake rehabilitation plan or post-earthquake disaster management plans.

Figure 6.5 shows the four zones of the Blenheim city. Zone 1 is located between Old Renwick Road in the north, Pollard Park in the east, Roselands in the west and Yelverton in the south. Zone 2 is surrounded by Alabama Road in the north and the south boundary of Blenheim city in the south. Churchward Park and the cemetery are located in the west of Zone 2, and in the east the zone is limited to agricultural farms. Zone 4 is limited by the Opawa River in the north, Alabama Road in the south, the Taylor River in the west and farms in the east. The Opawa River is in the north, and the east boundary of Zone 3. Pollard Park in the west and Main Street in the south also embrace Zone 3.

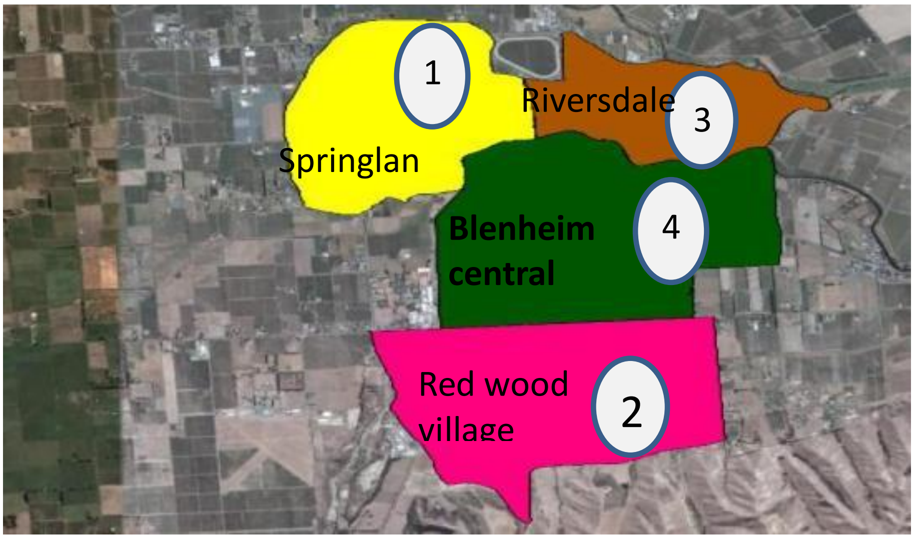


Figure 6.5: The 4 zones in Blenheim city (Google earth 2010)

Figure 6.6 (a) shows the distribution of different types of wastewater pipes in each zone of the Blenheim wastewater network. The wastewater pipes in each zone of the network are classified into ductile and brittle pipe types, as shown in Figure 6.6 (b).

It should be noted that the wastewater pipelines in Renwick are added to Zone 1 wastewater. Renwick is located out of Blenheim, far from the 4 zones in the city, but Renwick's wastewater network is connected to the Blenheim wastewater network. As a result, the wastewater pipelines in Renwich are added to Zone 1.

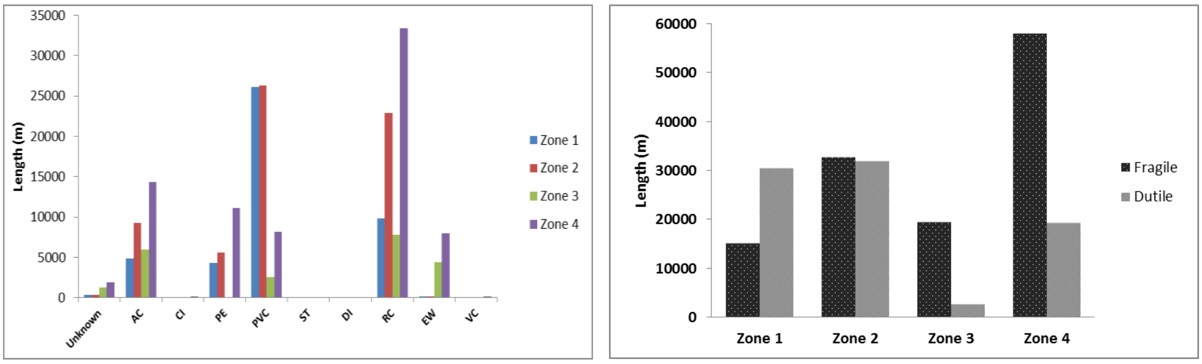


Figure 6.6: Pipes distribution in the 4 zones of the Blenheim wastewater network

## 6.3. Overall earthquake effect on Blenheim wastewater network

In order to show how vulnerable the Blenheim wastewater network is, Toprak's (1998) fragility curve is applied to show which part of the city is the most vulnerable in terms of wastewater network damage. To use the fragility curve, first, PGAs for different zones of the city are extracted, and then, the total expected damage is calculated. Figure 6.7 shows the expected damage in different parts of the network can be caused by 475 and 1,000 year return period earthquakes. As Figure 6.7 shows, the section of the Blenheim wastewater network located in the Zone 4 is the most vulnerable, and Zone 3 is the least vulnerable. This figure shows that similar to the two previous case studies there is little difference within expected damage by the 475 and 1000 year return period earthquakes. Expected damage in the earthquakes with 1000 year return period is about 27% more compared to the expected damage in the 475 year return period earthquakes.

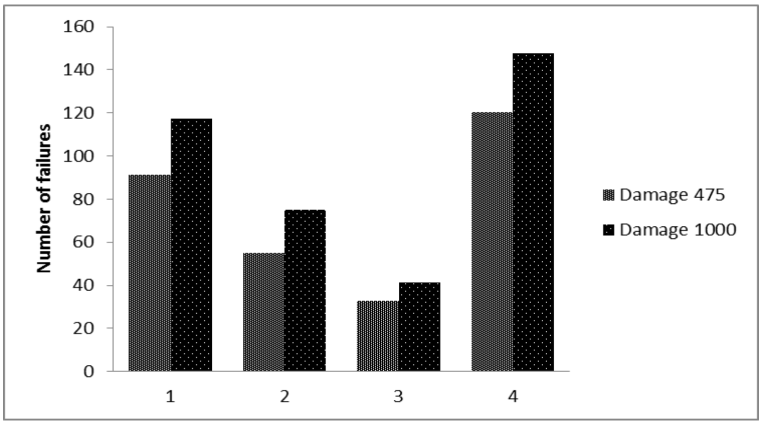


Figure 6.7: Earthquake effects on the Blenheim WW reticulation

## 6.4. Wave propagation effects of earthquakes on the Blenheim wastewater network

The methods applied to show the earthquake effects on the Blenheim wastewater network are similar to those adopted in the Hutt City and Gisborne case studies. First, wave propagation effects of earthquakes with annual probability of exceedance 1/1000 on pipelines of the Blenheim wastewater network will be calculated, and then the wave propagation effects of earthquakes with annual probability of exceedance 1/500 will be demonstrated.

Figure 7.8 shows calculated defects in Zone 1, 2, 3, and 4 of the Blenheim wastewater network which may be caused by wave propagation effects of earthquakes with annual probability of exceedance 1/1000. Figure 6.8 shows that the FEMA's equation (2003) estimates the maximum damage rate in most of the zones in the Blenheim wastewater network. However, it also should be added that the R-wave equation of O'Rourke and Deyoe (2004) estimates the maximum expected damage in Zone 2 where length of ductile and brittle pipes are almost the same. In this network, the equations suggested by Eidinger and Avila (1999), FEMA (2003) and O'Rourke and Deyoe (2004) (R wave) provide the maximum first three damage rates in each zone and show the same trend.

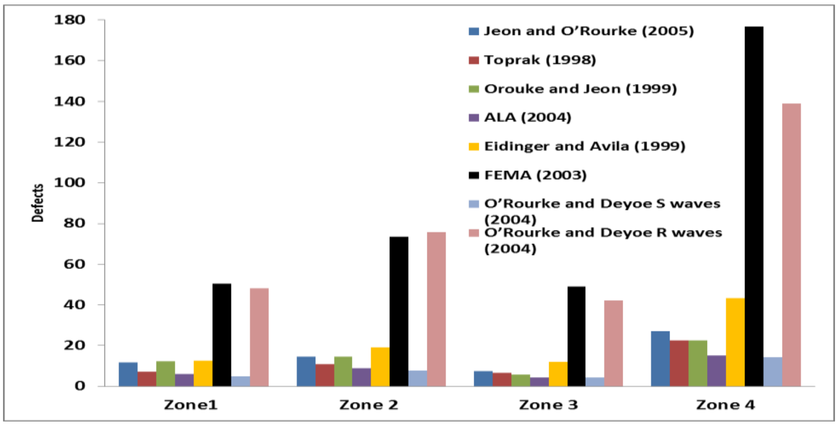
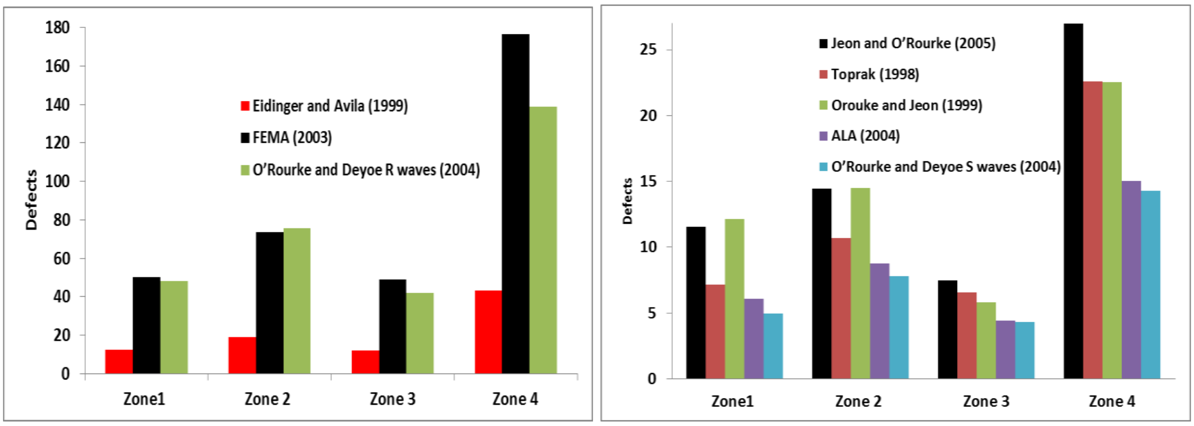


Figure 6.8: Earthquake damage in Blenheim wastewater network (annual probability of exceedance 1/1000)

Figure 6.9 (a) shows the differences within defects in each zone of the Blenheim wastewater network, calculated by the fragility curves of Group 1 and based on annual probability of exceedance 1/1000. As this figure shows both the equations of FEMA (2003) and the R-wave equation of O'Rourke and Deyoe (2004) almost provide the same number of defects in Zone 1 and 2.



a b

Figure 6.9: Earthquake damage in Blenheim wastewater network (graph (a) Group 1 and graph (b) Group 2) (annual probability of exceedance 1/1000)

Figure 6.9 (b) shows the seismic effects of earthquakes with annual probability of exceedance 1/1000 on the Blenheim wastewater network calculated by Group 2 fragility curves. This figure demonstrates that the fragility curves suggested by Jeon and O'Rourke (2005) provide the maximum damage rate in each zone of the Blenheim WWR, compared to other fragility curves.

If earthquakes with annual probability of exceedance 1/500 are taken into consideration, Figure 6.10 shows the differences between the estimated defects for each zone of the Blenheim wastewater network.

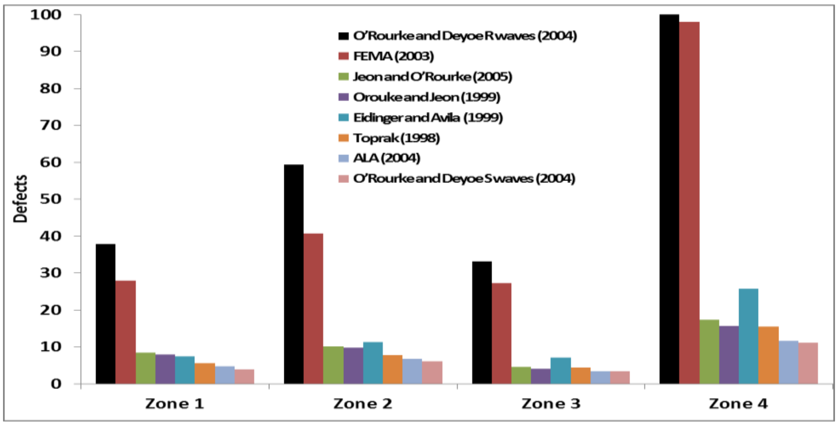


Figure 6.10: Earthquake effects on Blenheim wastewater network (annual probability of exceedance 1/500)

Comparison of calculated damage by earthquakes with annual probability of exceedance 1/500 can be classified into two main groups each of which will be explained in this section. The first part explains the variance between calculated defects in brittle pipes of each municipal zone and the second part will explain the differences expected in ductile pipes of each zone.

### 6.4.1. Comparison of calculated defects in brittle pipes of the Blenheim WWR in earthquakes with annual probability of exceedance 1/500

If calculated defects in brittle pipes of each municipal zone in the Blenheim wastewater networks are compared, the R-wave equation of O'Rourke and Deyoe (2004) estimates the maximum defects in all 4 zones. On the other hand, the ALA's equation (2004) provides the minimum number of defects in all 4 zones. If similar to the previous case studies calculated defects in brittle pipes of the network are divided into two groups, the significant differences between estimated defects by each group can be observed.

The fragility curves suggested by O’Rourke and Deyoe R-wave (2004), FEMA (2003) and Eidinger and Avila (1999) almost provide the first three maximum defects in the brittle pipes of each zone in the Blenheim wastewater network. The fragility curves developed by Jeon and O’Rourke (2005), O’Rourke and Jeon (1999), Toprak (1998), ALA (2004),and O’Rourke and Deyoe S-Wave (2004) as Group 2 fragility curves provide the maximum number of defects to the minimum number of defects in each zone of the network.

Comparison of results in Group 1 equations show the variance between the maximum and minimum estimated defects belong to Zone 1 and the difference is 2.3 times more than the minimum estimated defects. In Zone 1 RC and AC pipes cover 21.5% and 10.6 % of the total length of pipes in this zone, respectively. Furthermore, the percentage of RC pipes in Zone 1 is less than other zones. The maximum difference between the highest and lowest calculated defects belongs to Zone 2 with a difference of about 4.1 times the lowest estimated defects. Length of brittle pipes in Zone 3 is about 19 km that is the second smallest length within all four zones. As a result, the difference within the maximum and minimum estimated defects decreases as length of brittle pipes increases.

If calculated defects by the Group 2 fragility curves are compared to each other, Zone 2 comprises the lowest variance between the maximum and minimum defects.

### 6.4.2. Comparison of calculated defects in ductile pipes of the Blenheim WWR in earthquake with annual probability of exceedance 1/500

In ductile pipes of the Blenheim WWR, the R-wave equation of O’Rourke and Deyoe (2004) gives the maximum expected damages in all four zones, similar to brittle pipes of the Blenheim wastewater network. On the other hand, the S-wave equation of O’Rourke and Deyoe (2004) provides the minimum number of defects in all four zones in contrast to the brittle pipes and the ALA’s fragility curve (2004) shows the second lowest number of defects in all 4 zones.

On the ductile pipes the R-wave equation of O’Rourke and Deyoe (2004), FEMA (2003) provide the first two highest estimated defects. The comparison shows that the Jeon and O’Rourkes’ equation (2005) estimates number of defects more than the Eidinger and Avilas’ equation (1999) that usually estimates high number of defects compared to the fragility curve suggested by Jeon and O’Rourke (2005). However, due to short the length of ductile pipes in each zone, there is not notable difference between the estimated defects calculated by the

Jeon and O'Rourkes' fragility curve (2005) and Eidinger and Avilas' fragility curve (1999). The differences can be caused by type of pipe installed in the network. As explained here, the Blenheim wastewater network comprises the highest percentage of ductile pipes compared to Hutt City and Gisborne.

Similar to other case study cities there are a significant difference between the maximum and minimum calculated defects in each zone of the Blenheim wastewater network. The ratio of the maximum to minimum number of calculated defects varies form 10 in Zone 4 to maximum 14 in Zone 1.

## 6.5. Comparison of the calculated defects with observed damages

Comparison of the calculated defects with the recorded earthquake induced damages in Blenheim City provides further information regarding advantages and disadvantages of fragility curves for calculating seismic induced damage in unpressurised wastewater network.

As explained in Chapter 2 Blenheim is vulnerable to earthquake and can severely be affected by seismic shocks caused by its' nearby actives faults. Assessment of seismic activities in Blenheim based on the Geonet data base (2012) shows that Blenheim has experienced several weak to moderate earthquakes, but the city has not experienced strong earthquakes which could cause damage in its' wastewater network.

Reviewing the available sources of information shows that no report exists regarding seismic damage on buried pipelines networks including pressurised and unpressurised networks. Consequently, in this chapter the calculated damages are not compared to real earthquake induced damage in Blenheim.

## 6.6. Summary

Table 6.6 shows maximum and minimum expected defects in each zone of the Blenheim wastewater network in earthquakes with annual probability of exceedance 1/500 and 1/1000. Brittle pipes in Zone 4 will experience maximum number of 99 defects in earthquakes with annual probability of exceedance 1/500. Table 7.4 shows that there are notable differences between the maximum number of defects in brittle and ductile pipes in the Blenheim wastewater network; however, compared to Hutt City and Gisborne the same differences are less due to higher percentage of ductile pipes.

Zone 4 in the Blenheim wastewater network is the most vulnerable zone and Zone 3 is the least vulnerable zones. Numbers of defects in Zone 4 shows pre-earthquake rehabilitation plan should be taken into consideration in Zone 4 to reduce seismic damage after an earthquake. The minimum number of expected defects in Brittle pipes of zones 2 and 3 shows the post-earthquake restoration can be a good measure in these zones.

Comparison of the calculated defects in ductile pipes of the Blenheim wastewater network shows that zones 1 and 2 are the most vulnerable zones and Zone 3 is the least vulnerable zone in earthquakes with annual probability of exceedance 1/500. The number of defects in ductile pipes of the Blenheim wastewater network shows post-earthquake restoration plan can be the best measure to reduce seismic damage.

Seismic damage in the Blenheim wastewater network in earthquake with annual probability of exceedance 1/1000 shows that Zone 4 is the most vulnerable zone. Number of defects in Zone 4 increase much more than other zones compared to defects in earthquakes with annual probability of exceedance 1/500. The appropriate method to decrease seismic damage rate in Zone 4 can be pre-earthquake rehabilitation for its brittle pipes and post-earthquake restoration plan for its ductile pipes in earthquake with annual probability of exceedance 1/1000. Pre-earthquake rehabilitation of critical points and pipelines in zones, 1, 2 and 3 plus preparing post-earthquake restoration plan would be an applicable and appropriate method to decrease the seismic damage rate of earthquakes with annual probability of exceedance 1/1000.

Table 6.6: Maximum and minimum expected defects in each zone of the Blenheim wastewater network

| Blenheim Zones | Brittle (1/500) | | Ductile (1/500) | | Brittle (1/1000) | | Ductile (1/1000) | |
|---|---|---|---|---|---|---|---|---|
| | Max | Min | Max | Min | Max | Min | Max | Min |
| Zone 1 | 24 | 2 | 14 | 1 | 31 | 3 | 19 | 3 |
| Zone 2 | 46 | 4 | 13 | 1 | 59 | 6 | 17 | 2 |
| Zone 3 | 32 | 3 | 1 | 0 | 47 | 4 | 4 | 0 |
| Zone 4 | 99 | 10 | 10 | 1 | 161 | 12 | 16 | 2 |
| Total | 200 | 19 | 38 | 3 | 298 | 25 | 56 | 7 |

In this case study similar the other two case studies there are significant differences between estimated defects by the applied fragility curves. The differences again emphasise that fragility curves cannot accurately estimate expected post-earthquake damage in unpressurised networks.

Pipe material is the only factor which shows pipe specification in the applied fragility curves and other pipe specifications which can have effect on seismic vulnerability are not considered in these curves. Consequently, the differences in result show the fragility curves which are only based on pipe material cannot accurately assess seismic vulnerability in unpressurised wastewater networks.

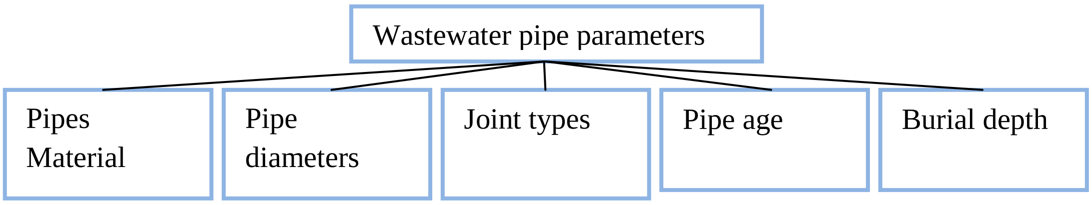
# Chapter 7. Wastewater pipelines damage: comparison and discussion

## Introduction:

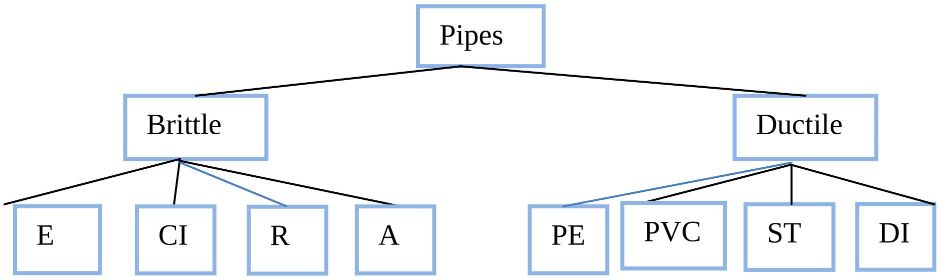
In this thesis, four case studies were studied in order to determine and exhibit the seismic vulnerability of wastewater network in New Zealand. One the studies reviewed revealed the damages and its effects caused by the 2010 Christchurch earthquake. This thesis investigated the limitation in applying the available fragility curves to estimating the defects. In this chapter, comparison of the case study wastewater networks and their seismic vulnerability will be discussed. Parameters that can affect seismic vulnerability analysis of wastewater network will be discussed and limitation of the applied fragility curves will be investigated. First, the similarities and the differences between each wastewater network in the four cases studies are compared.

## 7.1. Comparison of wastewater network in three case studies

Wastewater pipelines in each case study can be categorised according to the factors which can affect seismic vulnerability of the wastewater pipelines. In this study wastewater pipelines are classified by the pipeline material, diameter, joint, age, corrosion rate and buried depth, based on the available data on the wastewater network in each case study. Classifying wastewater pipes into two main groups of brittle and ductile types is the most usual manner. Figure 7.1 shows the general classification used in this thesis for the data analysis process. This figure shows the general types of pipe material, and various types of pipe diameter, investigated in this thesis.



Wastewater pipes classification by material



Wastewater pipes classification by diameter

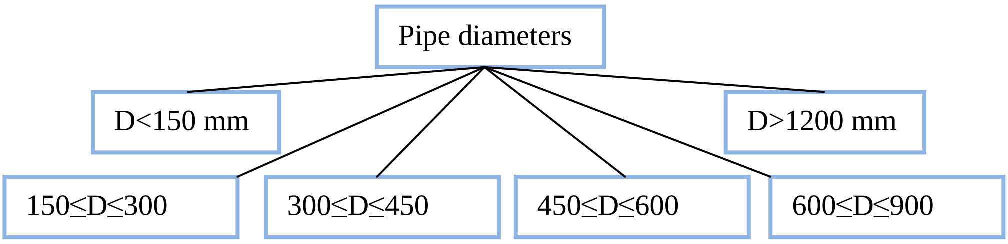


Figure 7.1: Wastewater pipelines classification

According to Chapter 2 the pipe material and diameter of the buried pipelines affects seismic vulnerability of wastewater pipes, and directly affects selection of rehabilitation and repair methods after an earthquake. The wastewater pipelines in Gisborne, Hutt City and Blenheim are compared in terms of the pipe material and diameter.

Wastewater pipelines in the three case studies comprised various types of pipes, but there are some common pipe types in each case study. For instance, concrete, asbestos and earthenware pipes are the three main types of sewers in Hutt City, Gisborne and Blenheim, covering 67.8%, 70.5% and 59.5% of the all the wastewater network in each city, respectively.

Polyvinyl chloride pipes are also one of the 4 major pipe types used in the Hutt City and Blenheim wastewater network. Polyvinyl chloride pipes cover 12.9% and 36.06% of the total length of the wastewater pipes in the Hutt City and Blenheim wastewater network, respectively. PVC pipes in the Gisborne wastewater network cover 7% of the total length. As noted in Chapter 2, these types of pipes (RC, AC and EW) are the most vulnerable to earthquake.

The pipes used in the wastewater networks are classified either as brittle or ductile pipes. The Gisborne wastewater network has the most brittle network among the three cities studied. The Blenheim wastewater network with 60% of brittle pipes has the most flexible network. Figure 7.2 shows the distribution of brittle and ductile pipeline in each case study.

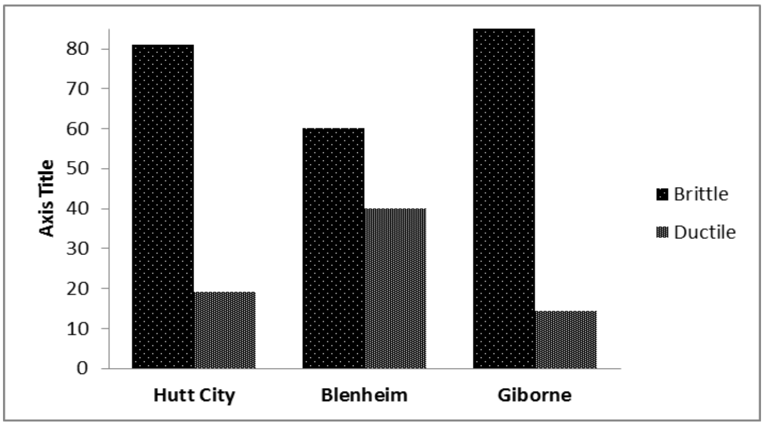


Figure 7.2: Ductile and brittle wastewater pipes in Hutt City, Blenheim and Gisborne

Table 7.1, shows the distribution of wastewater pipes in the Hutt City, Blenheim and Gisborne wastewater networks in terms of pipe diameters. The table indicates that the most common wastewater pipe types are the ones with diameters less than 750mm, and the most common types are 150mm diameter pipes in all the case studies. The 150mm diameter pipes cover at least 65% of the whole wastewater network in each case study.

Table 7.1: Sewers distribution in Hutt City, Blenheim and Gisborne

| Diameter (mm) | $D \leq 150$ | $150 < D \leq 300$ | $300 < D \leq 750$ | $750 < D \leq 975$ | $975 < D \leq 1800$ |
|---|---|---|---|---|---|
| Hutt City | **71.7 %** | **14.8%** | **8.5%** | **1.6%** | **3.4%** |
| Blenheim | **65.47%** | **14.68%** | **16.37%** | **3.21%** | **0.28%** |
| Gisborne | **73.05%** | **19.31%** | **7.64%** | **0.0%** | **0.0%** |

While Figure 7.3 shows the distribution of wastewater pipes in the Hutt City, Gisborne and Blenheim wastewater network in terms of pipe diameters.

