Hydropower Energy Recovery from the Water Supply Network: Feasibility, Risk Analysis, Optimisation and Implementation

By

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Declaration

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Summary

The provision of clean and reliable water supply is highly energy intensive. Recent research has focused on the development and application of energy and hydraulic efficiency measures to streamline water usage and improve energy security. This thesis presents the findings of an investigation of the potential for hydropower energy recovery within water supply networks (WSNs). Four core themes are presented and discussed; Feasibility, Risk Analysis, Optimisation and Implementation.

Firstly, a feasibility study was undertaken of the potential for hydropower energy recovery at existing locations within the WSNs of Ireland and Wales. Locations investigated included pressure reducing valves (PRVs), break pressure tanks and reservoirs. The total power generation available at the sites investigated was found to be 1.3MW. Flow rate variation, turbine selection and turbine costs were identified as key factors when determining the investment payback periods of potential projects.

Secondly, through an analysis of ten years of long term flow rate data, a methodology for future forecasting of flow rates at potential hydropower locations was developed. Long term flow variation due to changes in water demand will impact upon power generation capacity. Large changes in demand could render a turbine unsuitable in the future. Multiple linear regression and artificial neural network forecasting techniques were applied and compared as potential methods for long term forecasting of flow rates at hydropower locations. Using these models, future scenarios for flow rates and hence power outputs were forecast.

Thirdly, an optimisation algorithm was developed which can be applied by water service providers for the selection of new locations for hydropower energy recovery in a given WSN. Though existing locations in WSNs, such as PRVs, present opportunities for their replacement with a hydropower turbine, these points may not be optimal locations to install turbines for maximum power generation. The algorithm presented finds optimal locations to install turbines for maximised power generation. Three optimisation techniques were tested for their suitability, including genetic algorithms, non-linear programming and mixed-integer non-linear programming (MINLP). MINLP was found to be the most suitable method.

Finally, though technical solutions exist for hydropower energy recovery within WSNs there has not yet been widespread uptake of the technology by industry. To investigate the organisational, management and regulatory issues related to the implementation of this technology in practice, two case studies of previously installed turbines on water supply infrastructure were developed. Following these case studies, a framework for streamlining the installation of future MHP projects was also developed.

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List of Publications

Journal Publications:

1. **Corcoran, L.**, Coughlan, P., McNabola, A. (2013). Energy Recovery Potential using Micro Hydro Power in Water Supply Networks in the UK & Ireland. *Water Science & Technology: Water Supply*, Vol 13 No 2 p552560.

2. **Corcoran, L.**, Coughlan, P., McNabola, A. (2014). Innovation and the Water Industry: Case studies of innovation, education and collaboration. *TCD Journal of Postgraduate Research*, Vol 12 p8-23.

3. McNabola, A., Coughlan, C., **Corcoran, L.**, Power, C., Williams, A.P., Harris, I.M., Gallagher, J., Styles, D. (2013). Energy Recovery in the Water Industry using Micro-hydropower: An Opportunity to Improve Sustainability. *Water Policy*, Vol 16 p168-183.

Conference Publications:

1. **Corcoran, L.**, McNabola, A., Coughlan P. (2014). Optimisation of Water Supply Networks for Combined Leakage Reduction and Hydropower Generation, International Water Association (IWA): World Water Congress, Lisbon, Portugal, Sept 22-26 2014.

2. **Corcoran, L.**, Coughlan, P., McNabola, A. (2012). Energy Recovery Potential within Water Supply Networks of Ireland using Micro-Hydropower Turbines: Analysis of Case Studies, The IWA 9th International Symposium on Water Supply Technology, Yokohama, Japan, 20-22nd Nov 2012.

3. **Corcoran, L.**, McNabola, A., Coughlan P. (2012). Energy Recovery Potential of the Dublin Region Water Supply Network, IWA: World Congress on Water, Climate and Energy, Dublin, Ireland, May 13-18th 2012.

4. **Corcoran, L.**, Power, C., McNabola, A., Coughlan, P. (2011). Investigating of the Technical and Commercial Feasibility of Energy Recovery in the Water Industry. Dublin Region Higher Education Alliance, Future Voices Competition, Dublin Castle, 7th Sept 2011.

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A	Cross-sectional area (m^2)
C_d	Coefficient of discharge
e_0	Efficiency rate (%)
e	Elevation (m)
g	Acceleration due to gravity (m/s^2)
L	Leakage rate (m^3/s)
m	Mass (g)
p	Pressure head (m)
Q	Flow rate (m^3/s)
ρ	Density (kg/m^3)

List of Acronyms

ANN	Artificial Neural Network
BPT	Break Pressure Tank
CE	Engineer-in-Chief
CSO	Central Statistics Office, Ireland
DCC	Dublin City Council
DMA	District Metered Area
EU	European Union
GA	Genetoc algorithm
GHG	Greenhouse Gas
kW	Kilowatt
kWh	Kilowatt-hour
MHP	Micro-Hydropower
MW	Megawatt
MWh	Megawatt-hour
NPV	Net Present Value
OECD	The Organisation for Economic Co-operation and Development
ONS	Office of National Statistics, UK
PAT	Pump-as-Turbine
PB	Payback
PRV	Pressure Reducing Valve
REFIT	Renewable Energy Feed in Tariff
UK	United Kingdom
WSN	Water supply network
WSP	Water service providers
WW	Welsh Water

CHAPTER 1

Introduction

1.1 Background

World population is at an all time high at 7.243 billion as of the 1st of July 2014, and is set to grow significantly into the future, especially in urban areas. Currently the majority of world population live in urban areas. Since 1950, rapid urbanisation has seen urban population grow from 30% of total world population to 54% as of 2014. Furthermore, it is projected that by 2050, 66% of world population will be concentrated in urban areas (United Nations, 2014*a*). Rapid and unplanned urban growth threatens the sustainability of core services and infrastructure, such as water and wastewater services. Improvements in the efficiency and sustainability of water supply networks are vital to protect and maintain a safe and sustainable water supply in the future.

The theme of the 2014 UN World Water Day was 'Water and Energy', with the objective of highlighting the close links and interdependencies between water and energy. The water industry is a large user of energy. Globally, it is estimated that 8% of total energy generation is expended on pumping, treating and transporting water to consumers (United Nations, 2014*b*). In the UK, the water industry consumes approximately 3% of the UK energy demand with 8.5% of that generated by renewable energy resources

(Howe, 2009). Similarly in the US, the supply and conveyance of water is estimated to consume over 3% of total US electricity (King and Webber, 2008). In Japan, the total electricity consumption of water utilities accounts for 1% of total nationwide energy usage (Arai et al., 2014). Investigations have also shown that 45% of the total energy requirements for the provision of water services are associated with water distribution, with 29% associated with wastewater management and 26% for water treatment (Kwok et al., 2010). Developments in industrial, agricultural and domestic water use and in water quality regulation have greatly intensified energy requirements for the treatment and distribution of water (Rothausen and Conway, 2011). In addition, with global population at an all time high, the demand for water has never been greater, further increasing the total energy consumption of the water industry (Zilberman et al., 2008).

There has been much focus in recent years on the research and development of renewable energy sources due to rising oil prices and the introduction of climate change policies aimed at decreasing CO_2 emissions. The Kyoto Protocol, signed and ratified in December 1997, was a landmark piece of legislation recognising the global impact of climate change, and legally bound participating nations to reduce emissions and increase renewable energy generation. Moreover, the EU have set legally binding targets for all member states as part of Directive 2009/28/EC with the aim of reaching an overall target of 20% of all electricity generated to be from renewable sources by 2020. These status of these targets in 2004 and 2012 for each member state are shown in Figure 1.1. Historic climate change predictions, unlike many economic predictions, are today being proven to be true. There is an increasingly urgent need for all participating nations to meet these legally binding targets and to further incentivise both renewable energy and energy recovery incentives.

The water supply network in Ireland is becoming increasingly unsustainable from both an economic and an environmental perspective. Aging pipe networks are causing large water losses due to leakage. Tighter Irish and EU wastewater legislation and rising water and effluent charges is also putting the water industry under greater focus from an environmentally sustainable perspective and a cost perspective. In 2010, over €1.2 billion was spent on the public water supply in Ireland, with €715 million spent on operational costs with capital costs of over €500 million (Dept. of Communications, Energy and Natural Resources, 2012*a*). Aging pipe networks cause large water losses due to leakage. Tighter Irish and EU water and wastewater quality legislation is putting the water industry under further pressure from both an environmental and economic perspective. Water demand is also increasing as water supply stocks are depleting.

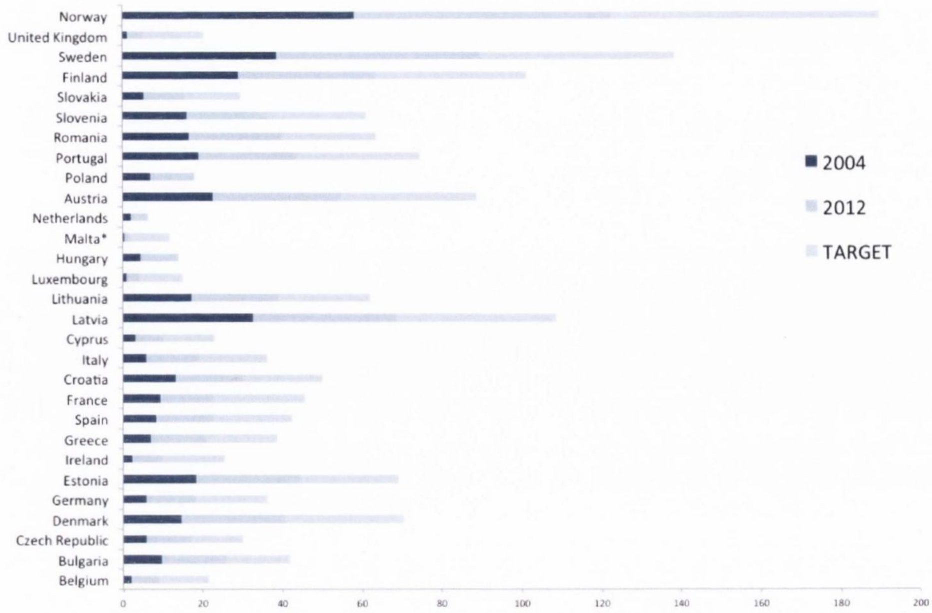

Fig. 1.1: Share of renewable energy in gross final energy consumption and target for 2020 (%) (Eurostat European Commission, 2011).

Table 1.1 shows Ireland to have the second highest percentage of unaccounted for water in Western Europe at 47% (Danilenko and Child, 2005). These leakage rates represent average water losses across each country, however water losses in districts within these countries may be even higher. Giugni et al. (2009) reported leakage rates as high as 60% in some parts of Italy.

The Central Statistic Office of Ireland (CSO, 2011) have released results from the 2011 census, which show that the population of Ireland has increased to 4.58 million, representing an increase of 8.2% over the previous five years. It also found there are 1.8 million people living in the Greater Dublin Area. The CSO predict the population of the Greater Dublin Area to increase to over 2.4 million by 2026 (CSO, 2008). The Dublin region is already under pressure in terms of its available water resources, and Dublin City Council have been examining various options of meeting this predicted increase in water demand. The favoured option involves the pumping of raw water from Lough Derg (River Shannon) to a storage reservoir in Garryhinch where it would then be treated and pumped to Dublin. In total water would be pumped a distance of more than 130km (DCC, 2011). Irish water supply is also undergoing major changes in its organisational structure. Up until January 2014, Irish water supply was managed by

Table. 1.1: Water performance data in Western Europe (Danilenko and Child, 2005)

Country	Consumption per capita (lpcd)	Unaccounted for water (%)
Austria	150	n/a
Belgium	108	n/a
Denmark	125	6
Finland	240	15
France	175	28
Germany	130	8
Greece	136	n/a
Ireland	135	47
Italy	278	27
Luxembourg	150	20
Netherlands	126	5
Norway	200	33
Portugal	n/a	50
Spain	238	25
Sweden	330	17
Switzerland	401	13
UK	150	23

the Water Services Authority, consisting of five city councils and twenty nine county councils. Currently, Ireland is undergoing an ambitious water reform process as part of the austerity measures imposed on the country according to the EU IMF bailout programme. This process primarily relates to the establishment of a new Water Services Authority, namely Irish Water.

One of the key tasks for Irish Water is to reduce the high rates of leakage from Irish water mains. In some local authorities over 50% of treated water is reported as 'unaccounted for'. Though a certain level of leakage will always exist, these rates are excessive. Pressure management has been proven to be an important measure for both leakage reduction and leakage prevention.

Water service providers are in agreement that there is a need to reduce their carbon footprint, reduce their energy bills and become more energy secure. In the UK, through Water UK, a voluntary target of 20% renewable energy generation has been agreed with all water companies (Howe, 2009). Energy can be recovered in the water industry during anaerobic digestion, co-digestion and sludge combustion, and also through the installation of hydropower turbines, wind turbines or solar panels on water industry infrastructure.

Hydropower is the most well established renewable energy resource worldwide. Though hydropower is the largest contributor (68%) to total renewable energy generation (Gaius-obaseki, 2010) and has produced over 16% of global electricity generation in 2008 (Helm et al., 2009) there has been a relatively small (+6%) increase in the amount of hydropower generation in the EU since 1999 (Eurostat European Commission, 2011). This is because most viable large scale resources have already been exploited or are now not feasible due to changes in environmental legislation. Small scale hydropower is therefore becoming more relevant. It is also lower in cost and has a lower environmental impact. Small-scale hydropower turbine technology exists and is relatively well-established for use in run-of-river schemes, which use the natural flow of a river to generate electricity. Hydropower turbines have also been used within water supply networks for many years, however historically they have mainly been installed in the raw water network, at reservoirs and inlets to water treatment works. More recently, hydropower turbines have been developed to be used within the treated water supply network to act as pressure reducing valves (PRVs). However, these turbines are only beginning to be introduced within the water industry and are not yet widespread. Also, thus far, they have primarily been installed on the raw water side and not on clean water distribution mains. One issue preventing the installation of turbines within water supply mains is the large variability of flow rates across the turbine which impacts upon turbine operating efficiencies.

The overall potential for hydropower energy recovery within water supply mains is unknown, however this untapped resource could play a vital role in the quest for an energy secure water supply.

1.2 Research Objectives

The proposed research for submission for Ph.D investigates the improvement of the sustainability of water supply networks through the installation of hydropower turbines. It investigates both engineering and organisational perspectives on this issue of societal and commercial relevance.

The underlying aim of this research is to develop a more sustainable water supply network from both engineering and organisational perspectives. Firstly, this research includes a feasibility study of the scope for energy recovery in the water supply networks of Ireland and Wales. Followed by a sensitivity analysis to investigate the different risks and uncertainties, including those related to varying feed-in tariffs and energy prices involved and their effect on project feasibility and investment payback periods. The next phase of research includes a long term (10 year) historic study of water flow rate variation at potential points for hydropower turbine installation and its effect on project viability. Further research includes the development of an optimisation model for selection of optimal new locations to install turbines in a given water supply network. The research concludes with the development of a framework to streamline implementation of hydropower projects in WSNs.

1.2.1 Research Question

The overall research question addressed in this thesis is: what is the potential for hydropower energy recovery from the water supply network? In order to investigate this potential, the following sub-questions will be addressed: what is the feasibility of energy recovery in the water supply network, how do uncertainties and variations affect feasibility, how can a water supply network design be optimised for energy recovery and how can energy recovery projects be implemented and replicated in practice from an organisational and operations management perspective?

The expected contribution of the proposed research will be to the theory and practice of hydropower energy recovery from the water supply network. This contribution has been captured in a series of papers, some published and some currently in the review process.

1. **Paper 1:** Energy recovery potential using micro hydro power in water supply networks in the UK and Ireland (published)
2. **Paper 2:** Historic study of long term flow variation: Patterns and predictability and its effect on project viability. (preparing for journal submission)
3. **Paper 3:** Optimisation of water supply networks for combined hydropopwer energy recovery and leakage reduction. (published)
4. **Paper 4:** Organising for Energy Recovery and for Future Collaboration: a cross-case analysis (preparing for journal submission)

The practical contribution will be enhanced by the collaborative nature of the research, by working on real data from local authorities, by dealing with practitioners in the area, who have been invited to give feedback and input.

The primary objectives of the research are the following:

1. To conduct a critical literature review of the following streams of research;
   - i Hydropower energy recovery in water supply networks
   - ii Flow rate variation and demand forecasting techniques in water supply networks
   - iii Optimisation and its application to water supply network design
   - iv Project implementation in the context of the water industry
2. To investigate the potential for hydropower energy recovery in the existing water supply networks in the UK and Ireland
3. To investigate the application of forecasting techniques for the long term prediction of water flow rates at potential hydropower energy recovery locations within water supply networks
4. To develop and apply an optimisation model for selection of optimal new locations to install hydropower turbines in a water supply network
5. To investigate organisational, management and regulatory issues associated with the implementation of hydropower energy recovery projects in water supply networks

The remainder of this thesis is structured as follows. **Chapter 2** presents a background to the research undertaken, a critical review of the current state of our knowledge on the feasibility and implementation of hydropower energy recovery projects on water supply infrastructure. A further critical review of key literature in the fields of water demand forecasting and the application of optimisation techniques to water supply network design. Building upon the research questions and objectives presented in Chapter 1, this review process identifies gaps and areas of research requiring further investigation. **Chapter 3** presents an overview of the research model and methodological approach underlying this thesis. The data types, sources and techniques selected and employed to conduct the research presented in this thesis are outlined.

As described in Section 1.2.1, there are four main research sub-questions addressed in this thesis, and each are presented in this thesis document as a chapter. **Chapter 4** presents the feasibility and economic analysis. **Chapter 5** addresses some of the key risks identified in Chapter 4 in relation to long term changes in flow rates at hydropower locations. **Chapter 6** presents the optimisation model developed and results of its application to sample water supply networks. Chapters 4, 5 and 6 approach the topic of hydropower energy recovery in WSNs from technical, feasibility and site selection perspectives. However, successful adoption of this technology by industry may also be dependent on organisational, management and regulatory issues. **Chapter 7** investigates these organisational, management and regulatory issues related to the application of MHP in water supply infrastructure through a comparative case study of two previously installed hydropower projects.

Chapter 8 presents a discussion of the primary results and findings presented in this thesis. The final chapter, **Chapter 9**, concludes this thesis with a summary of the key findings, recommendations and contribution to knowledge.

CHAPTER 2

Background

2.1 Introduction

This chapter will present a critical review of key literature in the fields of hydropower energy recovery in the water supply network, water demand and flow rate forecasting techniques and the application of optimisation techniques to water supply network design. An introduction to literature related to the implementation of hydropower projects in practice is also presented. The key drivers of this research are the need to improve the energy efficiency of water supply provision and also to reduce pressure in water supply mains, which in turn will reduce costs by reducing the amount of leakage, reducing the frequency of occurrence of burst pipes and through the reduction of expenditure on pumping and treating this excess water.

2.2 Energy Intensive Water Industry

The provision of a secure supply of clean and sufficient water is highly energy intensive. This secure supply of clean water is taken for granted by much of society today. Little thought is given to the large amounts of energy required to provide water and wastewater services. Globally, 2-3 % of energy usage is reported to be associated with the production, distribution and treatment of water (Kwok et al., 2010). In the United States, it is estimated that 5% of national energy consumption is associated with water services. At city level, 30-60 % of local government expenditure has been reported to be associated with water services, where the energy consumption requirement thereof is the single largest expense within budgets. Energy prices are rising and their effects on the cost of water supply have been highlighted in the literature (Zilberman et al., 2008).

The water industry is the fourth most energy-intensive industry in the United Kingdom, responsible for 5 million tonnes of CO_2 emissions annually and consuming 7.9TWh of energy in 2006 (Environment Agency, 2009). In Brazil, 60-80% of water services expenditure are reported to be associated with the distribution of water, consuming an estimated 9.6TWh annually at a cost of approximately $1 billion (€770 million) or 14% of the annual Brazilian electricity budget (Ramos et al., 2009). In smaller economies, such as that of Ireland, the operation of the water industry has been reported to cost over €600 million annually (Zhe et al., 2010).

In developed countries, the distribution of water typically accounts for 45% of energy use by the industry (Kwok et al., 2010). As mentioned in the Introduction, the supply and conveyance of water is estimated to consume over 3% of total US electricity (King and Webber, 2008). Furthermore, the pumping of water in California has been reported as the largest single use of electricity in the state (Lofman et al., 2002). Water is heavy and its transport over long distances against large rises in elevation is expensive and energy intensive. The remaining portion of energy consumption in the water industry is from wastewater management (29%) and water treatment (26%).

In Japan, the amount of electricity used by water utilities accounts for approximately 1% of the total nationwide electricity usage (Arai et al., 2014). Since 1995, the amount of water supplied in Japan has steadily decreased, however electricity usage has predominantly remained the same. The amount of energy used per unit water however has increased between 1995 and 2008. This is largely due to the intensification of treatment processes. Also, with more advanced treatment technologies due to be installed in the coming years, energy usage is expected to rise even further. Japanese researchers have highlighted the urgent need to find ways to reduce power usage by water service providers.

The increasing political efforts to improve water quality across the globe have caused water service companies to invest in high-tech, energy-intensive treatment facilities. Indeed, the ever-increasing stringency of, for example, the EU water quality directives has served to increase the energy consumption of the water industry over the past decade (Zakkour et al., 2002). These rising monetary and energy costs in the water industry require intensified research efforts to improve the sustainability of the process overall.

With our ever-changing climate and increasing population, water supply worldwide is under stress. As published in the IPCCs fifth assessment report, Europe has been warming faster than the global average of 0.27°C per decade. In northern European countries such as Ireland and the UK, temperatures have risen by 0.48°C per decade (WGI AR5, 2013). Climate change, as well as putting water sources under stress and influencing water usage patterns, has also led to the enforcement of carbon emissions reduction targets and renewable energy generation targets.

Rothausen and Conway (2011) reviewed the challenges facing water management today. Energy usage by water service providers has been further intensified in recent years due to developments in industrial, agricultural and domestic water use and in water quality regulation. The division of energy use by the water sector into two categories, construction and operation, was proposed. The construction category included infrastructure construction, manufacturing equipment, and operational processes including abstraction, conveyance, end-use and treatment. The multiple areas where energy is used by the water sector are illustrated in Figure 2.1.

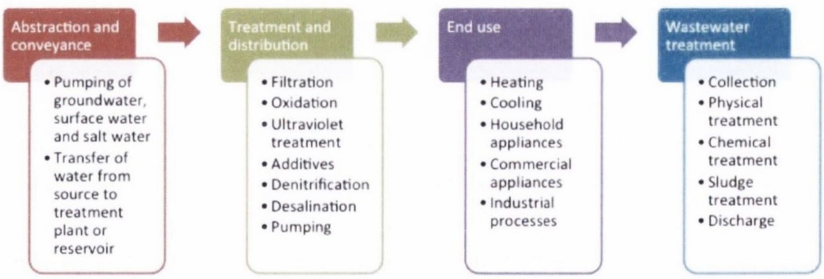

Fig. 2.1: A conceptual model of water-sector processes involving energy use. Adapted from Rothausen and Conway (2011)

Over recent years, water and wastewater treatment operators across Europe have been forced to use more energy intensive processes as a consequence of new water quality legislation. The water industry is governed by a number of regulations. In Ireland, the Department of Environment, Community and Local Government is responsible for policy and legislation related to water quality issues, and for the implementation of EU (Environ, n.d.). Some of the most influential of these water regulation directives on energy usage by WSPs are: The Drinking Water Directive (DWD) (80/778/EEC) which sets standards for drinking water quality at the tap; and the Urban Wastewater Treatment Directive (91/271/EEC) concerning the level of treatment at a works and the removal of nutrients and basic sanitary parameters. Its aim is to protect the environment from any adverse effects caused by the discharge of such waters. The requirements and timing of new water and wastewater legislation are the primary drivers for water supply investment programmes.