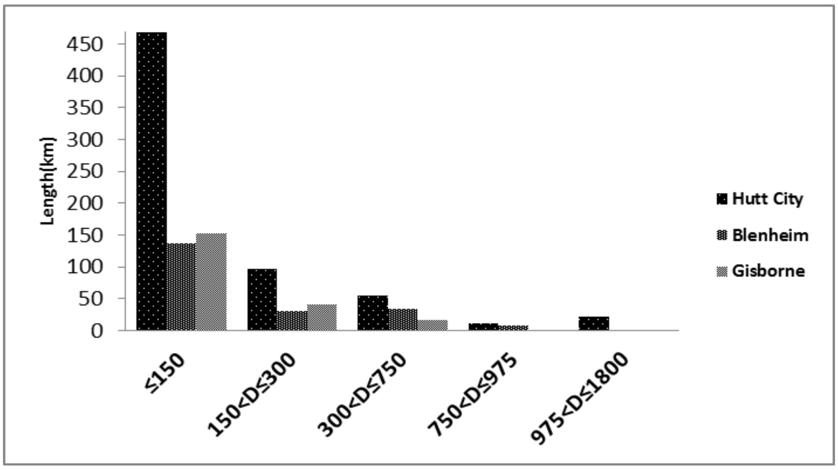


Figure 7.3: Sewer distribution in Hutt City, Blenheim and Gisborne

Comparing the three case studies indicates that at least 60 % of each wastewater network comprises brittle pipes compared to ductile pipe. Because fragile pipes are more vulnerable than ductile pipes, earthquakes cause damage on at least 60 % of each case study reticulation. The majority of sewers networks in each case study contain small diameter pipes which are more vulnerable to earthquakes compared to large diameter pipes.

The total instances of the damage in the Hutt City, Gisborne and Blenheim wastewater network calculated in Chapter 4, 5 and 6 were estimated as 817, 142 and 299, respectively, affected by earthquakes with 475 years return period in the overall approach method. Both wave propagation and permanent ground displacement of earthquakes were calculated by the PGA based equation. If the overall seismic effects of earthquakes with 1,000 years return period are taken into account, instances of the expected damage in the Hutt City, Gisborne and Blenheim wastewater network are 928, 204 and 382, respectively. The comparison of damage caused by two types of earthquake shows that the expected damage in earthquakes with 1,000 years return period is not significantly higher than damage caused by 475 years return period earthquakes. For instance, the expected damage in the Hutt City, Gisborne and Blenheim wastewater network affected by a 475 years earthquake is 12%, 30% and 22% respectively, which less than that of the 1,000 years earthquake, respectively.

In earthquakes with annual probability of exceedance 1/500, the brittle pipes in the Hutt City wastewater network will experience the maximum number of defects compared to Gisborne and Blenheim. However, if total damage in each city is divided into the total length of wastewater pipes, the obtained damage rate would indicate that Gisborne and Blenheim will experience more compared to Hutt City.

Table 7.2 shows the zones of each city with the maximum and minimum number of defects in brittle wastewater pipelines with an annual probability of exceedance 1/500. This table shows that Zone 5 in Hutt City, Zone 1 in Gisborne and Zone 4 in Blenheim should expect the maximum number of defects in brittle pipes in the wastewater network. On the other hand, in term of pre earthquake rehabilitation plan, Zone 5 in Hutt City, Zone 1 in Gisborne and Zone 4 in Blenheim are in the first priority.

Table 7.2 shows that Zone 6 in Hutt City, Zone 3 in Gisborne and Zone 1 in Blenheim will experience the minimum number of defects in brittle pipes. The table indicates that the number of defects after earthquakes with annual probability of exceedance 1/500 can be neglected in these zones in comparison with the number of damage in the zones with the maximum number of defects. Consequently, pre-earthquake network rehabilitation in the zones with the minimum number of defects in each case study is in the last priority and post-earthquake rehabilitation is recommended for these zones.

Table 7.2: Seismic damage on brittle pipes of case studies' wastewater network in earthquake with annual probability of exceedance 1/500

| City | Maximum number of defects | | Minimum number of defects | |
|---|---|---|---|---|
| | Zone | Number | Zone | Number |
| Hutt City | 5 | 189 | 6 | 1 |
| Gisborne | 1 | 95 | 3 | 3 |
| Blenheim | 4 | 100 | 1 | 2 |

As shown in Table 7.3 Zone 5 in Hutt City, Zone 4 in Gisborne and Zone 1 in Blenheim will experience the maximum number of defects in ductile pipes. Comparison of damage numbers in ductile and brittle wastewater pipes in each case study indicates that the number of expected damage in ductile pipes is much less than that of the brittle pipes, that is because of short length of ductile pipes to brittle pipes and also better behaviour of ductile pipes in earthquakes compared to brittle pipes. In the Gisborne wastewater network only one defect is expected in ductile pipes and the main concentration on post-earthquake rehabilitation plan should concentrate on brittle pipes.

Comparison of the minimum calculated defects in table 8.3 shows that the ductile wastewater pipelines in all zones in Gisborne, Zone 1, 2, 6, 7 and 8 in Hutt City and Zone 3 in Blenheim will experience the minimum number of defects. As a result, these zones can be in the last priority for pre and post-earthquake rehabilitation plans.

Table 7.3: Seismic damage on ductile pipes of case studies' wastewater network in earthquake with annual probability of exceedance 1/500

| City | Maximum number of defects | | Minimum number of defects | |
|---|---|---|---|---|
| | Zone | Number | Zone | Number |
| Hutt City | 5 | 34 | 1, 2, 6, 7 and 8 | 0 |
| Gisborne | 4 | 1 | 1, 2, 3 and 4 | 0 |
| Blenheim | 1 | 14 | 3 | 0 |

Table 7.4 shows the zones with the maximum and minimum number of defects in brittle pipes in the Hutt City, Blenheim and Gisborne wastewater networks. The maximum number of 160 defects are expected in Zone 1 at the Gisborne wastewater network and Zone 4 of the Blenheim wastewater network. Zone 1 in Gisborne and Zone 4 in Blenheim are the most vulnerable zones and as a result, both should be in the first priority for pre-earthquake rehabilitation. Table 7.4 shows the maximum number of defects in brittle pipes in earthquakes with annual probability of exceedance 1/1000 is at least 1.5 times more than expected defects in earthquakes with annual probability of exceedance 1/500.

Table 7.4: Seismic damage on fragile pipes of case studies' wastewater network in earthquake with annual probability of exceedance 1/1000

| City | Maximum number of defects | | Minimum number of defects | |
|---|---|---|---|---|
| | Zone | Number | Zone | Number |
| Hutt City | 5 | 289 | 6 | 1 |
| Gisborne | 1 | 160 | 3 | 3 |
| Blenheim | 4 | 160 | 1 | 4 |

Table 7.4 also shows that Zone 6 in Hutt City, Zone 3 in Gisborne and Zone 1 in Blenheim are the least vulnerable zones in each case study and expected to tolerate the earthquakes with minimum damage.

While Table 7.5 shows the zones with the maximum and minimum number of defects in ductile pipes in the case studies wastewater network affected by earthquakes with an annual probability of exceedance 1/1000. Table 7.5 indicates that ductile pipes in Zone 5 in Hutt City. Zone 4 in Gisborne and Zone 1 in Blenheim have the most vulnerable ductile pipes compared to other zones in each case study. However, comparison of Table 7.4 and Table 7.5 indicates that the number of defects in the zones with the highest number of defects in fragile pipe is much more than that of the defects in ductile pipes of the zones with the maximum number of defects in their ductile pipes.

Table 7.5: Seismic damage on ductile pipes of the case studies wastewater network in earthquake with annual probability of exceedance 1/1000

| City | Maximum number of defects | | Minimum number of defects | |
|---|---|---|---|---|
| | Zone | Number | Zone | Number |
| Hutt City | 5 | 56 | 1, 2, 6, 7, 8 | 0 |
| Gisborne | 4 | 8 | 1,2,3 | 0 |
| Blenheim | 1 | 18 | 3 | 0 |

## 7.2. Study limitations

Application of the available fragility curves used to evaluate wave propagation effects of earthquakes in regions without properly developed fragility curves can cause notable uncertainty in the estimated number of defects.

Pipe material, type of joints, buried depth, diameters, age and corrosion rate are other parameters which affect buried pipes vulnerability during an earthquake. Furthermore, the quality of construction, soil type, backfill soil, water table level, distance from faults and pipe direction towards the fault line can also affect seismic vulnerability of the buried pipelines.

In this study PGV was extracted from spectral acceleration and as a result was not extracted directly from fault lines and earthquake characteristics in New Zealand because the data was not available. However, the applied method used for calculating PGV is recommended by geologist to estimate PGV accurately (FEMA, 2003; Zhao, 2009).

Soil type is a substantial parameter that affects interaction between soil and buried pipes during an earthquake. Soil type directly affects the calculated PGV in each zone and consequently the damage. One or two types of soils out of the five general types were considered in order to calculate the expected damage. Each city can be comprised of several types of soil. As a result, PGV and estimated defects can change in each region with different soil type. This study took the different soil types into account as much as possible by using the latest geotechnical data per zone.

The comparison results calculated by the fragility curves in each case study show that the estimated defects by the fragility curves are sensitive to the length of pipe and short lengths of pipes can increase inaccuracy of the results.

Buried pipelines were classified into brittle and ductile pipes without any emphasise on the particular type. For instance, PVC pipes can be classified at least into three main types; PVC, UPVC and MPVC each of which with different specification especially in internal strengths to external loads. However, given the availability of equations, damage assessment can still contribute to emergency planning. For instance, the Christchurch earthquake showed damage rate in UPVC pipe is much more than that of the damage rates in PVC pipes. In order to apply PVC pipes in fragility curves all types of PVC pipes are classified as PVC which can affect the number of calculated defects. This classification may result less defects compared to defect calculation in each type of pipes.

Only few fragility curve like that of the Eidinger and Avila (1999) and Jeon and O’Rourke (2005) equations introduce specific equations for ductile pipes. As a result, accurate assessment of seismic effects on ductile pipes can be affected by application of fragility curves which are not developed for ductile pipe. In this study the Jeon and O’Rourke’s (2005) and Eidinger and Avila’s (1999) equations are applied to counteract this problem.

Poor installation quality can increase failure risk on buried pipeline during earthquake. For instance, if during installation, the excavated trench is not back filled with the selected suitable backfill soil, then soil-pipe interaction can increase seismic damage risk. It is difficult to gauge the effects of poor installation on damage so the assumption made in this study is to treat all pipes installation equally.

Pre-earthquake mitigation plans and post-earthquake recovery plans can be used to decrease earthquake rehabilitation efforts. Mitigation plans in wastewater network can be applied to decrease earthquake induced damage in buried pipelines. Accurate estimate of the number of defects in wastewater pipelines before an earthquake is critical to establish pre-earthquake mitigation plans and to assess post-earthquake recovery plans.

Because of the effects of earthquakes on pipelines and the behaviour of soil and buried pipes during an earthquake, estimating damage in sewers is challenging. Post-earthquake inspection of wastewater network can be used in parallel to damage assessment in order to determine the extent of earthquake induced damage in wastewater network.

Seismic vulnerability analysis based on the available fragility curves cannot provide accurate number of defects but can at least be used as a guide; therefore in this thesis the same is used in Hutt City, Gisborne, Blenheim and Christchurch in order to indicate the possible extent of the damages.

This thesis shows that the seismic vulnerability analysis by fragility curves can be applied to determine the most or least vulnerable parts of the wastewater network. Comparison of the calculated defects in three case studies shows that the applied fragility curves can appropriately be applied to show the most and the least vulnerable that the zones of each wastewater network.

The author's recommendation for protection of the wastewater network against earthquakes would be to strengthen the main pipelines or critical points within a city or alternatively, to strengthen the highly vulnerable zones within the entire network. Main pipelines can be classified by capacity or by diameter. However, some smaller pipelines that connect government or public buildings to the network can be determined as main pipelines because they are critical to infrastructure. For instance, protection of the pressurised pipeline which connects the treatment plant to the discharge point should be a priority in the Hutt City wastewater system.

The other vulnerable pipelines are the wastewater pipelines passing though bridges. As a result, these pipelines should be prioritised. This study also shows that concrete and asbestos wastewater pipes are highly vulnerable to seismic shocks and need pre-earthquake rehabilitation.

## 7.3. Summary

The brittle pipes mostly asbestos cement, concrete based and earthenware pipes are the predominant types of pipes in wastewater networks of New Zealand's cities. However, percentage of brittle pipe consumption in each city varies to other cities while brittle pipes at least cover 60% of total length in each case study.

Earthquakes with annual probability of exceedance 1/1000 have caused at least 1.5 times more damage in each case study compared to earthquakes with annual probability of exceedance 1/500. The differences between estimated defects by the two types of earthquakes in cities with total length of wastewater pipelines of about 600 km like Hutt City are more significant compared to Blenheim and Gisborne with about 208 km of wastewater pipelines.

Comparison of defects calculated by the fragility curves in each case study shows that there are significant differences between calculated defects in brittle pipes. However, the differences between calculated defects in ductile pipes of some zones in each case study can be neglected due to their short length. The significant differences in calculated defects in wastewater network of the four case studies show that the differences can vary from a case study to other because of mix application of brittle and ductile pipes in each case study.

Here, it is focused that the fragility curves can overestimate or underestimate the expected damage after a particular earthquake. As a result, the available fragility curves are not adequate tool in order to estimate the damage rate required for pre-earthquake or post-earthquake rehabilitation plan in the wastewater networks.

The differences within the calculated defects indicate that the available fragility curves cannot be applied accurately when calculating seismic damage in unpressurised wastewater pipelines. As explained, many parameters affecting seismic vulnerability on buried pipelines are ignored. The available fragility curves are only based on one or two independent parameters. Several other parameters explained in Chapter 2 affect seismic vulnerability of buried pipelines. Amongst others, these parameters include pipe diameter, durability, corrosion and burial depth. As a result, in the opinion of the writer, the applied fragility curves which are presently available have insufficient parameters.

Comparison of defects in brittle and ductile pipes for each zone within each case study indicates clearly that the total defects in ductile pipes are notably less than those of brittle pipes. As a management strategy, prior to earthquake occurrence, it is better to concentrate on critical points in the pipelines and zones of high seismic vulnerability than to prioritise rehabilitation of selected ductile pipes.

Earthquake impacts on brittle pipes of wastewater network almost significantly affect all zones within each city case study. Pre-earthquake rehabilitation of the most vulnerable zone or zones can reduce earthquake impacts on wastewater network of each case study. However, in some regions of small cities post-earthquake restoration or only pre-earthquake rehabilitation of critical points can sufficiently reduce earthquake hazard on a wastewater network.

# Chapter 8. Wastewater network damage and repair in the 2010 Christchurch earthquake

## Introduction:

It was during the writing of this thesis that an earthquake struck Christchurch on 4 September 2010. The earthquake caused major damage to the wastewater system and provided the opportunity to understand first-hand problems. In this chapter, the 2010 Christchurch earthquake and its general impact on the city’s wastewater system will be described. In terms of seismic vulnerability, this earthquake’s damage on the wastewater pipelines, wastewater pump stations and treatment plants proved positively the vulnerability of wastewater systems in New Zealand. This fact confirms the findings in Chapters 5, 6 and 7.

### 8.1. The 2010 Christchurch earthquake

Christchurch is the largest city in the South Island, located in the east of the South Island. The Christchurch area is bounded by the Pacific Ocean, and the Estuary of Avon and Heathcote rivers in the east. The south and southwest parts of the city are bounded by Port Hills, and the north by the Waimakariri River. Christchurch City was struck by 7.1 magnitude earthquake on 4 September, 2010. The epicentre was near town of Darfield, 40 km west of Christchurch City (GNS science, 2011). The 2010 Darfield earthquake damage was estimated to be 4 billion (Roome, 2010).

The 2010 Christchurch earthquake caused significant damage to the buried pipelines including the water, sewerage and storm water pipes. The total estimated damage in Christchurch and Waimakariri was $284 and $35.3 million, respectively. The 2010 earthquake in Christchurch caused $95 million damage on the water pipelines, $128 million on the wastewater pipelines and $61 million on the storm water pipelines (Gordon 2010). The damage on the water pipelines in Waimakariri District was reported $7 million, on the wastewater pipelines $22.5 million and on the storm water pipelines $5.8 million (Gordon, 2010).

Christchurch City was struck by the main earthquake and many aftershocks. The aftershocks caused moderate damage, compared to the main earthquake (Geonet Science, 2010). On the day of the main quake, 34 aftershocks were recorded with the magnitude of 4 to 4.9, while 87 aftershocks with the same magnitude were reported within 9 days after the main earthquake (Geonet Science, 2010).

## 8.2. The 2010 Darfield earthquake effects on the Christchurch wastewater system

The 2010 Christchurch earthquake affected the Christchurch wastewater system and Christchurch City as a whole. Its effects on Christchurch city can be divided into direct and indirect effects. Wastewater system failure in residential areas such as Avonside, Dallington and Bexley left 2,700 homes without a functioning sewer system. Restoration of the sewer system in the residential areas took more than 10 weeks in some cases (Mercer, 2010).

Tap water pollution, stream pollution, underground water pollution, soil pollution and increasing treatment costs were also effects of wastewater system failure due to the earthquake. The Christchurch earthquake caused blockages in the main sewer because of pipe collapse, which forced Christchurch City utility managers to discharge the flowing wastewater to the rivers with no treatment for 10 weeks (Harper, 2010). Public announcements were made warning the public for contaminated water. Figure 8.1 (a & b) shows the warning signs beside the contaminated river posted 3 weeks after the earthquake.

a b

Figure 8.1: River pollution after the Darfield earthquake in Christchurch (25 September 2010) (Source: Author's own)

The seismic force of the 2010 Christchurch earthquake caused underground breakages in the water and wastewater pipes. The wastewater due to pipes breakages spilled over the surrounding soil and other networks such as water pipelines. Even after the broken water pipelines had been repaired, tap water needed boiling for three minutes in regions where the wastewater pipelines were severely affected as a safety measure (Johnston, 2010).

Figure 8.2 (a) shows the damaged wastewater network to the nearby natural stream in Christchurch which contaminated the nearby stream. Figure 8.2 (b) in this figure shows the portable pumps used for transferring wastewater collected in the damaged manholes to the nearby natural streams in Christchurch city. It also shows the manhole uplift caused by the earthquake shocks.

a b

Figure 8.2: Direct discharge of wastewater to the stream 25 September 2010 Christchurch (Source: Author's own)

Figure 8.3 (a &b) shows the wastewater received in the reservoir of the wastewater pump stations being pumped in to the nearby river in Kaiapoi (Waimakariri District), because of severe damage in the main wastewater pump station. Figure 8.3 (a) shows how the earthquake forces have tilted the reservoir of the main wastewater pump stations. Figure 8.3 (b) shows the auxiliary pump installed beside the damaged pump station for transferring received wastewater. The received wastewater cannot be transferred because of the seismic damage on the mechanical equipment and connected pipelines.

Application of the auxiliary pumps in damaged pump stations not only is costly but it can also cause difficulties for the nearby buildings and affect traffic flow.

a b

Figure 8.3: Discharge of wastewater directly to the river due to WWPS failure (Kaiapoi 24 September 2010) (Source: Author's own)

Figure 8.4 (a &b) shows constraints portable pumps have on residential areas. The figure shows the installed pumps disrupt traffic flow and block access roads

a b

Photo 8.4: Application of series of mobile pumps in severely damaged region (Kaiapoi 24 September 2010) (Source: Author's own)

a b

Figure 8.5: Manhole uplift, Dallington, 25 September 2010 (Source: Author's own)

The earthquake damage from the 2010 Christchurch earthquake to wastewater pipelines caused collapse in the main and trunk pipes, and damaged connection pipes. Figure 8.5 (a &

b) shows the direct damage caused by the earthquake in the two components of the Christchurch wastewater network. Figure 8.5 (a) shows the damaged manhole in the main street while Figure 8.5 (b) shows the main sewer breakage which blocks the flow.

This earthquake caused damage to all parts of the wastewater system, not just pipelines. The Christchurch wastewater system served 150,000 households. The system had one main wastewater treatment plant and 5 auxiliary small treatment plants. It is comprised 1,600km of sewer mains and 950 km of laterals (Christchurch City Council, 2005). Domestic wastewater made 86% of the whole collected wastewater within the Christchurch region. The Christchurch wastewater network comprised 24,000 manholes and 91 wastewater pump stations (Christchurch City Council, 2010c).

There were 6 wastewater treatment plants in the Christchurch region, 5 of which were regional wastewater treatment plants and one main wastewater treatment plant. The main treatment plant in the Christchurch region was constructed in Bromley in 1988. Figure 8.6 shows the overall layout of the main wastewater treatment plant in Bromley (Christchurch), where the State Highway 74 passes though the treatment plant ponds.

Figure 8.6: The main wastewater treatment plant in Bromley, Christchurch (Google Map 2010)

The Christchurch WWTP was an advanced wastewater treatment plant comprised of three main treatment processes; the primary, secondary and tertiary treatment.

### 8.2.1. The earthquake effect on WWTPs in Christchurch

The Christchurch earthquake did not have significant effect on the wastewater treatment plants other than the Bromley site. In this section, only the earthquake effect on the Bromley treatment plant will be investigated.

The construction or upgrading dates of the WWTPs in the Christchurch region indicate all were constructed or upgraded in accordance with the recent building codes. According to the past earthquakes effects on wastewater system, the buildings and the equipment of WWTPs in Christchurch are expected to withstand earthquakes with annual probability of exceedance 1/500 (Zare et al., 2010).

Figure 8.7: The Christchurch wastewater treatment plant ponds (Google map)

Figure 8.7 shows the location of each treatment pond in the Christchurch wastewater treatment plant. The 2010 Christchurch earthquake affected the oxidation ponds and the pipes installed within the pond area such as the connecting pipelines between the ponds 2 and 3, and the bypass pipeline in the pond 1. Before the 2010 Christchurch earthquake, the treated wastewater was transferred to the pond 3 after passing ponds 1 and 2 through two 900mm diameter pipelines. The 2010 Christchurch earthquake damaged the under-road pipes which connected the treatment ponds on both sides of State Highway. Joint separations and pipe crushing in the joints and the body were the major damages inflicted on the pipes which connected the ponds. The connecting pipe failures created sink holes in the main road which closed state Highway 74 for two weeks. A subsequent pipe failure in the pond area bypass pipeline was also detected. Here, the joint separation and misalignment of the pipe were the main failures. Photo 8.8 shows the damage in the 900mm diameter concrete pipes after the 2010 Christchurch earthquake, which were replaced with new pipes. Joint separations, crushed joints by compression and pipe crushing in the body were the main types of damage in these pipes.

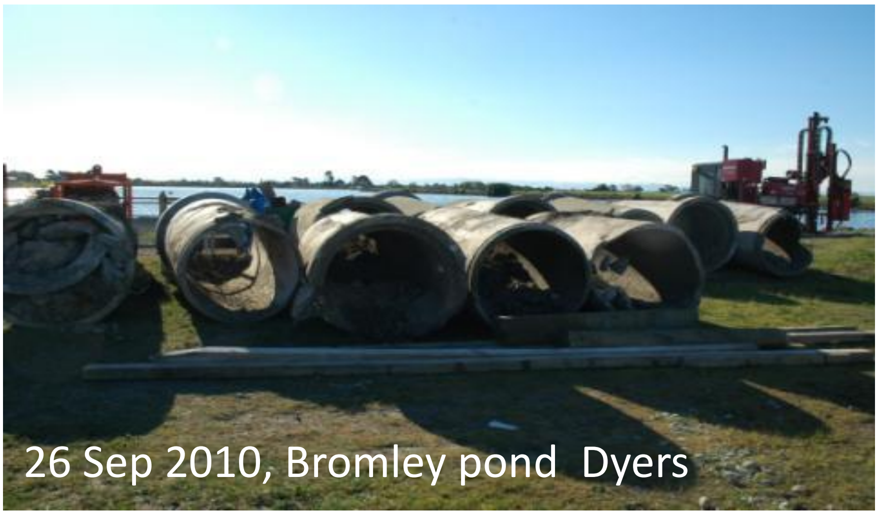


Photo 8.8: Connecting pipe damages in the Bromley pond (Source: Author's own)

Photo 8.9 shows a failure in the bypass pipeline in the Bromley WWTP in the pond 1 after the earthquake, here, the pipes are uplifted because of the earthquake forces and the pipes joints were separated.

Photo 8.9: Failure of bypass pipelines in the Christchurch WWTP (Source: Author's own)

The whole pipe replacement procedure took two weeks, which included preparation of the required pipes and materials. A delay was caused by preparing pipes in the North Island and shipping them to the damaged area. Preparing the required pipes took 10 days, and construction replacement procedure took 4 days to complete. Finally, the two 900 mm diameter pipelines were replaced with one 1,300mm diameter concrete pipeline. There was no particular pre-plan for replacing and repairing the bypass pipelines at the time the researcher visited the site.

A significant damage caused by this earthquake was on the ponds. After the earthquake, some stop banks slumped inside or outside the ponds, and caused severe cracks in particular points including State Highway 74. The dikes between the ponds 3 and 6, the ponds 4 and 5 and also the pond 1 and 2 were severely damaged. Figure 8.10 shows the damaged parts and repaired sections of the ponds in the Bromley treatment plant. In this figure the lines 1, 2 and 3 show the damaged stop banks, while line 3 shows the sheet piles line installed after the earthquake to protect the downstream ponds.

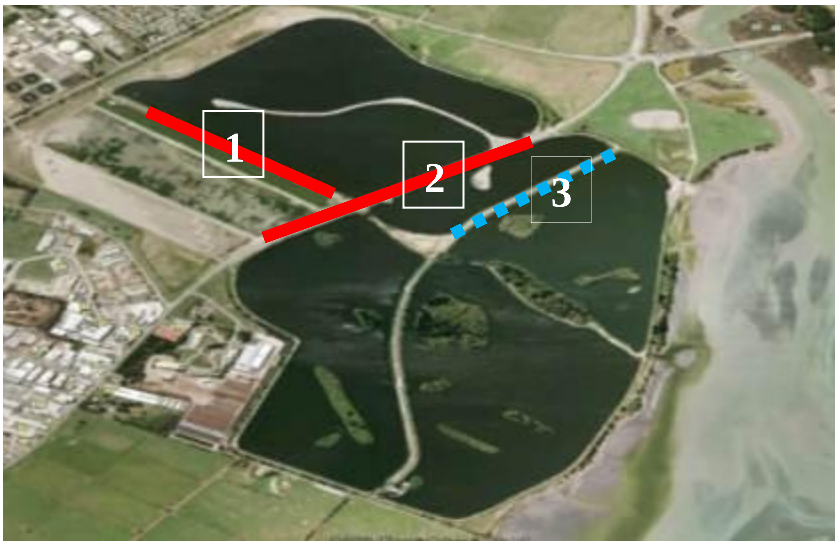


Figure 8.10: Damaged stop banks in the Christchurch WWTP ponds (Google map 2010)

After the earthquake, in order to decrease the risk of dike failure and to protect the downstream ponds, first, the water level in ponds 3 and 4 was decreased, and then two series of sheet piles with a length of 250 and 170 meter long were installed in the shared dikes 3-6 and 4-5, respectively. Some minor cracks were also seen near the recently constructed pump stations underneath pond 6. Photo 8.11 shows the sheet piles installed in the stop banks between pond 3 and 6. As indicated photo 8.11 due to the damage to the stop bank the bypass duct is blocked by sheet piles to prevent further damage.

Photo 8.11: Sheet piles installed in the Christchurch wastewater system (Source: Author's own)

Apart from direct damage in the ponds, a power outage affected the wastewater treatment plants (Gordon, 2010).

### 8.2.2. Wastewater pump station in Christchurch wastewater network

Damage caused by the Christchurch earthquake was also found at pump stations. Some of the wastewater pump stations, as the main components of the wastewater system, were affected by the earthquake, and some failed after the earthquake. Eighty percent of the Kaiapoi wastewater system failed, and all the WWPSs in Pines Beach and the Kairaki region also failed due to pump station damage.

The Christchurch wastewater system comprises of 123 wastewater pump stations which house 239 pumps (Christchurch City Council, 2010b). The WWPSs construction date is the main factor which can affect seismic vulnerability of WWPSs. According to the Christchurch GIS database, 39 % of all the WWPSs in Christchurch were built before 1976, and 15.4 % were built after 2004. Table 8.1 shows the construction dates of WWPSs in the Christchurch wastewater system.