2.3 Efficient and Sustainable Water Services

In order to reduce energy usage and expenditure by water services, much research has gone into the development and application of various energy efficiency measures. A number of actions that have been considered and employed by water service providers (WSPs) for both efficient provision of water supply and wastewater services are illustrated in Figure 2.2. These include, optimisation of pump schedules, pressure management schemes, development of renewable energy resources and anaerobic digestion.

The next three subsections describe research in three key areas of efficient water supply; pressure management, water supply optimisation and energy generation with a focus on hydropower.

2.3.1 Pressure Management for Leakage Reduction

Leakage from water supply mains is a major operational problem for WSPs worldwide. It is estimated that on average worldwide, 45-88 million m^3 of water is lost every day due to leakage from water supply infrastructure (Olsson, 2012). Leakage from water supply mains has large economic implications. As well as leakage leading to unnecessary expense in pumping and treatment costs, it may also trigger premature investment in the development of new sources or the expansion of system capacity to meet the increasing demand. In the Dublin region of Ireland for example, population has been pre-

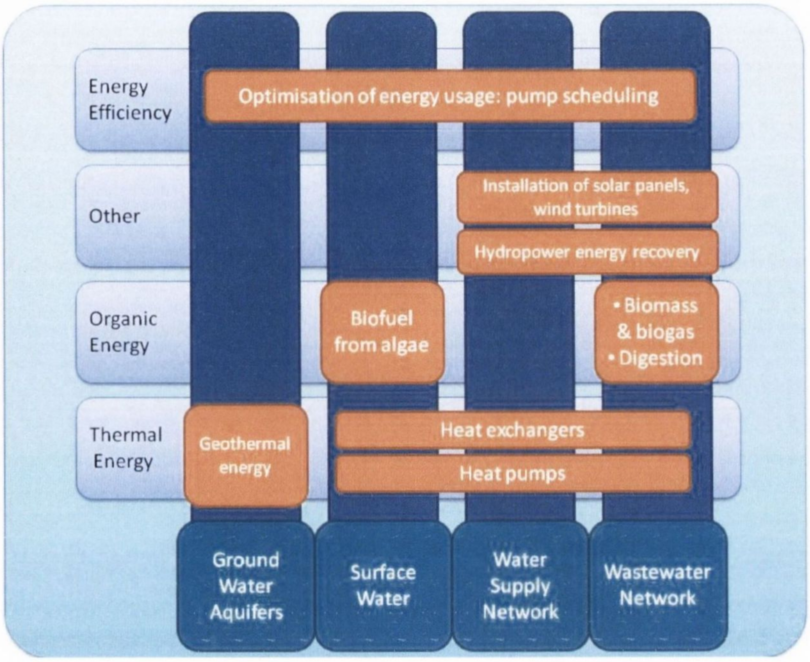

Fig. 2.2: Main categories of energy reduction and efficiency options for the water industry. Adapted from (Mol et al., 2011)

dicted to increase from 1.26 million in 2011 to over 2.4 million by 2026 (CSO, 2008). The Dublin area is already under pressure in terms of its available water resources, and Dublin City Council have been assessing various options, some extreme, of meeting this predicted increased water demand. The current favoured option involves the pumping of raw water from Lough Derg (River Shannon) to a storage reservoir in Garryhinch where it will then be treated to drinking water standards and pumped to Dublin. In total water would be pumped a distance of more than 130km (DCC, 2011).

WSNs can be large and complex, and as their complexity grows, it becomes increasingly difficult to maintain optimal pressure requirements (Sterling and Bargiela, 1984). Over-pressures, as well as resulting in increased pumping and energy costs, also lead to intensified leakage rates. It is well established that water leakage in distribution mains is directly related to the system water pressure (Jowitt and Xu, 1990; Carravetta et al., 2012). Lambert (2000) has described what is currently known about the pressure-leakage relationship. The principle of conservation of energy states that the velocity of a jet of water passing through an orifice varies with the square root of the pressure according to:

$$\text{Velocity} \quad V = C_{\mathrm{d}}(2gP)^{0.5} \tag{2.3.1}$$

Where C_d is a discharge coefficient, P is the pressure and g is acceleration due to gravity. It is often assumed that leakage also varies with the square root of pressure. However in practice, that is not the case. One reason for this is due to the fact that C_d can change depending on whether the flow at a leak is laminar, transitional or turbulent, and it may not remain constant. Futhermore, the rate of leakage (L) will also depend on the orifice area, A, according to:

$$L = VxA = C_{\mathrm{d}}A(2gP)^{0.5} \tag{2.3.2}$$

The orifice area will vary with pressure. For longitudinal splits in plastic PE and PVC pipes, if the area varies linearly with pressure then A will vary with $P^{1.0}$, and L will vary with $P^{1.5}$. If the split opens up in two dimensions - longitudinally and radially, then A will vary with $P^{2.0}$, and L with $P^{2.5}$. Lambert (2000) recommends the most appropriate general equation to use for simple analysis and prediction of the pressure-leakage relationship as:

$$L \quad \text{varies with} \quad P^{\mathrm{N1}} \quad \text{and} \quad L_1/L_0 = (P_1/P_0) \tag{2.3.3}$$

For the above equation N1 can vary between 0.50 and 2.50, depending on the type of leak present.

Figure 2.3 demonstrates the four primary leakage management activities. The central small square represents the volume of Unavoidable Annual Real Losses (UARL), and the larger square represents the volume of Current Annual Real Losses (Lambert, 2000). The current losses can be reduced by the four principal leakage management activities. With time, water supply networks will deteriorate further, increasing the amount of real losses. To prevent this, WSPs need to employ these four leakage management activities.

Large elevation drops throughout water supply districts can lead to large increases in network pressures. These high pressures can lead to burst pipes and water leakage problems. The control of pressure to prevent these problems is a top priority for WSPs. In order to maintain pressures between minimum and maximum pressure standards, pres-

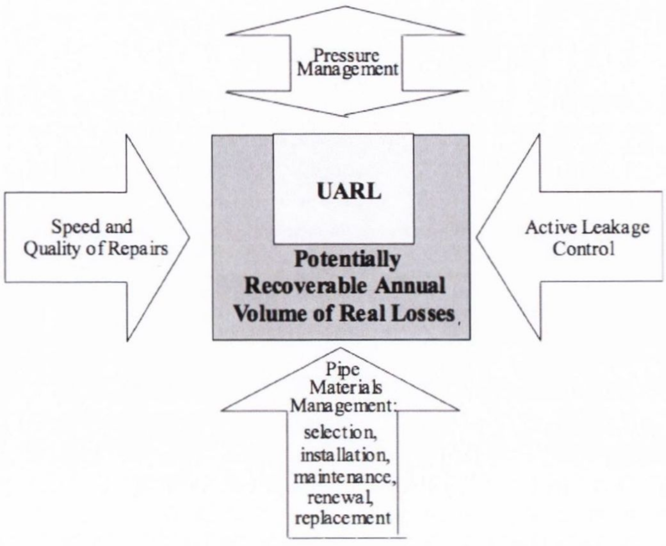

Fig. 2.3: Leakage management activities which constrain unavoidable annual real losses (Lambert, 2000)

sure management schemes must be implemented. Pressure management is one of the key components of any leakage management policy.

Pressure management can be achieved through a variety of methods such as reduction of pumping heads, establishing pressure zones through the use of pressure reducing valves (PRVs), and dissipating excess pressures at key locations within the WDN using PRVs or break pressure tanks (BPTs) (Jowitt and Xu, 1990; McNabola et al., 2011). A BPT reduces the pressure in the pipeline by creating a break in the pipeline where water falls into an open, unpressurised chamber. This break dissipates the pressure and kinetic energy contained in the flow. The water then flows on to the rest of the distribution network with only its potential energy left.

The purpose of a PRV is to maintain a constant pressure in the water distribution network downstream of the valve. PRVs are used extensively in networks characterised by hilly topography with gravity sources at high elevation (Savić and Banyard, 2011). PRVs automatically reduce the pressure in the water supply to a lower pre-set pressure. The most common type of PRV is a direct acting valve. Water enters a chamber within the valve, controlled by an adjustable spring loaded diaphragm and disc. The spring has a pre-set tension on the valve seat and adjusts the flow to a desired outlet pressure (Watts, 2010). A section cut through a PRV is shown in Figure 2.4.

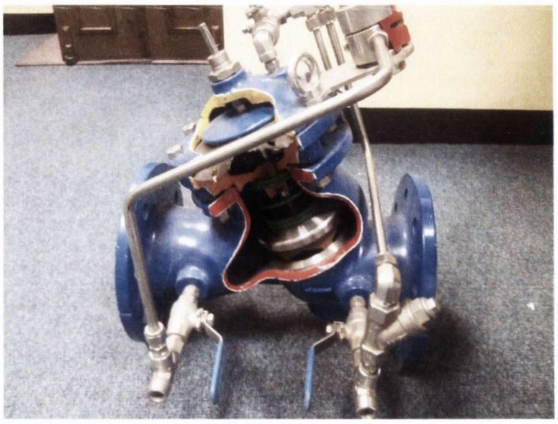

Fig. 2.4: Section View of a PRV

Both BPTs and PRVs present opportunities to recover energy through the installation of micro-hydropower turbines without interference in the level of service provided to downstream consumers. The energy dissipated at these points may also be used for energy production as is discussed further in Section 2.3.4. This recovered energy can be used to reduce the dependency of water service providers on external energy resources required to pump and treat water.

2.3.2 Water Supply Network Optimisation

In the past, WSPs predominantly relied upon trial and error approaches for WSN design. However, this would not guarantee an optimally designed network. The application of optimisation techniques to WSN design can ensure an optimally designed network. In recent years, optimisation techniques have been applied to improve the efficiency of a number of aspects of WSN design. There are numerous design objectives that WSPs can solve with optimisation techniques. Optimal WSN design could be the selection of the least cost combination of pipe diameters and types to meet demand, pressure and capacity constraints. Other design objectives that have seen increased research in recent years are the optimisation of energy usage in pumping and the optimal location and setting of PRVs for optimal pressure management and leakage reduction.

Optimal design of water supply networks has been extensively researched. Traditional mathematical optimisation methods such as linear programming have been used to optimise the installation of pressure control for leakage reduction (Sterling and Bargiela, 1984; Jowitt and Xu, 1990), to optimise the selection of pipe diameters in a WSN (Alperovits and Shamir, 1977) and to optimise pumping schedules in WSNs (Jowitt and Germanopoulos, 1992).

Optimisation of the location and setting of PRVs in a WSN for improved pressure management and leakage reduction has been researched widely. Early research in this field by Sterling and Bargiela (1984) saw the development of an optimisation algorithm for leakage minimisation based on the Simplex linear programming method. It was found that this method had low memory and processor time requirements for computational implementation. The algorithm was tested on a benchmark 25 node network as shown in Figure 2.5. The locations to install three control valves was assumed, and the optimisation algorithm calculated the optimal valve settings for maximised leakage reduction. Results indicated that a 20% reduction in the volume of leakages could be achieved through optimisation of valve controls.

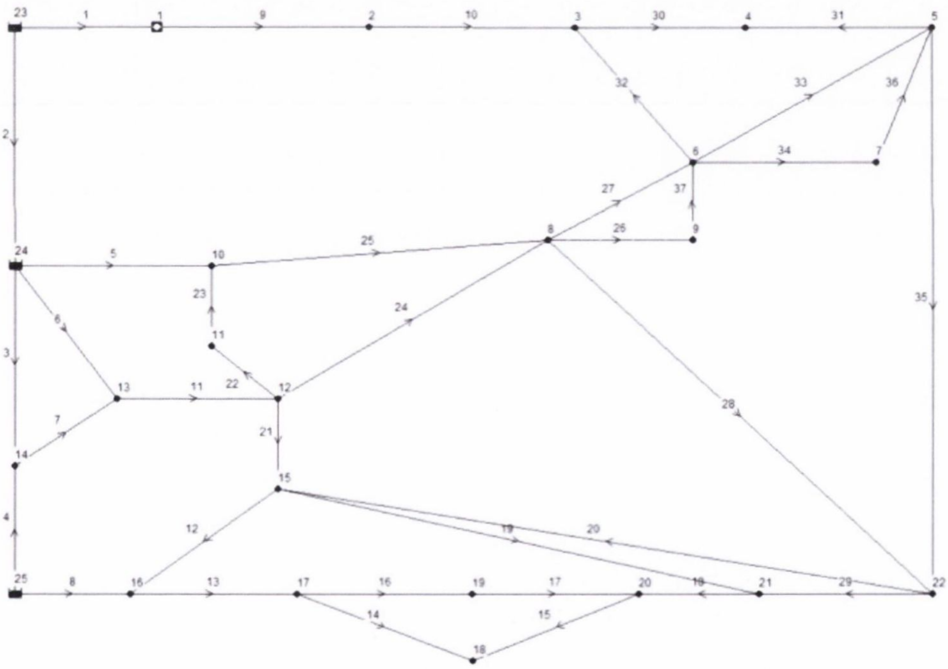

Fig. 2.5: Benchmark 25-Node network layout (Sterling and Bargiela, 1984; Eck and Mevissen, 2012; Nicolini and Zovatto, 2009)

Jowitt and Xu (1990) employed linear programming methodology to minimise leakage from a water supply network. For this analysis, the Hazen-Williams approximation for head loss was assumed. The non-linear relationship between leakage and average service pressure was approximated using the following formula:

$$QS_{\mathrm{ij}} = CL_{\mathrm{ij}}.(P_{\mathrm{ij}})^{\mathrm{N1}} \tag{2.3.4}$$

Where QS_{ij} is the water-leakage volume in pipe length L_{ij}; CL_{ij} is a coefficient that relates the leakage per unit length of pipe to service pressure and depends on the system characteristics, e.g. age and deterioration of pipes, soil properties etc; P_{ij} is the average service pressure across the pipe length ij. The value assumed for *N1* for this 25 Node Network was 1.18, this value would be in the lower range of possible values for *N1* as discussed by Lambert (2000), which can vary between 0.5 and 2.5. The objective function was to minimise the total system leakage through optimisation of the flow control valves in the network. In order to achieve this, a number of constraints had to be met. These constraints included the conservation of mass at each node, minimum head requirements at specified critical nodes and head loss across pipes according to the Hazen-Williams approximation. Germanopoulos (1995) used the same procedure as Jowitt and Xu (1990) however with a modified objective function.

Vairavamoorthy and Lumbers (1998) minimised the total volume of leakage in a WSN by solving a sequence of sequential quadratic (SQP) subproblems. Minor violations were allowed in the pressure constraints by adding a tolerance to the minimum allowable nodal pressure head according to:

$$H_{\mathrm{i}} \leq (H_{\mathrm{ij}}{}^{\mathrm{L}} - e_{\mathrm{i}}) \quad i = 1, ..., NPN \tag{2.3.5}$$

Where H_i is the nodal pressure head; $H_{ij}{}^{L}$ is a prespecified minimum allowable nodal head, for node i and e_i = degree (tolerance) by which the pressure at node i can violate $H_{ij}{}^{L}$. This optimisation model was tested in a benchmark 25 node network (Sterling and Bargiela, 1984; Germanopoulos, 1995), which had three flow control valves installed in links 11, 21 and 29. The target pressure at each node in the network was set at 30m, and the allowable deviation pressure tolerance was set at 5m. The Hazen-Williams head loss approximation was used. The model was optimised with the inclusion of allowable pressure head violations, and also without violations. It was found that leakage was further reduced by allowing violations, however this meant that some nodes were below their target pressure.

According to Savić and Banyard (2011), in recent decades, optimisation of water distribution systems has progressed from the use of traditional, deterministic, mathematical optimisation processes to the use of heuristics derived from nature such as genetic algorithms (GAs), amongst other evolutionary algorithms. These optimisation techniques allow for multi-objective optimisation.

A GA is an optimisation algorithm that uses a search process inspired by natural evolution theory. The algorithm begins with a randomly generated population of chromosomes and applies three operators, the selection, crossover and mutation operators to find the optimal model (Jalalkamali and Jalalkamali, 2011). Within the water resources field, GAs have been used to optimise pipe diameters for water supply networks (Dandy et al., 1996; Castillo and González, 1998; Gupta et al., 1999; Morley et al., 2001). GAs have also been used for leak detection and to optimise pumping performance (Barán et al., 2005; Wang et al., 2009).

A multi-objective GA approach was used by Nicolini and Zovatto (2009), to optimise the number, location and setting of PRVs in water networks. This optimisation model was tested on the benchmark 25 node network (Sterling and Bargiela, 1984; Eck and Mevissen, 2012). Behzadian et al. (2009) used both a multi-objective GA in combination with adaptive neural networks in research to determine the optimal locations for installing pressure loggers in a water distribution network. Jalalkamali and Jalalkamali (2011) also used a hybrid method of ANNs and GAs in their research to estimate the leakage rate of a water distribution network.

More recent research has seen the application of Mixed Integer Non-linear Programming (MINLP) techniques to the optimal design of water supply networks (Eck and Mevissen, 2012; Bragalli et al., 2012; Gleixner et al., 2012). Bragalli et al. (2012) employed the open-source MINLP solver, Bonmin, to optimise the water supply network design objective of the selection of optimal pipe diameters at minimum cost. Two modifications to Bonmin were tested in order to better handle the non-convexities of the design objective. One of these modifications has since been added to the new available version of the Bonmin solver. The decision variables for this problem were the flow in each pipe, the diameter of each pipe, and the hydraulic head at each junction. Head loss was modelled using the Hazen-Williams approximation. A preliminary continuous NLP model was solved first. It was concluded that the MINLP formulation resulted in good solutions found within reasonable computing times. Gleixner et al. (2012) applied MINLP to find an optimal pump schedule for minimised operating costs in a WSN. The

Darcy-Weisbach approximation for headloss was applied. The non-convex objective function was solved to global optimality using the open source MINLP solver SCIP.

Eck and Mevissen (2012) employed MINLP to find optimal locations and settings of PRVs on WSNs. The model was applied to the benchmark 25-Node network, reporting optimal PRV locations to be at links 1,5 and 11, differing from the optimal locations reported in prior research. An advancement in the development of a quadratic approximation for pipe headloss was also presented, enabling the use of a broader class of optimisation engines and increasing the network size solvable.

2.3.3 Renewable Energy Resources

The development of renewable energy resources on water supply network infrastructure is becoming an important activity of WSPs. Many water companies now have dedicated energy teams focused on the development of new energy sources. Options considered and available to WSPs for energy generation are the installation of solar panels, wind turbines or hydropower turbines on water supply infrastructure, through anaerobic digestion at wastewater treatment plants and through thermal heat recovery at different points in the network as illustrated earlier in Figure 2.2. The biogas generated through anaerobic digestion can be used directly on site as a fuel. In the UK, anaerobic digestion accounts for over 90% of the renewable energy generated by the water industry (Howe, 2009). The focus of this thesis research is on the incorporation of hydropower turbines within WSNs.

2.3.4 Hydropower

As discussed in Chapter 2, most viable large scale hydropower resources have already been developed, or are now not feasible due to increased environmental constraints. This has increased the focus on the identification and development of small scale hydropower resources. Locations for small-scale hydropower development can be found in water supply infrastructure. Potential locations include at PRVs, BPTs and inlets or outlets of tanks and reservoirs. This research will focus primarily on micro-hydropower projects with power outputs of less than 100 kW. To put this into context, according to the classifications of the International Energy Agency, hydropower plants with an electrical generating capacity of:

- 10 MW to 1 MW are described as small-scale
- 1 MW to 100 kW are termed mini installations
- 100 kW to 1 kW are described as micro-scale

As outlined earlier, the water industry is increasingly exploring the use of MHP as a means of energy recovery. The best available estimate of the hydropower potential in the UK water industry, for example, is 17 MW (Zakkour et al., 2002), with a capacity of 9 MW installed at present (Howe, 2009). In Germanym, the power generation potential has been assessed as 4.7 MW, which extrapolated to the European Union is estimated to be 28.5 MW (Carravetta et al., 2012). In this section, relevant research findings in the field of energy recovery in WSNs are outlined.

Hydropower turbines have been used within water supply networks for many years, however historically they have primarily been installed at inlets to reservoirs and water treatment works. Early research in this area was undertaken on the use of pumps as turbines (PAT) for micro-hydropower generation at a water treatment works (Williams, 1995, 1996; Williams et al., 1998). A PAT was installed in parallel to a PRV at a water treatment plant at Barnacre, Blackpool (Williams et al., 1998). From testing at this site, it was concluded that it is possible to recover energy at PRVs but that accurate prediction is required for the operating conditions and that the pressure head across the PAT should be maintained steady. Economically they concluded that when the energy recovery potential was greater than 30kW, the generator could feasibly be connected to the grid.

Wallace (1996) examined the possibility of micro-hydropower generation within water supply networks. The primary locations for energy recovery mentioned include at water treatment works, intermediate service reservoirs and break pressure tanks. Economic feasibility and the institutional framework were investigated as well as technical constraints. It was concluded that viability of investment would depend on the availability of on site electricity demand or the ability to obtain a power purchase agreement with an increased tariff for renewable energy generation. During feasibility stage, it was recommended that the economic virtue of the investment be evaluated using either present-worth (PW), net present value (NPV), or discounted cash-flow (DCF) methods. This is because the trading and payback period is sensitive to inflation rates, interest rates on borrowed capital, and tariffs applied. It was concluded that depending on the extent of the civil works required, many schemes can be economically viable, recovering the investment in relatively short periods of time, and thereafter generating revenue from energy that would otherwise have been wasted.

Gaius-obaseki (2010) reviewed the different options for energy generation on water supply infrastructure. The opportunities discussed include at PRVs, BPTs, sewage treatment outfalls and compensation flows from reservoirs. It was noted that smaller scale hydropower options are becoming more attractive as much of the larger scale sources have already been developed. Approximately 85% of the energy dissipated across a PRV could be recovered by replacing the PRV with a turbine. The capital cost for installation of small-scale hydropower turbines was reported to be in the region of £3,000 (€3,700) to £6,000 (€7,400) per kW installed.

Vilanova and Balestieri (2014) outlined a number of areas within water supply systems where hydropower energy recovery is possible. It was found that though hydropower is a mature technology and that its application in water supply networks is known, its application has yet to be widely implemented in WSNs worldwide. The options for hydropower energy recovery identified included raw water catchment locations at streams or rivers, at inlets to water treatment works, at regulation dams, pumped-hydro storage plants, at groundwater resources such as confined aquifers, in substitution for pressure reduction or management devices, and within the treated water distribution system. The issue of flow variation was also identified when installing turbines within the treated water distribution mains. High variability of flows and pressures due to varying consumer demands would influence the selection of turbine types.