Table 8.1: Wastewater pump station in the Christchurch wastewater network

| Date of Construction | Before 1935 | 1936<DC<1976 | 1977<DC<1993 | 1994<DC<2004 | After 2004 |
|---|---|---|---|---|---|
| **Number of WWPS** | **11** | **37** | **19** | **37** | **19** |
| **Percentage of WWPS** | **8.9** | **30.1** | **15.4** | **30.1** | **15.4** |

Availability of backup power generators was limited so wastewater pump stations were affected by the power outage. Portable power generators were installed to compensate with the power shortages. The power network was restored quickly after the earthquake, and power returned to the rest of the city on the same day, almost 12 hours after the earthquake. Power returned to 98% of the city two days after the earthquake, although Waimakariri District did not have power for several days after the earthquake, and thus the portable generators supplied power to wastewater pump stations.

The wastewater pump stations were affected in two distinct ways: the complete failures because of underground structure failure and temporary failures due to power outage. Approximately 90 % of all the WWPSs in Christchurch suffered temporary failures because of power outage. The power shortage and power network failures were more severe in Waimakariri District. For instance, 5 days after the earthquake, 350 homes still relied on power generators. Photo 8.12 shows the application of portable power generators in the wastewater pump stations. It should be noted that before using a power generator or returning a pump station to service, all types of earthquake-induced damage were checked including failures in reticulation pipeline connections to the pump station. Photo 8.12 indicates that even small and regional wastewater pump stations were affected by power outage and needed portable power generators.

Photo 8.12: Application of portable power generator after the earthquake in WWPS (Source: Author’s own)

Underground uplift was a common type of damage to the pump stations, especially ones located next to the river. For example, two wastewater pump stations were uplifted and failed near the Avon River in Christchurch. Photo 8.13 illustrates the uplifted wastewater pump stations in Avonside region in Christchurch after the earthquake. Figure 8.13 (a & b) shows the severity of the failure in wastewater pump stations.

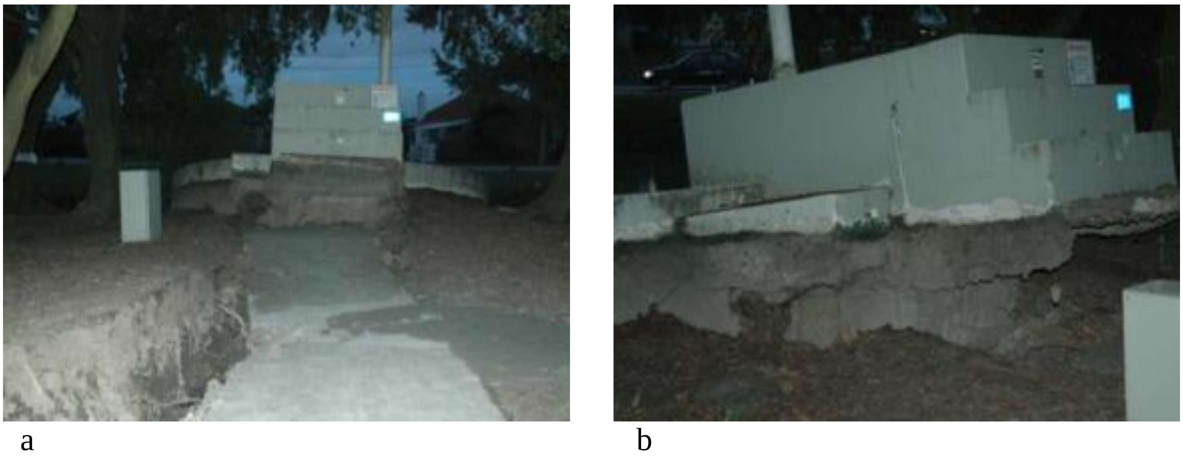


Figure 8.13: WWPS failure due to movement of underground structure (Source: Author’s own)

Photo 8.14 shows the slump of the river bank into the river after the earthquake in Chrsitchurch which caused the failure of the wastewater pump station.

Photo 8.14: WWPS failure due to underground structure replacement (Source: Author’s own)

Uplifting of wastewater pump station reservoirs is not restricted to WWPSs located beside the river banks. For instance, Figure 8.15(a & b) shows the uplift of the underground structures in two wastewater pump stations in Waimakariri District.

a b

Figure 8.15: Failure of underground strcuture in wastewater pumping stations in Waimakariri (Source: Author’s own)

In the earthquake affected regions in Christchurch or Waimakariri District, severity of damage in the WWPSs had a direct correlation with the distance from the riversides in the damaged WWPSs visited by the researcher. Failure and damage in the WWPSs buildings or underground structures decreased as the distance from the riversides increased. For instance, Figure 8.16 (a & b) shows less damage on the WWPSs located more than 100m away from the riversides. Figure 8.16 (a) shows a horizontal and vertical displacement of the underground structure, and Figure 8.16 (b) shows minor damage on the above ground structure in the WWPS in Waimakariri district (Kaiapoi).

a b

Figure 8.16: WWPS wastewater reservoir displacement (a) and above ground structure minor damages (b) in Waimakariri District (Kaiapoi 24 Sep. 2010) (Source: Author’s own)

The earthquake caused damage to the connected wastewater pipelines. Figure 8.17 (a & b) shows the pipeline replacement because of the severe damage to the pump station and the connected pipelines in Waimakariri District.

a b

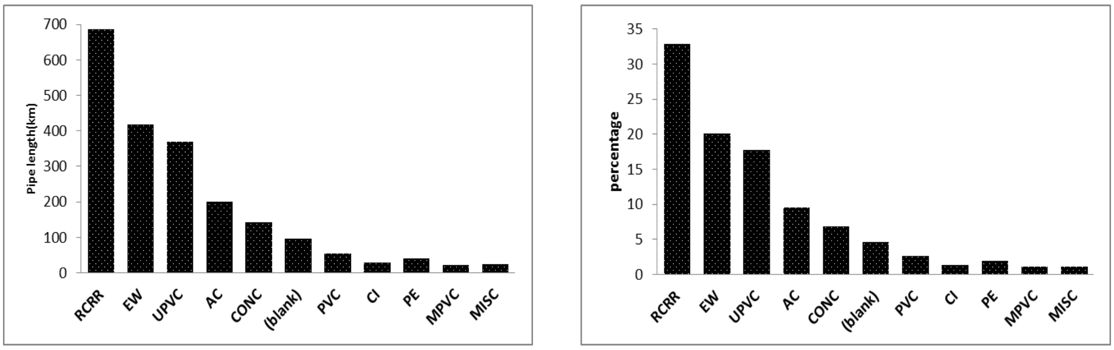
Figure 8.17: Main WWPS failure in Waimakariri District (Kaiapoi 24 September 2010) (Source: Author’s own)

The wastewater pump station failures in the regions affected by the 2010 Christchurch earthquake show how vulnerable the underground and above structures are.

### 8.2.3. The 2010 earthquake damage to the wastewater network in Christchurch

In this section the damage found in pipelines is discussed. The data source for the following analysis was the Christchurch GIS data base (2010), provided by the Christchurch City Council. Analysing the data source shows damage detail under different pipeline parameters.

According to the Christchurch GIS database (2010), here the wastewater network consisted of 30 types of different materials pipes. The wastewater pipelines in the Christchurch wastewater network consist of concrete, earthenware, asbestos, plastic, brick barrel and iron based pipe types. A small portion of the network was rehabilitated with rehabilitation methods such as cured in place. The plastic pipes in the Christchurch reticulation generally consisted of polyethylene and polyvinyl chloride pipes. Medium density polyethylene (100 and 80), high density polyethylene and corrugated polyethylene are the types of polyethylene pipes used in the Christchurch wastewater network. Reinforced concrete with rubber ring joints and concrete mortar pipes were also present. Iron based pipes comprise of cast iron, steel, ductile iron, wrought iron, concrete lined steel and concrete lined ductile iron pipes were in smaller quantities. Miscellaneous pipes comprise of any other type, and their percentage on length was less than 1% of the total length; Galvanised pipes, alkathene, acrylonitrile butadiene styrene, glass reinforced polymer, brick barrel, vertically cast concrete pipe, concrete lined iron, Novaflow and cured in place pipes comprised miscellaneous types of pipes. Figure 8.18 (a) illustrates the main types of pipes installed in the Christchurch wastewater network and their installed length as given by Christchurch GIS data base (2010). As illustrated, concrete, EW and UPVC pipes are the three main types of pipes in the Christchurch wastewater network, consisting of 685, 418 and 370 km of piping respectively.

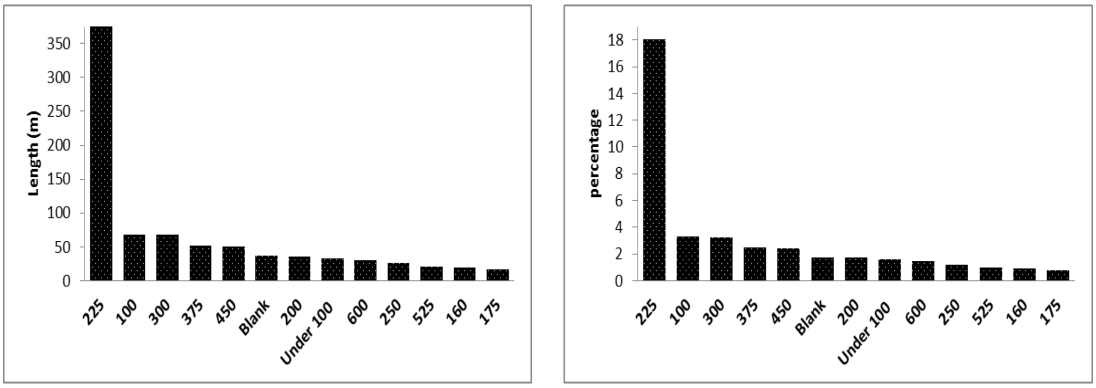


a b

Figure 8.18: Distribution of different pipe types graph (a) and percentage of each main pipe graph (b) in the Christchurch wastewater network (Compiled from Christchurch GIS data base (2010))

While Figure 8.18 (b) shows the percentage of each type of pipe used in the Christchurch wastewater network; that is, RCRR (32.9 %), EW (20.1 %) and UPVC (17.5 %) and AC pipes (9.6 %). As seen in this graph, only 4 types of pipes make 80% of the wastewater network, and the other 26 types make up the remaining 20% of the reticulation.

Analysing the GIS data base of the Christchurch wastewater network (2010) provide this information on various types of pipes in terms of the material and diameter. Diameters of the wastewater pipes in the network vary from 65mm to 2000mm. The 150mm pipes were predominant in the network, and were comprised of 56.2 % of the whole reticulation length (Christchurch GIS data base 2010). Figure 8.19 (a) shows the distribution of the most common diameters of pipes in the Christchurch wastewater network, except for the pipes with the 150mm diameter.

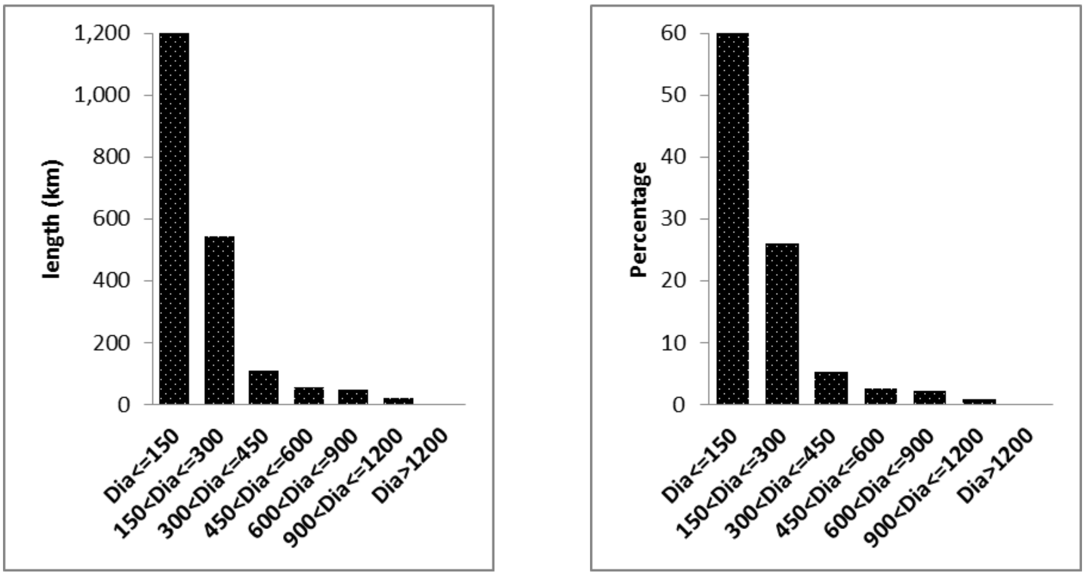


a b

Figure 8.19: Distribution of the most common pipe diameters graph (a) and percentage of sewer length versus pipe diameters (graph b) used in the Christchurch (Compiled from Christchurch GIS data base (2010))

Figure 8.19 (b) shows the percentage of each pipe length versus each pipe diameter; that is, the pipes with the 100, 225, 300 and 375mm diameters covered 29 % of the whole Christchurch wastewater network, whereas 85% of the Christchurch wastewater pipelines were pipes with the diameter less than 375mm.

If wastewater pipelines were classified into different categories in terms of the pipe diameter, as it can be seen in Figure 8.20, the pipes with the diameters less than or equal to 150mm, and with the diameter within 150 and 300mm were the most common types. Figure 8.20 (a) shows the length of each class of pipes in terms of the pipe diameter, and Figure 8.20 (b) describes the percentage of each class in the total sewer length.



a b

Figure 8.20: Categorised wastewater pipes in the Christchurch in terms of pipe diameter (Compiled from Christchurch GIS data base (2010))

The wastewater pipes in the wastewater network were laid at different depths, varying from close to the surface up to 5 meters deep. Figure 8.21 shows that the pipes were installed in depth of around 2m, and the most common buried depths were 1.8, 2.1 and 1.5m. As it is indicated here, except for the most common buried pipe depth, the total length of buried pipes for each depth was less than 80km.

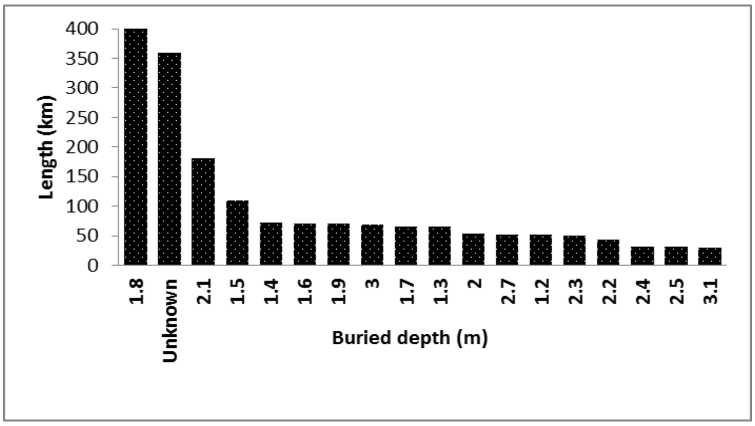
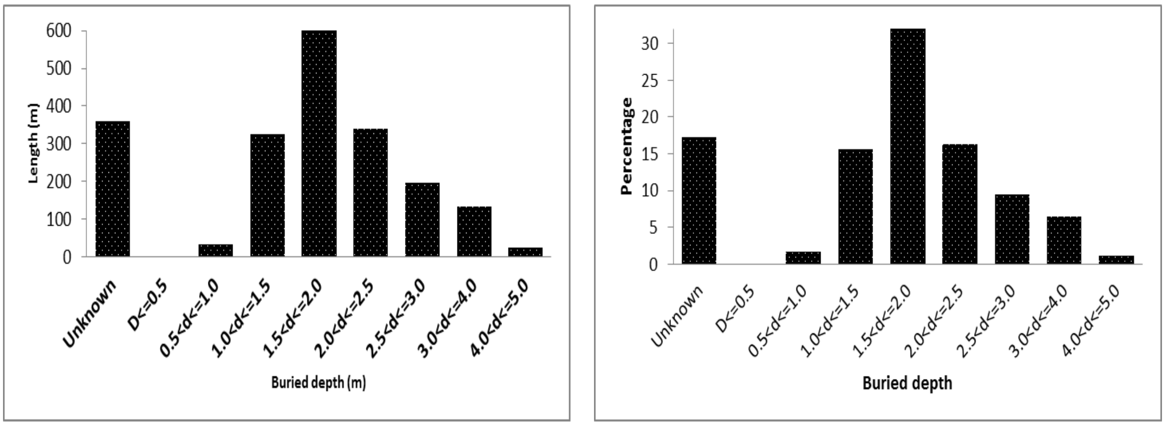


Figure 8.21: Distribution of sewer depths in the Christchurch WW network (Compiled from Christchurch GIS data base (2010))

The buried pipes depths in the Christchurch wastewater network were categorised into different depth ranges, as presented in Figure 8.22. Figure 8.22 (a) shows the length of wastewater pipes buried in different depth classes, and graph (b) shows the percentage of pipes laid in the different depth ranges.

Figure 8.22 (a & b) also shows that 64% of the pipes (1250 km) were installed in depth between 1 and 2.5m, but there is no adequate data in relation to the buried depth of 17.3% of 1250 km pipes previously installed in the Christchurch wastewater network.

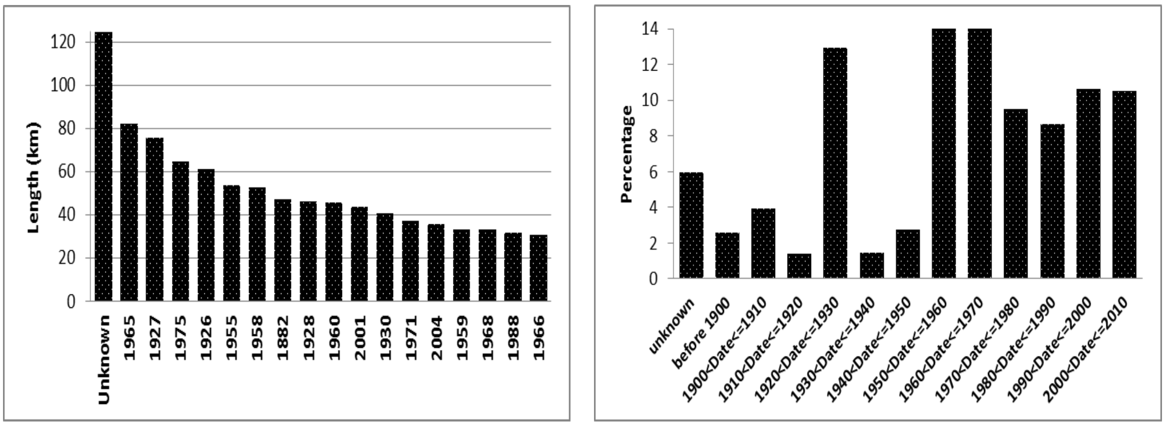


a b

Figure 8.22: Buried depth in the Christchurch sewer reticulation, sewer length (graph (a)) and percentage of total length (graph (b)) (Compiled from Christchurch GIS data base (2010))

In terms of time and repair cost, it should be noted that by increasing buried depth the required time and cost of repair can increase markedly. Therefore, this type of defect in pipes buried at increased depth can be more costly and require more time to repair compared to defects at less depth.

Wastewater network in every city is usually developed with the development of the city, and the Christchurch wastewater network is no exception. Analysing the Christchurch GIS data base (2010) shows the total length of the installed pipes during the development of wastewater network in Christchurch (see Figure 8.23). Figure 8.23 (a) shows that the maximum development of the Christchurch sewers was made in 1965 with more than 80 km (Christchurch City Council, 2010a). This graph further indicates that the maximum length of pipes were installed in the years 1965, 1927, 1975 and 1926 with the length greater than 60 km for each year.



a b

Figure 8.23: Installed sewers in Christchurch (graph (a)) and percentage of total constructed sewers in Christchurch (graph (b)) (Compiled from Christchurch GIS data base 2010)

According to the same figure approximately 69% of the Christchurch wastewater network was installed after 1950 and 31% before 1950. More than one third of the network was constructed between 1920 and 1930. Figure 8.23 (b) therefore indicates that about 46 % of the sewers in Christchurch were installed more than 50 years ago.

## 8.3. The Christchurch earthquake effects on the wastewater pipelines

Compared to other infrastructure in Christchurch, the wastewater network in Christchurch was severely affected by the Christchurch earthquake (Gordon, 2010). In the context of the pipeline damage 1273 repair points were reported to the Christchurch City Council by 20

October 2010 about 7 weeks after the earthquake occurred. The 2010 Christchurch earthquake damaged almost all types of pipes, irrespective of its material, diameter, depth and age. Photo 8.24 shows the damage in the main sewer in Christchurch, and the damaged pipes that are completely blocked and filled with surrounded soil.

Photo 8.24: Main sewer breakages in Avonside, 25 September 2010 (Source: Author's own)

Due to a failure in the pipe connections residential buildings were disconnected from the main sewer network (Zare, 2010b). For instance, the surrounding soil was washed into the collector pipes from the damaged connections, and the continuous groundwater flow was infiltrating the network before the connection was repaired (Zare, 2010b). Figure 8.25 (a) shows the damaged connection pipes in Waimakariri District by the earthquake and Figure 8.25 (b) demonstrates how the brittle pipes used in connection failed after the earthquake.

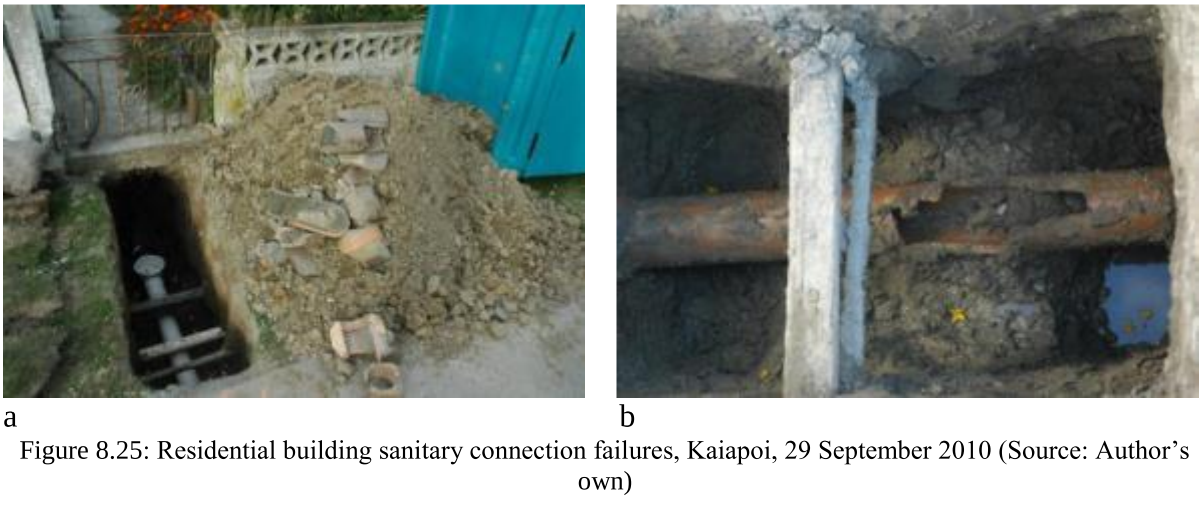


Figure 8.25: Residential building sanitary connection failures, Kaiapoi, 29 September 2010 (Source: Author's own)

The 2010 Christchurch earthquake damaged wastewater network in many regions in Christchurch City and Waimakariri District. Due to the extent of the damage, instead of point repairs, the whole damaged wastewater pipelines were replaced with new pipelines. In such a situation, damaged pipes are not removed, and new pipelines are laid next to them. The replacement of the whole pipeline due to damage in the old pipeline is shown in Photo 8.26.

Photo 8.26: Replacement of the whole wastewater pipeline, Avonside 25 September 2010 (Source: Author's own)

In this section the GIS data base is used for calculating and categorizing the earthquake damage on the Christchurch wastewater network. The dada based is developed by the Christchurch City Council after the 2010 earthquake in the city. The earthquake induced damage in the Christchurch wastewater pipelines is classified in terms of the pipe type, diameter, age and the buried depth. This classification provides an estimation of the most vulnerable type of pipes. Figure 8.27 (a) shows the most damaged pipes in the network in terms of the pipe material that were of the reinforced concrete with rubber ring joint (RCRR), the UPVC, the EW, the concrete (CONC) and the AC pipe types.

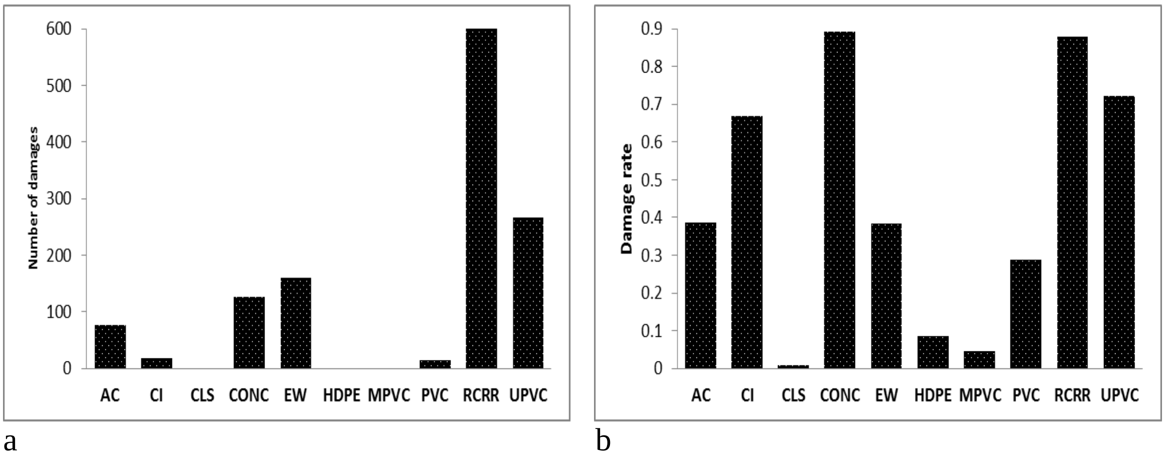
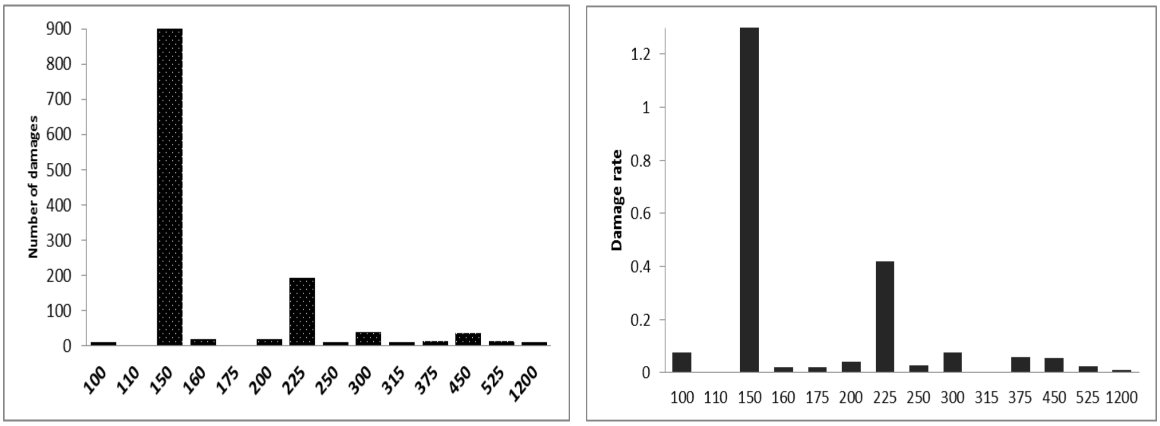


Figure 8.27: Distribution of damage level (graph (a)) and damage rates in the Christchurch wastewater network (Compiled from Christchurch GIS damage data base (2010))

The damage rate is a well-known factor which can be used for a better illustration of damage in different types of buried pipelines. The damage rate of each type of pipe is obtained by calculating the amount of damage in each type of pipe divided by the total length of each type of pipe. The RCRR, UPVC, EW, CONC, AC and CI pipes had the highest damage rates. The calculated damage rates from the GIS data base of damage in Christchurch wastewater pipeline (2010) are shown in Figure 8.27 (b) for pipe materials. This graph further indicates that the concrete pipes had the highest damage rate compared to the RCRR, UPVC and EW pipes. The RCRR pipes damage rate is shown as the second highest damage rate in the Christchurch pipes. The comparison of damage rate in concrete pipes shows that the damage rate in reinforced concrete pipes and concrete pipes was almost the same. Figure 8.27 (b) in Figure 8.27 shows the differences between damage rates in three types of PVC pipes. Different type of pipes even with the same base material can behave differently in an earthquake. For instance, damage rate in UPVC pipes are 2.5 times more than that of the PVC pipes and damage rates in PVC pipes in 6.4 times more than that of the MPVC pipes.