Researchers at Trinity College Dublin and Bangor University Wales have reported on the feasibility of hydropower energy recovery in water supply networks of Ireland and Wales (McNabola et al., 2011; Corcoran et al., 2012*a*, 2013; McNabola et al., 2013), and also on wastewater infrastructure of Ireland and Wales (Power et al., 2014). Furthermore, the life cycle costs of these schemes have been discussed (Coughlan et al., 2013). McNabola et al. (2011) reported on a technical and economic feasibility study of the hydropower potential of 7 BPTs in the County Kildare region of Ireland. This is a relatively small county with a total population of over 180,000. The power generation potential was modest with a range of 2kW to 27kW. Further research by Corcoran et al. (2013) reported on the hydropower energy recovery of the WSNs of Ireland and Wales. Of the 90+ sites investigated, two were found to have power generation capacities of over 100kW. Estimated investment payback periods were calculated for a variety of energy prices and feed-in tariff rates, A further sensitivity analysis was undertaken using Net Present Value (NPV) calculations, with projects deemed feasible if their NPV was positive by year ten.

Researchers in Portugal have published widely on various aspects of the application of renewable energy technologies within water supply infrastructure. Ramos and Borga (1999) reported that the use of a PAT instead of a turbine provides a cost efficient alternative for energy production within water supply networks. It was concluded, through experimental and computational analyses, that PATs can be used to replace PRVs (Ramos et al., 2010) and it was also recommended that new policies to encourage water supply providers to implement such energy recovery projects, be developed.

Carravetta et al. (2012) published research on the use of PATs for application in water distribution networks. The complications of selecting and installing a turbine in water distribution mains due to the large variation in flow and pressure head, depending on water demand were discussed. PATs were identified as an appropriate and economic turbine choice to suit the variable flows and pressures. An issue with calculating the Best Efficiency Point (BEP) of a pump operating as a turbine was highlighted. The three methods to find the BEP are either experimentally, requiring a large number of experiments over a range of flow speeds and generator speeds, or by using Computational Fluid Dynamics (CFD) or by a one-dimensional method. CFD was highlighted as a valid alternative method to lab experimentation due to good agreement between lab tests and CFD results in previous experiments. Standard design criteria for a PAT installed in a series-parallel configuration were presented. This method, named the variable operating strategy (VOS), allows the selection of suitable turbine geometry based on the flow and head conditions and the required network backpressure.

In further research by Carravetta et al. (2013), the control of PATs using either hydraulic regulation (HR) or electronic regulation (ER) was discussed. In order for PATs to act as PRVs they must be able to control the downstream pressure. HR is a hydraulic method of controlling the downstream pressure, by providing a bypass conduit and a PRV in series with the turbine. ER is an electrical method of controlling the output pressure through the use of an electronic inverter. This allows for the regulation of electrical voltage and frequency to vary the generator speed, thus providing the ability to control the torque and hence the outlet pressure from the turbine. The economic costs associated with both HR and ER were estimated and compared. A comparison between turbines with average pressure drops of 10, 20, 30 and 35 metres were calculated. The investment payback period of each option was estimated and the HR option was found to have a shorter payback period than the ER for each case. However, civil works and maintenance costs were not included in this calculation so the actual payback period of the entire hydropower installation would be longer.

Carravetta et al. (2014) reported on a Cost-Benefit Analysis (CBA) of hydropower production in water distribution networks using a PAT. PATs were identified as the most cost-effective solution, with the cost of a PAT estimated at €350/kW. The PAT installed was assumed to be running at a fixed rotational speed *N*, which was maintained through either mechanical or electrical regulation. A CBA was undertaken, assuming an annual maintenance cost of 10% of the total turbine cost. This study was applied on data for a network in Palermo, Italy. Installation costs were estimated based on the cost of the pipework required, the earthworks and installation required for each diameter pipe. No further economic costs were estimated, as they were considered to be case sensitive. The useful design life of the PAT was assumed as 30 years. 5 different cost scenarios were analysed based on different pipe size options. Three of the five scenarios reported positive net present values (NPV) by year 30, whilst two scenarios had not achieved investment payback by year 30.

In 2011, Sulzer, a pump and turbine manufacturer, published a technical review of the use of pumps in reverse to replace pressure reducing valves in industrial processes (Adams and Parker, 2011). One issue when selecting a pump to operate in reverse is that the efficiency point of the pump will not be the same for when it is operating as a turbine. It was found that the best efficiency point of the turbine was located at a higher flow rate and higher head, meaning the capacity was higher in turbine mode than pumping mode. Another point discussed was the potential for occurrence of cavitation. It was found that the susceptibility to cavitation was lower in turbine mode than pump mode, as the low pressure zone was at the runner outlet. At small power ratings, conventional pumps running as turbines were found to be an economical solution for pressure reduction.

Da Silva et al. (2011) undertook a case study of the potential for hydropower generation in Pato Branco, Brazil. 20 PRVs were investigated, with potential power outputs ranging from 2kW to 40kW. Flow rates and pressures were considered constant for 18 hours per day with the remainder of the day discounted for energy recovery as the water demand and hence flow rates were negligible. The limitations of this study are that variations of flow and pressure were not considered and also turbine efficiency was assumed to be constant and very high at 90%.

Kucukali (2011) reported on a similar case study of the small hydropower potential of the Edremit water supply network in Turkey. It was stated that the most convenient location to install hydropower turbines in a water supply network are on the water supply line before entering the water treatment works. The power generation at each of twelve pressure reduction tanks along the supply line was quantified. These power outputs ranged from 3kW to 95kW. An economic analysis was undertaken, with a payback period of two years estimated. The total installation cost was estimated at €1.118 million, with a 1% maintenance cost assumed. A detailed breakdown of this cost was not provided.

Beltran et al. (2014) completed a technical and economic analysis of the viability of micro-hydropower in a medium sized wastewater treatment plant. The hydropower potential at the inlet and outlet to a works in the Castello region of Spain was analysed. Assuming 70% system efficiency, the power outputs were calculated to be 1.67kW and 3.1kW respectively. For this power output calculation, an average annual flow rate was assumed. The total cost of each of the installations was estimated to be approximately €4,500. A breakdown of this cost was not provided. It was also noted that the installation could be eligible for a regional governmental renewable energy grant, which would provide a non-refundable grant covering up to 45% of the installation costs. The economic viability was then analysed using a number of methods; Payback (PB), Net Present Value (NPV) and Internal Rate of Return (IRR). The proposal was found to be viable under all methods however the installation costs assumed were low in comparison to costs of hydropower installations found in literature and also the assumed cost lacked detailed breakdown.

2.4 Hydropower in WSNs - Installation Issues

The environmental impacts of hydropower installations on water supply infrastructure are minimal. Requirements for other types of hydropower projects, such as run-of-river schemes, would include obtaining an abstraction license and undertaking flood defence studies. These, however, would not be required in the case of installations within water supply infrastructure. The primary environmental impact associated would be noise from the turbine generator, which can be minimised with acoustic insulation (Gaius-obaseki, 2010). The most important requirement when installing within water supply infrastructure is that the level of service for consumers is not interrupted. This can be successfully achieved by installing a bypass system around all turbines, with valves installed that will switch to the bypass if there is any error, problem or maintenance required at the turbine.

Variability of flow is a major issue of concern when installing hydropower turbines within the water supply network. Water demand varies on a diurnal, seasonal and location specific basis. The National Research Council (NRC) in Canada recently investigated the feasibility of small scale hydropower within the water distribution network from a probabilistic perspective in order to address this issue of demand variation (Colombo and Kleiner, 2011). This variability would be of particular concern when deciding on the feasibility of a potential installation, when estimating the predicted costs and payback periods associated. A number of other potential uncertainties were flagged, including long-term demand growth, diurnal and seasonal demand variations, pipe friction coefficients and future cost fluctuations. Colombo and Kleiner (2011) concluded that because demand is uncertain, a probabilistic framework should be used in calculations when deciding on the viability of a micro-turbine installation. They also concluded that diurnal flow rate variation had a substantial impact on the power generation capacity of a potential project and so recommended that comprehensive analysis of diurnal flow variations should be taken into account in the project feasibility stage. When diurnal flow rate variation and longer term flow rate predictions are made, a turbine can be selected that would operate at high efficiencies over this flow range. All turbines have best flow rate and pressure head operating conditions.

Giugni et al. (2014) reported that when installing turbines in WSNs, variations in demand patterns are significant especially in smaller distribution networks, for example between night-time and day-time demand, however for larger distribution networks, this variation is not as pronounced. It was recommended that an in-depth inquiry be performed to investigate the effect of daily and seasonal flow variation on both turbine and generator efficiency. It was also noted that though the option to install turbines in WSNs for pressure reduction is an attractive solution, an accurate cost-benefit analysis should be developed to determine the economic feasibility of suitable micro-generation equipment. Furthermore, Carravetta et al. (2014) noted that the lack of reliable evaluations of the cost benefits of hydropower energy projects on WSNs presents an obstacle to the widespread diffusion of this technology in practice.

The main issues affecting project feasibility that require addressing are how to handle fluctuations of flow rate (Colombo and Kleiner, 2011; Vilanova and Balestieri, 2014), and other sensitive variables such as the availability of government incentives (Gaius-obaseki, 2010; Beltran et al., 2014).

The literature has highlighted that it is possible to recover energy within water supply networks through the installation of hydropower turbines. Though some publications detail the potential for hydropower in WSNs, reliable project cost and long term project life information is not available. PATs have been suggested as a cost-effective alternative to a traditional hydropower turbine, however PATs operate less efficiently over a wider range of flow rates. It has also been noted repeatedly that flow variation and future cost fluctuations could impact project viability and also that new policies to incentivise the installation of hydropower turbines are required in order to increase project viability.

2.5 Flow Rate Variation

Water flow rate variation will directly impact the selection of a suitable turbine, and hence will impact the potential energy generation at sites for hydropower installation in WSNs. When undertaking a feasibility study for a hydropower installation, a key decision factor on whether to proceed with implementation is the investment payback period. The shorter the estimated investment payback period, the more attractive the project becomes. Payback periods of approximately ten years are the maximum considered feasible for many water supply providers. Any longer than this may not be deemed an attractive investment. When calculating these payback periods, it is important to consider the potential future fluctuations in water flow rates and pressures at these sites, as well as future fluctuations in energy prices and changes in policy.

Water flow rate and pressure are the two primary influential variables effecting power generation capability of installed hydropower turbines. The relationship between flow rate and pressure on power output is detailed in Equation 2.5.1. Where P is the power output, Q is the flow rate through the turbine, ρ is the fluid density, g is acceleration due to gravity, H is the head available at the turbine and e_o is the efficiency of the total system. Flow and pressure also both vary depending on the water demand in those sections of the water distribution mains. Short term diurnal variation is expected and largely predictable due to diurnal water demand patterns. However, longer term variation also occurs and is more difficult to accurately predict. This section details the current state of research and practice for both short term and longer term water demand forecasting.

$$P = Q\rho g H e_0 \tag{2.5.1}$$

Water demand forecasting is a central task for water supply operations and planning. It has also been extensively studied and remains a consistently active area of research and publication since the 1960s (Arbues et al., 2003). New methods and approaches are regularly developed and reported on. Early approaches addressed the task predominantly using traditional statistical models. More recently, computational intelligent approaches such as the use of artificial neural networks (ANNs) and genetic algorithms (GAs) have also been considered. Water demand forecasting can be either short-term or long-term. Short-term demand forecasting is necessary for day-to-day management and operation of a water supply network. However, longer term forecasting is necessary for strategic infrastructure investment planning, scheduling of maintenance and asset management.

The majority of the literature in this field is focused either on aggregate municipal demand or residential demand (Arbues et al., 2003). Forecasting for total municipal demand is required to ensure reservoirs are adequately supplied to meet future demands and for strategic infrastructure investment planning. Another stream of research is focused on residential water demand forecasting based on end-use analysis. For example, understanding how much water is used per household based on the characteristics of the users and uses. One issue with end-use forecasting is that it requires a large amount of detailed data and hence many assumptions (Khatri and Vairavamoorthy, 2009). The focus of this thesis research, is centred on flow rate fluctuation at valves within the distribution network, downstream of reservoirs, yet upstream of end-users.

The most frequently reported influential factors on water demand can generally be classified as either climatic or socio-economic. World population is at an all time high and set to grow significantly in the future, especially in urban areas. This will put our limited world water resources under stress. Climate factors such as temperature, rainfall and relative humidity have been shown to effect water usage (Mukhopadhyay et al., 2001; Goodchild, 2003; Arbues et al., 2003). Socio-economic factors such as population, age and employment status have also been shown to correlate with water demand (McDonald et al., 2011; Bennett et al., 2013).

Arbues et al. (2003) completed a state-of-the-art review of estimation of residential water demand. The focus of this paper however was on economic approaches to water demand estimation using econometric techniques to relate consumption to a measure of the price of water and also some explanatory variables. The key explanatory variables were summarised as reported in literature. These included income, weather variables, resident population/household composition, housing characteristics, frequency of billing and rate design and finally, indoor versus outdoor water use.

Khatri and Vairavamoorthy (2009) reported on uncertainties affecting future water demand. The uncertainties considered in this study included climate change, population growth and socio-economic changes. A scenario approach was employed using random sampling techniques, Monte Carlo simulation (MCS) and Latin Hypercube sampling (LHS), to describe uncertainties. Linear regression was used to correlate the affect of temperature and precipitation on water demand.

McDonald et al. (2011) reported the key determinant of water demand in a region to be population. The population factors considered included age, inward and outward migration, household size, class distribution and fertility differences between different societal groups. House-Peters and Chang (2011) reviewed urban water demand concepts and methods. The most common explanatory variables in literature were found to be temperature, precipitation, wind speed, evapotranspiration, water price, income and household size amongst others. Qi and Chang (2011) reported that there has been renewed interest in accurate water demand forecasting due to recent large economic fluctuations and their effect on large urban areas. They also reported that accurate prediction models should simultaneously consider variables related to climate change, economic development, and population dynamics.

Mukhopadhyay et al. (2001) undertook an analysis of water consumption patterns of private residences of Kuwait. Water consumption data was collected from 48 households over a period of one year. Both stepwise linear regression and ANNs were fitted to the observed data. Linear regression was used to determine the most influential factors. Water consumption was found to depend on the number of bathrooms and rooms in the residence, the size of the garden and the income level of the household, atmospheric temperature, relative humidity and the number of people in the residence. The best regression model was able to explain 63%-65% of the variation. The same inputs for that regression model were then used as inputs in an ANN. The ANN was able to explain 88% of the variation. It was also noted that measurements of weather variables and water demand are prone to error. It was suggested to resolve this issue with weather data, by using the average of data from multiple weather stations by a Thiessen polygon approach.

Goodchild (2003) investigated the impact of climate change on domestic water demand (DWD). The study reported on twenty years of climate data and water demand data for the Essex area in the UK. A multi-variate linear regression process was employed to investigate the key influential factors. The highest correlation was found to be with evapotranspiration (R^2 of 63%), water content in top 0.15m of soil (R^2 of 63%) and sunshine hours (R^2 of 53%). The lowest level of correlation was with rain (R^2 of just 19%). It was concluded that the relationship between weather variables and DWD can be effectively investigated using multi-variate stepwise linear regression. This method allowed for the production of an equation to predict demand, based on maximum temperature, evapotranspiration, days since significant (2mm) rainfall and temperatures greater than 25°C. No other influential factors, such as socio-economic factors, were considered.

Bennett et al. (2013) published research relating to the forecasting of residential end use demand based on a test data set of 205 households. Using linear regression analysis the key determinants for water demand end use forecasting were explored. For total internal demand, the key determinants were found to be, the income level, the number of adults, number of children, number of teenagers, toilet star rating, shower star rating, clothes washer rating and clothes washer loading. ANNs were then used to produce residential demand end use models. The ANN training set was developed containing 175 randomly selected samples, with the remaining 30 samples used for model validation. It was concluded that an ANN based methodology is a feasible means of producing residential water demand end use models. The best ANN model developed reported R^2 values ranging from 0.21 (bath) to 0.6 (shower) for different aspects of end uses. Total internal usage reported an R^2 of 0.47. All models reporting R^2 values of greater than 0.3 were deemed to provide a moderate forecast accuracy.

A summary of key research in the field of water demand forecasting is presented in Table 2.1. The use of ANNs for forecasting has been primarily adopted for short-medium term forecasting timeframes. MLR has been adopted for both short-term and long-term forecasts. The most commonly reported factors that have been found to influence water demand variation include climate factors, population factors and economic factors such as water price. Short-term water demand forecasts, for example 1 to 3 days ahead, can be accurately forecast, with some models reporting R^2 values of over 0.9. Longer term forecasting models however are considered moderately accurate with R^2 values of above 0.3-0.4.

Though water demand forecasting is an area of active research, the focus of prior research and publication has been predominantly on the forecasting of water demand for an entire water supply district or on forecasting total demand based on individual end usage. There has been no published research in the forecasting of water demand at

Table. 2.1: Water demand forecasting literature summary table

Source	Influential Factors	Analysis Timeframe	Method	Key Results
Mukhopad-hyay et al. (2001)	No. of bathrooms and rooms in house, garden size, income level, atmospheric temperature, relative humidity and no. of people in house	Model input of 1 year of weekly data	Stepwise MLR and ANNs	Best MLR model with R^2 of 0.65, best ANN model with R^2 of 0.80
Goodchild (2003)	Max temperature, evapotranspiration, days since significant (2 mm) rainfall and temperatures >25°C	Long term. 6 years of climate and flow data (1994-1999) used to predict 2020 scenarios	Stepwise MLR	R^2 of 0.54 and RMSE of 24.9
Babel and Shinde (2011)	Historical daily water demand; rainfall (mm); evaporation (mm); relative humidity; max, min, and mean temperature	Short term (1, 2, 3 days ahead) and Medium term (1, 3, 6 months)	Stepwise MLR followed by ANN	Climatic variables more influential for medium-term forecasts. Short term forecasts using only historic water demand found accurate to 98%
Adamowski et al. (2012)	Daily total precipitation, max temperature and past daily urban water demand	Short term- 1 day ahead	Comparison of MLR, non-linear MLR, ARIMA, ANNs and Wavelet ANNs	best MLR model R^2 of 0.76, best ANN model R^2 0.792 and best WA-ANN model R^2 0.896
Bennett et al. (2013)	Household income, no. of adults, no. of children, no. of teenagers, toilet star rating, shower star rating, clothes washer rating and loading	To forecast water savings following a retrofit of more efficient toilets, showers etc	MLR followed by ANN	R^2 values for total internal demand of 0.41
Haque et al. (2013)	total monthly rainfall, max temperature, water price, water conservation savings and water restriction savings	Long term	Comparison of MLR, non-linear MLR	MLR (R^2 of 0.88) found more accurate than MNLR (R^2 of 0.3)

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points within water supply mains, neither at reservoirs nor at end-users. Water flow rate estimation at these points is relevant when installing new infrastructure, such as hydropower turbines, at points within water supply mains. These installations require an initial capital investment and the payback period on that investment is dependent on the power generated. As mentioned previously, power generation at these points is dependent on the flow rate and pressure drop across the installed turbine. It is also particularly relevant when considering micro-hydropower (c. 1kW) because investment payback would be achieved over a longer period and as such, longer term variations could negatively impact future power generation and hence revenue, impacting upon investment payback and overall project feasibility.

2.6 Optimisation of Hydropower in WSNs

As has been discussed, optimisation techniques have been applied to many aspects of optimal and efficient water supply network design. However, the application of optimisation techniques to the installation of hydropower turbines in WSNs has not been extensively researched. The first time research in this field was published was by Afshar, in which an option for optimal allocation of pressure to potential hydropower turbines along water supply mains by employing a dynamic programming technique was presented and discussed. Dynamic programming (DP) involves the division of an optimisation problem into solvable stages. In this problem, a gravity fed WSN was divided into stages, each consisting of a length of pipe, a turbine at the downstream end, and the flow rate and pressure entering and exiting each stage. The problem was also subject to constraints such as meeting minimum water pressure standards and meeting water demand requirements at each stage. The Darcy-Weisbach head loss equation was calculated using the Newton-Raphson numerical technique. There were a number of limitations to this study. Firstly, the optimisation results were not verified using a hydraulic solver. Secondly, the water supply network analysed was simplistic, consisting of 45km of mains divided into seven stages, containing no network loops. Variations of flow rate and turbine efficiency were not considered, nor was the type of turbine installed.

Since the Afshar et al. (1990) publication, the majority of research to date regarding optimisation and hydropower in WSNs has primarily focused on the optimisation of pumped hydro systems. Vieira and Ramos (2008) discussed the optimisation of pumped hydro between water supply reservoirs through linear programming (LP) and non-linear programming (NLP). Both LP and NLP were employed to the optimal operation of a pumped-storage hydropower facility within a water supply system in Portugal. The system analysed, located on Madeira Island, consists of two reservoirs, a lower and an upper reservoir. There was a pumping station and a hydropower works at the lower reservoir. Water was pumped from the lower reservoir to the upper reservoir for six hours per day, during the remaining hours, water was discharged through the hydropower works. Several assumptions were made, for example in relation to water consumption and the inflow rates to the upper reservoir, due to lack of data. Further assumptions were made for modelling purposes. The results of the optimisation model were evaluated on EPANET to verify the hydraulics. The objective for the LP model was to minimise the total costs assuming pumping only occurred for 6 hours followed by turbine operation. The key decision variable was the water level of the upper reservoir. The NLP model contained another unknown, the hours of the day for pumping versus turbine operation. The results showed the NLP model increased profit by €100 per day compared to the LP model. A further optimisation model was developed for a hybrid considering both pumped-storage hydro and wind power. The addition of a wind energy park containing five wind turbines was then considered. The energy generated by the wind turbines could be used to power the pumps. The results of this model showed that profits were much higher, up to €5,200 per day. However, the costs of installation, operation and maintenance of a wind park were not included.

Further research by Vieira and Ramos (2009) included another investigation of the improvement of the energy efficiency of a WSN through the incorporation of a hydropower turbine, a wind turbine and through optimal pump scheduling. The WSN studied was a simplified realistic system, consisting of a water source feeding two populations, one at a higher elevation that requires pumping, and the other at a lower lever requiring pressure reduction via a PRV. The energy efficient method proposed included the installation of a hydropower turbine to replace the PRV. The turbine installed was assumed to have a constant efficiency of 83%. For the optimisation of the pumping schedule, a variable hourly electricity charge is considered. The optimisation model was implemented in Matlab and the hydraulics were verified using EPANET over a 24 hour period with 1 hour time steps. Simulations were run for a number of different design parameters. Simulations were run for three different diameters of the upstream and downstream reservoirs and also for three initial reservoir water levels. It was found that a daily economic benefit could be achieved through the installation of a hydropower turbine which was dependent on the initial water level of the downstream reservoir and it the reservoir volume. Maximum benefit was achieved for the lowest initial water level, as this meant that more water was required to flow through the turbine to meet the demand of that network. It was also concluded that through optimisation of the pumping schedule, that energy savings of 47% could be achieved.

Giugni et al. (2009) reported results of the application of a GA to find optimal locations for PRVs within a WSN in Naples, Italy. The replacement of these PRVs with PATs was then suggested and an economic analysis undertaken. Further to this analysis, Fontana et al. (2012) reported the results of the application of multi-objective GAs for the selection of optimal locations to install PRVs and optimal PRV settings to minimise leakage in a WSN. This optimisation approach was again tested on a WSN in Naples, Italy. The option of installing a PAT in place of these PRVs was also discussed. It was noted however that in order to optimise a WSN for energy production, the objective function should be modified. In recent research by Giugni et al. (2014), a modified objective function for maximised energy generation was presented, again applying a GA for solution of the optimisation problem. This optimisation model was applied to the benchmark 25 node network as discussed previously (Sterling and Bargiela, 1984; Nicolini and Zovatto, 2009; Eck and Mevissen, 2012).

2.7 Implementation

As demonstrated in the literature, though potential for hydropower energy generation within water supply mains exists, there have been very few of these types of projects implemented by industry. Some of the technical barriers were discussed previously, however there may also be organisational, political and socio-institutional barriers preventing widespread uptake of this technology by industry. The final research question addressed in this thesis aims to investigate barriers to project implementation from organisational and operations management perspectives. In order to approach this research question, some background theory and literature on how projects are managed was required. Key papers from both organisational and product development literature, as well as from water supply management literature were consulted.

Firstly, installing any new type of technology requires a change or movement from an old method to new, and it is widely accepted that there will be some barriers to this change (Brown and Farrelly, 2009). Across Europe and internationally, there has been a move from traditional water management practices towards sustainable urban water management (SUWM). With increased pressure on water services due to rapidly increasing urban population, depleting sources, and further duress due to the impacts of climate change, more sustainable water services are becoming increasingly vital. In Australia, researchers have investigated the barriers to the transition towards SUWM, amongst other related implementation issues. The installation of hydropower turbines on water supply infrastructure would be considered an SUWM initiative (Brown and Farrelly, 2009; Taylor, 2009; Taylor et al., 2011).

Brown and Farrelly (2009) published the results of an investigation into the barriers to implementation of SUWM in urban water supply. They found that though WSPs are beginning to move towards the SUWM approach, and are reflecting it in their water policies, this understanding is often not translated to implementation. In order to implement SUWM practices, it was noted that an integrated, adaptive, coordinated and participatory approach was required. An extensive literature review was undertaken by Brown and Farrelly (2009) of the existing body of urban water related literature and also from broader environmental management fields, to identify the key concepts and observed barriers to change. 36 barriers were identified, which were then consolidated down to 12 key barrier types, as follows:

1. Uncoordinated institutional framework;
2. Limited community engagement, empowerment and participation;
3. Limits of regulatory framework;
4. Insufficient resources (capital and human);
5. Unclear, fragmented roles and responsibilities;
6. Poor organisational commitment;
7. Lack of information, knowledge and understanding in applying integrated, adaptive forms of management;
8. Poor communication;
9. No long-term vision, strategy;
10. Technocratic path dependencies;
11. Little or no monitoring and evaluation, and
12. Lack of political and public will.

The barriers mentioned are largely socio-institutional rather than technical, and within this environment emergent leaders, also known as champions may come to the fore to push projects through to implementation (Taylor, 2009).

The implementation of environmental or sustainability initiatives in any industry can be difficult due to the need to get the industry as a whole on board. In recent years, a body of research has come to the fore investigating the need for a project champion to drive environmental change. As described in Taylor et al. (2011), while no widely accepted definition of champions exists within literature, there the consensus is that champions are emergent leaders who are centrally involved with effecting transformations within organisations or broader institutions. Taylor (2009) published results of a cross case analysis across six publically managed water companies in Australia, investigating the attributes of emergent leaders (or champions) working to implement more sustainable forms of urban water management. Ten key attributes were discussed including openness to experience, career mobility and work history, personal and positional power, strategic social networks and the organisational culture or context.

Another relevant research finding of Taylor (2009), was a definition for SUWM champions as adopted by practitioners within Australian water agencies. This definition is of emergent leaders with specific aptitudes such as: personality characteristics of creativity, persistence and resilience; a strong personal commitment to the project at hand and to environmental sustainability, a good general knowledge of the water industry and associated technology; advanced skills at exercising influence, high levels of personal power and key leadership behaviours such as identifying opportunities for influence, developing and encouraging colleagues and undertaking advance forms of social networking. This phenomenon of a project champion was also investigated throughout the case studies presented.

Jalba et al. (2014) reported on research undertaken of current water sector practices on interagency relationships. Through 51 semi-structured interviews with water utility and public health agency (PHA) staff from Australia, Canada the UK and the United States a strategy for developing and maintaining institutional collaborations was developed. In previous research by Jalba et al. (2010) six key components were identified that may be deficient in collaborations between water utilities and PHAs: (1) proactivity; (2) knowledge exchange; (3) trust; (4) regular communication; (5) joint training; and (6) a supportive regulatory environment. These deficiencies may also be present in other types of collaboration between water utilities, such as with turbine manufacturers or energy utilities as would be required for the implementation of MHP energy recovery in WSNs.

Wang et al. (2013) studied stakeholder involvement in drinking water supply systems. The primary aim of the study was to define who the major stakeholders were, and gain further understanding of their interests, influence, and relationships. Primary data collected included interviews and questionnaires with representatives of stakeholders in Shenzhen, China. Following the interviews, twenty groups of stakeholders were identified. Through further interviews with a group of 25 experts from industry, defined as people with more than ten years experience in the water industry, the final list of stakeholders was identified. This final list of key stakeholders consisted of water companies, governments, consumers, polluting companies, communities, experts and professional institutions, media and non-governmental organisations (NGOs). A stakeholder analysis was then undertaken. Stakeholders were rated and classified through use of a matrix model based on their level of legitimacy, power, urgency and impact. An interest-influence matrix was developed and is shown in Figure 2.6. The most important and definitive stakeholders in Chinese water supply were found to be water companies, governments, consumers and polluting companies. It was noted that governments play a vital role, affecting the system via legislation, regulation and compliance. Results also suggested that stakeholder involvement must be carefully managed.

Governments and legislative bodies play a vital role in incentivising renewable energy projects, through targeted policies, grants and incentives. The importance of these policies has been identified in much of the literature relating to small scale hydropower in WSNs, as was was highlighted in Section 2.4. Targeted renewable energy incentives exist such as renewable energy feed-in tariffs (REFITs). These schemes incentivise power generation from renewable energy resources by paying increased tariffs for the resulting electricity generated. Fluctuations in future energy prices will also affect the feasibility of small scale energy generation. In addition, scarcity of water is likely to increase water prices in the future and lower water demand, reducing flow and pressure in the network.

Stakeholder involvement in a development project raises the need to understand the process of interaction among stakeholders. Gehani (1992) introduced concepts for the description and understanding of different types of product or project development processes are presented. The three main approaches detailed are the serial 'relay race' approach, the iterative 'ping-pong match' approach and the parallel 'rugby' approach. Different functionalities or organisations involved in a project deliverable may operate independently like in a 'relay race' method. One stage is completed by one function/organisation with the results then passed on to the next. Alternatively, the development process may be more iterative, where the task passes between functions/organisations in a review and re-design process. One issue with this second type of approach is that it can cause delays. Finally the parallel or 'rugby' styled approach allows team members from all functions/organisations to be involved simultaneously in all stages of the project development process.

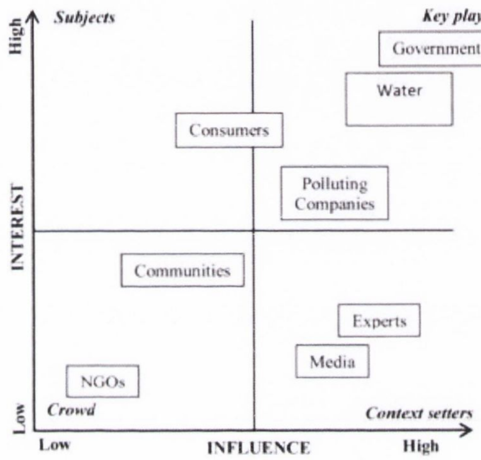


Fig. 2.6: Interest-influence matrix for the water supply system (Wang et al., 2013)

Integration and interaction between the various project stages for the development of a MHP project requires management. Cooper (2008) presented his original and updated stage-gate system as shown in Figure 2.7. The 'Stage-Gate' system is an established system or framework for driving new products to market. The key stages are detailed in Figure 2.7, beginning with the 'Discovery' stage where idea generation takes place and ending with the project launch. Following each stage is a go or kill decision point. This decision point on whether to move on to the next stage or not is based on defined criteria, deliverables or outputs. These gates allow projects to be accelerated quickly, or otherwise to prevent slow or infeasible projects from progressing early on in the process. Along the various stages there is a repeated feedback and review process between players at all stages. This Stage-Gate framework may be a useful framework to help accelerate the implementation of MHP projects within water supply infrastructure.

The development of new MHP projects on WSNs takes place in a strategic context.

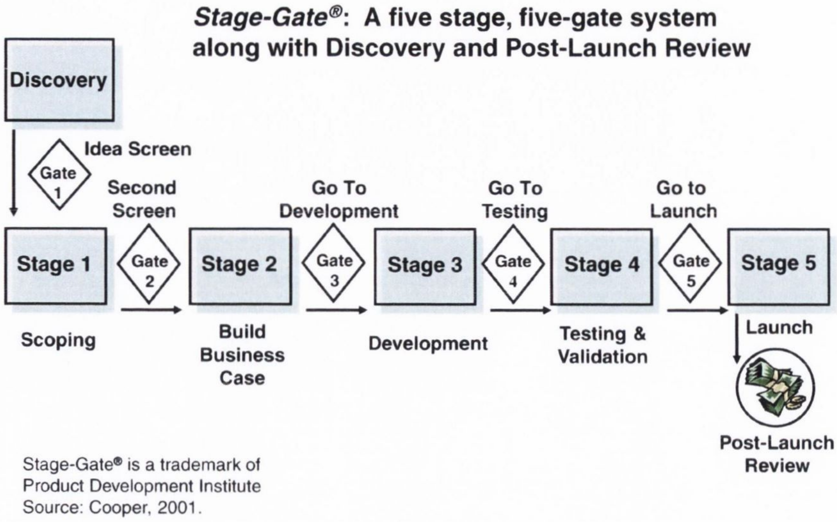


Fig. 2.7: A typical Stage-Gate system for major new product developments (Cooper, 2008)

Nystrom (1985) published a paper on the integration of technology and marketing as a basis for new product (or project) development strategy. The framework presented was tested in a number of different industries and was found widely applicable across a range of industries from pharmaceuticals, to steel manufacturers, to farm machinery. A method of measuring the success of these product (or project) development strategies was proposed in order to assess the relationship between the strategy and the project outcomes. Three performance dimensions were specified in this framework: technological, competitive and financial. The technological outcome is a rating of the level of technological innovation achieved based on the degree of novelty or uniqueness of the product. The competitive outcome is described as the interchangeability of the product with competing products already on the market. The financial outcome is a measure of the profitability of the product over its life cycle. These three performance measures could be employed for measurement of the success of MHP projects in WSNs.

Another relevant organisational aspect which could affect how MHP projects are implemented in the water industry is the organisational structure of the water companies involved. The organisational framework of water supply authorities can vary from a publically owned and operated organisation, to publically owned and privately operated, to completely privately owned and operated. Over 90% of the approximately 250,000 water service systems worldwide are municipally owned water and wastewater utilities, while only 8% are privately operated and/or owned (Kwok et al., 2010). In

OECD countries, the number of people relying on the private sector for water services ranges between 200 and 300 million (Perard, 2009), which represents about 17-25% of OECD members population. Figure 2.8 illustrates the break down of private sector participation in water supply for OECD countries.

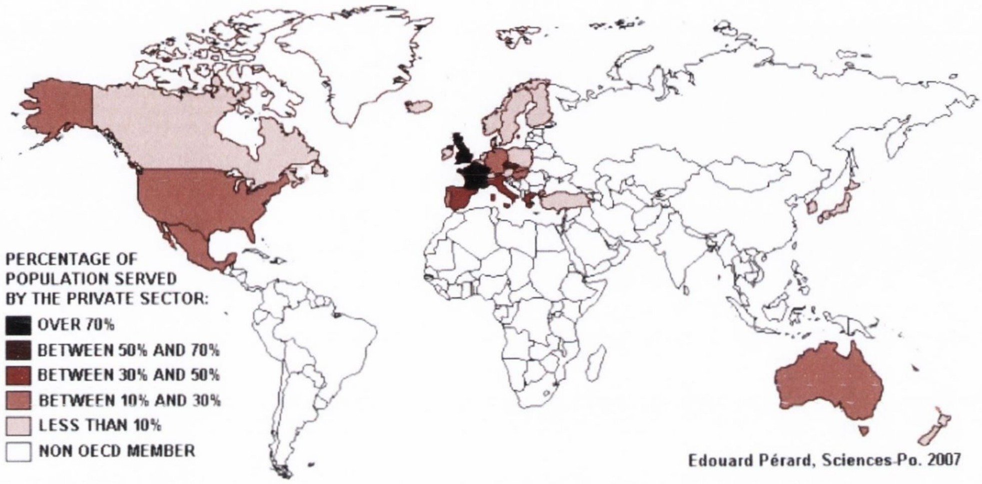


Fig. 2.8: Private sector participation in water supply in OECD countries (Perard, 2009)

2.8 Summary

It is clear from the literature that it is possible to recover energy within water supply networks through the installation of hydropower turbines. Though there are numerous published feasibility studies highlighting the potential and opportunities for hydropower within water supply infrastructure, there are few examples of penetration and adoption of this technology by the water industry.

A number of issues with the technology have been highlighted repeatedly in the literature and may be reasons as to why this technology has not yet been widely adopted. These issues are primarily related to the effects of flow variation and future cost fluctuations and their impact on project viability and also the absence of targeted policies to incentivise the installation of hydropower turbines in WSNs. This research will investigate the effect of flow variation and turbine efficiency as well as variations in energy prices and feed-in tariff prices on project feasibility and investment payback periods.

The issue of long term flow variation and its effect on successful investment return is another area requiring further investigation. When making long term strategic plans, water service providers use statistical techniques, such as multiple linear regression, and also computational intelligent approaches, such as ANNs and GAs for forecasting. The application of these techniques to be included in micro-hydropower feasibility studies will also be investigated.

The literature has highlighted that both mathematical and heuristic optimisation methods can be applied to the optimal design of a water supply network. However the application of optimisation methods, to the incorporation of hydropower turbines has not been extensively researched. This research will investigate the use of both traditional mathematical methods and heuristic optimisation methods for the optimal installation of hydropower turbines in WSNs.

Finally, for hydropower energy recovery to be widely applied in WSNs, further research is required into related organisational, political and operations management aspects, in the context of the water industry. This research will also investigate these potential organisational and political barriers preventing the uptake of this technology by industry.

CHAPTER 3

Experiment Design

3.1 Introduction

This chapter forms the conceptual background to the research study undertaken by outlining its context and the methodological approaches selected. This is not a mono-methodological research study. The research question and sub-questions as outlined in Chapter 1 require a number of different methodological approaches which vary from qualitative to quantitative in nature.

3.2 Research Model

The research model or framework underlying this thesis submission is illustrated in Figure 3.1. Hydropower energy recovery will be explored under four main themes: Feasibility, risk analysis, optimisation and implementation. The context within which these are explored is the water supply network.

The underlying opportunity explored in this research is that excess pressure exists

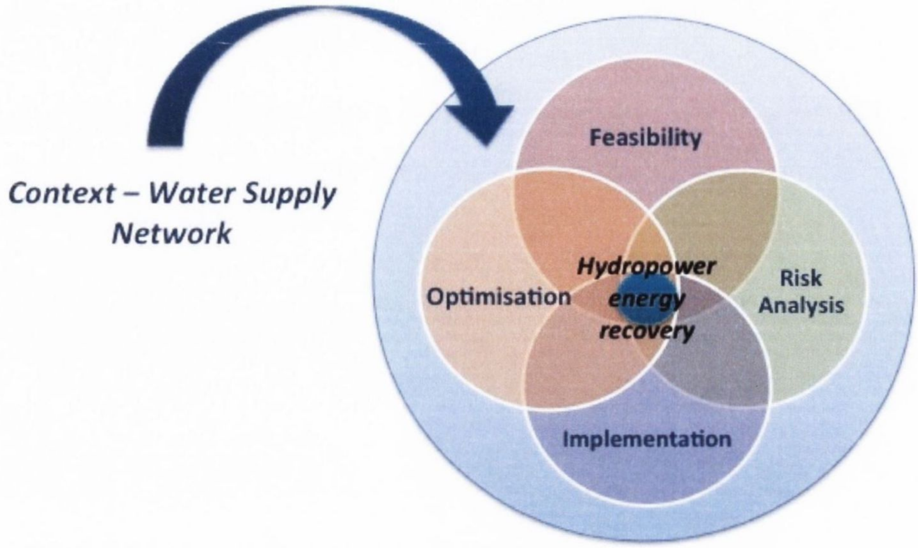


Fig. 3.1: Thesis research model

within pressurised water supply mains and that rather than dissipating this pressure, it could be used to drive a turbine and generate electricity. Initially the research was exploratory; exploring this phenomenon, its occurrence in industry and the feasibility for harnessing this energy. Following an initial critical literature review in this field as discussed in Chapter 2, a number of issues relating to the installation of hydropower turbines in place of PRVs were identified. These issues included the effect of large variations in flow rates across turbines, the impact of long term flow variation on the lifespan and investment payback period of an installed hydropower turbine, and finally organisational and political barriers to the widespread uptake of this technology by the water industry. These issues formed the basis of the research undertaken in Chapters 4, 5, and 7, under the themes of Feasibility, Risk Analysis and Implementation as illustrated in Figure 3.1. Another aspect of this research that was identified following the literature review was the application of optimisation techniques for optimal design of WSNs. The application of optimisation techniques to the optimal installation of hydropower turbines is an area that has seen very little research. In particular, the application of mathematical programming techniques. One final research theme addressed in this thesis is focused on optimisation techniques and their applicability to the installation of hydropower turbines in WSNs.

3.3 Methodological approach

This thesis research also contributes to and builds on a wider research project, the Hydro-BPT project. The Hydro-BPT project is a collaborative project between the Schools of Engineering and Business at Trinity College and also with the School of Environment, Natural Resources and Geography at Bangor University Wales, and it is part-funded by the INTERREG: Ireland-Wales Programme. INTERREG, meaning 'inter-regional', is an EU Community Programme that aims to strengthen economic and social cohesion by promoting international and cross-border co-operation (Interreg, 2014).The boundaries of the INTERREG Ireland-Wales region within which this research project is focused on are illustrated in Figure 3.2. The region consists of the eastern and south-eastern counties in Ireland, and western coastal areas of Wales.

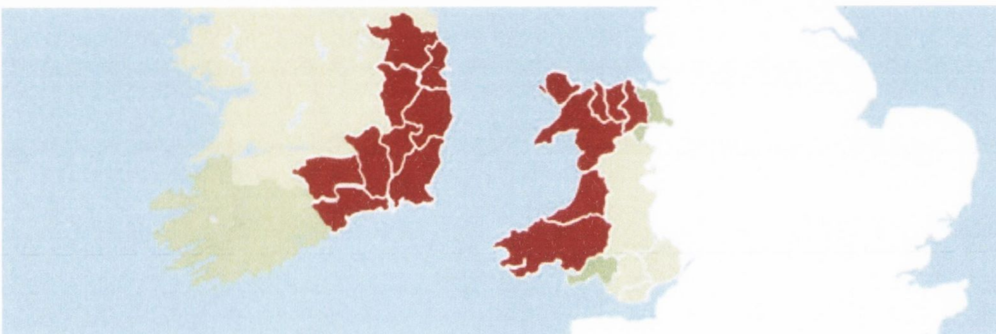


Fig. 3.2: INTERREG: Ireland-Wales region

This INTERREG region has defined the primary geographical areas for which data has been collected and analysed as presented in this thesis. The overall aim of the Hydro-BPT project is to investigate the improvement the sustainability of water services through the incorporation of hydropower energy recovery devices. The cross-disciplinary nature of the Hydro-BPT project is mirrored in this thesis research, for which both engineering and organisational research has been undertaken. Within the Hydro-BPT project, three PhD research projects are currently in progress, this project which is focused on water supply network infrastructure, another focused on wastewater infrastructure, and a final project focused on lab-scale testing and computational fluid dynamics (CFD) modelling of PATs for application as PRVs.

To approach the research question a number of different and varied methodologies were required. A summary of these methods are presented in Table 3.1.

Table. 3.1: Research Model Summary Chadwick et al. (2004)

	Data Type	Data	Analysis	Method
Feasibility	Quantitative	DCC, WW Flow, pressure data	Power output estimate; Economic analysis.	Data based analysis
Risk Analysis	Quantitative	Flow data; climate data; economic data.	MLR; ANN; Future scenarios	Data based analysis
Optimisation	Quantitative	Benchmark WSN input data	Optimisation algorithm formulation (within MATLAB).	Solvers - NLP, GA, MINLP
Implementation	Qualitative	Archival data; Interview transcripts	Comparative case studies	Case research

3.3.1 Feasibility

To investigate the potential for hydropower energy recovery within the water supply networks in Ireland and Wales, data was gathered from many of the local authorities and water service providers in these regions. Power generation estimates were quantified for these potential hydropower locations. An in-depth analysis of the economic feasibility at these sites was then undertaken, including a sensitivity analysis investigating a number of potentially sensitive variables such as energy prices, FIT rates and choice of discount rate, and their impact on overall project feasibility. As highlighted in Chapter 2, many previously published feasibility studies of the application of hydropower energy recovery in WSNs, have assumed average constant turbine efficiencies across average flow conditions. Also highlighted in the literature was the need to address the issue of flow variation across turbines, which will impact turbine efficiency. Building upon these issues identified in the literature, a further analysis was undertaken to estimate the power generation potential at sites based on variable turbine efficiency values dependent on the variable flow rates at these sites. Table 3.1 outlines the data types studied and analysis methods employed for this analysis.

3.3.2 Risk Analysis

Following on from the initial exploratory feasibility work, issues were raised relating to the effects of long term changes in water demand on flow rates through turbines. This was also identified as an area requiring further research during the critical literature review in the field presented in Chapter 2. Chapter 5, Risk Analysis, presents results of the application of forecasting techniques to predict the long term changes in flow rate at potential hydropower locations. Hydropower projects in WSNs are often decided on based on a predicted investment payback period. In general, hydropower projects in WSNs are deemed feasible if investment payback can be achieved within ten years. For this reason, flow rate and pressure data over a ten year period for a number of valves and reservoirs in both Ireland and Wales was analysed. The impact of climate data and socio-economic data on this flow rates at these sites was explored. Regression techniques and Artificial Neural Network (ANN) techniques were investigated as tools for long term flow prediction at these valves. These tools were used to predict future flow rate scenarios at these valves. Table 3.1 outlines the data type and analysis methods employed for this analysis.

3.3.3 Optimisation of WSNs

The next phase of research focused on the development of an optimisation model which could be applied to any water supply network to determine the optimum location, if any, to install a hydropower turbine. This research also aims to confirm or refute the proposition that an optimisation model could be used to design more economic and energy efficient water distribution networks. Previous research has focused on the feasibility of the installation of hydropower turbines at existing locations in WSNs, for example at PRVs, inlets to tanks or at service reservoirs. However, other suitable potential locations may exist on pipes in other parts of a WSN.