Figure 8.28 (a) describes the distribution of earthquake induced damage in the Christchurch wastewater network, with respect to the 150mm diameter pipes that are the most vulnerable type of pipes in terms of the pipe diameter. This graph shows the number of defects in pipes with 150 mm diameter compared to other pipes.

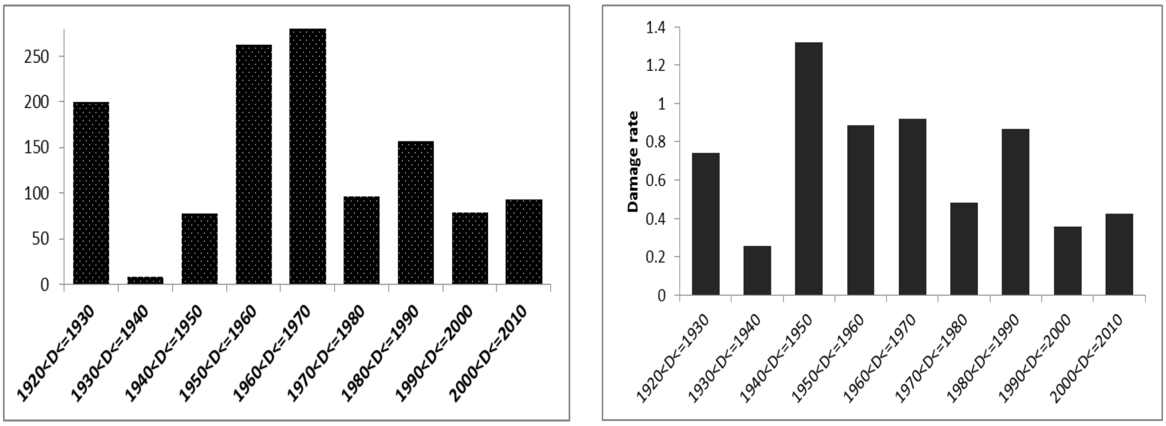


a b

Figure 8.28: Damage distribution on the Christchurch pipes in terms of the pipe diameter (mm) (graph (a)) and date of construction graph (b) (Compiled from Christchurch GIS damage data base (2010))

Figure 8.28 (b) shows distribution of damage rate in the Christchurch damaged pipes; that is, the damage rate in the 100mm diameter pipes is greater than that of the 160. 175, 200, 250, 375,525 and 1200mm diameter pipes, and the damage rate in the 300mm diameter pipes is greater than that of the 375,450, 525 and 1200mm diameter pipes. Figure 8.28 (b) further demonstrates that the pipes with 150 and 225 mm diameters have the maximum damage rates compared to other pipe diameters, although, damage rates in 150mm pipes are approximately three times more than damage rate in 225mm pipes.

Figure 8.29 (a & b) shows the numbers of defects and damage rates in pipes with different service lives in graph respectively. Figure 8.29 (a) illustrates that the Christchurch pipelines constructed between 1960 and 1970 suffered the maximum amount of damage after the earthquake. On the other hand, the pipes installed between 1930 and 1940 resist seismic shocks even better than pipes installed recently (between 2000 and 2010). Comparison of seismic damage in pipes with different service life indicates that installations in 1958, 1930, 1962, 1955 and 1965 recorded the most frequent damages

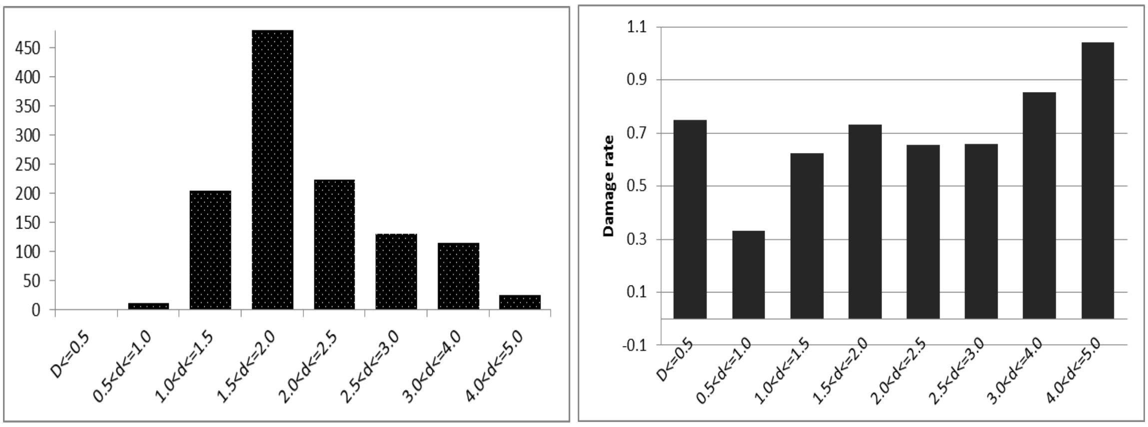


a b

Figure 8.29: Eearthquake induced damage on the Christchurch wastewater network installed in different decades; numbers of damage in graph (a) and damage rates in graph (b), (Compiled from Christchurch GIS damage data base (2010))

The numbers of observed damage and damage rates in different burial depths are shown in Figure 8.30 (a &b). Figure 8.30 (a) shows that the maximum number of defetcs occures in pipes which were installed in depth of 1.5 to 2.0 meters. This graph also shows that the number of damage decreses in pipes buried between 1.5 and 5 meters. This graphs shows that the numbers of damage in pipes buried between 4.5 and 5 meters are much less than other pipes, however, in terms of cost and time required for repair, this number of damages are notable comapred with others.

Figure 8.30 (b) indicates that the pipes installed between 4 and 5 meters experienced severe damage compared to other pipes installed in lower depths. This graph also shows that the earthquake caused the minumum damage in pipes installed between 0.5 and 1 meter.



a b

Figure 8.30: Earthquake induced damage on the Christchurch wastewater network installed in different depths decades; numbers of damage in graph (a) and damage rates in graph (b), (Compiled from Christchurch GIS damage data base (2010))

## 8.4. Calculation of seismic effects on the Christchurch wastewater network

The Christchurch wastewater network similar to other three case studies is divided into 10 zones in order to make damage rate calculating accurate on different types of pipes and soil types in each zone. Here the main roads and main collector pipelines are used as boundaries. Each zone almost covers one particular zone in Christchurch. Figure 8.31, shows the different zones in Christchurch which are used to calculate seismic effect.

Figure 8.31 Christchurch zones (extacted from google earth 2011)

As mentioned in Chapter 2, pipe specifications can impact seismic vulnerability. As a result pipe specifications in each zone are extracted from available data. Required data to calculate seismic damage in each zone is extracted from the GIS data base. Figure 8.32 reveals the distribution of brittle and ductile pipe in each zone of the Christchurch wastewater network.

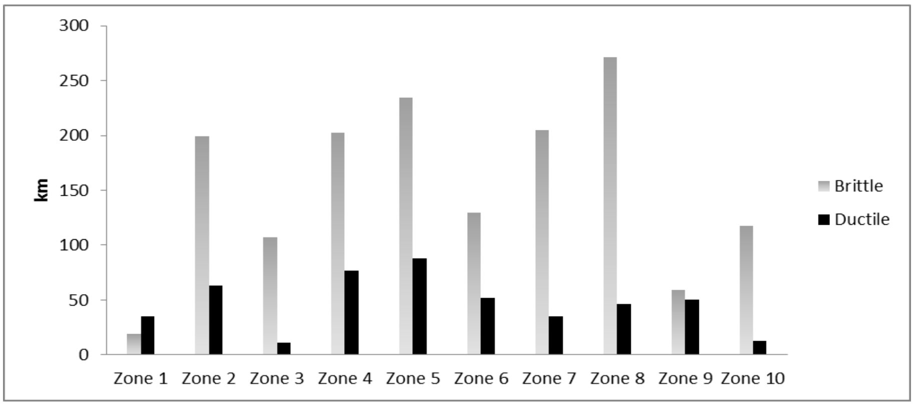


Figure 8.32: Distribution of brittle and ductile pipes in the Christchurch wastewater network (extracted from Christchurch City Council data base (2010))

The same procedure used in calculating the seismic damage on the wastewater networks of Hutt City, Gisborne and Blenheim, is applied in this case study as well. Figure 8.33 shows the expected wave propagation damage in each one of the 10 zones in the Christchurch wastewater network caused by earthquakes with annual probability of exceedance 1/500.

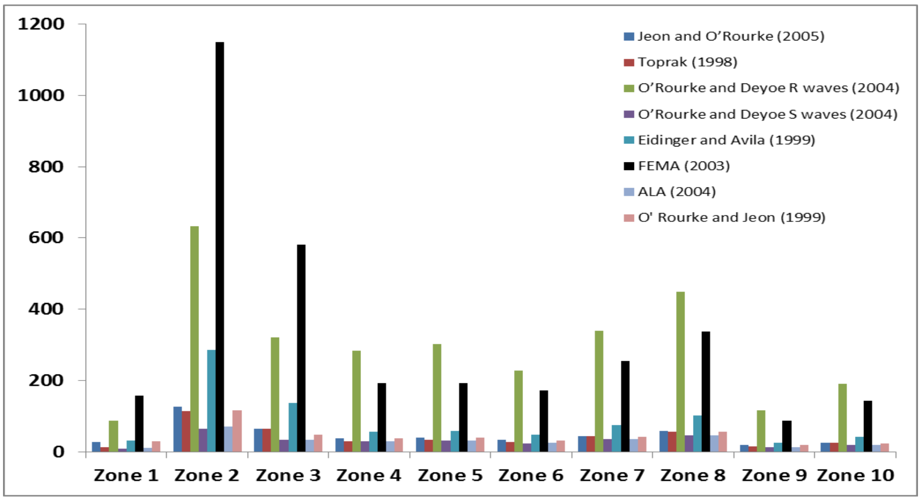


Figure 8.33: Seismic damage in the Christchurch wastewater network (earthquakes with annual probability of exceedance 1/500)

Figure 8.33 shows that Zone 2 is the most and Zone 9 is the least vulnerable among all zones. The maximum estimated number of defects is Zone 2 is 1149 calculated by the FEMA's fragility curve (2003) and the minimum number of defects in Zone 2 with 65 defects is calculated by the S-wave equation suggested by O'Rourke and Deyoe (2004). The maximum defects number in Zone 9 as the least vulnerable zone is 116 calculated by the R-wave equation suggested by O'Rourke and Deyoe (2004) and the minimum expected damage is 12 estimated using the S-wave equation given by O'Rourke and Deyoe (2004).

Figure 8.34 shows the wave propagation effects of earthquakes with annual probability of exceedance 1/1000 on each zone of the Christchurch wastewater network. Figure 8.34 shows that Zone 2 is the most and Zone 9 is the least vulnerable zone among all zones in Christchurch. The maximum estimated number of defects in Zone 2 is 2073 calculated by the FEMA's fragility curve (2003) and the minimum number of defects in Zone 2 with 83 defects is calculated by the S-wave equation of O'Rourke and Deyoe (2004). The maximum defects in Zone 9 as the least vulnerable zone is 139, calculated by the R-wave equation of O'Rourke and Deyoe (2004) and the minimum expected damage is 14, as estimated by the S-wave equation of the same researchers.

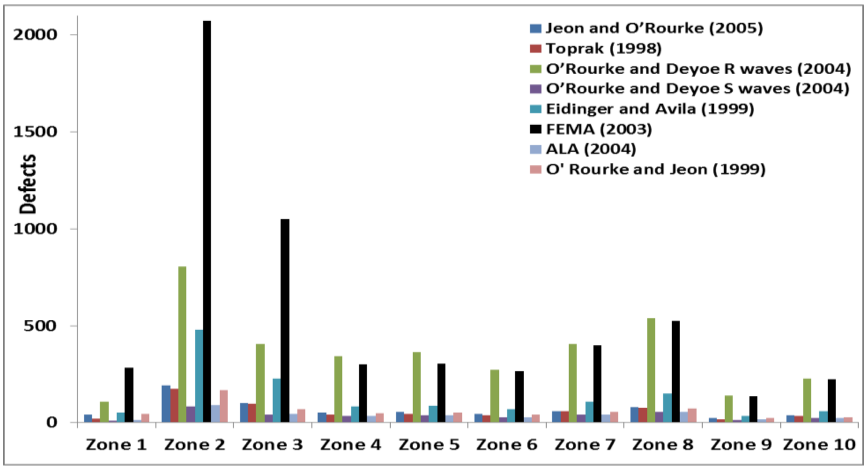


Figure 8.34: Seismic damage in the Christchurch wastewater network (earthquakes with annual probability of exceedance 1/1000)

The Christchurch City Council collected all the damaged points in the wastewater network and made the GIS data sources after the 2010 earthquake. This GIS data source is used to identify the damaged points in each zone. The data base was updated on a daily basis by the Christchurch City Council and the data set used here is updated up to September 30, 2010.

The total number of seismic defects in the Christchurch wastewater network is distributed into the 10 zones according to location of each defect. This distribution provides an opportunity to compare the calculated defects to the detected defects in each zone. Figure 8.35 shows Zone 2 suffered the maximum damage in 809 points.

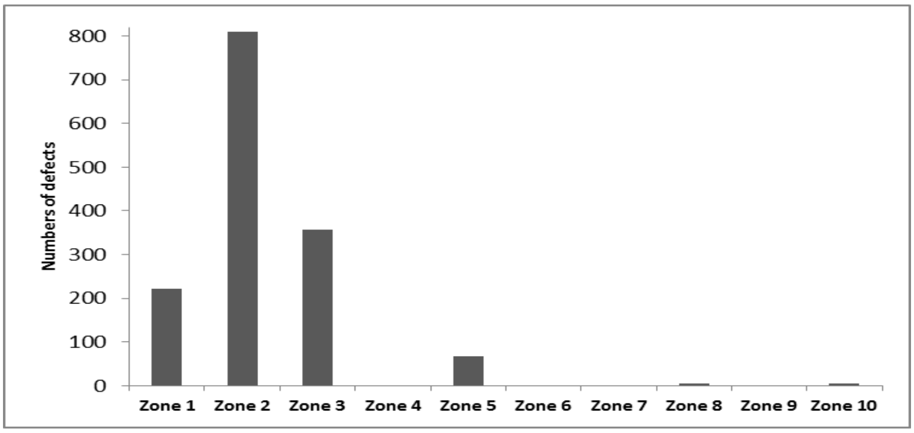


Figure 8.35: Distribution of damage in 10 zones (extracted from the christchurch wastewater network damage data base (2010))

Comparison of the detected and calculated damage in each zone indicates both methods estimate the same zones as the most and least vulnerable zones. Figure 8.35 shows that zones 2, 3, 1 and 5 experienced more damage than that of the zones 4, 6, 7 and 9.

Comparison of the calculated and recorded defects in the Christchurch wastewater network shows numbers of defects in some regions such as 4, 6, 7 and 9 are less than calculated defects in each zone. As explained in this chapter as well as Chapter 2, inspection of defects in wastewater pipelines is costly and time consuming due to the especial characteristics of wastewater network. The applied data base used to show the detected damages do not cover inspected defects after August 20th 2010. On the other hand, number of detected defects in some regions can be more than the calculated defects, because these numbers can be summation of wave propagation and liquefaction defects.

## 8.5. Correlation of the seismic damage to the earthquake parameters

Reviewing the literature revealed that the seismic damage on buried pipelines can be inflicted by two seismic factors: wave propagation and permanent ground deformation. As it was explained, wave propagation is the main cause of damage on the buried pipeline in most earthquakes.

Chapter 4, 5, 6 and also section 8.4 of this chapter showed and explained the seismic vulnerability of wastewater network in Hutt City, Gisborne, Blenheim and Christchurch respectively. The seismic vulnerability analysis showed that there are significant differences in the calculated defects when the available fragility curves are applied to calculate seismic damage. The previous chapters presented some explanations on the differences.

The significant difference in calculating the seismic damage in wastewater pipelines has encouraged the author to compare the real seismic damage with the calculated damage. In Section 8.4 the calculated damage was compared to real damage after dividing the city into different key areas. The result shows that the calculated seismic damage cannot show the real damage distribution within the city. It should be emphasised that the data relevant to the wastewater pipe damage was collected within a month after the earthquake and the number of detected defects had to increase by progressing the network assessment.

The analysis of seismic damage on wastewater buried pipelines not only indicated the significant differences within the calculated defects but also the lack of appropriate match among the calculated seismic defects and the real seismic damage. As explained in Chapter 2 the PGV is the main earthquake parameter used in calculating the wave propagation effects in the available fragility curves. According to these curves an increase in the PGV value results in higher damage rate in buried pipelines.

### 8.5.1. Analysis process

Here the real damage inflicted on the wastewater pipelines is correlated to the PGV in order to show how accurate is the trend of damage rate and PGV presented by the fragility curves. Is the application of the available fragility curves an appropriate tool in investigating the damage in wastewater pipelines in New Zealand? This is one of the main questions in this thesis and the adequate answer of which is sought for in this study.

In order to show the correlation between earthquake parameters and damage rate after the Darfield earthquake in Christchurch the provided GIS data by the Christchurch City Council was applied. The GIS database provides information required for analysis regarding the location of damaged pipes and the required information in the damaged points on the wastewater network. GNS science provided the earthquake relevant parameters. The GNS science data provides required information collected by the seismographs around the New Zealand especially for the Canterbury region after the Christchurch earthquake in 2010.

According to Chapter 2, the researchers have recommended some equations for demonstrating the correlation between PGV and damage rate in buried pipelines in the available fragility curves. The fragility curves used in this study show that the higher PGV higher the damage rate. As a result, damage rate versus PGV should show a similar trend. In order to find and extract the trend after the earthquake in Christchurch the damage rate in different PGVs are calculated.

Here the first step is to divide Christchurch City into areas with similar PGVs. As a result, the earthquake parameters extracted from the measurement points after the earthquake in 2010 are used to define the areas with similar earthquake parameters. For instance, Canterbury area including Christchurch City was divided into 20 regions, each region with the particular PGV value. Zhao 2010 form GNS science provided earthquake parameters for 86 sites, where seismographs were installed. The seismographs are located at about 10 km to 350 km away from the epicentre. In each site, type of soil, distance from epicentre, horizontal and vertical components of PGA, PGV and PGD are extracted.

The PGV available information on each site is added into the GIS data in order to define areas with similar PGV value. As mentioned before the ARCMAP software is used for drawing and analysing the damage caused on the Christchurch wastewater network. Interpolating PGV value for the sites within or in the vicinity of Christchurch defines the areas with the same PGV value. The number of damages in the wastewater network located in each area is calculated by dividing the number of damage to the length of network in each area.

As a result, damage rate and PGV value for each area is determined. If the damage rate and PGV value for each region are drawn in a graph, the extracted trend can be compared to the previous works.

Figure 8.36 shows the damage rate in areas with various PGV. This graph shows that no particular trend is found to show the correlation between PGV and damage rate. The damage rate varies from the maximum of 2.5 to about 0.2 in the Christchurch wastewater network. The maximum damage rate was seen in PGV a 460 cm/s whereas the minimum damage rate occurred in PGV at about 525 cm/s, and this indicates that the minimum damage rate occurred at the maximum PGV.

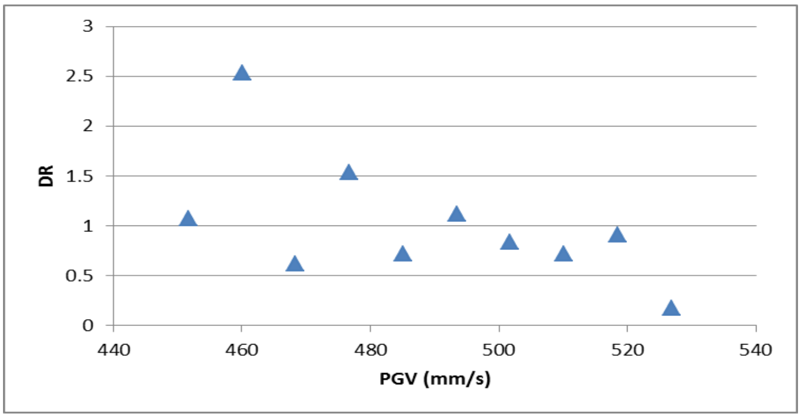
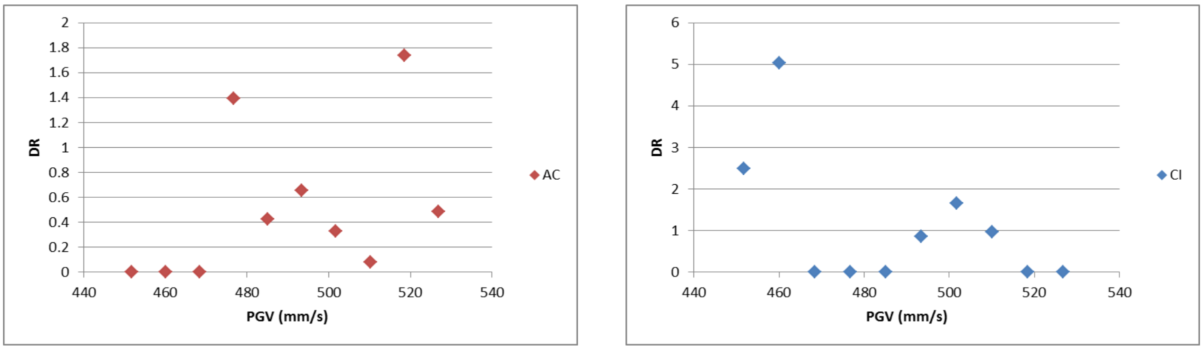


Figure 8.36: Total damage rate in the 2010 Christchurch earthquake

Reviwing the literature and the previous chapters showed pipe material can affect seismic vulnerability. As a result, the damage rate for each type of pipe is extracted for different PGV. Figure 8.37, 8.38 and 8.39 show the distribution of damage rate and PGV for different type of pipes.

Figure 8.37 (a) shows there is no dmage on asbestos cement pipes in PGV less than about 480 cm/s and in this range the AC pipes experinecd the secund highest damage rate at 1.4 per kilometer in the network. By incresing the value of PGV damage rate decreases to a minimum of about 0.007 in PGV at 510 cm/s. The damage rate in PGV of about 520 cm/s is the highest damage rate, about 1.8 while again the increasing PGV lead to decrease in the damage rate to about 0.5 defect per kilometer.

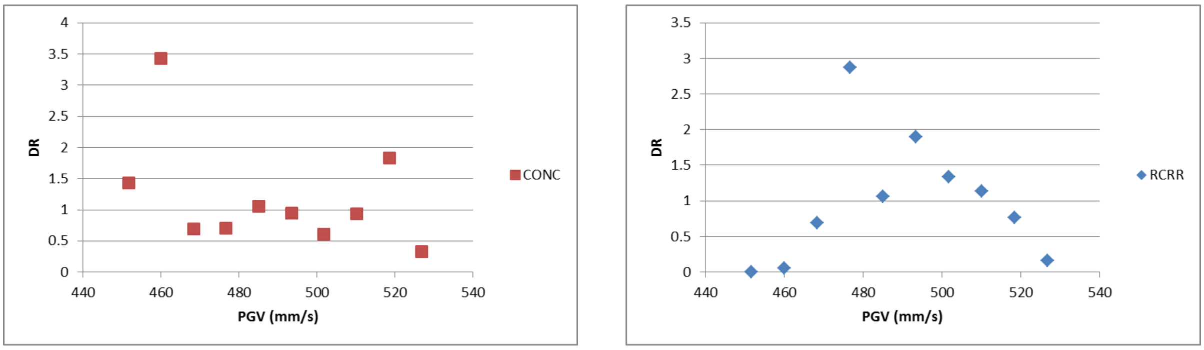
While Figure 8.37 (b) shows that no particular trend is followed for CI pipes. Comparison of the two graphs in this figure shows that damage inflicted to CI pipes was before that of the AC pipes and the PGV of 450 cm/s can cause damage rate of about 2.5, which is higher than the highest damage rate in AC pipes. The highest damage rate in CI pipes was recorded 5 in PGV of 460 cm/s.



a b

Figure 8.37: Damage rates in AC (graph (a)) and CI pipes (graph (b)) versus PGV

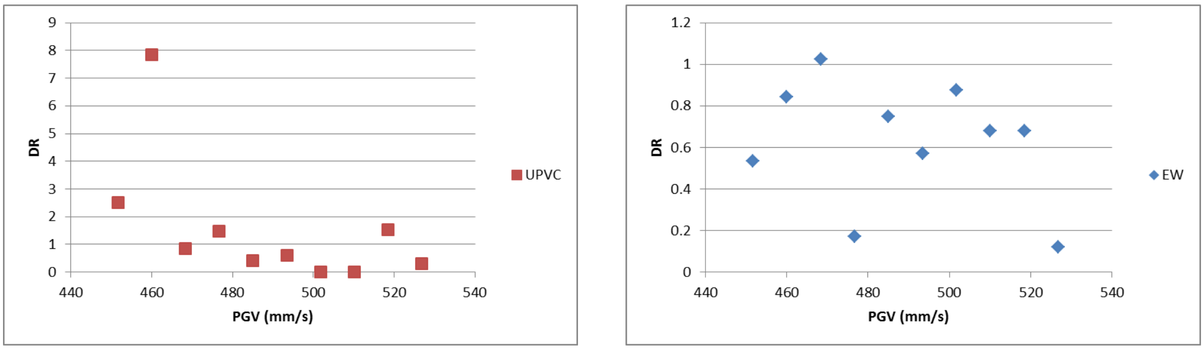
The trend between damage rates and PGV in concrete and reinforced concerete pipes are illustrated in Figure 8.38 (a &b). The figure shows that there exists no particular trend to show a correlation between damage rate and PGV; however, Figure 8.37 (b) shows that a increase in PGV leads to decrease in damagae rate in PGV more than 490 cm/s which is completely different from what has been reported by the researchers to date.



a b

Figure 8.38: Comparison of damage rates after the 2010 Christchurch earthquake in concrete pipes on the left and in reinforced concrete pipes on the right

Comparison of the trends in Figure 8.39 shows that both the UPVC and EW pipes are vulnerable to earthquake at PGV greater than 450 mm/sec. Figure 8.39 (a) shows the maximum damage rate of about 8 at PGV of 460 mm/s, while Figure 8.39 (b) illustrates the maximum damage rate of about 1 in EW pipes. Both the graphs cannot provide any specific trend to show how an increase or decrease of PGV could affect damage rates.



a b

Figure 8.39: Comparison of damage rates after the 2010 Christchurch earthquake in Unplasticised Polyvinyl Chloride pipes on the left and Earthern Ware pipes on the right

Comparison of the damage rate in various types of pipes shows that there exists no appropriate trend to show a correlation among PGV and damage rate in the wastewater network after the 2010 Christchurch earthquake.

Reviewing the literature indicated that, although the wave propagations effects of seismic shocks constitute the bases for seismic damage in buried pipelines, permanent ground deformation somewhere can cause significant damage.

The previous section showed that there exists no specific correlation to show damage rate at PGV in the Christchurch earthquake. According to Chapter 2, the seismic damage in buried pipelines can probably be affected by permanent ground deformation, and ground deformation can show damage rate; stronger ground deformation cause greater damage rate.