As discussed in Chapter 2, optimisation techniques can be applied for a number of design objectives relating to optimal WSN design. For example, for the least cost selection of pipes to meet WSN demands, for the minimisation of pumping costs or for optimal location of PRVs in a network to reduce network pressures. An optimisation algorithm for the optimal location of new hydropower turbines in a WSN was then developed and implemented in MATLAB. Three optimisation techniques were applied to solve this optimisation design objective; a genetic algorithm (GA), a non-linear programming (NLP) technique and a mixed integer non-linear programming (MINLP) technique. A comparison of the effectiveness of these optimisation solvers was also included, between the traditional mathematical techniques and techniques derived from heuristics.

3.3.4 Implementation

The potential for hydropower energy recovery within WSNs is known and has been reported on in much literature as discussed in Chapter 2. However, very few projects have been implemented in practice. Though technical solutions exist, potential organisational or institutional barriers may be preventing their widespread implementation. To investigate the organisational, management and regulatory issues associated with the implementation of MHP in practice, a cross case analysis was undertaken of two completed MHP projects. This qualitative approach required the gathering and analysis of archival data and also the undertaking of semi-structured interviews with key players involved in these projects.

3.4 Summary

In summary, in accordance with the key research question and objectives as outlined in Chapter 1, this research requires a number of different methods and approaches. These different methods vary from quantitative to qualitative. The four key research themes will be addressed in the following chapters; Feasibility (Chapter 4), Risk Analysis (Chapter 5), Optimisation (Chapter 6) and Implementation (Chapter 7).

CHAPTER 4

Feasibility

4.1 Introduction

The water industry is under increased environmental and economic stress, as was previously discussed in Chapter 2. With water quality regulations becoming more stringent, water demand increasing and energy prices rising, the cost of supplying clean water is becoming increasingly unsustainable. Furthermore, Ireland and the UK are legally bound to increase their renewable energy generation under Directive 2009/28/EC. In order to address all of these issues, water supply providers (WSPs) today aim to minimise their energy usage, improve energy efficiency and to develop new renewable energy resources on water supply infrastructure to become more sustainable and self-sufficient. Some of these energy efficiency options available to WSPs have also been discussed in Chapter 2.

This thesis research is focused on one particular method of improving water supply sustainability and energy efficiency- the incorporation of hydropower turbines within water supply networks (WSNs). The option to install any hydropower turbine in a WSN depends upon the technical and economic feasibility of the installation. As identified in Chapter 2, there has not yet been a widespread uptake of this technology by WSPs.

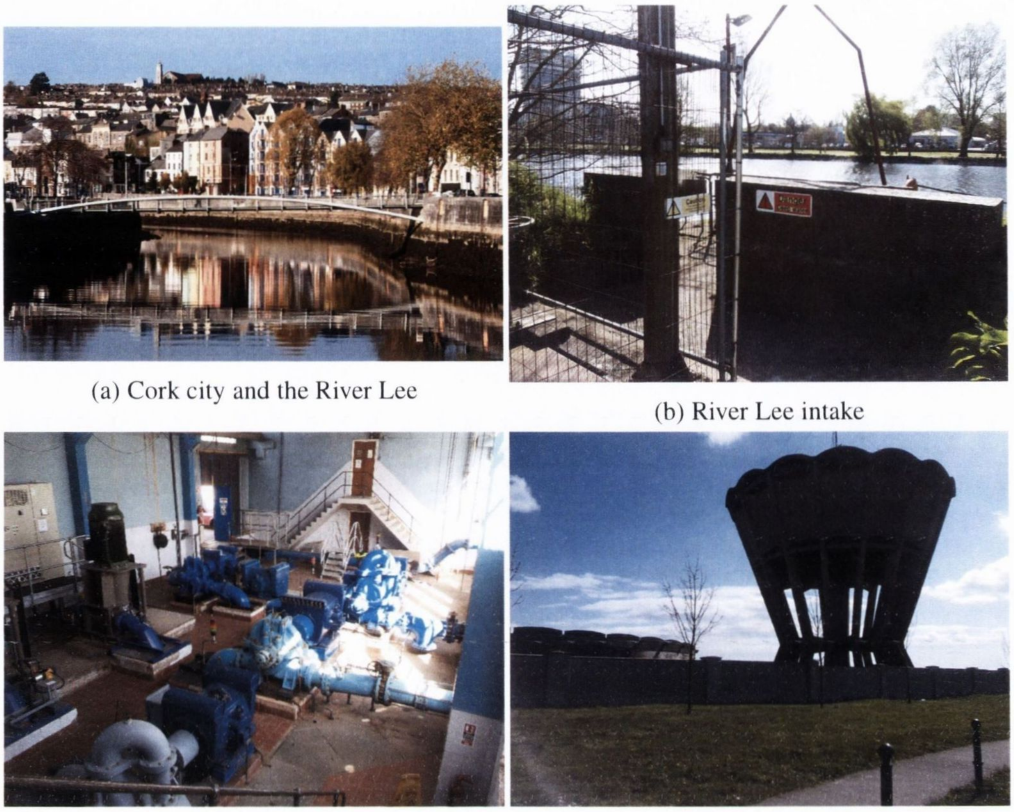
Technical issues such as the effect of variable flow rates on turbine efficiency, as well as the absence of detailed cost evaluations were identified as potential barriers and as areas requiring further research and discussion. This chapter begins with an introduction to the context of hydropower energy recovery in WSNs, followed by an assessment and discussion of the technical and economic issues affecting hydropower installation feasibility. Water supply network infrastructure in Ireland and Wales was studied to assess the potential and feasibility of hydropower energy recovery.

4.2 Background

Water supplied in our distribution mains is predominantly designed to be gravity fed, thus reducing the need for expensive pumping of water. Service reservoirs are usually located at higher elevations so that water can then flow by gravity throughout the water supply district. However, there may be points in a network where some pumping is necessary, but WSPs aim to reduce the amount of pumping to as little as possible. Moreover, as discussed in Chapter 2, efforts are now being made to minimise pumping costs by optimising the scheduling of pumping so that pumps only operate at times when energy tariffs are lowest. Within both gravity-fed and also pumped water supply districts, water pressure must be carefully managed. Water pressure at water taps in all homes and businesses must be at a minimum required pressure. In the UK, this minimum pressure is 7 metres as defined by OFWAT the UK water supply regulators (OFWAT, 2014).

One of the primary sources feeding Dublin city is Vartry Reservoir, located in the Wicklow mountains. This mountainous location, at a high elevation, means that water can be gravity fed from the source to the adjacent treatment works and can again be gravity fed along approximately 30km of large diameter water mains to Stillorgan reservoir, a large service reservoir close to Dublin city. Water is then gravity fed throughout District Metered Areas (DMAs) in the Dublin city region. Pressure reduction is then required on both the trunk mains and the distribution mains within the city centre. This pressure reduction is achieved through the installation of control valves (primarily on trunk mains) and PRVs (primarily on distribution mains).

In contrast, in Ireland's second largest city, Cork City, more pumping is required. Cork is a city of hills and valleys. The city boundaries have spread over time from the low-lying areas near St Patrick's Street, over the surrounding suburbs located on hills to the



(a) Cork city and the River Lee

(b) River Lee intake

(c) Cork city water supply - pumping station

(d) Water tower at high elevation

Fig. 4.1: Case example: Cork city water supply: Energy usage and pressure build up

north and south. The primary city water source is the River Lee. With the city surrounding both sides of the Lee, pumping is required to meet the water demands of these areas. The intake and treatment works are located on the North side of the river, with the service reservoirs and water tower located up on top of a hill above the city. Water must be pumped from the intake up to these service reservoirs before it then flows by gravity through the city's distribution mains. The presence of large elevation drops in the Cork city WSN means water pressure builds up quickly, increasing the risk of burst pipes and intensifying the magnitude of leakage.

Pressure management is therefore a key responsibility for Cork City Council, and is achieved through a variety of measures. The city water supply is divided into DMAs with the inflow and outflow to each carefully monitored. There are also a number of PRVs located at various points within the distribution mains. Despite the current pressure management activities in place, 55% of water supplied in Cork City in 2012 was unaccounted for. As has been discussed previously, hydropower turbines have been identified as a potential alternative pressure reduction measure, with the added benefits of producing much needed electricity, and reducing the dependency on fossil fuels, reducing the carbon footprint of water industries.

4.2.1 Energy Recovery Locations

As illustrated in the previous two case studies in Cork and Dublin, water supply network designs vary. The location of the source determines whether the water in that network is supplied by gravity or by pumping. Head loss through a network will also vary depending upon the topography of the region and the location of the pipes. Figure 4.2 shows a typical pipeline between two storage reservoirs. During WSN design, the minimum and maximum pressures along this pipeline must be found to ensure the pipes have a sufficient pressure rating. Water pressures in a system are generally maintained between a maximum (about 70m head) and a minimum (about 20m head) value (Chadwick et al., 2004).

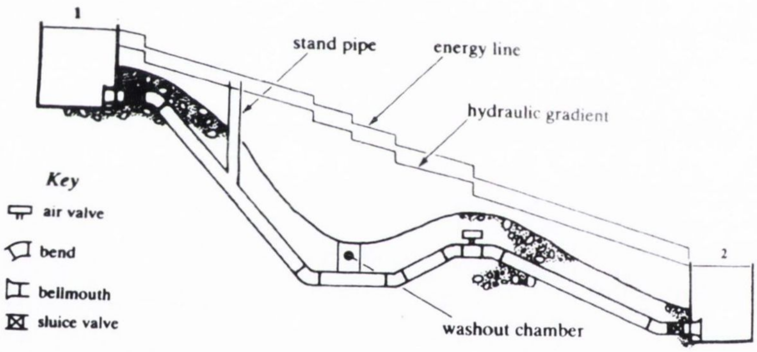


Fig. 4.2: Design of a simple piped WSN (Chadwick et al., 2004)

According to Bernoulli's equation, at any point between the reservoirs:

$$\frac{p}{\rho g} + \frac{V^2}{2g} + z = \text{height of the energy line} \tag{4.2.1}$$

and:

$$\frac{p}{\rho g} + z = \text{height of the hydraulic gradient} \tag{4.2.2}$$

Where p is the pressure (m), V is the velocity (m^2/s) and z is the elevation (m). The energy line begins and ends at the water level in the upper and lower reservoirs, while the hydraulic gradient is always a distance $V^2/2g$ below the energy line. The maximum and minimum pressures can be found by finding the maximum and minimum heights between the pipe and the hydraulic gradient (Chadwick et al., 2004).

If the hydraulic gradient is below the pipe, then there is sub-atmospheric pressure at that point. This condition must be avoided as cavitation may occur. Furthermore, if there are any leaks in that pipeline, external matter may be sucked into the pipe, potentially polluting the water supply.

In order to prevent excess pressures, a number of pressure reducing methods (e.g. PRVs and BPTs) may be installed in a network, as has previously been discussed in Chapter 2. These locations, illustrated in Figure 4.3, present opportunities for potential turbine installation. However, output pressures at these turbines may also require accurate control, dependent on the topography later on in the network, to prevent both excessively high pressures and the occurrence of sub-atmospheric pressures.

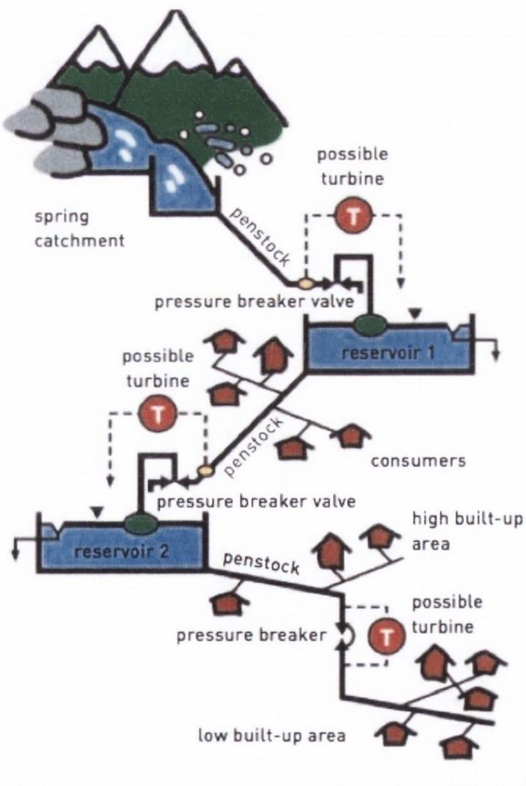


Fig. 4.3: Potential hydropower energy locations in a WSN (ESHA, 2010)

4.2.2 Context within WSN

Hydropower energy recovery, as discussed in Chapter 2, has previously been applied within water distribution networks. However, installed industry examples are primarily found on the water transmission (WT) side of the distribution network, and not within the distribution mains themselves. The difference between these two contexts for hydropower energy recovery was illustrated clearly by Carravetta et al. (2012) as shown in Figure 4.4. Installing turbines on the WT nodes, such as at water treatment works, tanks or reservoirs, or the end of large transmission lines (trunk mains) present opportunities for energy recovery. These locations have more regular and predictable flow rate patterns, and are not heavily effected by seasonal variations.

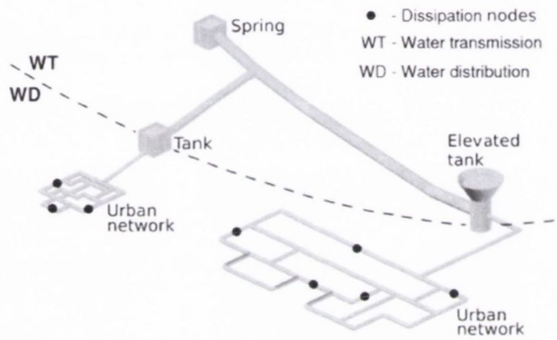


Fig. 4.4: Water supply system showing potential energy recovery points (dissipation nodes)(Carravetta et al., 2012)

PRVs however which are primarily located on distribution mains within the urban water distribution (WD) network, also present opportunities for energy recovery. These locations are highly dependent on the local water demands of the areas they are located in, and as such see more extreme levels of daily and seasonal variation. For a larger uptake of hydropower generation within WD mains, further research is required on the impact of these flow variation patterns on turbine selection and turbine operating efficiencies.

This chapter presents results of an analysis of the potential for hydropower energy recovery on both water transmission and water distribution mains of Ireland and Wales. However, the focus is primarily on the largely untapped resource and more technically challenging opportunity of electricity generation at PRVs on WD mains.

4.2.3 Turbine Technology

Hydropower turbines are classified into two broad categories; impulse turbines or reaction turbines. At an impulse turbine, water hits the blades at atmospheric pressure. Water is directed at the blades through a nozzle, which converts the kinetic energy of the water into mechanical energy. In reaction turbines, water hits the blades at a higher pressure than atmospheric, and also at high speed. Fluid enters the turbine containing both kinetic and potential energy, which the turbine then transfers into mechanical energy. Another practical wat of classifying hydropower turbines is in three groups depending on the relative head and flow (Rodriguez and Sanchez, 2011): high head and low flow turbines; medium head and medium flow turbines; and low head and high flow turbines.

4.2.3.1 High-head low-flow turbines

The two most well-known high-head and low-flow turbines are the Pelton turbine and the Turgo turbine. These are both impulse type turbines. The Pelton turbine is the oldest and most commonly used turbine in the world. It is reliable, robust and highly efficient machine. Micro, mini and pico Pelton turbines are currently manufactured, with one jet or multiple jets depending on the head and flow conditions (Rodriguez and Sanchez, 2011). Pelton turbines operate under high head (60-1000m) and with relatively small discharges (less than $100m^3/s$) (Zu-yan, 1991).

4.2.3.2 Medium-head and medium-flow turbines

The Francis turbine and the cross-flow (Michell-Banki) turbine are the most well known in this range. Pumps-as-turbines (PATs) would also apply in this head-flow range. The Francis turbine is a reaction type turbine. Its design can be varied to operate over a large performance range. Different shaft speeds can be obtained for a specific head and flow, depending on the design of the blades (Rodriguez and Sanchez, 2011). Francis turbines also usually have a set of guide vanes which can control the flow rate depending on the energy requirements. A cross-flow turbine is an impulse type turbine. The turbine spins following the impact of a water jet on its blades. Unlike the Pelton and Turgo turbines, the cross-flow turbine has a larger rectangle nozzle.

In recent years, PATs have come to the fore as a cost-effective alternative to traditional hydropower turbines. The most commonly employed PATs are centrifugal pumps with high specific speeds. The main advantage of a PAT is that pumps can be bought off the shelf, ready for installation and are usually cheaper than equivalent turbines. Turbines however must be ordered and manufactured to suit specifications.

One issue with buying a pumps off the shelf for installation as a PAT is that the turbine performance curves are unknown. The pump performance would be known, however a PAT will not operate at the same performance as the equivalent pump operation. In order to establish the characteristics of a PAT, testing must be undertaken. Figure 4.5 shows the characteristic curves of a Sulzer centrifugal pump operating in both pump mode and turbine mode. Testing of this pump operating as a turbine showed that the best efficiency point (BEP) of the turbine was located at a higher flow rate and higher head. That means the capacity is higher in the turbine mode than when pumping (Adams and Parker, 2011).

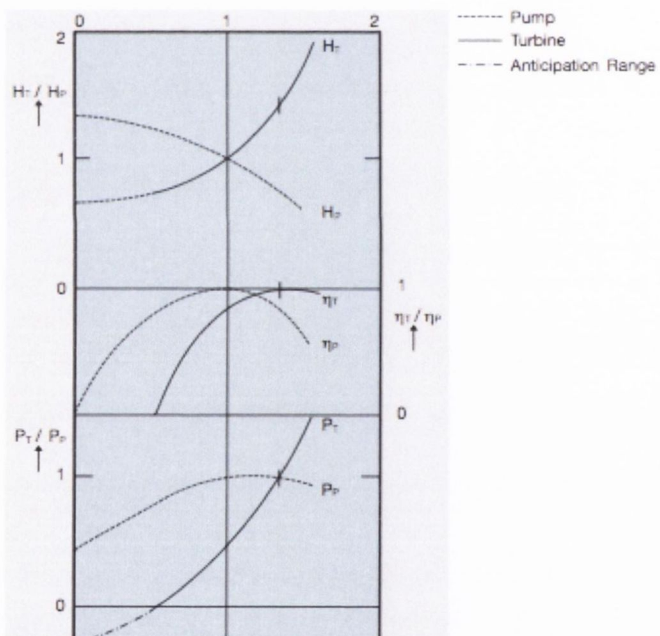


Fig. 4.5: Characteristics of a pump impeller in pump and turbine mode (n=constant, D=constant). The best efficiency point (BEP) of the turbine is shifted to higher head and flow rate. (Adams and Parker, 2011)

4.2.3.3 Low-head, high-flow turbines

Axial-flow turbines operate over low-head, high-flow sites. These are reaction type turbines. Different shaft speeds can be obtained by changing the positioning of the blade angles. The Kaplan and propeller turbines are two types of axial-flow turbines. They operate similar to a ship's propeller, except in reverse. Kaplan turbines have adjustable blades that can be changed depending on the flow and pressure conditions. These blades can be regulated automatically in response to the energy generation requirements. A propeller turbine however has fixed runner blades.

4.2.4 Turbine Selection

Hydropower turbine selection is site specific, depending upon the water flow rates and pressures as well as the nature of the site. Different potential locations for hydropower energy recovery in WSNs as discussed in the previous sections, may have different site requirements. Within the distribution mains, the output pressure of a turbine would have to be fixed at a specified value to guarantee downstream demand requirements are met. However, at inlets to tanks or reservoirs, the output pressure of the turbine would not need to be fixed.

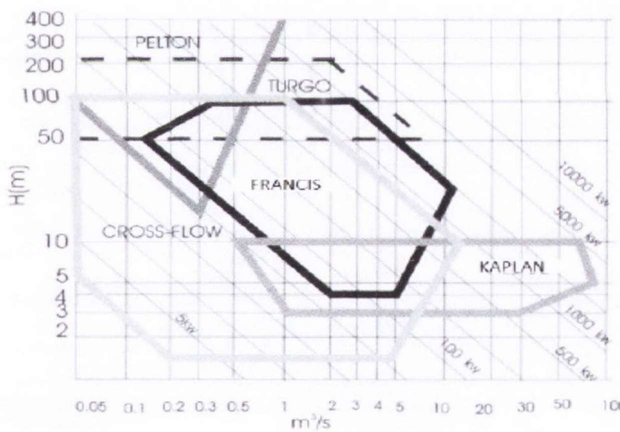


Fig. 4.6: Small scale hydropower turbine selection chart (Morales et al., 2014)

In order to select the most appropriate turbine to suit the site conditions present, turbine selection charts such as Figure 4.6 are often employed. Every turbine type has an optimal operating range where it performs most efficiently. In order to obtain exact efficiency behaviour of a turbine, it is essential to test the turbine. In order to test a turbine, a booster pump with sufficient power must be used to provide the inlet flow and high inlet pressure. The output power of the turbine must be measured with a calibrated generator, torque meter, or dynamometer. Measurements of power, flow, and pressure are used to calculate turbine efficiency (Adams and Parker, 2011). To avoid cavitation at the turbine outlet, backpressure at the outlet must be controlled.

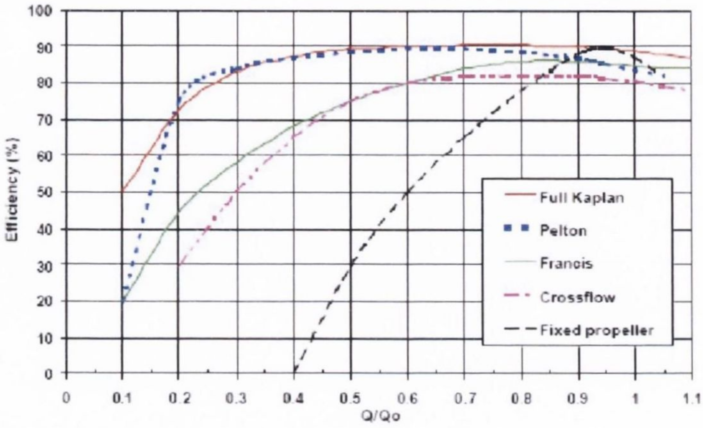


Fig. 4.7: Efficiency of different turbines over range of flow rates (Gatte and Kadhim, 2012)

Turbine efficiencies will also vary with reductions in the rated design flow of the turbine. Some typical efficiency curves are shown in the Figure 4.7. As can be seen in this graph, the Pelton and Kaplan turbines retain very high efficiencies when running below their design flow. In contrast the efficiency of the Crossflow and Francis turbines decreases more sharply if they operate below half their design flow. The fixed-blade propeller turbine perform poorly below 80% of full flow. A summary of the discharge requirements of some of the primary turbine types are detailed in Table 4.1. This Table also details the flow rate control options of each device. The Pelton, Francis and Kaplan turbines all have adjustable elements which can be used to control the flow rates. The fixed propeller type turbine and the PAT however do not.

4.2.5 Turbine Specific Speed

For detailed design and analysis, it is important to examine how a turbine will perform under the specific site head and flow conditions, and how it will perform with variations in these conditions. Within water supply infrastructure, the flow rate is more likely to vary than the head. In some cases the head may vary, so it is important to understand how a selected turbine will perform under variable loads.

Table. 4.1: The four primary turbine types and their flexibility ESHA (2010)

Turbine Type	Discharge Control Device	Minimal Discharge
Pelton	1-5 adjustable nozzles	At least 15% of the nominal discharge per nozzle
Francis	Adjustable guide vanes	Approx. 50% of the turbine nominal discharge
Kaplan	Fixed or adjustable guide vanes, adjustable runner blades	At least 20% of the turbine nominal discharge
PAT	No device, would require additional valves to regulate flow	85-90% of the machine nominal discharge

To investigate the technical characteristics of a turbine it is necessary to perform a number of controlled tests over a range of conditions to measure the parameters accurately. The turbine law of similarity is then employed, whereby the running conditions of a large generating turbine are compared to its laboratory counterpart (Rodriguez and Sanchez, 2011). The most frequently used parameters of the laws of similarity are:

- Specific speed based on power:

$$N_S = \frac{N\sqrt{P}}{H^{5/4}} = N\frac{\sqrt{(\rho g Q \eta)/K}}{H^{5/4}} \tag{4.2.3}$$

- Specific speed based on flow:

$$N_q = \frac{N\sqrt{Q}}{H^{3/4}} \tag{4.2.4}$$

Where N_S and N_q are the specific number of revolutions of power and flow, respectively; N is the rotation speed of the turbine in revolutions per minute (rpm); P is the power of the turbine (HP or kW); Q is the design flow of the turbine (m^3/s) and H_n is the net head (m).

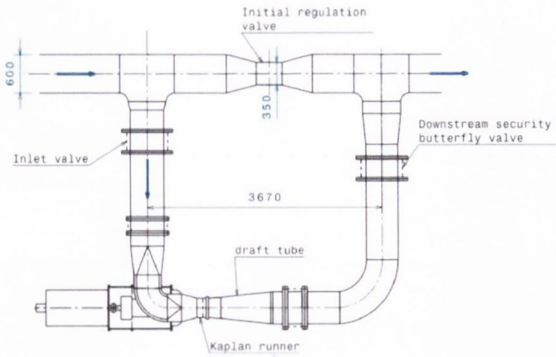
Table. 4.2: Main characteristics of hydraulic turbines (adapted from (Rodriguez and Sanchez, 2011))

Turbine	N_S	Q (m^3/s)	H (m)	P (kW)	Max. e_0 (%)
Impulse					
Pelton	1 jet 25; 2 Jet 25-50; 4 Jet 25-50; 6 Jet 43-60	0.05-50	30-1,800	2-300,000	91
Turgo	50-225	0.025-10	15-300	5-8,000	85
Michell-Banki	34-138	0.025-5	1-50	1-750	82
Reaction					
Rotodynamic pump	25-147	0.05-0.25	10-250	5-500	80
Francis	Low N_S: 50-130; Normal N_S: 130-215; Fast N_S: 215-345;	1-500	2-750	2-750,000	92
Kaplan and propeller	250-690	1,000	5-80	2-200,000	93

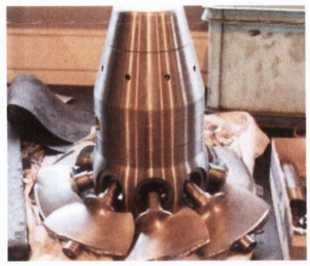
4.2.6 Turbines in WSNs

Traditional reaction type turbines are most suited for PRV sites as they sit in the full flow of the water, therefore can operate in pressurised pipes where an output pressure must be maintained. Impulse type turbines, such as the Pelton turbine, are more suited for end of pipeline locations, such as at the inlet to tanks or reservoirs. This is due to the fact that a Pelton runner operates in air (at atmospheric pressure). However, when the output pressure is required to be a certain value, a counter pressure Pelton turbine could be employed (ESHA, 2010). The runner of a counter pressure Pelton turbine rotates in an air volume maintained at the required output pressure.

Reaction type turbines, such as a Francis turbine or a Kaplan turbine can be directly installed as a bypass to a PRV. Francis turbines however are not efficient in coping with large variations of head, especially in the lower head range Zu-yan (1991). Figure 4.8 depicts an existing valve in a drinking water network (Poggio Cuculo plant) at Arezzo, Italy (ESHA, 2010). This Kaplan turbine installation has a nominal discharge of 0.38 m^3/s and a gross head of 28m. The power output of the plant is 44kW.



(a) Kaplan turbine configuration



(b) Kaplan turbine runner

Fig. 4.8: The Poggio Cuculo Kaplan turbine installed in the Arezzo WSN, Italy (ESHA, 2010)

One issue to consider when installing a Kaplan turbine is the potential for the occurrence of cavitation. Though a Kaplan turbine has a higher specific speed and operates more efficiently than a Francis turbine in the low-head range, it has poorer cavitation characteristics. This is because the runner exit kinetic energy is higher than that of a Francis turbine (Zu-yan, 1991).

4.3 WSNs in Ireland and Wales

The focus of this assessment is on WSNs in Ireland and Wales. During this assessment, Irish water supply was operated by the Water Services Authority (WSA). The WSA consisted of five city councils and twenty nine county councils. In contrast, the majority of water supply services in the UK, including Wales, are run by private water companies as shown in Figure 4.9.

Pressure and flow data from Irish and Welsh water supply networks were gathered for a range of potential energy recovery locations such as PRVs, control valves, reservoirs and break pressure tanks (BPTs). However the focus of this thesis research is on the installation of turbines within WSNs as pressure reduction devices, therefore the majority of the data collected was for PRVs. This Chapter presents results from a variety of different types of water supply schemes ranging from small rural group water schemes (GWS), to small-medium sized town WSNs, up to large urban water supply networks. The schemes examined in Ireland were located around the greater Dublin area, Dublin city and county, Kildare and Wicklow in the east and also southern areas such as Cork

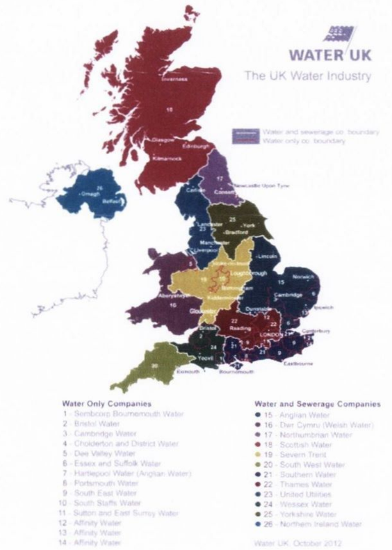


Fig. 4.9: UK water companies UK Water (2014)

City, Tipperary and Waterford. A small group water scheme located in Co.Monaghan was also investigated. The schemes examined in Wales were all located under the jurisdiction of Welsh Water. Welsh Water were responsible for water services for the majority of Wales, except for a small section on the east which is governed by Severn Trent Water. In total, 174 sites were studied. These site locations are shown in Figure 4.10 with further site information included in Appendix B for reference.

The nature and amount of the data obtained varied for each location. Twelve months of telemetry data readings taken every 15 minutes from control valves and PRVs within the Dublin city water supply network were analysed to identify suitable locations for energy recovery. These readings were taken for the period June 2010 to August 2011. A sample of reservoirs and PRVs from within the greater Dublin area and from Cork City were also analysed to estimate their potential for energy recovery. The data obtained for the South Dublin Region, under South Dublin County Council, were average flow rates taken over the period 16th to the 23rd of April 2012. The inlet and outlet pressure values were obtained for both the daytime and night-time settings for their PRVs. A number of PRVs in the Fingal County Council jurisdiction in North Dublin were also studied. Data for three PRVs in Bray, Co.Wicklow, a large town on the Dublin city commuter belt, were also studied. The population of Bray town was recorded at 31,872 in the 2011 census, making it the 9th largest urban area in Ireland. Reservoirs and tanks in the Co.Kildare region were also investigated. Average daily values for inlet and outlet flows and pressures were analysed.

Fig. 4.10: Location of Sites Assessed in Ireland and Wales

Rural areas and small towns outside of Dublin were also investigated, including the Clonmel town water supply in south Co. Tipperary. Clonmel is the largest town in Co.Tipperary, with a population of 17,908 people recorded in the 2011 census (CSO, 2011). On a smaller scale again, three valves from the Waterford County Council region were also investigated.

Another important sector of water supply in Ireland is the Group Water Schemes (GWS) sector. GWS serve approximately 10% of the total population of Ireland. GWS are independent community-owned supplies serving two or more houses and are generally located in remote, rural locations. Further insight into the role of the GWS sector in Ireland can be found in Brady and Gray (2010).

One private GWS, Tydavnet, in the north of Co.Monaghan was investigated as part of this research. The location of this scheme can be seen in Figure 4.10 as the northernmost point plotted. The total local population in the Tydavnet region was recorded at 298 in the 2011 census, with a total housing stock of 129 (CSO, 2011). Flow rate readings from the inlet and outlet of their primary supply reservoir, taken every 15 minutes from October 2010 to March 2012, were analysed. This reservoir is fed by gravity from the source and treatment works.

The Welsh Water data consisted of flow and pressure information for all PRVs on mains of over 150mm diameter. The flow data consisted of average daily flows for the month of February 2011.

4.4 Renewable Energy Feed-in Tariff Schemes

As required by the EU Directive 2009/28/EC, Ireland must have 16% of its electricity generated by renewable sources by 2020. The primary incentive in Ireland to help to achieve this target is the Renewable Energy Feed-in Tariff (REFIT) scheme. This scheme was designed to enable the addition of 4,000 MW of new renewable electricity to the Irish electricity grid, and more specifically to incentivise the development of new renewable generation from onshore wind, small hydro and biomass landfill gas technologies (Dept. of Communications, Energy and Natural Resources, 2012*a*). The first REFIT scheme was launched in Ireland in 2006, and March 2012 saw the launch of REFIT 2.

Depending on the site in question, there may also be the option of using the electricity on site rather than connecting to the grid. This is even more economically viable as the cost of buying electricity is considerably higher than the REFIT price paid, currently in Ireland the cost to buy electricity is €0.19 per kWh. The European average electricity price for small users is €0.18 per kWh but this varies considerably from as low as €0.08/kWh in Bulgaria to as high as €0.29/kWh in Denmark (Eurostat European Commission, 2011). It is clear therefore that the local energy costs and incentives will significantly influence the viability of a micro-hydro energy recovery project. This investigation examines the effect of different REFIT tariffs and energy prices on project viability.

Currently the tariff for hydropower schemes in Ireland with a generation capacity of less than 5MW is approximately €0.088 per kWh (€87.89 per MWh). These REFIT tariffs are adjusted annually by the increase, if any, in the consumer price index in Ireland, and are available for 15 years. The plant must be operational by the end of 2015 to be eligible so the current tariffs can therefore not extend beyond 2030 (Dept. of Communications, Energy and Natural Resources, 2012*b*).

REFIT schemes are available in many countries and the tariffs available in Ireland are small in comparison with some other countries. Figure 4.11 illustrates the range of some of the FITs available in Europe. Germany, the UK, Greece, Luxembourg, the Netherlands, Hungary, Slovakia and Slovenia all offer variable rates dependent on the power generation capacity of the site, the mid-point of these rates has been used for illustration purposes. The UK feed-in tariff scheme for example currently offers a variable rate depending on the power output of the site. The maximum rate of £0.21(€0.26) per kWh is offered for the smallest micro-scale hydropower sites (<15kW), while a rate of £0.04 (€0.06) per kWh is offered for larger schemes with power outputs ranging from 2MW to 5MW (Ofgem, 2012). The UK REFIT scheme offers further incentives, because as well as receiving the tariff per kWh of renewable energy generated, operators can also use the electricity on site, recovering the energy costs that would have previously been paid for.

However there are some drawbacks to the UK FIT banding. The fact that better prices are paid for the smaller generation sites means that WSPs may decide to reduce the capacity of the installed turbine in order to achieve a quicker investment payback. One potential measure to avoid this would be for the FITs to have a linear relationship be-

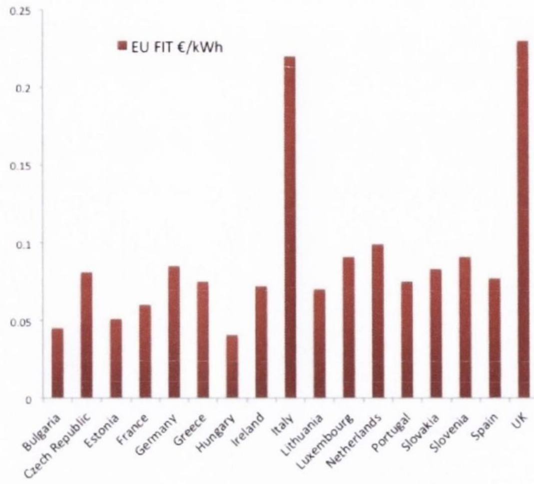


Fig. 4.11: European FIT Rates Comparison

tween the tariff and the turbine capacity, ensuring projects which are at the margins would not downsize to fit the smaller capacity FIT rate (Millington, 2014).

In order to receive a UK FIT rate, a micro-hydropower scheme must first be accredited by the Microgeneration Certification Scheme (MCS). UK REFIT tariffs for hydropower generated from water that has previously been pumped is not considered renewable by Ofgem and therefore these sites cannot qualify for a REFIT. This is currently a grey area in this new market of installing hydropower turbines within water mains in place of PRVs. In most water supply networks, though water may be primarily gravity fed, there is likely to have been some prior pumping somewhere in the network. For example small booster pumps to feed water to a high elevation site, or even a very small amount of seasonal pumping if reservoir levels are low. If there has been any pumping at all previously in the network, the site does not qualify for a REFIT.

4.5 Methods

Several methods were employed for this feasibility study. These include calculation of the power generation potential, an economic analysis including an estimation of the investment payback periods and an analysis of the impact of flow rate variation on turbine selection was undertaken. Each of the methods applied for this study are described in the following sections.

4.5.1 Power Generation Potential

Initial power output estimates were calculated using average flow rates and pressure conditions. For comparative purposes, the effect of applying an assumed constant turbine efficiency versus a variable turbine efficiency dependent on the variable flow conditions was also investigated.

The estimated power output from potential hydropower installations at existing infrastructure sites was calculated using Equation 4.5.1:

$$P = Q\rho gHe_0 \tag{4.5.1}$$

Where P is the power output, Q is the flow rate through the turbine, ρ is the fluid density, g is acceleration due to gravity, H is the head available at the turbine and e_o is the efficiency of the total system. For initial calculations, a conservative constant system efficiency (including turbine, generator and transmission losses) value of 65% was assumed. This is consistent with the efficiencies assumed in previous research and also with the efficiency range as reported by Paish (2002) for micro-hydropower systems. Paish (2002) reported that the best turbines would operate at efficiencies in the range of 80-90%, while micro-hydro systems tended to have efficiencies in the range of 60-80%.

Power outputs were then calculated for a number of potential turbine types and the most suitable turbine was selected. These turbine efficiencies varied with the flow rate for more accurate power generation estimates.

An economic analysis of these potential installations was then undertaken. Two evaluation methodologies were applied, payback period and Net Present Value. A sensitivity analysis was undertaken, considering sensitive variables such as the feed-in tariff available, the use of energy on site, variable energy prices and the selection of an appropriate discount rate to predict future cashflows.

As well as the generated hydropower resulting in an economic benefit, there is also an environmental benefit. In order to quantify this environmental benefit, equivalent CO_2 emissions were calculated using guidelines published by the UK Department for Environment, Food and Rural Affairs (DEFRA) and the Department of Energy and Climate

Change (DECC) (AEA, 2012). The conversion factor for equivalent CO_2 emissions for purchased grid electricity is 0.52037 kg of CO_2 emissions per kWh generated.

4.5.2 Economic Analysis

In general, water companies will rule out investments in hydropower projects with payback periods of greater than ten years. Gilkes, a hydropower consulting company and turbine manufacturer, have stated that they would consider implementing projects with payback periods of up to 12 years (Crompton, 2010). The cost of installing small scale hydropower turbines is largely site specific. Costs can vary significantly depending on the amount of civil works required and also on the proximity to the electric grid. However, some general estimates for the cost of installation of small scale hydropower can be found in published research and reports.

The capital cost of small-scale hydropower installed on water supply infrastructure has been said to be in the region of £3,000 (€3,700) to £6,000 (€7,400) per kW installed (Gaius-obaseki, 2010). For an energy efficiency case study undertaken by Gonçalves and Ramos (2012), a capital cost of €2,500 for a hydropower turbine was assumed with an additional €250/year (10% of the turbine cost) assumed as the annual maintenance cost. This cost, however, was for the turbine only and did not include other necessary civil and electrical works. An American supplier and installer of hydropower turbines for water pipes estimated a cost of \$3,500 (€2,800)- \$7,000 (€5,600) per kW installed and also an additional estimated annual maintenance cost of \$2,000 (€1,600) (Colombo and Kleiner, 2011).

Aggidis et al. (2010) developed empirical formulae for the total installation cost estimation of electro-mechanical equipment and different turbine types for small hydropower installations. The formulae as shown in Equations 4.5.2 and 4.5.3 were developed through statistical analysis of cost data obtained from various turbine manufacturers. These formulae estimate the turbine and electro-mechanical equipment costs (in GB£) based on the rated power output P (kW) of the installation and the hydraulic head H (m). A calibration of the application of these formulae to the cost estimation of hydropower projects in wastewater infrastructure has since been undertaken by Power et al. (2014). Using installation costs as published for five previous installed projects, the cost per kW of these projects versus the estimated cost per kW using Equations 4.5.2 and 4.5.3 were compared. These calibration estimates showed a slight variation in the region of

+940€per kW to -265€/kW for two projects.

$$C_{\mathrm{pr}} = 25,000 \quad x \quad \left(\frac{P}{H^{0.35}}\right)^{0.65} \quad \text{For heads from 2-30m} \tag{4.5.2}$$

$$C_{\mathrm{pr}} = 45,000 \quad x \quad \left(\frac{P}{H^{0.3}}\right)^{0.6} \quad \text{For heads from 30-200m} \tag{4.5.3}$$

Ogayar et al. (2009) produced empirical formula based on head and power output, for the estimation of the cost of the electro-mechanical equipment (turbine-alternator) for a number of common turbine types including Pelton, Kaplan and Francis turbines. A further investigation of the variability of turbine costs will also be investigated using these formulae.

Carravetta et al. (2014) published a cost-benefit analysis for hydropower production in water distribution networks. This analysis assumed the installation of a pump-as-turbine (PAT) and also assumed that the PAT would operate at a fixed speed, through the installation of a PAT control and isolation valve before flow enters the PAT. The net benefit (NB) was defined as:

$$NB = I_{\mathrm{a}} - C_{\mathrm{a}} \tag{4.5.4}$$

Where I_a is the annual revenue produced through energy generation, and C_a is the total costs including construction and maintenance costs, and was calculated by:

$$C_{\mathrm{a}} = C_0 \frac{r(1+r)^{\mathrm{n}}}{r(1+r)^{\mathrm{n}} - 1} + \alpha C_0 \tag{4.5.5}$$

Where C_0 is the total initial investment, r is the discount rate, α is the percentage of total cost spent on maintenance, assumed to be 10% in this case, and n is the project lifespan (years).

One way to reduce overall turbine costs is through the use of a PATs. Pumps used in reverse as turbines, as has been discussed previously, present a cost-effective alternative to a traditional hydropower turbine. This is primarily because centrifugal pumps are mass-produced worldwide in variety of sizes to operate over a wide range of flows and pressures (Laghari et al., 2013; Carravetta et al., 2013). The cost per kW of a PAT has been estimated to reach €350/kW, with overall payback periods of less than one year (Carravetta et al., 2013).

4.5.2.1 Investment Payback

In order to progress a hydropower project from the feasibility stage to implementation, a decision needs to be made by WSPs as to whether the project is a viable investment. There are a number of different evaluation techniques employed by WSPs at the feasibility stage of new projects and operations. Three methods commonly used to evaluate new projects are Net Present Value (NPV), Payback period (PB) and Internal Rate of Return (IRR) (Savić and Banyard, 2011; Beltran et al., 2014). For comparison, two of these evaluation techniques, PB and NPV, have been applied to estimate the economic viability of hydropower projects in WDNs.

An initial PB analysis was undertaken by the author with some preliminary results published in Corcoran et al. (2012*b*). For this thesis research two cost calculation models were tested, the first using a cost/kW model with cost estimates employed as reported by Gaius-obaseki (2010) and Colombo and Kleiner (2011). The second cost method employed used the empirical formulae as proposed by Aggidis et al. (2010) and later adopted and calibrated by Power et al. (2014).

The first project evaluation method employed was the payback period approach, whereby the payback period (PB) was calculated by the formula:

$$\mathrm{PB} = \frac{\text{Total Investment Cost}}{\text{Net Annual Revenue}} \tag{4.5.6}$$

A comparison of the payback periods for differing feed-in tariffs, energy prices and using the electricity on-site was also undertaken.

4.5.2.2 Sensitivity Analysis

A further project evaluation method using NPV calculations was then undertaken. The variables investigated included the impact of different discount rates and cost/kW models on project feasibility. Three cost per kW installation cost estimates were trialled, €3,000/kW, €5,000/kW and €7,000/kW. Results of this analysis were previously published by the author (Corcoran et al., 2013).

NPV calculations were selected as an evaluation method because they more accurately calculate and compare future investment payback periods. The key additional sensitive variable investigated with these calculations was the addition of an annual maintenance cost, which was taken to be €1,600 as reported by turbine manufacturers (Colombo and Kleiner, 2011). A comparison of the initial investment required per kW installed, between the minimum estimated €3,000 per kW and maximum estimated €7,000 per kW was also tested. NPV calculations were carried out for a sample 20kW, 50kW and 100kW site. A project was considered feasible when the NPV was positive before year ten. The NPV was calculated according to Equation 4.5.7 below:

$$NPV(i) = \sum_{t=1}^{N} \frac{R_t}{(1+i)^t} \tag{4.5.7}$$

Where R_t is the net cash flow at a given time t, and i is the discount rate.
The choice of a discount rate requires the exercise of judgement and the inclusion of sensitivity analysis to explore the effects of different rates on project feasibility (Savić and Banyard, 2011). For this sensitivity analysis, discount rates of 5%, 7.5% and 10% were tested and compared.

4.5.3 Flow and Turbine Efficiency Variation Analysis

Power output and annual power generation were calculated using average flow and pressure values as well as using more accurate varying flow rates and pressures. Finally, turbine selection and variable turbine efficiency and its effect on power output estimation were also investigated. Annual power generation was estimated using a constant assumed average annual flow rate and compared to the annual power generation according to variable flow rates dependent on the characteristic efficiency curves of a number of turbine types. A comparison between these methods was made to assess the risk of assuming average flow rates and pressures when undertaking project feasibility studies, as is the case in many previous investigations (see Chapter 2). An estimate of the cost of different turbines was also undertaken based on empirical formulae developed by Ogayar et al. (2009). These formulae, dependent upon the power output and the hydraulic head at each turbine, were applied to calculate the cost (in €/kW) of the electromechanical equipment. The cost for a Kaplan turbine was calculated using:

$$Cost = 31,196.P^{0.41662}.H^{-0.113901} \tag{4.5.8}$$

Similarly for a Francis turbine, the cost was calculated according to:

$$Cost = 25,698.P^{0.439865}.H^{-0.127243} \tag{4.5.9}$$

The cost (€) of a Fixed Propeller turbine was calculated according to:

$$Cost = 19,498.P^{0.41662}.H^{-0.113901} \tag{4.5.10}$$

Finally, the cost per kW (€) of a PAT was also considered, which has been previously reported to cost as little as €350/kW (Carravetta et al., 2013). The use of a PAT in place of other small scale hydropower turbines could have a large impact on initial investment costs, however a PAT does not operate as efficiently as other turbine types over a wide range of flow rates which could lead to a reduction in generated revenues. The relative benefits of each of these options was investigated as part of this analysis.