PGD was subjected to the same procedure as that of the PGV to show the correlation between the damage rate and permanent ground deformation. Areas with similar ground deformation were recognised and for each area the length, numbers of defects and specification of each damage point were classified. Finally, damage rate and relevant PGD were plotted in a graph to show the correlation between damage rate and permanent ground deformation.

Figure 8.40 shows that no seismic damage is reported in PGD of 150 mm on the Christchurch wastewater network. While, the maximum damage rate is 4.6 caused by a PGD of 400mm. The damage rates reported for PGD greater than 460 mm are less than damage rate reported for a PGD of 400mm. The graph generally shows there exists no specific correction between PDG and damage rate in the wastewater network after the 2010 earthquake in Christchurch

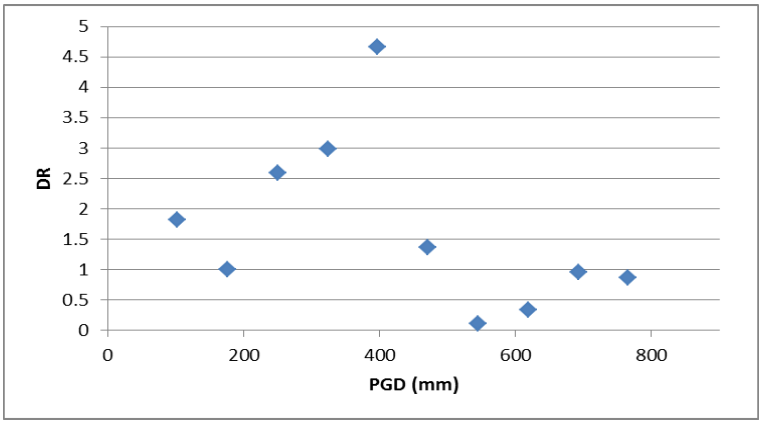


Figure 8.40: Peak Ground Deformation and damage rate in the 2010 Christchurch earthquake

## 8.6. Rehabilitation process for wastewater networks in New Zealand

The expected damage discussed in chapters 4, 5 and 6 and the demonstrated damage from the 2010 Christchurch earthquake in Chapter 7 raise the question of how to mitigate and manage damage? This section describes the process of pipeline rehabilitation following the 2010 Christchurch earthquake

Recognizing the various types of failures and applying the most appropriate rehabilitation methods is required after an earthquake. The rehabilitation procedure for unpressurised pipelines affected by an earthquake is different from the normal rehabilitation procedure. Restoration of affected wastewater pipelines after an earthquake is of concern of city councils and private companies who run the wastewater network.

Rehabilitation of wastewater pipelines is complicated, time consuming and costly. Wastewater pipelines are connected to each other in series; therefore, a failure affects a large area, especially if the main pipeline fails. Wastewater pipelines consist of various types of pipes in terms of material, diameter, joints, corrosion rate, and age. Likewise, wastewater pipes are laid in different depths of different soil types.

The first step in rehabilitation of wastewater pipelines is the prioritization, is identifying the critical areas such as the pipe passing under highways, main roads, bridges and critical infrastructure facilities. For example, the failure of the effluent pipes in the Christchurch wastewater treatment plant damaged the main highway, which had to be closed for two weeks. The second priority is determining the capacity of each wastewater pipeline. Typically, the critical pipes are trunk and main wastewater pipelines which carry a high volume of wastewater and thus require rapid repair.

Large diameter pipes are usually built using higher quality material than that of small diameter pipes due to their expected high carrying capacity and installation criteria in transferring wastewater.

### 8.6.1. Sewage pipe post-earthquake rehabilitation methods:

Different methods of wastewater pipelines repair and rehabilitation were demonstrated after the 2010 Christchurch earthquake. The 2010 Christchurch earthquake thus provided a unique opportunity to assess different wastewater rehabilitation methods used for different pipe types.

Open-cut methods are common in pipe installation, repair and rehabilitation. Soil excavation is the main operation in open-cut methods, where depending on the underground conditions, different excavation methods can be applied.

Trench excavations in open-cut methods can be categorised as excavation below the groundwater level, and excavation above the groundwater level in regions with high groundwater table (Abu Dhabi water and electricity authority, 2005).

### 8.6.1.1. Open-cut rehabilitation and repair

In excavation of a pipe trench, when the bottom of the trench is above the groundwater level, the trench can be directly excavated with appropriate machinery with a stable soil slope. The side slopes of the trench can vary from a vertical, in a very stable environment, such as rock, to a very wide slope, in loose sand. For rehabilitation and repair of buried pipelines by open-cut methods, ordinary exaction machinery is usually used after determining the pipe depth and nearby infrastructure facilities which may be affected by excavation. The main concern with open-cut methods is excavation nearby buried infrastructures such as gas pipelines (Government of Western Australia, 2005).

Before excavation the nearby buried infrastructure and pipe specifications should be derived from available utilities maps, although sometimes there are no available maps, or the maps are not accurate enough for the excavation operation (Government of Western Australia, 2005). Consequently, detection equipment should be used to detect any buried infrastructures such as water and gas pipe, power cables, communication cables, etc. (Government of Western Australia, 2005).

The significant and most costly parts of excavation in earthquake affected areas with loose sand and a high groundwater level are dewatering and stabilization of the excavated trench (Najafi, 2005). Groundwater control or dewatering in trench excavation is the main issue in excavation of the below groundwater table. Consequently, the groundwater level should be controlled carefully before excavation, and should always be kept below the trench bottom especially in point repairs (Department of Defense United States of America, 2004).

Most damaged pipelines were located in areas with loose sand with high groundwater table and supporting device is required during repair process.  Consequently, sheet piles and steel boxes were applied as Method 1 and 2 for repairing seismic damage in wastewater pipes.

In both methods when applied below the water table, dewatering is required. Application of drainage rods is the applied method used for dewatering of trench during the repair process. Dewatering trench by drainage rods comprises of some steps; first drainage rods are drilled on both sides of a damaged pipe to an adequate distance required for excavation and the repair process. Each drainage rod is connected to a drain collector which is located on each side of the trench. Both drain collectors are connected to a mobile pump, and when the pump is operated , the underground is drained into the drain collector, and the water table is decreased locally around the repair point. Dewatering procedure is almost the same in both open-cut excavation methods. When underground water level decreases to a proper level on both sides, underground pipes will be exposed for repair.

Application of open-trench methods in regions with a high groundwater level is restricted to dewatering and groundwater controlling. The dewatering procedure in open-cut methods is a continuous operation that should start before excavation and continue after pipe installation especially in the soft soil. Figure 8.41(a) shows the sucker rods used for dewatering, and Figure 8.41 (b) describes the sucker rods installed before excavation.

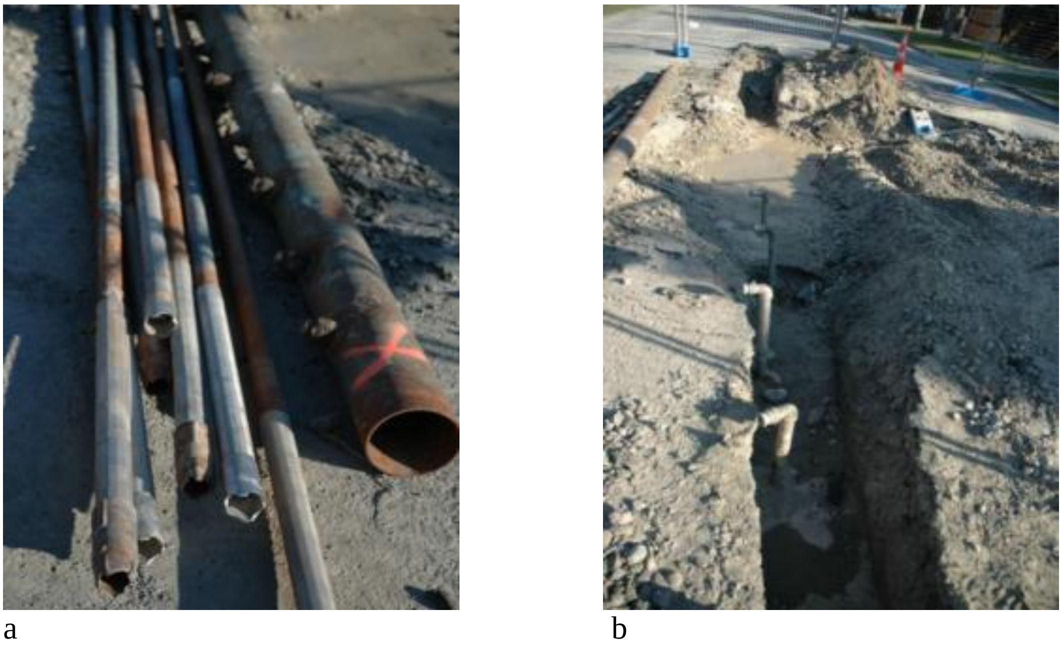

a b

Figure 8.41: Drainage sucker rods and operation (25 Sep. 2010 Christchurch) (Source: Author’s own)

The difference between the two open-cut methods used in loose soil with high water table is in the methods used in excavation. In Method 1, a trench is excavated after installing sheet piles around the damaged pipe. However, in Method 2, the trench is excavated to a certain depth before installing the support box subjected to the stability of the trench soil type. In the first method, sheet piles are used to stabilise each side of the trench, but in the second method, the support box is used instead of sheet piles. Method 1 is more reliable than the second because installation of the support box is usually time consuming especially in the unstable soil, although the first method is usually more expensive compared to the second method (Zare, 2010a). Figure 8.42 shows the open-cut method where the groundwater level is below the trench bottom. This method can be applied without support even in a very loose soil when a trench is not deep. Furthermore, Figure 8.42 (b) shows that the trench walls are not stable even in shallow excavation depth without high ground water level and therefore require support.


a b

Figure 8.42: Open-cut pipe replacement above groundwater level (Sep. 2010, Kaiapoi) (Source: Author's own)

The buried depth is one of the main factors in selection of repair methods. For instance, the open-trench method without any trench support is usually the most appropriate method for repairing damaged buried in the depths less than 1.5m, but due to a high ground water level in very soft soil, the open-trench method without support can be very expensive and dangerous. Figure 8.43 shows the replacement of slumped soil caused during excavation using the open-trench method in the depth of about 1 meter.

Photo 8.43: Excavation and backfilling due to use of open-cut method in very loose soil (Sep 2010 Kaiapoi) (Source: Author's own)

Figure 8.44 (a &b) shows the sheet piles used for repairing of damaged wastewater pipes. The sheet piles are usually used in areas with a very soft soil and with high water table. Figure 8.44 (a) shows the trench will be excavated after installation of sheet piles and drainage rods. Figure 8.44 (b) shows the last stage of pipe repair which is backfilling the trench with suitable soil. After this stage the installed sheet piles will be pulled out.

Figure 8.44: Open-cut pipeline replacement in loose sand and high water table (Sep. 2010, Waimakariri district-Kaiapoi) (Source: Author's own)

The earthquake caused damage in different type of pipes installed in various depths. As a result, different methods of repair and rehabilitation had to be used. Repair and rehabilitation with open-cut methods include installation of sheet piles and drainage rods, soil excavation and soil replacement and re-pavement which require heavy equipment. Application of construction equipment not only causes some types of pollution and traffic disturbance but it also increases repair cost. On the other hand, application of trenchless techniques not only do not need sheet piles for soil stabilization they also have little or no disturbance to the neighbouring areas alongside of pipelines. As a result, traffic disturbance and cleaning costs are less when trenchless methods are adopted.

Application of adjustable steel box is another type of open-cut method which is used for pipe repair in areas with soft soil and high water table. Photo 8.45, shows the application of the adjustable steel box in an area with high water table. Application of this method in areas with high water table requires dewatering during excavation and pipe repairs. Application of the adjustable steel box is limited to the repairs of buried pipes laid maximum in 2.5 meter depth, although a special box can be designed and constructed for greater depths.

Photo 8.45: Wastewater pipe localised open-cut repair with supporting box in loose sand under water table (25 Sep. 2010, Avonside, Christchurch) (Source: Author's own)

Adjustable steel box can be applied to support both sides of the opened trench in an area with loose soil above water table. Application of adjustable steel box can be limited in vicinity of other buried networks such as water and gas networks. Photo 8.46 shows that how unexpected extra excavation is required in application of the adjustable steel box. This photo also shows that how application of this method can be limited by being close to other lifelines.

Photo 8.46: Wastewater pipe localised open-cut repair with supporting box in loose sand and dry soil (25 Sep.2010, Darlington Christchurch, Source: Author's own)

### 8.6.2. Trenchless rehabilitation and repair

The second category of sewer repair and rehabilitation methods is trenchless based methods. Unlike open-cut methods, the trenchless based methods do not need excavation of the soil surrounding the pipe although some trenchless methods require access pits that should be excavated at particular points for installation.

#### 8.6.2.1. disadvantages of open-cut methods in ordinary repair and rehabilitation

Application of conventional open-cut methods in urban areas, especially in populated areas, can increase the travel time and disturb routes for residents due to limited access or road closures. The number of heavy vehicles and machinery used in open-cut methods is greater than in trenchless methods, and application of heavy machinery equipment produces noise and dust pollution (Najafi, 2005). Excavation in open-cut methods is costly and time consuming because of the high density of other buried facilities on top or in vicinity of wastewater pipelines, including power, telephone, data lines and private television cables as well as water and gas pipelines. Excavation in regions with several underground networks not only is expensive and time consuming but it can also damage or cause failure in normal operations of other facilities. Figure 8.47 shows the numerous underground networks in urban areas which restrict application of ordinary open-trench methods.

Figure 8.47: Difficulties in open-cut methods in crowded urban areas (Source: Author’s own)

In terms of safety, the working environment in trenchless methods is safer than in open-cut methods because there is less excavation and open areas. Removing spoil and damage in the pavement are other disadvantages of open-cut methods. Backfilling, compaction and reinstatement of the ground and the pavement makes 70 % of the project costs in some open-cut projects. Furthermore, life expectancy of the pavement after excavation decreases by up to 60 % (Najafi, 2005). Finally, in terms of aesthetics, trenchless technology is preferable to open-cut methods.

### 8.6.2.2. Factors affect post-earthquake repair process

After the 2010 Christchurch earthquake, some roads and streets in the city were severely damaged. Consequently, it did not matter what techniques was used due to highly damage to pavements and traffic disturbances. However, cost-benefit analysis of rehabilitation methods needs to carry out in order to select the most appropriate method. Figure 9.48 (a & b) shows the road damage after the 2010 Darfield earthquake.


a b

Figure 8.48: Damage to road pavement (left Christchurch and right Kaiapoi, Sep 2010) (Source: Author's own)

Road damage not only limited access to the affected areas but it also hindered repair and rehabilitation process. Figure 8.49 (a & b) shows the inaccessible road due to severely damaged roads after the earthquake. The earthquake not only caused buried pipes breakages and pavement damage but it also caused severe mess in roads because of water pipe breakages, as well as the spoil of sand from the nearby residential buildings. Figure 8.49 (b) shows the mess caused by the earthquake in the liquefied regions in the city.

a b

Figure 8.49: Inaccessible roads, 6 Sep. 2010 Christchurch (left severe road damage, right pipe breakages and debris) (Source: Author's own)

Building debris and safety issues also limited access to the seismic affected regions particularly in business areas where the buildings are constructed alongside the roads. Figure 8.50 (a & b) shows that how building debris after the Christchurch earthquake can hinder the repair process in the nearby road.

a b

Figure 8.50: Closed streets because of building debris and building unsafely (6 Sep. 2010 Christchurch CBD) (Source: Author's own)

### 8.6.2.3. Factors restrict application of trenchless techniques after an earthquake

Technical feasibility is the main and initial parameter that significantly affects selection of trenchless repair and rehabilitation methods after an earthquake. Burial depth, type of pipe, pipe diameter, required accuracy, length of drive, ground condition, numbers and location of laterals and interference with other utilities affect the technical feasibility.

An accurate slope is required in unpressurised wastewater pipes to satisfy design criteria, such as the flow rate and minimum flow velocity (Bizier, 2007). For instance, some trenchless techniques, such as Horizontal Directional Drilling (HDD), pipe ramming and pipe bursting, cannot provide the required accuracy for wastewater pipe relining. However, some advanced and new HDD machineries can be applied to relining of low diameter pipes in an unpressurised networks (Najafi, 2005).

Typically, main wastewater pipelines are buried underneath other utility lines in a high burial depth, and thus application of trenchless methods in this case is more convenient than of open-cut methods, especially in congested regions of an affected city as was seen in Christchurch.

Observation from Christchurch showed that earthquake induced damage in unpressurised wastewater pipes can be divided into two groups; damage in the body and joint of pipe without any change in the pipe alignment and elevation; and damage in the pipe alignment and elevation. In the first group, buried pipes are affected by earthquake forces such as compression and tension without any change in the pipe alignment and elevation. In the second group, earthquake forces change the pipe alignment and the pipe elevation, from their designed locations.

First group of damage can be fixed by both open-cut and trenchless method because of the point damage. In this group, time and cost are the two main items that are considered in selection of appropriate method, but in the second group of damage the feasibility of each method should be assess first before time and cost analysis. Application of trenchless techniques is limited in repair of damages caused by misalignment of pipes.

The Christchurch wastewater network was affected by the both groups of damage. The first group of damage is observed within all regions in Christchurch but the second group of damage is mainly observed in areas affected by liquefaction. Avonside and Darlington regions in Christchurch, as well as some regions in Waimakariri District experienced the second group of damage. In these affected regions, pipes between two or more manholes were misaligned, and the pipe elevation in some points was changed. As a result, the affected wastewater pipeline was failed or adversely affected the network.

Earthquakes can change the pipe slope due to an uneven settlement alongside of pipeline. If earthquake forces change the slope of pipe line, all the affected pipes between two manholes should be replaced and reconstructed even if there is no individual pipe damage. For instance, the Christchurch earthquake caused severe change of pipe slope and blocked some wastewater pipeline and caused back flow. Running wastewater flow can accumulate on the back of damaged points and this may cause wastewater overflow into house connections.

Earthquake induced damage to other components of the wastewater networks such as manholes and pump stations can also cause damage into the wastewater pipelines. Figure 8.51 (a &b), shows damage in wastewater pipes because of the earthquake damage to other wastewater system components. Figure 8.51 (a &b), shows the pipe damage in the inlet and outlet of the damaged manholes, while Figure 8.52 (a &b) shows the pipe damage caused to the inlet and outlet of the wastewater pumping stations.

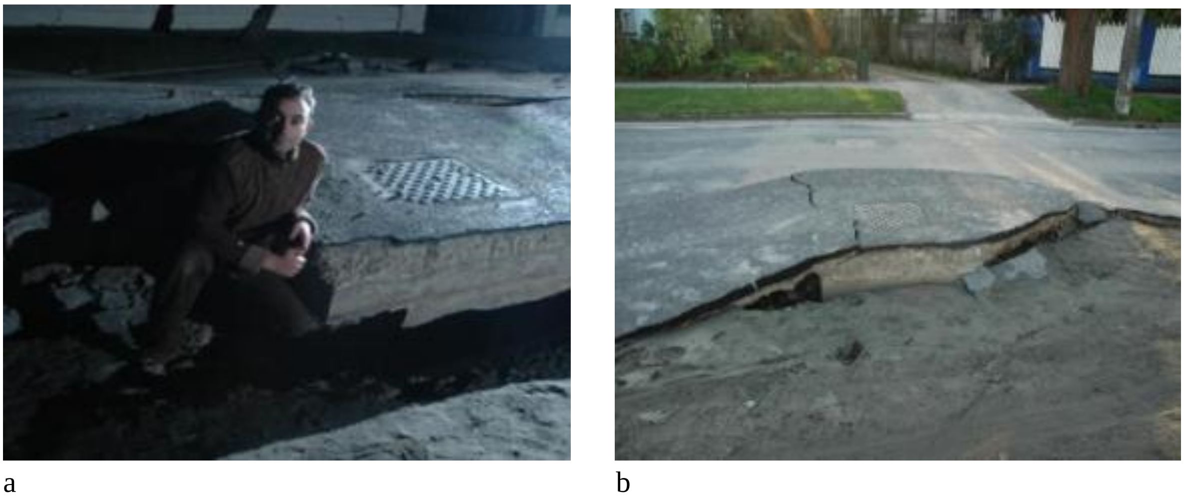

a b

Figure 8.51: Pipe failure in manholes (6 Sep 2010, Avonside Christchurch) (Source: Author’s own)

Earthquake induced damage in the pipes connected to the damaged manholes or pumping stations is classified as the second group of damage. This damage causes misalignment and changes elevation of the damaged pipe. Figure 8.52 (a & b) shows the misalignment of the connected pipes in the damaged pump stations after the earthquake. Due to type of damage shown in this figure, open-cut methods are the only applicable method of repair in this type of damage.

a b

Figure 8.52: Pipeline failure in inlet and oulets of WWPSs (Sep. 2010, Avonside, Christchurch) (Source: Author’s own)

As explained in Chapter 2, earthquakes cause immediate or long term effects on unpressurised wastewater pipelines. Pipe collapse is the immediate effect of an earthquake on unpressurised networks. Type of damage in unpressurised networks is one of the differences between seismic damage in pressurised and unpressurised networks. Most damages in pressurised networks affect the system functionality whereas, the same type of defects in unpressurised network do not impact on the system function in unpressurised networks sometimes for a long period. Most of the seismic damage in pressurised networks needs to be repaired immediately while similar defects in unpressurised wastewater networks can be left without repairs for a long time. On the other hand, some instances of pipe damage if left without repair, will cause a collapse and system failure. Photo 8.53 shows a severely damaged wastewater pipe which did not collapse or block the flow immediately after the earthquake, but if left without repair it will eventually collapse.

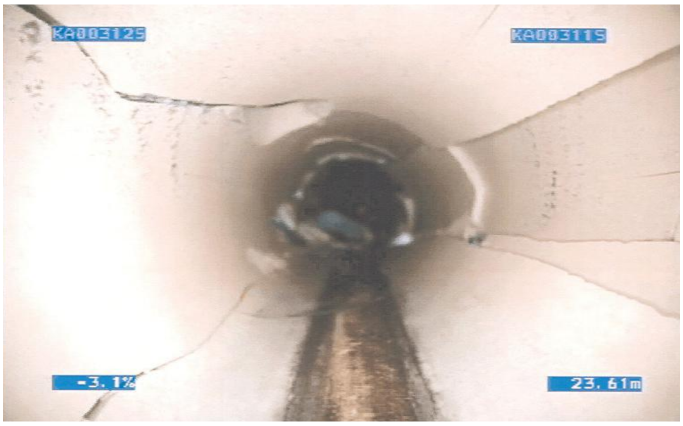


Photo 8.53: Severely damaged wastewater pipe (Source: Christchurch (Gordon, 2010))

Two types of damage inspection were sued in the seismic affected areas in order to detect earthquake induced damage in the unpressurised wastewater network; visual and closed-circuit television inspections. Visual inspection was applied to find the visible earthquake induced damage in the unpressurised wastewater networks. The visual inspection used to find displacement of manholes and pump stations, water invasion and direct damage in visible pipes underneath of bridges. Photo 8.54 shows water invasion in a pressurised wastewater pipeline in the Christchurch wastewater network.

Photo 8.54: Visual inspection (water invasion) (Source: Author's own)

The visual inspection method usually shows the location of damage. Therefore, other methods are required to detect type and location of damage in the pipes. Close-circuit television camera or CCTV is the method used as the second type of inspection method for detecting type and location of damage inside the damaged pipes.

Reviewing CCTV camera films in Christchurch and Waimakariri revealed that concrete pipes suffered minor to severe damage after the 2010 earthquake. Pipes misalignment damage, body or joints damage, and a combination of misalignment and body damage are types of damage detected by CCTV method.

The inspection showed that joints in wastewater pipelines were severely damaged. Joints were affected by compression and tension forces caused by the earthquake. The compression and tension forces damaged different types of joints such as bell and spigot joints. The compression force crashed or caused cracks in the joints and the tension force pulled apart joints.

Damaged joints cause infiltration or exfiltration. Joint separation in wastewater pipes located in areas with high water level, such as areas beside rivers, can increase the flow in damaged pipelines because of groundwater infiltration. On the other hand, wastewater leakage from damaged joints can infiltrate into the surrounding soil and pollute nearby soil and groundwater.

Joint damage can be repaired using localised trenchless methods if pipe alignment is not affected by earthquake forces. Grouting, point Cured In Place Pipe (CIPP) and internal seal point methods are common types of repair method. Grouting techniques for controlling leakages should be evaluated carefully before their application, especially in high ground water level regions and loose sand, such as regions located in vicinity of natural streams. For instance, grouting methods did not appear to be suitable for the severely affected regions after the 2010 Christchurch earthquake. The affected regions in Christchurch are located in vicinity of rivers where the groundwater level is high and loose sand is the predominant type of soil. In case of a wastewater pipe collapse after an earthquake, open-cut methods are usually the best repair option.

The localised CIIP repair method was the only trenchless method used in some earthquake affected regions after the 2010 Christchurch earthquake. Figure 8.55 (a &b), shows the application of CIPP point repair used for repairing damaged wastewater pipe after the 2010 Christchurch earthquake.


A b

Figure 8.55 Localised CIPP repair team (Waimakariri District, Sep. 2010) (Source: Author's own)

## 8.7. Summary

This chapter shows and confirms seismic vulnerabilities of wastewater networks in New Zealand which were discussed and explained in Chapter 5, 6 and 7 for the three case studies.

The intensive impact of the wastewater system failure on the Christchurch City and also the cost and time spent for restoration of damaged wastewater pipelines calls for the need for pre-earthquake rehabilitation plan in areas with high susceptibility to earthquakes. Extensive damage calls for the need for post-earthquake restoration plan for wastewater networks in seismic prone cities.

Comparison of damage in different types of pipe in the Christchurch wastewater network revealed that the concrete pipes are the most vulnerable ones used in the wastewater network. Unplasticised polyvinyl chloride, cast iron and asbestos cement pipes are also considered as vulnerable pipes. On the other hand, concrete lined steel, modified polyvinyl chloride and high density poly ethylene pipes are the least vulnerable pipes in the wastewater network. In terms of pipe diameter, small diameter pipelines, especially the 150mm pipes are the most vulnerable type of pipes in the Christchurch wastewater network.

Comparison of calculated damage in the Christchurch wastewater and volume of damage after the 2010 earthquake shows that fragility curves can estimate which areas are expected to have the maximum and minimum number of defects. The number of calculated defects in each zone is different compared to the number of detected defects after the 2010 earthquake.

As explained in Chapter 2 and this chapter accurate inspection is required to identify all type of defects in a wastewater network after an earthquake in all zones of the Christchurch wastewater network in order to determine total number of defects. Complete inspection in the network not only is costly but requires long time to complete. As a result, the number of defects introduced in this chapter does not reflect the total number of defects in Christchurch after the 2010 Darfield earthquake. Incomplete damage inspection in some zones can lead to the differences between calculated defects and recorded defects in each zone.

This chapter indicates that no trend exists between damage rate and PGV and PGD as earthquake parameters. On the other hand, the equations of fragility curves are based on a specific trend. This trend shows that when there is an increase in the volume of PGV and PGD there is a comparable increase in damage rates. These facts indicate that application of fragility curves which are based on a specific trend, developed by other researchers, cannot be applied in New Zealand particularly in areas with liquefiable soils such as that of the experienced in Christchurch.

Observation of post-earthquake restoration operation after the earthquake in the Christchurch indicates that the adopting of open-cut methods for repair and replacement is faster than trenchless techniques. Furthermore, this method is the only applicable method in repairing some types of damages such as connection points between pipelines and manholes or pump stations.

# Chapter 9. Conclusion

## Introduction

This chapter shows how the objectives of the thesis are achieved and bring the thesis to a conclusion. This chapter also includes recommendations to improve application of fragility curves on wastewater network in New Zealand. This study contribution to knowledge is also explained in this chapter.

As discussed in the first chapter, the overarching aim of this thesis seeks to understand the limitations of the present fragility curves in evaluating seismic vulnerability of unpressurised wastewater pipelines in New Zealand. Therefore, in order achieve the overarching aim 6 questions which are explained in Chapter 1 are asked that finding appropriate answers for the questions satisfy the overarching aim of this thesis.

In order to find appropriate answers for the thesis questions, for each question an appropriate objective is developed. Some methods explained in Chapter 3 are applied for each objective to satisfy the objective.

## 9.1. How the objectives were achieved

The objectives of the thesis are explained in the first chapter and investigated in Chapter 2 to 8. The objectives and the final conclusions are discussed here.

### 9.1.1. Demonstrate the extent of seismic damage on the unpressurised wastewater networks in New Zealand in order to show the necessity of pre or post-earthquake measures in the wastewater networks.

As explained in Chapter 2, few researchers have considered damage on unpressurised wastewater networks due to earthquakes within the context of severe effects of past earthquakes on buried pipe networks. The subjects of this thesis are four case studies within New Zealand which examine earthquake effects on unpressurised wastewater network. Earthquakes with 475 and 1,000 years return periods are applied in calculating the holistic and wave propagation effects of earthquakes in each case study.

The earthquake effects on the Hutt City wastewater network are presented in Chapter 4. The seismic assessment in Chapter 4 demonstrates that this network is vulnerable to earthquakes and can be affected severely. Earthquakes with 475 and 1000 year return periods are used in Hutt City to show the seismic vulnerability of the wastewater network. The available fragility curves were applied in Hutt City to demonstrate the wave propagation effects of earthquakes with an annual probability of exceedance 1/500 and 1/1000. The assessment shows the number of expected defects in each wastewater network.

From the analysis it can be deduced that a 475 years return period earthquakes can cause a total number of 818 instances of damage in the Hutt City wastewater network. The study conducted in this thesis showed that Zones 5, 8 and 3 are the most vulnerable parts of Hutt City in terms of earthquake induced damage in the wastewater network. The amount of the expected damage in Zone 5, 8 and 3 covers 36%, 17.7% and 14.5%, respectively, of the total expected damage that can be caused by a 475 years return period earthquake. As expected, an earthquake with 1,000 year return period can cause more damage than a 475 year return period earthquake; however, the difference is not significant in Hutt City. The former can inflict a total number of 928 instances of damage in the Hutt City wastewater network. The expected damage caused by the 475-year return period earthquakes, Zones 5, 8 and 3 have the most vulnerable wastewater network, and Zone 6 has the least vulnerable wastewater network in Hutt City. The expected instances of damage in Zones 5, 8 and 3 are 328, 152 and 147, respectively. The percentage of the total damage in each of the most vulnerable zones is almost the same as the percentage of damage that can be caused by 475-year return period earthquakes. The expected damage caused by a 475-year return period earthquake is only 11.85% less than that of the total amount caused by a 1,000-year return period earthquake.

Both types of the periodic earthquakes can cause the same percentage of total damage in each municipal region of Hutt City. It should be noted that, although both effects of an earthquake were considered in the formula applied, significant ground displacement like landslides, should be taken into consideration separately for more accurate results.

In order to extend the research findings the seismic vulnerability of the Gisborne and Blenheim wastewater networks was taken into consideration in Chapter 5 and Chapter 6, respectively. In the Gisborne and Blenheim wastewater networks both holistic and wave propagation effects of earthquakes with different return periods were applied to show seismic vulnerability of the wastewater networks similar to Hutt City. Earthquakes with 475 and 1000 year return periods were applied to show the earthquakes effects on the Gisborne and Blenheim wastewater network.

The seismic vulnerability analysis in Chapter 5 and 6 showed that the Gisborne and Blenheim wastewater networks are also vulnerable to the expected earthquakes. Earthquakes can cause significant damage to the Gisborne and Blenheim wastewater network. Comparison between the calculated defects in Gisborne and Blenheim indicates similarities compared to the Hutt City wastewater network.

The total instances of the damage in the Gisborne and Blenheim wastewater network calculated in Chapter 5 and 6 were estimated to be 142 and 299, respectively, if affected by earthquakes with 475 years return period in the overall approach method. Both wave propagation and permanent ground displacement of earthquakes were calculated by the PGA based equation.

If the overall seismic effects of earthquakes with 1,000 years return period are taken into account, instances of the expected damage in the Gisborne and Blenheim wastewater network are 204 and 382, respectively. The comparison of damage caused by two types of earthquake shows that the expected damage in earthquakes with 1,000 years return period is not significantly greater than damage caused by 475 years return period earthquakes. For instance, the expected damage in the Gisborne and Blenheim wastewater network affected by a 475 years earthquake is 30% and 22% respectively, which is less than that of the 1,000 years earthquake.

Evaluating and analysing the Christchurch earthquake damage on the wastewater network in Chapter 8 explained the real earthquake damage on the wastewater pipeline. The seismic damage induced after the earthquake were classified in terms of quantity, location, and damage characteristics of each damaged point and the wastewater pipes were compared in terms of seismic vulnerability. The damaged pipes were evaluated in terms of materials, diameters, buried depths and durability.

As explained in Chapter 8 the brittle pipes were damaged significantly compared to ductile pipes. AC, CI, Conc., RCRR, EW, PVC and UPVC pipes were damaged significantly. The significant difference here compared to the previous studies in terms of seismic damage is that the PVC and UPVC pipes can be affected significantly by seismic shocks to a greater extent than the pipes previously mentioned by the other researchers. Damage rates in PVC and UPVC pipe are much more than that of the rates in the other type of pipes. Ductile pipes such as different types of PE and ST pipes were not affected seriously and they could stand seismic shocks in an appropriate manner.

### 9.1.2. Identify the parameters which need to be included in fragility curves to achieve more accurate results.

As explained in Chapter 2, there are two main methods for evaluating earthquake damage. The first method is the application of the available fragility curves, and the second is the application of the analytical methods. The second method is costly and time consuming, and is typically used to evaluate critical buried pipelines in seismically vulnerable areas. Application of analytical models is usually limited to construction of new main pipelines. As a result, the use of fragility curves is the only applicable method; however, many factors can affect the calculated results when fragility curves are applied. In this thesis the parameters which can affect the results of seismic assessment by fragility curves are taken into account.

Application of the available fragility curves used for seismic vulnerability analysis is not only affected by factors ignored in fragility curves but are also affected by factors during the analysis process that can cause some other limitations.

Reviewing the literature in Chapter 2 revealed that various fragility curves can be applied in earthquake effects evaluation on buried pipelines. Many parameters can affect seismic vulnerability of buried pipelines but only a few of them are used in available fragility curves. In Chapter 2 the parameters that could affect the seismic vulnerability of buried pipelines are extracted and discussed.

The 2010 Darfield earthquake effect on the Christchurch wastewater network was taken into consideration in Chapter 8 for further evaluation of parameters affecting seismic vulnerability of unpressurised wastewater pipes. In Chapter 8 some significant parameters are discussed. Comparison of damage in different wastewater pipes in Christchurch shows some significant similarities and dissimilarities compared to the literature review. For instance, Chapter 8 indicates that the UPVC pipes were severely damaged in the earthquake similar to damages in brittle pipes. Furthermore, UPVC pipes should not be classified as ductile pipes or even classified with the same type of PVC.

This study indicates that the seismic vulnerability in wastewater pipelines can be affected by three groups of parameters. The first group comprises pipe relevant parameters such as pipe material, diameter, joint type, corrosion rate, durability and wall thickness. Second group of the affecting factors are earthquake relevant parameters such as earthquake intensity, magnitude, angle of pipelines direction to fault line, number and intensity of aftershocks, and the focal depth. It should be added that the 2010 Christchurch earthquake showed numerous aftershocks with notable intensity can affect the number and extent of damage.

Other notable parameters are the miscellaneous parameters like soil type, burial depth, water table, quality of installation, number of connections, number of manholes, and closeness to other lifelines especially water pipelines. The studies conducted on the 2010 Christchurch earthquake showed that damages on water pipelines installed in the vicinity of wastewater pipelines can increase vulnerability of wastewater pipelines.

Chapter 4, 5, 6 also show the factors such as length of the pipes, type of the pipes and combination of different type of pipes can affect accuracy of the results during seismic analysis by fragility curves. As explained in the case study chapters, estimation of the required data in analysis can cause uncertainties in results. For instance, estimation of the required earthquake parameters, soil type and pipe classification are uncertainties which can alter the results.

Comparison of the calculated defects by the applied fragility curves reveals the differences within the estimated defects. As a result, the differences reveal that fragility curves are not accurate enough to use for accurate seismic assessment in buried pipe networks, particularly in unpressurised networks. On the other hand, many factors can affect seismic vulnerability in buried pipes which are not considered in fragility curves.

### 9.1.3. Compare the estimated defects in unpressurised wastewater pipes by a range of fragility curves.

Wave propagation effects of earthquakes with an annual probability of exceeding 1/500 and 1/1000 were calculated for the Hutt City wastewater network in Chapter 4. The calculated defect by different fragility curves were divided into two groups to show the differences between fragility curves of each group more accurately. Wave propagation effects of the earthquake on the Hutt City wastewater network shows that the R-wave equation suggested by O’Rourke and Deyoe (2004) estimated the maximum amount of damage, while FEMA’s fragility curve (2004) estimated the minimum amount of damage in Group 1 fragility curves. The maximum average estimation is about 52.5% more than that of the minimum estimation, and Eidinger and Avilas' (1999) equation is about 7.7% more than the average of the estimated values in Group 1 fragility curves.

In Group 2, the fragility curves suggested by Jeon and O'Rourke (2005) estimates the maximum amount of damage within all 5 fragility curves, while ALA's equation (2004) estimates the minimum amount of damage in the Hutt City wastewater network. In contrast with Group 1 equations, in Group 2 equations, the changes between estimation of different fragility curves in each region are less than that of the Group 1 equations. In Group 2 equations, the distribution of the maximum and minimum amount of the predicted damage is different within each zone.

Similar to Hutt City, wave propagation effects of earthquakes with different return periods were applied to show seismic vulnerability of the wastewater network in Gisborne, Blenheim and Christchurch each city. Earthquakes with annual probability of exceedance 1/500 and 1/1000 are used to show the differences in calculated defects caused by wave propagation on the wastewater network of Gisborne.

If the damage in the Gisborne wastewater network is calculated by Group 1 equations, the equation suggested by Eidinger and Avila (1999) provides the maximum amount of damage assessment, while FEMA's fragility curve (2003) estimates the minimum damage in Group 1 fragility curves. In contrast with Hutt City and Blenheim, the fragility curves developed by Eidinger and Avila (1999) and the O'Rourke and Deyoe's (2004) (R-Wave) provide almost the same amount of damage assessment.

Comparison of results in Gisborne shows that if more than 60% of the total length of each municipal zone comprises of brittle pipes, usually the R-wave fragility curve suggested by O'Rourke and Deyoe (2004) and the Eidinger and Avilas' equations (1999) provide almost the same results. The fragility curve suggested by Eidinger and Avila (1999) estimates the least amount of damage amongst Group 1 equations in regions with more flexible pipes usually more than 20 %.

If the average amount of damage in each group of fragility curves is compared to each other, the average estimation of damage calculated by Group 1 equations is 8.2 times greater than that of Group 2. This ratio of damage is 7.4 in Hutt City, and 5.0 in Blenheim. This means that with an increase in the percentage of brittle pipes in each wastewater network the difference between average estimation of Group 1 and 2 increases.

The Blenheim wastewater network with different seismic vulnerability and different types of wastewater pipelines is discussed in Chapter 6 as the third case study. Nine fragility curves were applied to investigate wave propagation effects of earthquake with annual probability of exceedance 1/500 and 1/1000.

Comparison of the calculated defects in the Blenheim wastewater network was discussed in Chapter 6. Similar to the Hutt City wastewater network, in the Blenheim wastewater network, the R-wave equation suggested by O'Rourke and Deyoe (2004) provides the maximum expected damage. In the Blenheim damage estimation, the FEMA's fragility curve (2003) provides the second highest amount of damage after the O'Rourke and Deyoes' R-wave equation (2004), this in contrast to the predicted damage in the Hutt City wastewater network.

In earthquakes with annual probability of exceednace 1/500, the amount of damage estimated by the Eidinger and Avilas' (1999) fragility curve is 4.6 times less than that of the R-wave equation of O'Rourke and Deyoe (2004). This shows that the type of pipe material can significantly affect results in Group 1 equations. The S-wave equation suggested by O'Rourke and Deyoe (2004) estimates the minimum number of defects in the Blenheim wastewater network. The amount of damage calculated by the O'Rourke and Deyoes' R-wave equation (2004) is 9.6 times more than that of the S-wave equation. As to the calculation of earthquake damage in the Blenheim wastewater network, Eidinger and Avilas' equation (1999) can be significantly affected by the types of pipes applied in each wastewater network. Consequently, in regions with more ductile pipes, the fragility curve recommended by Eidinger and Avila (1999) estimates the lowest amounts of damage in Group 1 equations, and the results are more similar to the results of Group 2 equations.

The Christchurch wastewater network with different specifications compared to other case studies is discussed in Chapter 8 as the fourth case study. Nine fragility curves were applied to investigate wave propagation effects of earthquake with annual probability of exceedance 1/500 and 1/1000. Comparison of the calculated defects by the fragility curves in the Christchurch wastewater network is discussed in Chapter 8.

Comparison of the calculated defects in the Christchurch wastewater network shows that estimated defects by different fragility curves have a notable difference with one another, which is similar what was experienced in the Hutt City, Gisborne and Blenheim.

### 9.1.4. Recommend the most effective application of fragility curves in damage estimation of unpressurised networks.

The literature review shows the fragility curves are recommended by many researchers in order to assess seismic vulnerability in buried pipe networks. Seismic assessment of buried pipe networks by the other method requires intensive effort which is time consuming and costly. As a result, fragility curves are recommended as an appropriate tool to assess seismic vulnerability in buried pipe networks.

On the other hand, the number of defects calculated by the applied fragility curves indicates that there are differences between the calculated defects by each fragility curve and in the zones of the four case studies. As a result, the fragility curves developed overseas from pressurised water networks cannot be an appropriate tool to calculate accurate number of defects in unpressurised wastewater networks, but utility managers do not always look for an accurate number of defects in the affected network.

Application of each fragility curve shows which zone has the highest or the lowest seismic vulnerability. All fragility curves applied in Chapter 4, 5, 6 and 8 show similar results. Thus application of fragility curves is a useful tool for utility managers who are looking for the overall impact of earthquakes on wastewater networks for general purposes. They are interested to know which are the most and the least vulnerable zones to seismic shocks and fragility curves give a clear indication of this.

The comparison of calculated defects in each case study in Chapter 4, 5, 6 and 8 indicates that if the minimum number of expected defects in each type of pipe is notable, the pre-earthquake mitigation plan should be used to decrease earthquake effects on wastewater pipelines. Here the small number of calculated minimum defects in each zone indicates that the post-earthquake mitigation plan is the appropriate adopted method.

For instance, in the Blenheim wastewater network Zone 4 is the most vulnerable zone. As a result, pre-earthquake rehabilitation plan should be considered in Zone 4. The minimum number of expected defects in brittle pipes of zones 2 and 3 shows that the post-earthquake rehabilitation can be a good measure in these zones. The examination of the calculated defects in ductile pipes of the Blenheim wastewater network shows that in earthquakes with annual probability of exceedance 1/500 zones 1 and 2 are the most vulnerable zones and Zone 3 is the least vulnerable zone. Number of the calculated defects in ductile pipes of the

Blenheim wastewater network indicates that the post-earthquake restoration plan can be the best measure to reduce seismic damage.

### 9.1.5. Identify the relationship between damage rate and earthquake variables in unpressurised wastewater networks.

Reviewing the literature showed that the available fragility curves are common tools used to calculate seismic damage in buried pressurised networks. The same fragility curves were also recommended for calculating seismic damage on unpressurised networks such as wastewater networks.

Chapters 2 discussed the earthquake impacts on buried pipelines during the past earthquakes and also the previous works on the seismic vulnerability analysis of buried pipelines by the fragility curves. These two chapters revealed the significant parameters which may affect seismic vulnerability of buried pipelines.

Some of the available fragility curves introduced in Chapter 2 were used in Chapter 4, 5, 6 and a section in Chapter 8 in order to calculate seismic damage in areas that contain 31.81% of the total population with medium to high potential earthquake risk. The comparison of the calculated seismic damage in the chapters mentioned above revealed that there are significant differences among the calculated damage in each city. The significant differences indicated that the application of fragility curves should be limited in some applications, as explained before. Selection of adequate the fragility curves should be based on location. In highly populated areas the selected fragility curve should be the one that could estimate the higher damage rate. Estimation of higher damage rates decreases the probability of severe damage after applying the pre or post-earthquake measures.

The above discussion is based on the fact that the damage rates follow the same trend as the curve’s formula. The trend applied in the available fragility curves shows that by increasing the PGV the calculated damage rates should increase. The 2010 earthquake in Christchurch provided an opportunity to investigate the applicability of the trend applied in the available fragility curve.

The comparison of the calculated defects and the affected PGV after the 2010 earthquake showed that there is no particular trend to show the changes in damage rate by particular change in PGV. The trend plotted for the various types of pipes indicated that the pipe material cannot affect the trend. The plotted trends even showed the reverse trend. For instance, the plotted damage rates and PGV showed an increase in PGV will lead to a lower damage rate.

All of the above explanations indicate that the application of available fragility curves should be limited in New Zealand, especially in areas like Christchurch with soils susceptible to liquefaction.

### 9.1.6. Discover the limitations in application of trenchless techniques for post-earthquake repair and rehabilitation in unpressurised wastewater networks.

The 2010 Christchurch earthquake caused damage to the wastewater system and provided opportunities to find out the difficulties in post-earthquake repair and rehabilitation. In this study different types of repair and rehabilitation methods were explained after the earthquake. Open-cut methods were the main technique used in rehabilitating the damaged wastewater network after the 2010 Darfield earthquake. The damage which occurred in the wastewater network fell into the following categories; crushed joints, joint separation (pull out), body cracks, crushed body, pipe elevation changes and pipes moving out of their designated locations. The Christchurch earthquake in 2010 serves as a lesson on how theory and practice overlap, but more crucially the earthquake showed the vulnerability of these systems in action.

The major advantage of both the trenchless repair and the rehabilitation methods based on trenchless techniques is that the lower pavement and traffic disruption when compared to the ordinary open-cut methods. The case study in Christchurch showed that if the pavement is severely damaged, open-cut methods should be considered as the most applicable method.

When seismic shocks change the designated location of affected pipes or damage network connections in wastewater pipelines, application of trenchless methods can be limited. For instance, when damage occurs in pipes connected to manholes and to pumping stations application of trenchless methods cannot help.

## 9.2. Contribution to knowledge

In this section the main contributions of this study are classified into three main categories and then the significant contributions of each category are briefly discussed and compared with the previous studies.

The first category demonstrates limitations of current fragility curves for seismic vulnerability analysis of unpressurised networks. The study shows the significance impacts of parameters which are ignored in available fragility curves. Analysing the impacts of these parameters is the second main contribution of this study. The third main contribution of this study is defining new thresholds for appropriate application of fragility curves in seismic vulnerability analysis of unpressurised networks.

The contribution to knowledge for this study shows that many researchers have conducted studies in pressurised buried pipelines network affected by seismic shocks. FEMA (2003) and ALA (2004) recommended use of fragility curves for both pressurised and unpressurised networks. As no study exists showing the limitations of fragility curves to unpressurised networks, there is a need for research to be conducted. This thesis addresses this gap.

The thesis is the first study conducted in New Zealand to demonstrate the application of fragility curve as an estimation tool for calculating seismic damage in unpressurised pipeline networks.

This study compares the damages in the unpressurised wastewater networks within various fragility curves and compares the calculated damage with the observed damage. These comparisons provide contributions of this study by commenting on seismic assessment of fragility curves in unpressurised wastewater network. These finding can be used to improve future fragility curves for seismic assessment of both buried pressurised and unpressurised networks in order to achieve accurate damage estimation.

This study reveals that the seismic assessment by the applied fragility curves should be limited to calculating seismic damage on unpressurised wastewater pipelines. Furthermore, the fragility curves can only be applied to give a holistic estimation and cannot be applied as an accurate estimation tool in unpressurised wastewater networks.

The study illustrates that there are fundamental differences between damages calculated by the applied fragility curves and these differences can limit the application of the curves in all types of buried pipelines including pressurised and unpressurised networks. As a result, fragility curves which are developed overseas cannot be accurately applied in New Zealand for seismic assessment in buried pipe networks. The difference in calculated damage rates can adversely affect decision making processes in pre and post-earthquake measures.

The fragility curves cannot be used in areas with soil susceptible to liquefaction even to show the areas with the highest or lowest damage rates. The fragility curves can be applied in areas which do not have soil susceptible to liquefaction in order to show the regions with highest or lowest potential post-earthquake damage rate. This is because each of the applied fragility curves shows the same zone registering the highest and lowest seismic vulnerable region in all four case study cities.

This study demonstrates that interaction between soil and buried pipes after seismic shocks are complicated enough to limit the application of fragility curves with only one independent variable. This study not only shows that pipe material is a critical factor in accuracy of the fragility curve but also shows that many other factors should be considered and applied in the fragility curves for an adequate estimation of seismic damage rate.

This study shows concrete and reinforced concrete pipes have the highest damage rates compared to other type of pipes. This fact confirms the study of Kawashima et al. (1985) regarding the most vulnerable type of pipes. These types of pipe are the most vulnerable in unpressurised networks. This research also shows that seismic vulnerability of concrete reinforcement pipes is similar to seismic vulnerability of concrete pipes. Reinforcement of concrete pipe does not affect seismic resistance. Seismic resistance in these types of pipes are similar in spite of differing joints used in each type of pipe. The similarities in various types of concrete pipes show that joint type cannot affect resistance to seismic shocks in concrete pipes. Furthermore, this fact indicates that pipe weight in concrete pipes increase seismic vulnerability of this type of pipe compared with other types of pipe, particularly in small diameter pipes.

The most vulnerable types of pipes are different between unpressurised and pressurised networks. This study demonstrates that concrete pipes are the most vulnerable type of pipe in unpressurised wastewater networks in contrast with AC and CI pipes in pressurised pipes stated by Katayama et al., (1975), Cooper, (1984) and Pender, (1987). Furthermore, the finding here confirms the work of Allouche and Bowman (2006) that showed damage rate in CI pipes is higher than in AC pipes. This finding is disputed by other researchers.

Pender (1987) noted that during the Edgecombe earthquake almost every earthenware wastewater pipe was damaged severely and replaced but, these findings show that earthenware pipes accommodate seismic shocks better than concrete, RCRR, UPVC and CI pipes. This fact illustrates that the same type of pipes can behave in different ways depending on the earthquake. Furthermore, the fragility curves which are only based on pipe material can show inaccurate results.

In order to analyse seismic pipeline vulnerability it is common for researchers to divide pipes into two main groups, although a few researchers divide pipes into three main groups (FEMA, 2003 & Zhao et al. 2008). Because of notable damage rate differences amongst various types of pipes, this study demonstrates that pipe classification into the two broad classes of fragile and ductile cannot appropriately represent pipe vulnerability in unpressurised networks. It also shows that even pipes with the same base material can resist seismic shocks to notably different degrees.

Comparison of damage rates in PVC-based pipes confirms that there are remarkable differences between seismic vulnerability of different types of PVC pipes. According to this study, pipe flexibility is the main pipe specification that affects seismic vulnerability of buried pipelines. For instance, MPVC pipes resist seismic movement better than PVC and UPVC pipes. Furthermore, UPVC pipes with less flexibility compared to PVC-based pipes experience intensive seismic damage even compared to brittle pipes such as AC and CI pipes. As a result, a specific factor is recommended to apply in fragility curves in order to show differences between various types of pipes with the same basis or different fragility curves use for each type of pipe.

Comparison of damage rates in pipes with varying service lives show that the minimum damage rate belongs to the pipes installed between 1930 and 1940 and the maximum rate reports in the pipes installed between 1940 and 1950. This study demonstrates that recently installed pipes suffer more damage compared to those installed between 1930 and 1940. However, the pipes installed between 1990 and 2010 resist seismic shock better than pipes installed before that period with the exception of the pipes installed between 1930 and 1940. This fact reveals that the service life of pipes can affect seismic vulnerability in unpressurised networks but this is not the only factor, other parameters should also be taken into account.

Comparison of damage rates in terms of burial depth roughly indicate that pipes installed in deeper trenches suffer more damage than those in previous studies. The comparison shows wastewater pipelines experience the lowest damage rates in depths of 0.5 to 1.0 meter and this depth would be the appropriate burial depth for unpressurised wastewater networks in seismic prone areas. Furthermore, the pipe installed close to the ground surface and those installed in depths greater than 3 meters experience higher damage than other pipes.

Comparison of the earthquake damage on the Christchurch wastewater network reveals that damage rates in 150 and 225mm pipes are notably higher than other pipe diameters. Damage rate increases gradually in 160mm to 200 mm pipes and had a step change in 225mm pipes. On the other hand, damage rate decreases gradually by increasing pipe diameters in pipes with diameters greater than 300mm. This fact shows damage rates vary in pipes with different diameters but there is no particular trend.

Seismic damage threshold can be used to investigate regions with seismic damage on buried pipelines. Consequently some researchers recommended different thresholds which shows when seismic damage in buried pipelines occurs (O’Rourke and Jeon, 1999, Isoyama et al., 2000 and Maruyama & Yamazaki, 2010). As explained in Chapter 2, O’Rourke and Jeon (1999), Isoyama et al. (2000) and Maruyama and Yamazaki (2010) respectively recommended a PGV threshold of 10, 15 and 20 cm/sec in pressurised CI pipes.

This study suggests different thresholds for the most vulnerable types of pipes in unpressurised networks. The PGV thresholds in unpressurised wastewater networks show that PGV equal and greater than 45 cm/sec can cause damage in most types of fragile pipes including CI, EW, Concrete, and UPVC pipes. Reinforced concrete pipes can withstand PGV up to 46 cm/sec which is slightly more than what unreinforced concrete pipes can withstand. This study shows that AC pipes can tolerate more seismic forces compared to other fragile pipes. As a result, AC pipes seem to perform better than other pipes with a PGV less than 48 cm/s.

## 9.3. Practical implication

The findings of this thesis can be used in four main areas; seismic vulnerability analysis, design process in seismic prone areas, pre-earthquake rehabilitation and post-earthquake rehabilitation.

The thesis findings can be used for design of a new wastewater network or development of available wastewater networks. The findings can be applied for more accurate pipe selection and pipe installation in areas with high seismic vulnerabilities. For instance, use of concrete and reinforced concrete pipes should be restricted in seismic vulnerable areas.

In terms of seismic vulnerability analysis, the thesis showed that the application of available fragility curves should be limited in unpressurised networks. The thesis findings showed what type of fragility curves provides more accurate results and also showed the appropriate application of fragility curves in seismic vulnerability analysis of unpressurised wastewater networks.

The critical points and the most damaging types of pipes in unpressurised wastewater networks are shown in this study. These findings can be appropriately applied during pre-earthquake rehabilitation planning and scheduling. In terms of post-earthquake rehabilitation, the thesis findings can be applied for planning and scheduling required for inspection and selection of repair and rehabilitation methods.

## 9.4. How the overarching aim of the thesis is achieved

As explained in Chapter 1 the overarching aim of this thesis is to highlight the limitations of the present fragility curves in evaluating seismic vulnerability of unpressurised wastewater pipelines.

To achieve this aim, first the fundamental elements required for this study are gathered by the literature review in Chapter 2. This literature review reveals how buried pipelines have been affected by seismic shocks particularly in unpressurised networks. This part of the literature review shows that buried pipe networks have been severely affected by earthquakes. This finding has critical relevance for probable severe damage to lifelines which applies in particular to wastewater networks.