These cost formulae are for the cost of the electromechanical equipment only, and as such, do not include estimations for other costs incurred during installation, such as civils and construction work, design fees and grid connection costs. However, the turbine and electromechanical costs are one of the major primary investment costs required for any hydropower project. Using these cost estimates, the effect of the choice of turbine on projected investment payback periods was also evaluated.

4.6 Results

The results of this feasibility study are presented in the same sequence as each of the methods were previously presented. Beginning with the results of a preliminary estimation of the power generation potential at each site, followed by an investment payback analysis and a sensitivity analysis using NPV calculations. The feasibility study concludes with an analysis of the impact of short term flow variation on power generation potential and investment payback.

4.6.1 Power Generation Potential

The total 174 potential sites considered for energy recovery included 154 PRVs, 4 control valves, 3 BPTs and 13 reservoirs. A summary of the results of this investigation are presented in Table 4.3 and highlight that significant potential exists for energy recovery. The power output values in Table 4.3 were calculated using average pressure and flow values. In practice however, flow and pressure, and hence power output, vary considerably. The majority of the sites (70.11%) investigated yielded power outputs of less than 5 kW. This 70% majority of the sites yielded 14.4% of the total power generation. The majority of the power output was found to be generated from the sites greater than 100 kW (at 25%). The total power generation potential estimated in this study was 1.3 MW.

Table. 4.3: Overview of Estimated Power Outputs

Estimated Power Output	Number of Sites	% of Sites	Total Power Output (kW)	% of Power Output
<5 kW	122	70.11	188.51	14.41
5 - 10 kW	26	14.94	187.11	14.30
10 - 20 kW	14	8.05	191.02	14.60
20 - 50 kW	8	4.6	244.15	18.66
50 - 100 kW	2	1.15	170.20	13.01
>100 kW	2	1.15	327.25	25.01
Total	**174**	**100**	**1,308.24**	**100**

Table 4.4 presents power outputs, total annual generation and annual emissions savings for the 15 sites studied with the highest generation potential.

Table. 4.4: Power output and generation estimates for 15 sites with highest power output

Site Location	Power Output (kW)	Annual Generation (kWh/yr)	Annual CO_2 emissions savings (tonnes)
	Dublin City Valve Data		
Control Valves			
Thomas Court	212.26	1,859,414	967.6
Blackhorse Bridge	98.56	863,386	449.3
Stillorgan Road	33.6	294,189	153.1
Merrion Gates	33.4	292,568	152.2
PRVs North City Centre			
Poplar Row	17.42	152,604	79.4
PRVs South City Centre			
Rainsford Street	28.04	245,631	127.8
Rialto Bridge	46.97	411,429	214.1
Slievebloom Park	26.92	235,797	122.7
	Tanks and Reservoirs		
Dublin			
Saggart	115	1,007,323	524.2
Cookstown	71.64	627,566	326.6
Kildare			
Old Kilcullen	30.1	263,611	137.2
South Dublin			
Belgard	56.7	496,300	258.3
	Welsh Water PRVs		
Horeb Road	19	166,440	86.6
Berrymead Rd. Cardiff	24.3	212,868	110.8
Llanrhos Church	18.3	160,308	83.4

4.6.2 Investment Payback

Investment payback was also estimated for each site. Two cost models were employed for comparison. The first cost method assumed a minimum installation cost of €3,000 per kW installed and maximum installation cost of €7,000 per kW. These were based on the cost per kW estimates as found in the discussed literature. A minimum base total investment of €50,000 as was also assumed by Power et al. (2014) was used. Hydropower energy recovery project costs are highly case and site specific. These cost per kW models provide conservative total project cost estimates. For example, in practice some sites may require more civil works than others. Cost per kW estimates do not take into account the price differential between the different turbine types which would further impact upon total costs. They also do not take into account the extra expenditure required if the turbine is to be grid connected. This extra expenditure would include connection fees, cabling and regulating equipment. However, cost per kW models do provide a preliminary conservative estimate of total project costs, which can give a quick indication of whether or not a project is likely to be feasible (payback period <10 years). Following on from a cost per kW analysis, further more detailed site-specific project costs should be calculated.

Installation costs were then estimated using the empirical formulae developed by Aggidis et al. (2010). These formulae estimate total cost based on the flow, head and power output characteristics of the site. The annual revenue was initially calculated assuming a renewable energy feed-in tariff scheme was in place such as the REFIT scheme in Ireland. These payback periods were then compared with the potential payback period if no REFIT scheme was available. For this a rate of €0.04 per kWh was assumed based on the average price paid for electricity sold to the Irish grid for non-renewable resources. Finally, the investment payback period was calculated for situations where the electricity could be used on site, without needing to connect to the grid.

The possibility of future fluctuation in energy prices was also identified as a sensitive variable. Energy prices are rising and are predicted to increase further into the future. This will increase the attractiveness of future investment in renewable energy resources by increasing the net benefit of an installation. Howley et al. (2012) details how Ireland's energy prices have changed between 2005 and 2012. The water industry, as an industrial energy user, has faced a 47% increase in electricity prices between 2005 and early 2012. The fuel mix in Ireland for electricity generation is one key influential factor on this price variation. Ireland has the highest overall dependency on fossil fuels in the OECD EU-15 countries. Ireland, Luxembourg and the Netherlands each have 64% and Italy has 58% of electricity generated by gas and oil. Oil prices for Irish industrial users increased by 55% between 2005 and early 2012 (Howley et al., 2012). This represented the largest increase in real terms of the EU-15 countries, with an average 22% increase in Europe. Electricity prices in Ireland over the last three years are detailed in Table 4.5. Predictions for future fuel mix prices in Ireland have been analysed by the SEAI with results published in Howley et al. (2012). The worst case scenario predicted a 22.22% increase in fuel costs from 2011 to 2020 (Power et al., 2014). This predicted price increase was applied to the 2011 data and projected electricity prices of €0.157/kWh by 2020 (Table 4.5).

Table. 4.5: Electricity prices for Irish Industrial Users (Eurostat, 2014)

Year	2011	2012	2013	2020*
Price (€/kW)	0.129	0.14	0.137	0.157

* 2020 cost adapted from (Howley et al., 2012; Power et al., 2014)

A summary of the results of these payback period calculations are presented in Tables 4.6 and 4.7. Table 4.6 presents the payback periods using the Aggidis et al. (2010) cost model, while Table 4.7 presents the payback periods using the €7,000/kW cost estimations. Further details on the top 20 highest power generation sites in Ireland are presented in Table 4.8.

Table. 4.6: Payback Period with total costs calculated as per the Aggidis et al. (2010) empirical formulae*

	No. of sites feasible (payback period <10 years)	% of sites feasible
With Irish FIT rate	20	11.5
With UK FIT rate	159	91.4
Using electricity on-site (Ireland)	54	31
Using electricity on-site (UK)	35	20.1
Using electricity on-site (Ireland - 2020)	78	44.8
Selling to grid without REFIT (Ireland)	2	1.1

* Note: Each FIT rate was applied to all sites for comparison

From Table 4.6, it is clear that the UK FIT scheme is an effective incentivising scheme, with over 90% of the projects evaluated as feasible based on having payback periods of less than 10 years. However, many of these sites, according to the current UK FIT rules, may not qualify as renewable energy resources. Using the electricity on-site, according to 2013 industrial charges for electricity proved to be a favourable alternative, with over 30% of sites deemed feasible according to the potential electricity savings in Ireland, and 20% in the UK. Without a REFIT tariff in place, only two sites were found to be feasible.

A similar analysis was undertaken according to a €7,000/kW total installation cost estimate. This was the highest estimate in the range of estimated costs found in literature and as such would represent a worst-case-scenario approximation. No sites were deemed feasible if electricity was sold to the grid with no FIT secured. In this case, the UK FIT scheme proved the most incentivising with 61.5% of projects deemed feasible. It was found that should electricity prices rise as forecast by 2020, a further 24 sites would be feasible according to the Aggidis et al. (2010) cost approximation formulae.

Table. 4.7: Cost per kW Payback Period - €7,000/kW

	No. of sites feasible* (payback period <10 years)	% of sites feasible
With Irish FIT rate	42	24.1
With UK FIT rate	107	61.5
Using electricity on-site (Ireland)	58	33.3
Using electricity on-site (UK)	54	31
Using electricity on-site (Ireland - 2020)	59	33.9
Selling to grid without REFIT (Ireland)	0	0.0

* a minimum baseline installation cost of €50,000 was assumed

A further comparison was made of investment payback periods across Europe. Payback periods were calculated using the Irish, UK, Bulgarian and the average European feed-in tariff rates for a 20kW hydropower scheme at an installation cost of €5,000/kW. These were chosen because the Bulgarian tariff is the lowest European FIT on offer, while the UK tariff is the highest. These payback periods were compared to the estimated payback periods if the electricity were to be used on site, using 2013 industrial electricity price data sourced from Eurostat (2014). The average European industrial electricity tariff was estimated to be €0.113/kWh, while the average European FIT tariff was estimated at €0.091/kWh. Using the electricity on-site removes the need to buy electricity from the grid at these industrial tariffs. Payback periods varied considerably between the different tariffs. These payback periods (in years) are illustrated in Figure 4.12.

Table. 4.8: Costs and investment payback estimates for the top 20 Irish energy recovery sites using (Aggidis et al., 2010) formulae

Name	Head (m)	Flow (l/s)	Power (kW)	Annual CO_2 emissions savings (tonnes)	Cost (€)	Annual Revenue * (€)	PB* (yrs)	Annual savings ** (€)	PB** (yrs)	Annual Revenue *** (€)	PB *** (yrs)
Thomas Court	73.5	452.9	212	975,449	651,293	155,837	4	353,289	3	74,377	9
Saggart	10.2	1768	115	528,442	401,319	84,424	5	191,391	3	40,293	10
Blackhorse Bridge	56	276	99	452,910	431,637	72,357	6	164,035	4	34,534	12
Cookstown	15	749	72	329,221	270,274	52,596	5	119,238	3	25,103	11
Rialto Bridge	62	118.8	47	215,836	271,662	34,482	8	78,171	5	16,457	17
Stillorgan Rd	41.7	126.3	34	154,331	238,580	24,656	10	55,896	6	11,768	20
Merrion Gates	19.5	268.6	33	153,481	155,043	24,520	6	55,588	4	11,703	13
Old Kilcullen	20.4	231.5	30	138,290	143,428	22,093	6	50,086	4	10,544	14
Rainsford St.	63	69.8	28	128,858	198,779	20,586	10	46,670	6	9,825	20
Slievebloom Park	39.6	106.6	27	123,699	210,873	19,762	11	44,801	7	9,432	22
Giltspur reservoir	51	69.6	23	103,994	181,584	16,614	11	37,665	7	7,929	23
Poplar Row	76.1	35.9	17	80,056	144,402	12,790	11	28,995	7	6,104	24
Bayside Boulevard	27.8	97.5	17	79,426	93,208	12,689	7	28,767	4	6,056	15
Brunswick St.	71.6	37.3	17	78,259	144,020	12,503	12	28,344	7	5,967	24
Ballygoran	20.4	104.2	14	62,231	85,353	9,942	9	22,539	5	4,745	18
Castlewarden	20.4	104.2	14	62,231	85,353	9,942	9	22,539	5	4,745	18
Stillorgan	6	306	12	53,801	102,561	8,595	12	19,486	7	4,102	25
Rathlin Rd.	26	66.1	11	50,360	70,380	8,046	9	18,240	5	3,840	18
Gallanstawn WSA	27	62	11	48,571	68,301	7,760	9	17,591	5	3,703	18
Allen	30.6	51	10	45,636	121,447	7,291	17	16,528	10	3,480	35

* Revenue with Irish REFIT

** Electricity savings by using electricity generated on site

*** Annual revenue selling to Irish grid with no REFIT

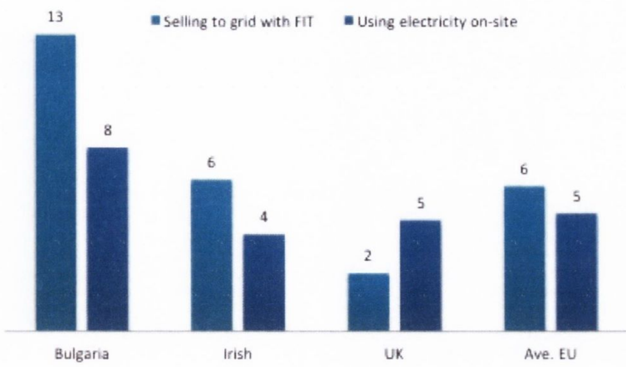


Fig. 4.12: Renewable Energy FIT Comparison

Again, it is clear from this comparison that the UK FIT scheme is the most incentivising scheme with the total investment cost paid back through revenue generation by year two. In the UK, if it can be secured, it is more economic to obtain a FIT. However, in Ireland and Bulgaria it is more economic to use the electricity on site where possible.

4.6.3 Sensitivity Analysis

NPV calculations were used to predict future cash flows. These assumed a constant FIT rate of €0.087 per kWh generated as per the Irish REFIT scheme. An annual maintenance cost of €1,200 was added each year. Three discount rates were compared, 5%, 7.5% and 10%. Installation costs of €3,000, €5,000 and €7,000 per kW installed were compared for both a 20kW installation and a 100kW installation. The results of these calculations are shown in Figures 4.13 and Figure 4.14. Figure 4.13 shows the results of the NPV calculations at a discount rate of 7.5%. The NPV of all of the cases investigated was positive by year ten. The highest valued investment site, the 100kW installation at an installation cost of €7,000 per kW delivered a return on investment after year 9, while the 20kW installation for the same installation cost delivered a return on investment after year 10.

A further NPV analysis was undertaken to investigate a variation in the discount rate. Three discount rates were tested, 5%, 7.5% and 10% for the highest total installation cost/kW model of €7,000 per kW. Investment payback was achieved by year 9 for all cases.

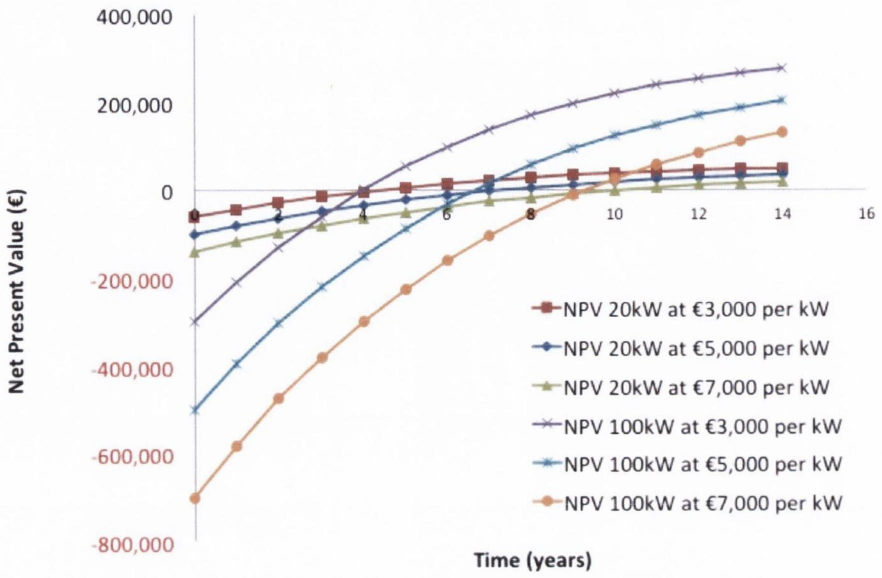


Fig. 4.13: NPV calculations for discount rate of 7.5%

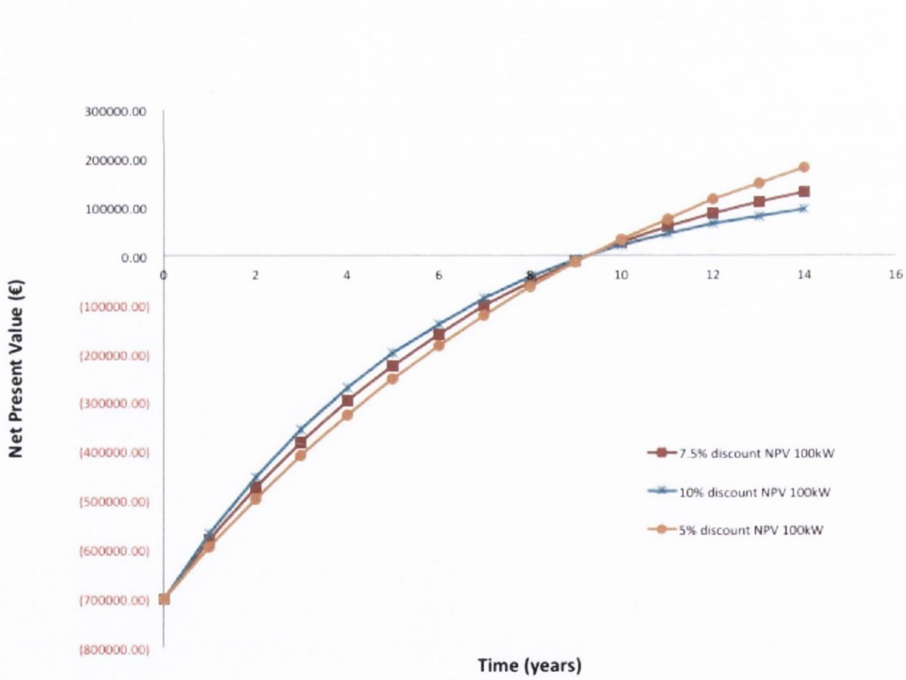
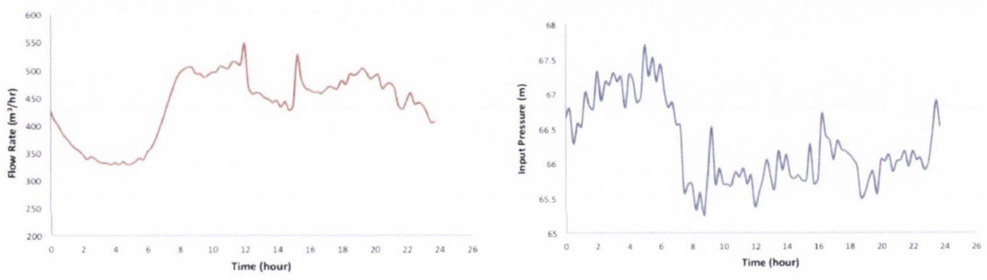


Fig. 4.14: NPV calculations for variable discount rates for a €7,000 per kW installation

4.6.4 Flow and Turbine Efficiency Variation Analysis

All of the previous cost estimates were calculated assuming a constant average flow rate and a constant average pressure drop across the installed turbine. In practice however, flow rates and pressures across the turbine will vary. The resolution of the DCC PRV and control valve data obtained enabled further analysis of the effects of flow rate and pressure variations. A sample diurnal flow rate variation for the Dublin City PRV at Slievebloom Terrace on the 1st of December 2010 is shown in Figure 4.15a. The increase in demand can be seen by the increased flow rates at certain times of the day. These diurnal flow patterns reflect domestic water use and are largely predictable, with a peak in the early morning (high shower use etc.), and again in early evening when people return home, for cooking, cleaning etc. Diurnal variation will also differ depending on the demands in that local area, for example the industrial and commercial mix, water intensive industries, the number of households and sports grounds etc.



(a) (b)

Fig. 4.15: Diurnal (a) Flow variation and (b) Pressure Variation at Slievebloom PRV

Pressure at the inlet to these PRVs and control valves also varies throughout the day. However, this pressure variability is generally not as pronounced as the flow variability. Figure 4.15b illustrates the diurnal inlet pressure at the Slievebloom PRV. The sharp increase in flow rate at 8am resulted in a corresponding drop in the inlet pressure. This pressure varies from approximately 65.5m to 67.5m.

As discussed in Section 4.2.3, turbines must be selected for each site based on the site conditions, including both the flow rate and pressure through the turbine. Turbine selection charts exist for many turbine types. A turbine selection chart for the most common turbine types, including reaction type turbines (the Francis and Kaplan) and impulse type turbines (the Pelton, Turgo and cross-flow) are shown in Figure 4.16. The Pelton turbine is most suited to sites with high pressures and low flow rates. The Kaplan turbine is most suited to mid range head sites over a wide range of flow rates, while the Francis turbine performs best at higher head sites and flow rates greater than $10m^3/s$. Further details on turbine selection and operating efficiencies can be found in Section 4.2.3.

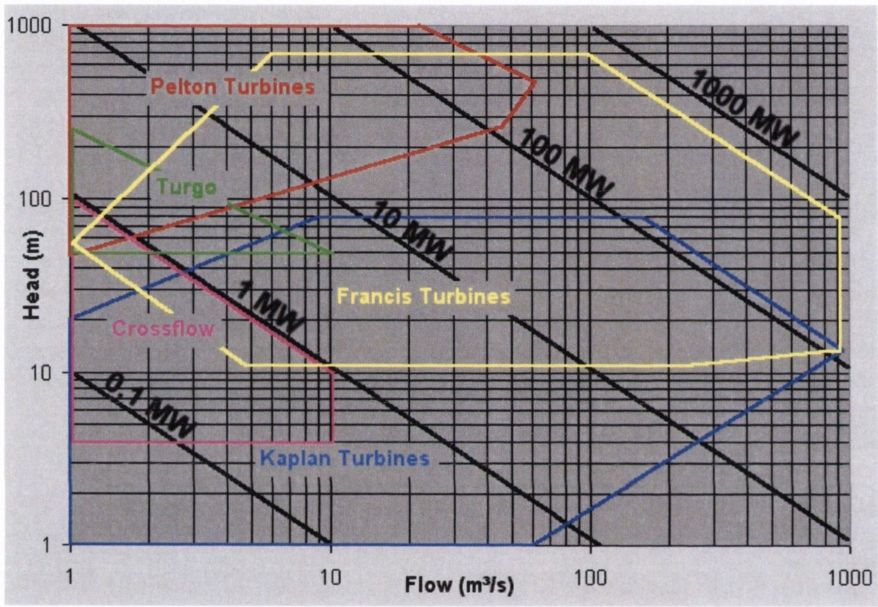


Fig. 4.16: Typical hydropower turbine selection chart

Reaction turbines are more suitable for PRV sites as they sit in the full flow of the water and can operate in pressurised pipes where the output pressure must be maintained. Impulse turbines are more suited for end of line locations or at inflows to tanks or reservoirs, as the water dissipates to atmospheric pressure at the turbine.

Figure 4.17 shows the typical performance of different hydropower turbines over varying flow rates. With the flow variability present in water supply mains, turbine efficiencies will vary considerably. If the flow rate moves below the performance band for the installed turbine, turbine efficiency and hence power output would be reduced. As is evident in Figure 4.17, the Kaplan and Pelton turbines operate at between 80% and 90% efficiency over a wide range of flow conditions, whereas the propeller turbine operates at a high efficiency close to the average flow rate, with the efficiency then dropping significantly as flow drops below average.