Furthermore, the limitations of fragility curves as a seismic assessment tool in unpressurised pipe networks in New Zealand are demonstrated by the use of the literature review. This is achieved by comparing calculated defects with one another and by comparison of calculated defects with the observed damages. Each case study within this thesis demonstrates that the damage calculated in any one fragility curve can have notable differences in comparison with the damages calculated by other fragility curves in the same area. Comparing the calculated defects by fragility curves and the observed damage illustrates that notable differences do exist within the calculated and observed defects. Therefore, the application of present fragility curves should be limited. It is the recommendation of this study that fragility curves in New Zealand be used only to broadly estimate seismic damage in unpressurised networks.

It is also cautioned that fragility curves cannot accurately be applied to estimate seismic damage in unpressurised networks. The relevance of achieving this aim is also explained in the contribution to knowledge section.

## 9.5. How the thesis’s findings can be applied for improvement in fragility curves

This study demonstrates that many factors can affect seismic vulnerability in buried pipelines including unpressurised wastewater networks. On the other hand, reviewing the literature shows that only pipe material is used to calculate seismic damage in buried pipe networks as an affecting factor in many fragility curves. Therefore, in order to estimate accurate damage in buried pipe networks more parameters such as those mentioned in Chapter 2 and 7 should be considered in fragility curves.

The literature review shows fragility curves have been developed overseas to assess seismic damage on pressurised pipeline networks. On the other hand, fragility curves cannot accurately estimate expected damage in unpressurised wastewater network. Therefore, new fragility curves are required to calculate seismic vulnerability in unpressurised wastewater networks. It is recommended that new fragility curves need to be developed for New Zealand to estimate seismic vulnerability of unpressurised wastewater networks more accurately.

This study shows that categorizing buried pipes into two broad groups i.e. brittle and ductile, can adversely affect the estimated damage in buried pipelines. As a result, it is recommended that different fragility curves need to be developed for pipes of different material and even for pipes of the same base material.

Application of fragility curves should be limited to a specific range of PGV and PGD values. This study shows that fragility curves cannot be applied to the full range of PGV which can be experienced in a specific earthquake. This particularly applies to low PGV values. As explained in Chapter 8, seismic damage in unpressurised networks is only observed in areas with minimum levels of PGV. The PGV levels differ depending on various types of unpressurised pipe.

## 9.6. Recommendations

This study introduces critical arguments for and against the application of the available fragility curves to estimate earthquake damage in unpressurised wastewater networks. This thesis shows that the application of the available fragility curves should be restricted. Numerous issues should be taken into account such as pipe classification, length and combination of pipe types used in seismic vulnerability analysis by fragility curves.

The 2010 earthquake in Christchurch showed that the available fragility curves cannot be applied in areas with any soil type susceptible to liquefaction. This study shows that the wastewater networks in New Zealand are highly vulnerable to earthquakes. As a result, pre and post-earthquake measures should be applied to increase resistance to seismic forces in wastewater networks.

Earthquakes can directly or indirectly damage wastewater network and the surrounding environment. Indirect effects of a wastewater system failure after an earthquake can affect many environmental issues and the population.

Application of the fragility curves cannot accurately estimate the expected damage rate, especially in areas without any established fragility curves. As shown in the study the estimated damage, caused by wave propagation effects, was divided into two main groups. The differences between calculated damage by Group 1 and Group 2 fragility curves are significant and Group 1 equations estimate the damages to be several times higher than Group 2 equations. The differences between the estimated amounts of damage can vary to more than 12 times when the estimations are gained by applying various fragility curves.

As a result, two scenarios of damage estimation are recommended in wastewater network if the estimated damage by fragility curves is applied. Scenario one is based on estimated damage by Group 1 equation and scenario two comprises damage calculated by Group 2 equations. It is recommended to use Group 1 equations to estimate damage in highly populated, regions with corroded pipes and important areas such as regions with several governmental buildings, hospitals, schools and critical buildings, because failure of wastewater networks in these areas can cause severe direct and indirect damage as explained. Damage in wastewater pipelines are expected to be higher compared to pressurised pipelines such as water and gas.

Application of the available fragility curves should be restricted in areas with liquefiable soils. This study shows that even those fragility curves which are recommended in liquefied soil are not applicable. The recommended fragility curves in this condition are those based on permanent ground deformation. This study indicates that no trend can be seen to demonstrate the correlation between permanent ground deformation and damage rate.

If the fragility curves are used in areas without liquefied soil type, they can be appropriate tools to show the most and least seismic vulnerable region for wastewater pipelines. However, other methods such as numerical methods should be applied for accurate seismic vulnerability analysis of wastewater buried pipelines. Fragility curves should be used as an initial tool for seismic vulnerability analysis in order to recognise the most vulnerable regions. Therefore, it is recommended that the vulnerable regions undergo careful analysis.

The 2010 earthquake in Christchurch revealed how vulnerable concrete and UPVC pipes are. Damage rate in UPVC pipes was high to a degree that it became close to the damage rate in brittle pipes. UPVC pipes behaved similar to brittle pipes in spite of their ductile characteristics. As a result, UPVC pipes should not be classified as ductile pipes in wastewater network. However, more research should be conducted to accurately evaluate the vulnerability of UPVC pipes.

Damage rate in wastewater pipelines in contrast with the pressurised pipelines can increase by buried depth. Main wastewater pipelines are usually buried in deeper trenches compared to smaller pipes. The cost of rehabilitation and repair in wastewater pipelines increases with burial depth. As a result, the protection of the main wastewater pipelines as a pre-earthquake measure should be carried out.

Damage in wastewater pipelines affected by seismic shocks can occur in different PGV intensities based on the type of pipe. Some types of pipes can be affected with low PGV intensity and others can tolerate more PGV. Peak ground velocity greater than 45 cm/s can cause damage in EW, UPVC, CI and concrete pipes and PGV greater than 48 cm/s can cause damage in RCRR and AC pipes.

Pre-earthquake protection plans and post-earthquake recovery plans for wastewater networks are recommended to increase resistance in earthquake prone cities. Post-earthquake rehabilitation plans are more applicable in regions without accurate estimation of seismic effects on wastewater network. As explained in the course of this thesis, accurate estimation of expected damage rate in wastewater pipelines is almost impossible. Moreover, fragility curves cannot estimate the location and type of defects in buried pipelines required for pre-earthquake protection measures. It is therefore recommended that more detailed studies be conducted in order to find better methods of damage assessment.

## 9.7. Future Work

This thesis contributes to the development of a better understanding of earthquake impact on unpressurised wastewater pipelines. It also contributes to an understanding of the limitations of fragility curve application as an estimation tool for calculating seismic damage on unpressurised networks.

After the 2010 Christchurch earthquake, the government, the private sector and the researchers in New Zealand noticed the significance of seismic effects on buried pipelines especially on unpressurised networks. The Christchurch earthquake provides unique opportunities for researchers to concentrate on the concerns and topics discussed in this thesis. Future research topics can be classified in four main categories; working on the applicable fragility curves for New Zealand, providing appropriate method for estimating seismic damage on wastewater pipelines, considering pre and post-earthquake measures in order to decrease seismic effects of the earthquake, and investigating the most vulnerable regions in earthquake prone cities according to soil type and seismological characteristics.

The applied fragility curves in the thesis are developed by correlating seismic damage in pressurised networks in other countries. In these curves only few parameters are considered as independent variables and many factors are ignored. This study illustrates that many factors can cause distortion in application of the available fragility curves, such as the network layout, different pipe types, the quality of construction, the corrosion rate, the buried depth etc. Consequently, the first issue is to develop the appropriate tool for estimating seismic damage in unpressurised buried pipelines in New Zealand for the recognised earthquake parameters.

In order to develop fragility curves in New Zealand, both pressurised and unpressurised pipes should be investigated, and for each category, a different fragility curve should be developed based on the most accurate data. Another significant research in New Zealand should concentrate on types of damage in wastewater pipelines for different earthquake intensities.

Developing the available fragility curves with respect to the factors which can affect seismic vulnerability of buried pipelines can improve accuracy of the available fragility curves. More affecting factors need to be added to the available fragility curves in order to achieve more accurate results. Moreover, finding the correlation between the damage rate and the affecting factors can improve accuracy of the available fragility curves. Matching the theory to practice would greatly improve the breakage prediction in wastewater pipe networks.

Calculating and plotting damage rates in different types of wastewater pipes versus various PGVs diagrams after the Christchurch earthquake in 2010 showed that there is no specific correlation that could estimate the rate by PGV. As a result, other methods should be taken into account for damage estimation in wastewater buried pipelines.

Wastewater systems are required to provide cities with a good standard of health. Improving these systems requires extensive research and understanding of complex interactions of nature on systems.

Working on geological and seismological specifications through micro zonation in seismic prone cities can improve seismic damage estimation on buried pipelines. This type of research should concentrate on regions with a high density of buried pipeline networks. In micro zonation, permanent ground deformation, which may occur after earthquakes, should be investigated. The micro zonation study can be coupled with the process of finding the appropriate fragility curves for New Zealand.

## References:

Abu Dhabi water and electricity authority. (2005). Excavation Guidelines (pp. 22): Abu Dhabi water and electricity authority (ADWEA).

ALA. (2004a). Wastewater System Performance Assessment Guideline, part 1 Guideline Draft (U.S. Department of Security Homeland, Trans.) (pp. 50): Federal Emergency Management Agency and National Institute of Building Sciences

ALA. (2004b). Wastewater System Performance Assessment guideline, part 2 commentary (U.S. Department of Security Homeland, Trans.) (pp. 152): Federal Emergency Management Agency and National Institute of Building Sciences

Allouche, E. N., & Bowman, A. L. (2006). Holistic approach for assessing the vulnerability of buried pipelines to earthquake loads. *Natural Hazards Review, 7*(1), 12-18.

Allouche, E. N., & Parhami, A. (2003). *A simplified approach for selecting a trenchless construction method for municipal sewers.* Paper presented at the New pipeline technologies, security and safety, Baltimore Maryland.

Baratz, B. (2010). Addressing key environmental hazards after an earthquake. *Disaster Risk Management in East Asia and the Pacific, Working paper series No. 11*. Retrieved from http://reliefweb.int/node/379928

Barenberg, M. E. (1988). Correlation of pipeline damage with ground motions. *Journal of Geotechnical Engineering (ASCE), 114*(6 ), 706-711.

Bascand, G. (2011). *Subnational Population Estimates: At 30 June 2011*. Statistics New Zealand.

Berryman, K., Marden, M., Palmer, A., & Litchfield, N. (2009). Holocene rupture of the Repongaere Fault, Gisborne: Implications for Raukumara Peninsula deformation and impact on the Waipaoa Sedimentary System. *New Zealand Journal of Geology and Geophysics, 52*(4), 335-347.

Bevin, J. ( 2000). *Seismic microzoning of the ground shaking hazard from soil geotechnical properties, Gisborne, New Zealand*. Paper presented at the Fourth Australia New Zealand Young Geotechnical Professionals Conference 2000, Australia, Perth, WA: University of Western Australia.

Bizier, P. (2007). *Gravity sanitary sewer design and construction* (secound ed.): American Society of Civil Engineers : Environmental & Water Resources Institute.

Blenheim City Council. (2004). Blenheim sewage treatment plant operation and management plan (pp. 14). Blenheim: Marlborough District Council.

Brown, L. J. (1981). Late Quaternary geology of the Wairau Plain, Marlborough, New Zealand. *New Zealand Journal of Geology & Geophysics, 24*(4), 477-490.

Bruke, E. (1999). Recommendations for disaster preparedness of water and sanitation systems in pacific small island developing states *SOPAC Miscellaneous Report 356* (pp. 25): Pacific Islands Applied GeoScience Commission.

Capacity Company, & Hutt City Council. (2008). Wastewater Assessment Hutt City (pp. 137). Hutt City Capacity Company and Hutt City Council.

Capacity infrastructure services. (2007). Wastewater Asset Management plan 2007 (pp. 195). Hutt City: Capacity infrastructure services and Maunsell Ltd.

Capcity infrastructure services. (2007). Water Supply Asset Management Plan (pp. 206). Hutt City: Capacity infrastructure services and Hutt City Council.

CH2M Beca Ltd. (2007). Assessment of Environmental effects for upgrading of the Blenheim sewage treatment plant (pp. 160). Blenheim: Marlborough District Council and CH2m Beca.

Chen, W.-F., & Scawthorn, C. (2003). *Earthquake Engineering Handbook*. Boca Raton, Florida: CRC Press.

Chen, W. W., Shih, B.-j., Chen, Y.-C., Hung, J. H., & Hwang, H. H. (2002). Seismic response of natural gas and water pipelines in the Ji-Ji earthquake. *Soil Dynamics and Earthquake Engineering, 22*(9-12), 1209-1214.

Christchurch City Council. (2005). Water & sanitary services assessment (pp. 357). Christchurch: Christchurch City Council.

Christchurch City Council. (2010a). Christchurch wastewater network GIS database [Arcmap files: vwWwAcess.shp, vwWwpipe.shp]. In Christchurch City Council (Ed.). Christchurch: Christchurch City Council.

Christchurch City Council. (2010b). Facts and figures Retrieved October 2010, from http://www.ccc.govt.nz/homeliving/wastewater/wastewatercollection/factsandfigures.aspx

Christchurch City Council. (2010c). Wastewater collection  Retrieved October 2010, from http://www.ccc.govt.nz/homeliving/wastewater/wastewatercollection/index.aspx

Clarke, A. M. (1998). The qualitative-quantitative debate: moving from positivism and confrontation to post-positivism and econciliation. *Journal of Advanced Nursing, 27*(6), 1242-1249.

Cooper, J. D. (1984). *Lifeline earthquake engineering : performance, design, and construction* Paper presented at the proceedings of a symposium / sponsored by the Technical Council on Lifeline Earthquake Engineering of the American Society of Civil Engineers in conjunction with the ASCE National Convention, October 4-5, 1984 San Francisco, California.

Creswell, J. W. (2002). *Educational research : planning, conducting, and evaluating quantitative and qualitative research* N.J.: Upper Saddle River.

Crossan, F. (2003). Research philosophy: towards an understanding. *Nurse Researcher 11*(1), 46-55.

Dash, S. R., & Jain, S. K. (2007). An overview of seismic considerations of buried pipelines. *Journal of Structural Engineering (Madras), 34*(5), 349-359.

Davison, R. M. (1998). *An action research perspective of group support systems: How to improve meetings in Hong Kong.* PhD Dissertation, City University of Hong Kong.

Dawson, C. (2002). *Practical research methods a user-friendly guide to mastering research techniques and projetcs*. Oxford: How to Books

Dellow, G. D. (2009a). Gisborne amplification behaviour (pp. 3). Lower Hutt: GNS science.

Dellow, G. D. (2009b). Wellington amplification behaviour (pp. 7). Lower Hutt: GNS science.

Dellow, G. D., Read, S. A. L., Begg, J. G., & Dissen, R. J. V. (1992). Distribution of geological materials in Lower Hutt and Porirua, New Zealand a component of ground shaking hazard assessment. *Bulletin of New Zealand National Society for Earthquake Engineering, 25*(4), 13.

Department of Defense United States of America. (2004). Unified facilities criteria, dewatering and groundwater control (pp. 161): Departments of the army, the navy and the air force.

Doyal, l. (1993). *Discovering knowledge in a world of relationships. In: Kitson A (Ed) Nuring. Art and Science* London: Chapman and Hall.

Duran, O., Althoefer, K., & Seneviratne, L. D. (2002). State of the art in sensor technologies for sewer inspection. *IEEE Sensors Journal, 2*(2), 73-81.

ECLAC. (1991). Manual for Estimating the socio-economic effects of natural disaster (pp. 288). Santiago, Chile,: United Nation Economic Commission for Latin America and the Caribbean.

Edwards, C. (2005). Preparing for disasters. *Public Works, 136*(7), 47-48.

EERI. (2004). Learning from Earthquakes, Preliminary Observations on the Bam, Iran, Earthquake of December 26, 2003 (pp. 12): Earthquake Engineering Research Institute.

EERI. (2010). The MW 8.8 Chile earthquake of February 27,2010 *EERI special earthqauek report* (pp. 20): Earthquake Engineering Research Institute.

Eguchi, R. (1991). Early post-earthquake damage detection for underground lifelines *Final report to National Science Foundation, Dames and Moore P.C.* Los Angeles, California.

Eguchi, R. T., Taylor, C., & Hasselman, T. K. (1983). Earthquake performance of water and natural gas supply system *Seismic component vulnerability models for lifeline risk analysis* (pp. 66). California: J.H. Wiggins company.

Eidinger, J. M. (1998). Water distribution system. In A. J. Schiff (Ed.), *The Loma Prieta, California, Earthquake of October 17, 1989 - Lifelines Performance of Built Environment - lifelines* (pp. A63-A80). Washington: United States Geological Survey.

Eidinger, J. M., & Avila, E. A. (1999). Guidelines for the seismic evaluation and upgrade of water transmission facilities *Technical Council on Lifeline Earthquake Engineering Monograph* (pp. 199): American Society for Civil Engineers,.

Eisenhardt, K. M. (1989). Building theories from Case Study research. *The Academy of Management Review, 14*(4).

El Hmadi, K., & O'Rourke, M. J. (1990). Seismic damage to segmented buried pipelines. *Earthquake Engineering & Structural Dynamics, 19*(4), 529-539.

Ellis, J. B. (2001). *Sewer infiltration/exfiltration and interactions with sewer flows and groundwater quality*. Paper presented at the INTERURBA II Lisbon, Portugal.

Environmental Canterbury Regional Council. (2011). Water contamination in the Christchurch area after the 22 February and 13 June 2011 earthquakes  Retrieved October 2010, 2010, from http://ecan.govt.nz/services/online-services/monitoring/pages/water-contamination-christchurch-post-22-feb-2011-earthquake.aspx

Environmental Services. (2006). Sewer and Drainage Facilities Design Manual (pp. 206). Portland: City of Portland-Bureau of Environmental Services-Engineering Services.

Evans, N., & Wells, J. (2008). Gisborne earthquake impacts on buildings and lifelines (pp. 50 ): OPUS international consultants.

Falvey, C. (1996). Alternative sewers: a good option for many communities *Pipeline* (Vol. 7, pp. 8). Morgantown: West Virginia University.

Feagin, J. R., Orum, A. M., & Sjoberge, G. (1991). *A case for the case study*. Chapel Hill: University of North Carolina Press.

Feeney, C. S., Thayer, S., & Martel, K. (2009). White paper on condition assessment of wastewater collection systems (pp. 74). Washington: United States Environmental Protection Agency.

FEMA. (2003). HAZUS-MH MR3 technical manual *Multi-hazard loss estimation methodology earthquake model* (pp. 699). Washington , D.C.: Department of Homeland Security Emergency Preparedness and Federal Emergency Management Agency.

Flyvbjerg, B. (2006). Five misunderstandings about case-study research. *Qualitatative Inquiry, 12*(2), 219-245.

Ford-Gilboe, M., Campbell, J., & Berman, H. (1995). Stories and numbers: Coexistence without compromise *Advances in Nursing Science, 18*(1), 14-26.

François-Holden, C., Bannister, S., Beavan, J., Cousins, J., Field, B., McCaffrey, R., McVerry, G., Reyners, M., Ristau, J., Samsonov, S., & Wallace, L. (2008). The mw 6.6 Gisborne earthquake of 2007: preliminary records and general source characterisation. *Bulletin of the New Zealand Society for Earthquake Engineering, 51*(4), 266-277.

Frankfort, C., & Nachmias, D. (2000). *Research methods in the social sciences*. London: Edward Arnold.

Geonet Science. (2010). The 2010 Darfield earthquake  Retrieved October 2010, from http://www.geonet.org.nz/earthquake/historic-earthquakes/top-nz/quake-13.html

Gisborne District Council. (2009a). Our 2009-2019  ten year plan (pp. 66). Gisborne: Gisborne District council.

Gisborne District Council. (2009b). Our district, 2009, from http://www.gdc.govt.nz/our-district/

Gisborne District Council. (2010). Wastewater activity management plan (pp. 103). Gisborne: Gisborne District Council.

GNS science. (2009). New Zealand active faults database (Map).  Retrieved 2009, from GNS science http://maps.gns.cri.nz/website/af/viewer.htm

GNS science. (2011). 2010 Darfield (Canterbury) Earthquake, 2011, from http://www.gns.cri.nz/Home/News-and-Events/Media-Releases/Most-damaging-quake-since-1931/Canterbury-quake/Darfield-Earthquake

Gordon, M. (2010). Canterbury earthquake 4 September 2010 lifelines coordination and infrastructure recovery (pp. 50). Christchurch: AECOM.

Gould, N. C. (2003). Understanding the language of seismic risk analysis, 2010, from http://www.irmi.com/expert/articles/2003/gould07.aspx

Government of Western Australia. (2005). Excavation (pp. 115). West Perth: Government of Western Australia, Commission for occupational safety and health.

Harper, P. (2010). No more sewage into quake-hit river. *NZHERALD*  Retrieved November 12, 2010, from http://www.nzherald.co.nz/nz/news/article.cfm?c_id=1&objectid=10687067

Harrison Grierson Company. (2010). Municipal vacuum sewer system, an environmentally sustainable first for NZ, from http://www.harrisongrierson.com/view/story/municipal-vacuum-sewer-system-an-environmentally-sustainable-first-for-nz/m/302/

Heubach, W. F. (2002). *Seismic screening checklists for water and wastewater facilities*: American Society for Civil Engineers.

Hossain, D. M. (2011). Qualiatative reserch process. *Postmodern Openings, 7*, 143-156.

Hutt City Council. (2008a). Hutt City wastewater network GIS data base [ArcMap files: HCC_Sewerpipe_Network.shp]. Hutt City.

Hutt City Council. (2008b). Wastewater reticulation GIS database [ArcMap files]. Hutt City Hutt City Council.

Isenberg, J., & Taylor, C. E. (1984). Performance of water and sewer lifelines in the May 2, 1983 Coalinga, California earthquake (pp. 176-189): U.S. Geological survey.

Isoyama, R., Ishida, E., Yune, K., & Shirozu, T. (2000). Seismic damage estimation procedure for water supply pipelines. *Water supply, International water services association and Japan water 18*(3), 63-68.

Jeon, S. S., & O’Rourke, T. D. (2005). Northridge earthquake effects on pipelines and residential buildings. *Bulletin of the Seismological Society of America, 95*(1), 294-318.

Johnston, M. (2010). Keep boiling tap water, residents urged. Retrieved from http://www.nzherald.co.nz/christchurch-earthquake/news/article.cfm?c_id=1502981&objectid=10671612

Jorgensen, D. L. (1989). *participant observation: A methodology for human studies*. Newbury Park, CA: Sage publication

Katayama, T., Kubo, K., & Sato, N. (1975). Earthquake damage to water and gas distribution systems. *Proceedings of the US National Conference Earthquake Engineering*, 396-405.

Kawashima, K., Obinata, N., Gotoh, K., & Kanoh, T. (1985). *Seismic damage of sewage pipes caused by the 1983 Nihonkai-Chubu earthquake.* Paper presented at the Pressure vessels and piping conference.

Kim, J., O’Connor, S., Nadukuru, S., Lynch, J. P., Michalowski, R., Green, R. A., Pour-Ghaz, M., Weiss, W. J., & Bradshaw, A. (2010). *Behavior of fullscale concrete segmented pipelines under permanent ground displacements.* Paper presented at the SPIE.

King, P. V., & Betz, J. M. (1972). Earthquake damage to a sewer system *Journal of the Water Pollution Control Federation, 44*(5), 859-867.

Kuraoka, S., & Rainer, J. H. (1996). Damage to water distribution system caused by the 1995 Hyogo-ken Nanbu earthquake. *Canadian Journal of Civil Engineering, 23*(3), 665-677.

Lau, D. L., Tang, A., & Pierre, J. R. (1995). Performance of lifelines during the 1994 Northridge earthquake. *Canadian Journal of Civil Engineering, 22*(2), 438-451.

Lund, L., Laughlin, J. M., Edwards, C., Laverty, G., Cornell, H., Guerrero, A. R., Cassaro, M., Godshack, A., Brodt, G., Ballantyne, D. B., Eguchi, R., Pickett, M., Abu-Yasein, O., Lay, C., Schiff, A. J., Blacklock, J. R., & French, S. (1998). The Loma Prieta, California, earthquake of October 17, 1989: Performance of the built environment, lifelines, water and wastewater systems. *Professional Paper 1552-A The Loma Prieta, California, Earthquake of October 17, 1989—Lifelines*, a47-a62.

Marlborough District Council. (2009). Blenheim wastewater reticulation GIS data base [ArcMap files:Blenheim_Sewer_pipes.shp]. In Marlborough District Council (Ed.). Blenheim: Marlborough District Council.

Maruyama, Y., & Yamazaki, F. (2010). *Damage estimation of water distribution pipes following recent earthquakes in Japan*. Paper presented at the Joint conference proceedings, 7th international conference on urban earthquake engineering & 5th international conference on earthquake engineering, Tokyo, Japan.

Mercer, G. (2010). The Christchurch earthquake. *The Christchurch earthquake*, 2010, from http://www.civicassurance.co.nz/bigone.htm

Najafi, M. (2005). *Trenchless technology, pipeline and utility design, construction and renewal*: McGrow-Hill.

NZS 1170.5. (2004). Structural design actions part 5: Earthquake actions New Zealand *NZS 1170.5* (pp. 80). Wellington, New Zealand: Standards New Zealand.

O'Rourke, M., & Ayala, G. (1993). Pipeline damage due to wave propagation. *Journal of Geotechnical Engineering - ASCE, 119*(9), 1490-1498.

O'Rourke, M., & Deyoe, E. (2004). Seismic damage to segmented buried pipe. *Earthquake Spectra, 20*(4), 1167-1183.

O'Rourke, T. D., & Jeon, S. S. (1999). Factors affecting the earthquake damage of water distribution systems. *Technical Council on Lifeline Earthquake Engineering Monograph*(16), 379-388.

O'Rourke, T. D., Stewart, H. E., & Jeon, S. S. (2001). Geotechnical aspects of lifeline engineering. *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering, 149*(1), 13-26.

Orense, R. P. (2010). Liquefaction-induced damage during the 2010 Darfield earthquake (pp. 22). Auckland: Department of civil and environmental engineering, University of Auckland.

Pan American Health Organization. (1998). Natural disaster mitigation in drinking water and sewerage systems, guidelines for vulnerability analysis *Disaster mitigation series* (pp. 90). Washington: Pan American Health Organization, Regional Office of the World Health Organization.

Pender, M. J., Robertson T. W. (1987). Edgecombe earthquake: reconnaissance report. *Bulletin of the New Zealand national society for earthquake engineering, 20*(3), 201-248.

Pervan, G., & Maimbo, H. (2005). *Designing a case study protocol for application in IS research.* Paper presented at the The Ninth Pacific Conference on Information Systems

Petty, I. (2008). The December 20, 2007 Gisborne earthquake, post disaster responsibilities (pp. 64). Gisborne: Gisborne District Council.

Philliber, S. G., Schwab, M. R., & Samsloss, G. (1980). *Social reserch: Guides to a decision-making process*. Itasca, IL: Peacock.

Pickett, M., & Laverty, G. L. (1998). Lessons learned by water and wastewater utilities. *US Geological Survey Professional Paper*(1552 A), A87-A97.

Pineda-Porras, O., & Ordaz, M. (2007). A new seismic intensity parameter to estimate damage in buried pipelines due to seismic wave propagation. *Journal of Earthquake Engineering, 11*(5), 773-786.

Ponterotto, J. G. (2005). Qualitative research in counseling psychology: a primer on research paradigms and philosophy of science. *Journal of Counseling Psychology, 52*(2), 126-136.

Rahman, S. (2004). *State of art review of municipal PVC piping products.* Paper presented at the International Pipeline Conference 2004, Calgary, Alberta Canada.

Ramsay, J. (1998). Problems with empiricism and the philosophy of science: Implications for purchasing research'. *European Journal of Purchasing and Supply, 4*(2), 163-173.

Read, S. A. L., & Sritharan, S. (1993). Reconnaissance report on the Ormond earthquake-10 August 1993. *Bulletin of the New Zealand National Society for Earthquake Engineering, 26*(3), 292-308.

Rentoul, J. (2008, July 2008). [Effects of the 2007 Gisborne earthquake on the Gisborne wastewater system].

Rick, T., P., A. J., Farid, S., Amalia, R., & Mike, L. (2001). Review of construction inspection procedures on accelerated sewer repair program at the bureau of contract administration (pp. 29). Los Angeles: City of Los Angeles office of the controller.

Robertson, E. J., & Smith, E. G. C. (2004). *A seismic site response and ground-shaking hazard assessment for Blenheim, New Zealand*. Paper presented at the 2004 New Zealnad Society for Earthquake Engineering Conference, New Zealand.

Roome, D. (2010). Stress management after the Christchurch earthquake of 2010, from http://www.suite101.com/content/stress-management-after-the-christchurch-earthquake-2010-a293877#ixzz12YAZDr6N

Sasaki, T., Koseki, J., Matsuo, O., Saito, K., & Yamashita, M. (1999). Analyses of damage to sewer pipes in Shibetsu Town during the 1994 Hokkaido-Toho-Oki earthquake. *Technical Council on Lifeline Earthquake Engineering Monograph*(16), 247-256.

Saunders, W. (2011). How long is your piece of string - are current planning timeframes for natural hazards long enough? Retrieved from www.planning.org.nz

Scawthorn, C., Miyajima, M., Ono, Y., Kiyono, J., & Hamada, M. (2006). Lifeline aspects of the 2004 Niigata Ken Chuetsu, Japan, earthquake. *Earthquake Spectra, 22*(SUPPL. 1), S89-S110.

Schiff, A. J. (1995). *Northridge earthquake: lifeline performance and post-earthquake response*: American Socoety of Civil Engineers.

Schiff, A. J., Abrahamson, N., Matsuda, E., Tang, A., Lau, D. T., O'Rourke, M., Zhao, J., Hu, D., Lee, C.-H., & Yashinsky, M. (2000). Chi-Chi, Taiwan, earthquake of September 21, 1999 lifeline performance. In A. Schiff & A. K. Tang (Eds.), *Technical Council on Lifeline Earthquake Engineering Monograph No. 18* (pp. 217): American Society of Civil Engineers.

Shavelson, R., & Townes, L. (2002). *Scientific reseach in education*. Washington DC: National Academy Press.

Shinozuka, M. (1995). The Hanshin-Awiji earthquake of January 17, 1995: performance of lifelines (pp. 316). Buffalo, N.Y.: National Centre for Earthquake Engineering Research.

Shuji, T. (2005). Measures for recovery against seismic damage to wastewater systems (pp. 14). Tsukuba Japan: National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure and Transport.

Sirtharan, S. A. L. R. a. S. (1993). Reconnaissance report on the Oromond earthquake, 10 August 1993. *Bulletin of the New Zealand national society for earthquake engineering, 26*(3), 292-308.

Smith, W. D., & Berryman, K. R. (1983). Revised estimation of earthquake hazard in New Zealand. *Bulletin of the New Zealand National Society for Earthquake Engineering, 16*(4), 259-272.

Statistics New Zealand. (2009a). National population estimation at 30 June 2009, 2009, from http://search.stats.govt.nz/search?w=population

Statistics New Zealand. (2009b). Population mobility of urban/rural profile areas, from http://www.stats.govt.nz/store/2009/01/mobility-urban-rural.htm

Statistics New Zealand. (2011). Population estimates.  Retrieved June 2012, from Statistics New Zealand www.stats.govt.nz/inforshare/selectVariable.aspx?pxid=1820d911-ac2e-4b59-847b-923bab10b0ff

Stirling, M. W., McVerry, G. H., & Berryman, K. R. (2002). A new seismic hazard model for New Zealand. *Bulletin of the Seismological Society of America, 92*(5), 1878-1903.

Strachan, C. M., & Glogau, O. A. (1966). Report on damage in the Gisborne earthquake  1966 (Vol. 194, pp. 35-45): Department of Scientific and industrial research.

Sudman, S., & Bradburn, N. M. (1982). *Asking questions: a particular guide to questionnaire design market reseach, political polls, social and health questionnaires*. San Francisco: Jossey-Bass.

Takada, S., Hassani, N., & Fukuda, K. (2002). Damage directivity in buried pipelines of Kobe city during the 1995 earthquake. *Journal of Earthquake Engineering, 6*(1), 1-15.

Tan, W. (2004). *Practical reserach methods*. Singapore: Pearson Prentice Hall.

Tang, A., Cooper, T., Duenas, L., Eidinger, J., Fullerton, B., Imbsen, R., Kempner, L., Kwasinski, A., Pyrch, A., Schiff, A., & Wang, Y. (2010). Preliminary report, 27 February 2010 $M_W$8.8 offshore Maule, Chile earthquake (pp. 25): American Society of Civil Engineers, Technical Council on Lifeline Earthquake Engineering.

Tobin, P. K. J. (2006). *The use of stories and storytelling as knowledge sharing practices : a case study in the South African mining industry* DPhil, University of Pretoria, Pretoria.

Toprak, S. (1998). *Earthquake Effects on Buried Lifeline System.* Doctor of philosophy, Cornell University, New York.

Toprak, S., Koc, A. C., Cetin, O. A., & Nacaroglu, E. (2008). *Assessment of buried pipeline response to earthquake loading by using GIS*. Paper presented at the The 14th world conference on earthquake engineering, Beijing, China. http://www.iitk.ac.in/nicee/wcee/article/14_06-0077.PDF

Toprak, S., & Taskin, F. (2007). Estimation of earthquake damage to buried pipelines caused by ground shaking. *Natural Hazards, 40*(1), 1-24.

Trochim, W. (1985). Outcome patern matching and program theory. *Evaluation and Program Planning, 12*, 355-366.

USGS. (2010). Explantion of Parameters, from http:/geohazards.usgs.gov/deaggint/2002/documentation/parm.php

Van-Dissen, R. J., J.Taber, J., Stephenson, W. R., Sritheran, S., Read, S. A. L., McVerry, G. H., Dellow, G. D., & Barker, P. R. (1992). Earthquake Ground Shaking Hazard Assessment for the Lower Hutt and Porirua Areas, New Zealand. *Bulletin of New Zealand National Society for Earthquake Engineering, 25*(4), 286-301.

Wang, L. R. L., Ishibashi, I., & Wang, J. C. C. (1991). GIS applications in seismic loss estimation model for Portland, Oregon water and sewer systems *Earthquake hazards in  the Pacific Northwest of the United States* (pp. 71): United States department of the interior geological survey.

Webb, C. (1989). Action research: philosophy, methods and personal experiences. *Journal of Advanced Nursing, 14*(5), 403-410.

Webb, T. H., Wesnousky, S. G., & Helmberger, D. V. (1985). A body-wave analysis of the 1966 Gisborne, New Zealand, earthquake. *Tectonophysics, 113*(3-4), 271-282.

Wellington City Council. (2010). Sewerage & Wastewater, from http://www.wellington.govt.nz/services/sewerage/history/history.html

Wells, T., Melchers, R. E., & Bond, P. (2010). Factors involved in the long term corrosion of concrete sewers (pp. 12). Newcastle, Australia: SCORe.

Western Bay of Plenty District Council. (2011). Small Communities Wastewater, Frequently Asked Questions For Pressur eSewer, from http://www.westernbay.govt.nz/Documents/Projects/Small%20Communities%20Wastewater/FrequentlyAskedQuestionsForPressureSewer.pdf

Willing, C. (2001). *Introducing qualitative research in psychology*. Buckingham: Open University Press

Winkler, G. (2008). Ground effect on the Gisborne earthquake (pp. 61). Gisborne: Land Development & Exploration Ltd.

Yetton, M., Traylen, N., & McCahon, I. (2011). 2010 Canterbury earthquake liquefaction report (pp. 32). Christchurch: Geotech Consulting LTD.

Yin, R. K. (2003). *Case study research : design and methods*. Thousand Oaks, California: Sage Publications

Zare, M. R. (2008). [Gisborne wastewater system site visiting and personal communication].

Zare, M. R. (2010a). [Comparison of sewer reapir methods after the 2010 Christchurch earthquake].

Zare, M. R. (2010b, September 2010). [Damage observation in the Christchurch wastewater reticulation after the 2010 earthquake].

Zare, M. R., Wilkinson, S., & Potangaroa, R. (2010). Vulnerability of wastewater treatment plants and wastewater pumping stations to earthquake. *International Journal of Strategic Property Management, 14*(4).

Zhao, J. (2009). [PGV estimation in New Zealand].

Zhao, J. X., Cousins, J., Lukovic, B., & Smith, W. (2008). *Critical factors for restoration of water supply pipelines in the Hutt City, New Zealand after a magnitude 7.5 earthquake from the Wellington fault.* Paper presented at the The 14th World Conference on Earthquake Engineering, October 12-17, 2008, Beijing, China.

**Appendices:**

## Appendix 1: list of paper

- Zare, M. R., Wilkinson, S., & Potangaroa, R. (2011). Earthquake damage in wastewater systems and post-earthquake repair methods; limitation and practice. Paper presented at the Australian Earthquake Engineering Society Conference, Novotel Barossa Valley Resort, South Australia.

- Potangaroa, R., Wilkinson, S., Zare, M., & Steinfort P. (2011). The Management of Portable Toilets in the Eastern Suburbs of Christchurch after the February 22, 2011 Earthquake. Australasian Journal of Disaster and Trauma Studies, 2, 35-48.

- Zare, M. R., Wilkinson, S., & Potangaroa, R. (2010). Vulnerability of wastewater treatment plants and wastewater pumping stations to earthquakes. Vulnerability of wastewater treatment plants and wastewater pumping stations to earthquakes, 14, 408-420

- Zare, M. R., Wilkinson, S., Potangaroa, R., & Heays, K. (2010). Seismic vulnerability of wastewater systems in urban areas, a case study in New Zealand. Paper presented at the Sustainable Building 2010 Brazil (SB10Brazil) conference, São Paulo, Brazil.

- Zare, M. R., & Wilkinson, S. (2010). Resilience of Wastewater Pipelines in Earthquakes Paper presented at The 9th US National & 10th Canadian Conference on Earthquake Engineering, Toronto, Canada

- Zare, M. R., Wilkinson, S., & Potangaroa, R. (2010). Vulnerability of Wastewater Treatment Plants and Wastewater Pumping Stations to Earthquakes. Paper presented at the 18th CIB World Building Congress, Salford, United Kingdom.

- Zare, M. R., & Wilkinson, S. (2010). Earthquake effects on metropolitan cities lifelines Paper presented at the Fourteenth International Water Technology Conference, IWTC 14 2010, Cairo, Egypt.

- Zare, M. R., & Wilkinson, S. (2009). Wastewater pipeline vulnerability to earthquakes. Paper presented at the AEES 2009, New Castle, Australia.

- Zare, M. R., & Wilkinson, S. (2009). Earthquake effects on Waste Water Systems. Paper presented at the NZSEE 2009, Christchurch, New Zealand.

- Zare, M. R., & Wilkinson, S. (2009). Wastewater system vulnerability to earthquake. Paper presented at the 1st International Postgraduate Conference on Infrastructure and Environment, Hong Kong.

- Zare, M. R., & Wilkinson, S. (2008). Earthquake effect on buried pipelines. Paper presented at the AOUTLE postgraduate conference, Auckland, New Zealand.

# Appendix 2: the spectral shape factor

Table 1: $C_h(T)$ the spectral shape factor (NZS 1170.5)

| Soil type | Strong rock | Shallow soil | Deep or soft soil | Very soft soil |
|---|---|---|---|---|
| T(s) | A&B | C | D | E |
| 0 | 1.89 | 2.36 | 3 | 3 |
| 0.1 | 1.89 | 2.36 | 3 | 3 |
| 0.2 | 1.89 | 2.36 | 3 | 3 |
| 0.3 | 1.89 | 2.36 | 3 | 3 |
| 0.4 | 1.89 | 2.36 | 3 | 3 |
| 0.5 | 1.6 | 2 | 3 | 3 |
| 0.6 | 1.4 | 1.74 | 2.84 | 3 |
| 0.7 | 1.24 | 1.55 | 2.53 | 3 |
| 0.8 | 1.12 | 1.41 | 2.29 | 3 |
| 0.9 | 1.03 | 1.29 | 2.09 | 3 |
| 1 | 0.95 | 1.19 | 1.93 | 3 |
| 1.5 | 0.7 | 0.88 | 1.43 | 2.21 |
| 2 | 0.53 | 0.66 | 1.07 | 1.66 |
| 2.5 | 0.42 | 0.53 | 0.86 | 1.33 |
| 3 | 0.35 | 0.44 | 0.71 | 1.11 |
| 3.5 | 0.26 | 0.32 | 0.52 | 0.81 |
| 4 | 0.2 | 0.25 | 0.4 | 0.62 |
| 4.5 | 0.16 | 0.2 | 0.32 | 0.49 |

# Appendix 3: Sub soil class in New Zealand (Source: NZS 1170.5 (2004)

Class A – Strong rock

Class A is defined as strong to extremely-strong rock with:

(a) Unconfined compressive strength greater than 50 MPa; and

(b) An average shear-wave velocity over the top 30 m greater than 1500 m/s; and

(c) Not underlain by materials having a compressive strength less than 18 MPa or a shear wave velocity less than 600 m/s.

Class B – Rock

Class B is defined as rock with:

(a) A compressive strength between 1 and 50 MPa; and

(b) An average shear-wave velocity over the top 30 m greater than 360 m/s; and

(c) Not underlain by materials having a compressive strength less than 0.8 MPa or a shear wave velocity less than 300 m/s.

A surface layer of no more than 3 m depth of highly-weathered or completely-weathered rock or soil (a material with a compressive strength less than 1 MPa) may be present.

Class C – Shallow soil sites

Class C is defined as sites where:

(a) They are not class A, class B or class E sites; and

(b) The low amplitude natural period is less than or equal to 0.6 s; or

(c) Depths of soil do not exceed those listed in Table 3.2.

The low amplitude natural period may be estimated from four times the shear-wave travel time from the surface to rock, be estimated from Nakamura ratios or from recorded earthquake motions, or be evaluated in accordance with Clause 3.1.3.7 for sites with layered subsoil, according to the hierarchy of methods given in Clause 3.1.3.1.

Class D – Deep or soft soil sites

Class D is defined as sites:

(a) That are not class A, class B or class E sites; and

(b) Where low-amplitude natural period is greater than 0.6 s; or

(c) With depths of soils exceeding those listed in Table 3.2; or

(d) Underlain by less than 10 m of soils with an undrained shear-strength less than 12.5 kPa or soils with SPT N-values less than 6.

The low amplitude natural period may be determined in accordance with Clause 3.1.3.4.

Class E – Very soft soil sites

Class E is defined as sites with:

(a) More than 10 m of very soft soils with undrained shear strength less than 12.5 kPa; or

(b) More than 10 m of soils with SPT N-values less than 6; or

(c) More than 10 m depth of soils with shear-wave velocities of 150 m/s or less; or

(d) More than 10 m combined depth of soils with properties as described in (a), (b) and (c) above.

## Appendix 4: Soil type (Source: GNS science (2009))

### ***Wellington and Hutt City Amplification Behaviour***

### *Class A: Strong Rocks*

Geological units assigned strengths greater than 20MPa in the Wellington area include:

1. Graded sandstone and siltstone;
2. Channelled sandstone;
3. Graded sandstone and siltstone, minor siltstone, and conglomerate;
4. Siliceous mudstone, siltstone, and sandy siltstone;
5. Basal breccia, conglomerate, pebbly siltstone to muddy sandstone, alternating sandstone and siltstone;
6. Poorly bedded sandstone, graded sandstone and mudstone;
7. Volcanic rocks;
8. Greywacke;
9. Deformed and indurated sandstone and mudstone;
10. Argillite and basalt;
11. Basalt;
12. Basalt and chert;
13. Basalt and limestone;
14. Melange;
15. Volcanics;
16. Deformed and indurated sandstone and mudstone;
17. Argillite and basalt;
18. Argillite, chert, and basalt;

19. Chert;

20. Limestone; and

21. Melange, argillite, and chert.

*Class B: Weak Rocks*

Geological units assigned strengths between 1MPa and 20MPa in the Wellington area include:

1. SmectiticMudstone;

2. Sandstone and mudstone and olistostrome;

3. Calcareous mudstone with discontinuous limestone and tuffbeds, silty sandstone and sandy siltstone with minor grit, and bedded;

4. Calcareous mudstone with discontinuous limestone and tuffbeds;

5. Silty sandstone and sandy siltstone with minor grit, and bedded sandstone and calcareous sandstone with minor conglomerate;

6. Quartzite, glauconitic sandstone, and minor siltstone;

7. Pebbly greensand, calcareous mudstone, well sorted sandstone, concretionary mudstone, limestone, and sparse tuff beds;

8. Shelly limestone, coqina, and calcerous sandstone, and siltstone and sandy mudstone with minor grit and conglomerate;

9. Limestone; and

10. Mudstone.

*Class C: Shallow Soils*

Geological units assigned to the shallow soil class are:

1. Beach deposits;

2. Alluvium;
3. Fan, scree, and colluvial gravels;
4. Mixed lithology;
5. Fan gravels;
6. Loess-covered fan gravel; and
7. Landslide debris.

*Class D: Deep Soils*

Geological units assigned to the deep soil class are:

1. Floodplain gravels;
2. Mixed lithology;
3. Beach gravels with sand and mud;
4. Mobile dunes;
5. Fixed dunes;
6. Fill -rubbish;
7. Alluvial gravel with sand and silt;
8. Alluvial gravel with sand;
9. Alluvial gravel, silty clay, and peat;
10. Marine gravel with sand;
11. Loess-covered fan gravel, alluvial gravel, and lacustrine silt;
12. Undifferentiated deposits;
13. Marine and marginal marine gravel, sand, and silt;
14. Loess-covered alluvial gravel; and

15. Alluvium, silt, peat, and loess.

*Class E: Soft Soils*

Geological units assigned to the soft soil class are:

1. Swamp deposits; and
2. Fill - construction.

### ***Blenheim Amplification Behaviour***

*Class A: Strong Rocks*

Geological units assigned strengths greater than 20MPa in the Blenheim area include:

1. Marlborough Schist.

*Class B: Weak Rocks*

Geological units assigned strengths between 1MPa and 20MPa in the Blenheim area include:

1. Freshwater conglomerate with brown to grey poorly sorted boulders to pebbles in brown coarse sand.

*Class C: Shallow Soils*

No geological units are assigned to the shallow soil class.

*Class D: Deep Soils*

Geological units assigned to the deep soil class are:

1. Gravel coasre sand and sand of Boulder Bank;
2. Gravel coarse sand and sand of inland older beach ridges and swales;

3. Gravel coarse sand and sand of beach ridges and swales;

4. Sand dunes;

5. Sand silt and mud lagoon deposits better drained and less saline;

6. Sand and gravel of present beach;

7. Glacial outwash of weathered brown-yellow gravel sand and silt; covered by 2 to 3 m of loess;

8. Alluvial gravel sand and silt at higher level and well-drained;

9. Alluvial gravel sand and silt;

10. Fluvial (glacial outwash) brown and blue gravel sand and silt; and

11. Glacial outwash of weathered to slightly weathered brown gravel sand and silt; covered by a layer of loess up to 1m thick.

*Class E: Soft Soils*

Geological units assigned to the soft soil class are:

1. Sand silt and mud lagoon deposits;

2. Sand silt and mud lagoon deposits liable to flooding;

3. Unstabilised river silt sand and gravel in present river beds;

4. Mud silt and sand of inter-tidal river channels and lagoons;

5. Swamps between beach ridges;

6. Man-made ground includes railway and road embankments and a coastal breakwater; and

7. Alluvial silt adjacent to present or historic river flood or man-made diversion channels and reclaimed or drained swamps.

## ***Gisborne Amplification Behaviour***

*Class A: Strong Rocks*

No geological units are assigned strengths greater than 20MPa in the Gisborne area.

*Class B: Weak Rocks*

Geological units assigned strengths between 1MPa and 20MPa in the Gisborne area include:

1. Smectitic claystone (c.f. bentonite), greensand, and siliceous mudstone melange forming lower Tertiary (and upper most Cretaceous); and

2. Mudstone, sandstone, limestone, and agglomerate forming upper Tertiary bedrock.

*Class C: Shallow Soils*

Geological units assigned to the shallow soil class are:

1. Compacted gravels, sand, mud, and tephra forming Mangatuna Formation.

*Class D: Deep Soils*

Geological units assigned to the deep soil class are:

1. Gravel, sand, silt, and pumice forming remnant river terraces;

2. Gravel, sand, and silt forming higher alluvial terrace remnants (<6000yrs);

3. Sand and minor gravel forming modern beach deposits and unstabilised dunes;

4. Silt, sand, and gravel forming modern beach deposits and modern river bed;

5. Sand forming stabilised sand dunes and beach sand; and

6. Sand forming stabilised sand dunes and beach sand overlain by pumice.

7. Gravel, sand, silt, and pumice forming flood plain deposits.

### *Class E: Soft Soils*

Geological units assigned to the soft soil class are:

1. Gravel, sand, and silt forming river channels subject to flooding in historic time;

2. Gravel, sand, silt, and pumice forming older river channels subject to flooding in historic time;

4. Rock boulders, gravel, sand, and soil forming reclamation;

5. Saturated mud forming swamp deposits; and

6. Unstabilised river silt, sand, and gravel forming modern river bed.

# Appendix 5: Z-VALUES AND SHORTEST MAJOR FAULT DISTANCES *D* FOR NEW ZEALAND LOCATIONS (Source: NZS 1170.5 (2004))

**Table1: The Z values for New Zealand's cities**

| # | Location | Z | D |
|---|---|---|---|
| 1 | Kaitaia | 0.13 | - |
| 2 | Paihia/Russell | 0.13 | - |
| 3 | Kaikohe | 0.13 | - |
| 4 | Whangarei | 0.13 | - |
| 5 | Dargaville | 0.13 | - |
| 6 | Warkworth | 0.13 | - |
| 7 | Auckland | 0.13 | - |
| 8 | Manakau City | 0.13 | |
| 9 | Waiuku | 0.13 | - |
| 10 | Pukekohe | 0.13 | - |
| 11 | Thames | 0.16 | - |
| 12 | Paeroa | 0.18 | - |
| 13 | Waihi | 0.18 | - |
| 14 | Huntly | 0.15 | - |
| 15 | Ngaruawahia | 0.15 | - |
| 16 | Morrinsville | 0.18 | - |
| 17 | Te Aroha | 0.18 | |
| 18 | Tauranga | 0.2 | - |
| 19 | Mount | 0.2 | |
| 20 | Hamilton | 0.16 | - |
| 21 | Cambridge | 0.18 | - |
| 22 | TeAwamutu | 0.17 | |
| 23 | Matamata | 0.19 | - |
| 24 | Te Puke | 0.22 | |
| 25 | Putaruru | 0.21 | - |
| 26 | Tokoroa | 0.21 | - |
| 27 | Otorohanga | 0.17 | - |
| 28 | Te Kuiti | 0.18 | |
| 29 | Mangakino | 0.21 | - |
| 30 | Rotorua | 0.24 | - |
| 31 | Kawerau | 0.29 | - |
| 32 | Whakatane | 0.3 | - |
| 33 | Opotiki | 0.3 | - |
| 34 | Ruatoria | 0.33 | - |
| 35 | Murupara | 0.3 | - |
| 36 | Taupo | 0.28 | - |
| 37 | Taumarunui | 0.21 | - |
| 38 | Turangi | 0.27 | - |
| 39 | Gisborne | 0.36 | - |
| 40 | Wairoa | 0.37 | - |

## Table1: The Z values for New Zealand's cities (continued)

| # | Location | Z | D |
|---|---|---|---|
| 41 | Waitara | 0.18 | - |
| 42 | New Plymouth | 0.18 | |
| 43 | Inglewood | 0.18 | - |
| 44 | Stratford | 0.18 | - |
| 45 | Opunake | 0.18 | - |
| 46 | Hawera | 0.18 | - |
| 47 | Patea | 0.19 | - |
| 48 | Raetihi | 0.26 | - |
| 49 | Ohakune | 0.27 | - |
| 50 | Waiouru | 0.29 | - |
| 51 | Napier | 0.38 | - |
| 52 | Hastings | 0.39 | - |
| 53 | Wanganui | 0.25 | - |
| 54 | Waipawa | 0.41 | - |
| 55 | Waipukurau | 0.41 | - |
| 56 | Taihape | 0.33 | - |
| 57 | Marton | 0.3 | - |
| 58 | Bulls | 0.31 | - |
| 59 | Feilding | 0.37 | - |
| 60 | Palmerston North | 0.38 | |
| 61 | Dannevirke | 0.42 | 10 |
| 62 | Woodville | 0.41 | |
| 63 | Pahiatua | 0.42 | 8 |
| 64 | Foxton/Foxton | 0.36 | |
| 65 | Levin | 0.4 | - |
| 66 | Otaki | 0.4 | - |
| 67 | Waikanae | 0.4 | 15 |
| 68 | Paraparaumu | 0.4 | 14 |
| 69 | Masterton | 0.42 | 6 |
| 70 | Porirua | 0.4 | 8 |
| 71 | Wellington CBD | 0.4 | 2 |
| 72 | Wellington | 0.4 | 0 |
| 73 | Hutt Valley– | 0.4 | 0-.4 |
| 74 | Upper Hutt | 0.42 | |
| 75 | Eastbourne–Point | 0.4 | 4-8 |
| 76 | Wainuiomata | 0.4 | 5 |
| 77 | Takaka | 0.23 | - |
| 78 | Motueka | 0.26 | - |
| 79 | Nelson | 0.27 | - |
| 80 | Picton | 0.3 | 16 |
| 81 | Blenheim | 0.33 | 0 |
| 82 | St Arnaud | 0.36 | |
| 83 | Westport | 0.3 | - |
| 84 | Reefton | 0.37 | - |

## Table1: The Z values for New Zealand’s cities (continued)

| # | Location | Z | D |
|---|---|---|---|
| 85 | Murchison | 0.34 | - |
| 86 | Springs Junction | 0.45 | |
| 87 | Hanmer Springs | 0.55 | 2-6 |
| 88 | Seddon | 0.4 | 6 |
| 89 | Ward | 0.4 | 4 |
| 90 | Cheviot | 0.4 | - |
| 91 | Greymouth | 0.37 | - |
| 92 | Kaikoura | 0.42 | 12 |
| 93 | Harihari | 0.46 | 4 |
| 94 | Hokitika | 0.45 | - |
| 95 | Fox Glacier | 0.44 | |
| 96 | Franz Josef | 0.44 | |
| 97 | Otira | 0.6 | 3 |
| 98 | Arthurs Pass | 0.6 | |
| 99 | Rangiora | 0.33 | - |
| 100 | Darfield | 0.3 | - |
| 101 | Akaroa | 0.16 | - |
| 102 | Christchurch | 0.22 | - |
| 103 | Geraldine | 0.19 | - |
| 104 | Ashburton | 0.2 | - |
| 105 | Fairlie | 0.24 | - |
| 106 | Temuka | 0.17 | - |
| 107 | Timaru | 0.15 | - |
| 108 | Mt Cook | 0.38 | |
| 109 | Twizel | 0.27 | - |
| 110 | Waimate | 0.14 | - |
| 111 | Cromwell | 0.24 | - |
| 112 | Wanaka | 0.3 | - |
| 113 | Arrowtown | 0.3 | - |
| 114 | Alexandra | 0.21 | - |
| 115 | Queenstown | 0.32 | - |
| 116 | Milford Sound | 0.54 | |
| 117 | Palmerston | 0.13 | - |
| 118 | Oamaru | 0.13 | - |
| 119 | Dunedin | 0.13 | - |
| 120 | Mosgiel | 0.13 | - |
| 121 | Riverton | 0.2 | - |
| 122 | Te Anau | 0.36 | |
| 123 | Gore | 0.18 | - |
| 124 | Winton | 0.2 | - |
| 125 | Balclutha | 0.13 | - |
| 126 | Mataura | 0.17 | - |
| 127 | Bluff | 0.15 | - |
| 128 | Invercargill | 0.17 | - |