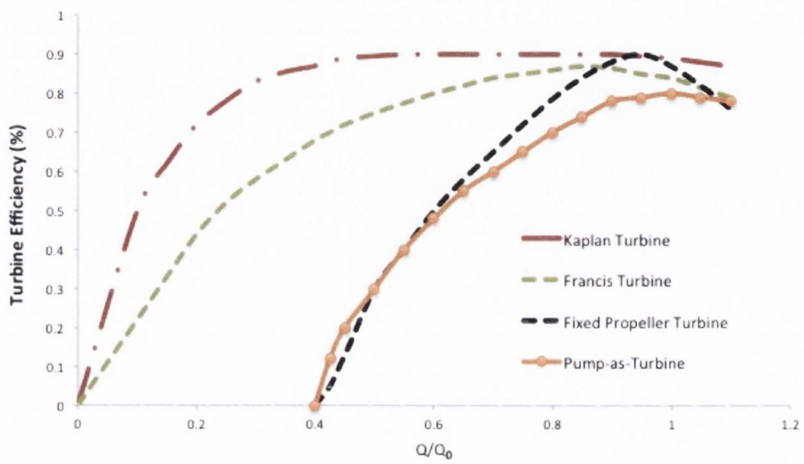


Fig. 4.17: Typical turbine efficiencies (adapted from Gatte and Kadhim (2012) and Orke (2010))

Flow variation and its impact on turbine efficiency and hence the power generation potential were assessed in more detail focusing on three PRVs in Dublin city. PRVs were selected for this analysis rather than control valves. The flow across PRVs on distribution mains would be expected to be more variable than at control valves on trunk mains, due to the more direct impacts of the local demand served. Three sites were selected for analysis. A 3kW site at Ringsend Park was selected as a case to represent the smaller generation sites. Approximately 70% of the 174 sites studied were found to have power generation capacities of less than 5kW. The Poplar Row PRV was also selected as an example of a mid-range power generation site, with estimated power output of 17kW. Finally, the Bellvue PRV was selected as this PRV serves the local Guinness brewery and as such would see increased variability highly dependent on the requirements of the brewery. The annual estimated power generated, as discussed in the Results section 4.6 was then compared to the estimated power output using variable turbine efficiencies across varying flow rates and pressure drops. Again, the data for these PRVs consisted of a full year from July 2010 to July 2011 of inlet pressure, outlet pressure and flow rate measurements taken every 15 minutes.

Turbine efficiency curves over variable design flow conditions (Figure 4.17) were applied for a more accurate estimation of power output and these were incorporated into total system efficiencies. Other system losses (such as transmission and generator losses) were represented by applying an additional constant efficiency of 80% to these power generation estimates. Table 4.9 details the annual power generated at each PRV using a Kaplan turbine, Francis turbine, a fixed propeller turbine and a PAT. In all cases, the Kaplan turbine was found to operate most efficiently over the variable flow rates present, followed by the Francis turbine. Assuming a constant conservative system efficiency of 65% led to an underestimation of the power generation capacity at the Ringsend Park PRV (+8.2%) and the Poplar Row PRV (+6.5%).

However in terms of investment payback period, despite the reduction in power generation capacity, the PAT presented the most cost-effective option of the four turbine types evaluated. Investment payback was achieved for all three valves within one year. The application of a PAT at the Ringsend Park PRV and the Poplar Row PRVs resulted in reduced power generation of between 8.6% and 10.2%. However, the cost of a PAT in comparison to the other turbine costs was significantly cheaper. For the Ringsend PRV site, the PAT cost was estimated to cost just 5% of the total turbine cost for the next cheapest option of a fixed propeller turbine. For the Bellvue PRV, a PAT was estimated to cost 14.4% of the next cheapest option.

For the Bellvue PRV, flow variation was found to have a more significant impact on the actual power generation and the overall investment payback periods. The flow rates at this PRV varied more, which is likely due to the industrial water usage by the local brewery. Calculating the annual power generation using a constant turbine efficiency overestimated the generation potential by between 36.8% and 56.1%, depending on the turbine selected. For this valve, again the Kaplan turbine operated the most efficiently of the turbines tested. However the actual power rating of the site, using variable turbine efficiency was found to be 5.5kW as opposed to the estimate 8.3kW estimated using a constant assumed turbine efficiency of 65%.

Table. 4.9: Turbine selection and efficiency variation results

Site	Turbine Type	Annual Generation (kWh)	% Difference*	Power Output (kW)	Cost** (€)	Payback Period (years)	Annual Revenue*** (€)
Ringsend Park PRV	Constant Turbine Efficiency	32,167		3.7			2,827.21
	Kaplan Turbine	34,810	+8.2	4.0	40,410.92	13.2	3,059.51
	Francis Turbine	32,213	+0.1	3.7	32,014.03	11.3	2,831.25
	Fixed Propeller	30,602	-4.9	3.5	23,937.38	8.9	2,689.63
	PAT	29,410	-8.6	3.4	1,175.07	0.5	2,584.94
Poplar Row PRV	Constant Turbine Efficiency	152,543		17.4			13,407.35
	Kaplan Turbine	162,516	+6.5	18.6	64,313.52	4.5	14,283.85
	Francis Turbine	150,471	-1.4	17.2	51,734.67	3.9	13,225.22
	Fixed Propeller	142,730	-6.4	16.3	38,080.64	3	12,039.51
	PAT	136,981	-10.2	15.6	5,472.98	0.5	12,039.51
Bellvue PRV	Constant Turbine Efficiency	72,457		8.3			6,368.37
	Kaplan Turbine	47,963	-33.8	5.5	40,872.28	9.7	4,215.52
	Francis Turbine	41,587	-42.6	4.7	31,250.31	8.5	3,655.16
	Fixed Propeller	31,256	-56.9	3.6	21,371.71	7.8	2,747.12
	PAT	31,797	-56.1	3.6	1,270.44	0.5	2,794.73

* % differences were calculated between the assumed constant turbine efficiency and turbine efficiency variations
** Turbine costs were calculated using Ogayar et al. (2009) and Carravetta et al. (2012)
*** Annual revenue (€) calculated assuming Irish REFIT tariff in place

Table 4.10 presents a comparison of the calculated investment payback periods between using variable turbine efficiency dependent on variable flow rates versus a constant assumed turbine efficiency. For both the Ringsend and Poplar Row PRVs, the estimated payback periods were similar. Payback was estimated to be achieved one year sooner when assuming a constant turbine efficiency for the Francis turbine option at Ringsend. However, the variations for the Bellvue PRV were more pronounced. Assuming a constant turbine efficiency during the feasibility study, would have underestimated the actual investment payback period. Investment payback would have been expected three years earlier for the installation of a fixed propeller turbine, and two years earlier for the installation of both a Kaplan turbine and a Francis turbine.

Table. 4.10: Payback period comparison using constant turbine efficiency of 65% versus variable efficiency dependent on flow rate

Site	Turbine Type	Payback Period (Variable Efficiency)	Payback Period (Constant Efficiency)
Ringsend Park PRV	Kaplan Turbine	13.2	13.8
	Francis Turbine	11.3	11.3
	Fixed Propeller Turbine	8.9	8.6
	PAT	0.5	0.5
Poplar Row PRV	Kaplan Turbine	4.5	4.7
	Francis Turbine	3.9	3.9
	Fixed Propeller Turbine	3.0	2.9
	PAT	0.5	0.5
Bellvue PRV	Kaplan Turbine	9.7	7.6
	Francis Turbine	8.5	6.3
	Fixed Propeller Turbine	7.8	4.8
	PAT	0.5	0.5

* Costs calculated according to Ogayar et al. (2009) and Carravetta et al. (2013)
** Annual revenue (€) calculated assuming Irish REFIT tariff in place

4.7 Discussion

Of the top 20 power generation sites from both the Irish and Welsh data, 10 of the sites were PRVs, 4 were control valves and 6 were at reservoirs or tanks. Though there are many examples in practice of turbine installations at reservoirs and treatment works, there are very few examples of turbines installed to replace PRVs. It is clear that there is a large untapped energy resource that requires further research and development to further market penetration.

The majority of the high power generation sites were located in the more densely populated regions, large urban areas, and commuter towns. Smaller generation sites in the region of 1-5kW were found in smaller towns and less populated areas. Though these are small generation sites, with estimated longer investment payback periods, they should not be discounted for development. With many turbines having a design life of more than 25 years, though investment payback may not be achieve within ten years, it could be achieved within 15 years, well before the turbine would require replacement. It should also be noted that though the turbine may require replacement after 25 years, much of the installation work would not need replacement, such as the civil engineering work required for site access etc. Furthermore, the use of a PAT in place of other turbine types would significantly reduce the total installation costs at these smaller generation sites, improving the economic success of these installations. Finally, small water supply schemes located in hilly or mountainous regions, with large elevation variation would see larger pressure drops which would increase the power generation potential, despite the lower flow rates due to reduced demand.

Strongly evident in the economic evaluations of potential projects was the impact of the presence of renewable energy FITs. These significantly increase the possibility for cost-effective implementation of turbines on small scale power generation sites. The UK FIT, with its tariff banding scale to further improve the economic success of the smallest power generation capacity sites, was found to be the most effective tariff structure. However, discrepancies exist in the rules for awarding FITs to hydropower sites which discount many WSN applications, where there may have been pumping elsewhere in the network. The key objective of the REFIT schemes, is to incentivise generation of electricity from renewable resources to help meet the EU 2020 target of 16% generation of electricity by renewables. The author would argue that these hydropower energy efficiency measures should be considered for FIT rate qualification, as they also help to reduce our dependency on non-renewable energy resources and help to reduce carbon emissions. The objective of both is ultimately the same, whether it is a run-of-river scheme, or a turbine at a reservoir, the water flows by gravity and without a turbine installed, precious energy is being wasted.

Possible methods to suit these schemes to the REFIT scheme would be to calculate the volume of pumping in the network as a proportion of the entire water supplied. This ratio of gravity fed versus pumped water could be used to proportionally reduce the REFIT tariff. Alternatively, as is the case on some sites, if pumping is only seasonal (e.g. only pumping day a week in the Winter), to pay the REFIT rate only on the days of the year when there is no pumping. Another option, would be the development of a new Energy Efficiency FIT, to award improved energy prices to novel, energy efficient schemes which provide electricity to the grid.

4.7.1 Flow Variation

Flow variation directly impacts the amount of power generated. Flow varies on a diurnal, seasonal and location specific basis, depending on the demands in the area. The Rialto Bridge PRV for example is on a pipeline serving St. James Hospital. The most intensive water usage at the hospital would relate to cleaning processes, such as the Central Sterilization Supply Department (CSSD) and the endoscope washing unit. Aside from the patients themselves, the other large water user would be the canteen. Peak water demand in the hospital is between 7am to 2pm, with demand lessening after 2pm as shown in Figure 4.18.

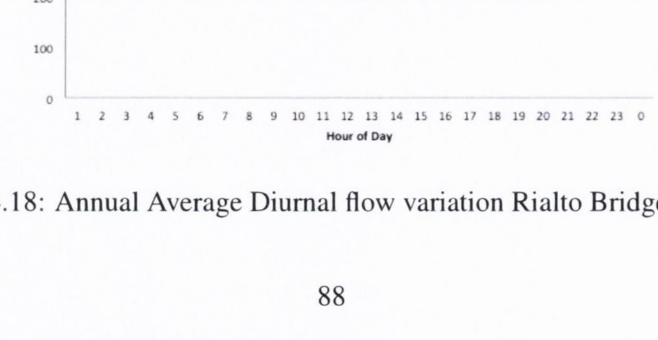


Fig. 4.18: Annual Average Diurnal flow variation Rialto Bridge PRV

Seasonal variation in the data was also investigated. The PRVs and control valves data analysed were recorded over the period July 2010 to July 2011. The most obvious seasonal variations apparent from this data were the disruptions to service that occurred over the period of November 2010 to January 2011. This period, commonly referred to by the media as The Big Freeze, saw the temperature drop severely across the country, with a record low of -18°C recorded. These cold temperatures put Dublins aging pipe network under stress. Water demand also reached a peak of 634 million litres per day, when average daily demand was 540 million litres per day. This large peak was attributed to burst pipes, and also to people leaving their taps running to prevent their pipes freezing. Many of the valves saw closures during this period, with very low or no flow for periods during maintenance etc. Approximately 370 repair works were carried out in Dublin city centre over this period (DCC Personal communication, 2012). These periods of closures would have resulted in lost running time of any turbines installed, impacting on the power generation capacity and hence the predicted investment payback.

Water mains also undergo year-round relief and maintenance works which could interrupt the flow. When improvements are made to the water supply mains, some valves may be bypassed and rendered useless as potential energy recovery sites. While a Government schedule for works exists for 2012-2014, these programmes can and do change, especially given the current economic climate in Ireland.

The risks of assuming a constant turbine efficiency for feasibility studies was highlighted in the results of the turbine selection and efficiency variation analysis. In the case of one PRV studied, assuming a constant turbine efficiency lead to power generation overestimations of between 36% and 56%. Following on from this, it would have led to the actual investment payback being achieved between 2 and 3 years later than predicted. For projects evaluated as feasible based on achieving investment payback by year ten, this could lead to projects being implemented which may not achieve investment payback until year 12 or 13.

The relative cost of a PAT in comparison to other micro-hydropower turbine costs was found to be significant. Though PATs perform less efficiently at low flow rates, their relatively low economic cost makes them an attractive option for installation. For the 17.4kW PRV at Poplar Row, an estimated seven PATs could be purchased for the same cost as a fixed propeller turbine. One option to improve the efficiency of PATs over variable flow rates could be to install two or more PATs in parallel. These PATs could be sized to suit the variations in flow present. Flow rates could then be split to operate across these PATs to maximise the net power generation of the multiple PAT system. A further cost analysis of the total turbine and electrical costs of PATs should be investigated.

When estimating the economic feasibility of a hydropower project, short-term and long-term demand uncertainties will need to be taken into account in order to more accurately forecast the duration of the investment payback period. For some valves analysed, their location will affect the level of demand uncertainty. Valves located on the trunk or arterial mains tend not to have as many closures or interruptions to flow as the valves located on the smaller distribution mains. Furthermore, there will also generally be a consistent level of flow on the trunk mains, whereas the minor distribution mains would see highly reduced flow rates at particular times of the day based on demand, i.e. at night time.

4.8 Summary

The focus of this chapter was to present the results of an analysis of the potential for small scale hydropower energy recovery in water supply networks in Ireland and Wales. The results of the initial analysis show that there is potential for energy recovery in the water supply networks of Ireland and the UK with some sites having an estimated power output of over 100kW. The larger capacity sites were primarily found in large cities, which would see increased flow rates and pressures due to the increase in local water demand. However, small scale generation potential could also be found on smaller rural networks. For these sites, it is recommended that where possible, electricity should be used on site in order to maximise the economic benefits and reduce the overheads involved in grid connection. The types of infrastructure locations for hydropower energy recovery that were investigated would be typical of water supply networks worldwide, and, so, the conclusions from this study would find application in other settings.

From the initial analysis of data, the issue of flow rate variation through installed turbines due to fluctuations in water demand was identified. This directly impacts upon the operating efficiency of any installed turbine. In order to investigate the effects of variable flow rates on turbine operating efficiencies, power generation potential was also calculated using variable turbine efficiency values dependent on the variable flow rates present. The aim of this analysis was to investigate the effect of flow variation, turbine efficiency and variations in energy prices and feed-in tariffs on project feasibility.

Turbine selection and turbine efficiency were shown to have a significant effect on the amount of power generated. Using actual flow, pressure and turbine efficiency variations compared with assuming an average value for all three, showed overestimation of power generation capability in all cases, with some cases showing highly overestimated generation in the region of 36-56%. It is therefore recommended that variable turbine efficiencies be used for future project feasibility calculations. It is also recommended that both short-term and long-term flow rate uncertainties be taken into account during the economic feasibility study of a hydropower project. Particularly if investment payback was estimated to be achieved after 8-10 years, as flow rates could vary considerably over a longer period.

From the investment payback analysis it was found that the presence of renewable energy incentives such as FIT schemes had a major effect on financial viability of small scale hydropower projects. It was also concluded that, for the majority of the cases investigated, the most economic option, where possible, is to use the electricity generated on site, however this was also found to vary across different countries depending on electricity and REFIT prices. It is recommended that governments and legislative bodies consider the introduction or improvement of renewable energy feed-in tariff schemes to incentivise small scale hydropower.

More accurate investment payback periods were calculated using NPV calculations. Other sensitive variables that could affect investment payback are long term changes in water demand, population growth, REFIT tariff changes and the effects of climate change. It is recommended that long term water demand, population and climate data be studied and a further sensitivity analysis, using NPV calculations, be undertaken with these taken into account. This further study would strengthen the investment case through the thorough analysis of longer term, more detailed data.

CHAPTER 5

Risk Analysis

5.1 Introduction

The focus of this chapter is an exploration of the impact of long term changes in flow rates on installed hydropower capacity. While future flow rates are uncertain, large fluctuations in flow rates could have a major impact on turbine operation and efficiency. Water demand forecasting is an area of consistent research and development. Sensor technology, data processing techniques and computing capabilities are all areas that have seen much research and development in recent years. As well as the increased quality and quantity of data available today, much research has also gone into the development and application of models to use this data so as to improve decision making.

Though the future is unknown and unknowable, predictive methods can be applied to estimate future values, based on historic data. In the past, statistical techniques were commonly applied. More recent research has seen the development and application of learning algorithms coming to the fore as predictive tools.

Long term changes in water flow rates and pressures at hydropower locations could render an installed turbine unsuitable, requiring its removal, replacement or by-pass. To reduce this risk, it is therefore essential that long term changes in flow conditions at potential turbine locations be considered at the initial feasibility and design stages. Using historical data over a ten year period, this chapter examines the effects of changes in climatic and socio-economic factors, amongst other influential variables, on the water flow rates at potential turbine locations in Ireland and Wales. Two predictive methods were tested, multiple linear regression analysis (MLR) followed by artificial neural networks (ANNs). This research evaluates the performance of these techniques to predict water flow rates at locations for hydropower energy recovery in WSNs. It then applies these models to predict the impacts of changes in these influential factors on future flow rates and hence the estimated hydropower generation at these points.

5.2 Background

Hydropower generation is directly linked with water supply and availability. Recent years have seen an increase in the installed capacity of hydropower turbines on water supply infrastructure. Water availability and water demand will affect hydropower capacity at these locations. Water availability will impact the generation capacity of large-scale hydropower plants at reservoirs, while changes in water demand will impact smaller scale hydropower capacity at turbines on water distribution mains. As was discussed in Chapter 2, one barrier that has been reported in previous research that could impact the viability of MHP installations in WSNs was reported to be the effects of flow variation (Colombo and Kleiner, 2011; Corcoran et al., 2013; McNabola et al., 2013; Giugni et al., 2014). As described in Chapter 4, flow rates in WSNs vary considerably both diurnally and seasonally. Water demand can also change dramatically over longer periods, with changes in population, climate and economies. Long term changes in water demand would impact flow conditions at turbines. These changes could see large increases or decreases in flow rates, rendering turbine operating efficiencies decreased or turbines unsuitable.

Having calculated typical investment payback periods for feasible hydropower generation sites in WSNs, the smaller scale hydropower sites (e.g. <5kW) often achieve investment payback after longer periods of 10+ years. Over these 10 year periods, it is important to consider how flow conditions through installed turbines may change. Changes in flow rate will affect turbine operation and efficiency.

Water demand forecasting is a central task for water supply operations and planning. Forecasting water demand is also a consistently active area of research. New methods and approaches are regularly developed and reported on. Early approaches focused on

Forecast Type	Forecast Horizon	Applications
Very-Short-Term	Hours, days, weeks, up to two weeks	Optimizing, managing systems, operations, pumping
Short-Term	Years, 1-2 years	Budgeting, program tracking and evaluation, revenue forecasting
Medium-Term	Years to a decade, 7-10 years	Sizing, staging treatment and distribution system improvements, investments, setting water rates
Long-Term	Decades, 10-50 years	Sizing system capacity, raw water supply

Fig. 5.1: Types of Water-Demand Forecasts and Applications (Billings and Jones, 2008)

the use of traditional statistical models. More recently, intelligent approaches such as the use of ANNs and genetic algorithms have also been considered and applied. Water demand forecasting can be either short-term or long-term. Short-term demand forecasting is necessary for day-to-day management and operation of a water supply network. However, longer term forecasting is necessary for strategic infrastructure investment planning and asset management. As the forecast horizon is extended, uncertainty and the risk of errors increases. This is a challenge for WSPs, because over-design or large scale expansion would lead to increased capital costs. However, if long range demand changes are not adequately addressed, there is a risk of water shortages which in turn leads to extra costs (Billings and Jones, 2008). Figure 5.1 details the types forecasting related to different water supply operations and planning decisions.

The majority of the literature and research in this field has focused on the forecasting of water demand for the water supply district as a whole to ensure reservoirs are adequately supplied. This chapter however focuses on long term flow rate variation at specific valves downstream of reservoirs and the application and implications of these predictions for hydropower design.

The most frequently reported influential factors on water demand can generally be classified as either climatic or socio-economic. As described in Chapter 2, climate factors such as temperature, rainfall and relative humidity have been shown to affect water usage. McDonald et al. (2011) reported the key determinant of water demand in a region to be population. House-Peters and Chang (2011) reported the most common explanatory variables in previous research in the field to be: temperature, precipitation, wind speed, evapotranspiration, water price, income and household size, amongst others. The consensus in the field is that short-medium term water demand forecasting is usually dependent on weather variables, whereas longer term water demand variance is more likely be determined by socio-economic factors (Donkor et al., 2014). Qi and Chang (2011) reported that there has been a renewed interest in accurate water demand forecasting due to recent large economic fluctuations and their effect on large urban areas. It can be concluded that long-term water demand forecasting should simultaneously consider variables related to climate change, economic development, and population dynamics (Qi and Chang, 2011). Research developments in this field were discussed in more detail in Chapter 2.

Another factor selected for inclusion in this analysis that may explain water flow variation at valves was the rate of leakage from the water supply network. Leakage rates were identified as a potential predictor of flow rates within water networks because as water mains are upgraded or repaired, leakage and hence water flow rates should also reduce. There will however always be some level of leakage from water supply mains, known as the economic level of leakage. Repairs and upgrades to reduce leakage below this level are not considered economically feasible as further investment in repairs does not significantly improve the amount of water saved.

This chapter presents the results of an investigation of the long term flow variation at potential hydropower locations in the Irish and Welsh water supply networks. An analysis of water flow rate variation based on ten years of high resolution historic water flow data is presented with key influential factors on this water flow rate variation investigated. An initial exploratory analysis of data is presented in Section 5.3. Two forecasting models were then tested, an MLR model and an ANN model. These models were applied to the ten year valve data. The performance of these models was then compared using performance measures such as the R^2, adjusted R^2 and Root Mean Squared Error (RMSE). These prediction models were then applied to forecast future flow rates at these potential hydropower sites to investigate the future operation of an installed hydropower turbine.

5.3 Exploratory Data Analysis

In this section, an exploratory analysis of the data employed in this analysis is presented with the aim of providing a better understanding of the modelling problem. This long term flow variation analysis was undertaken on a dataset over a ten year period from 2002 to 2012. A ten year analysis timeframe was chosen because for a water supply hydropower project to be deemed feasible, investment payback is generally required to be achieved within ten years. If flow rates could be predicted accurately for this ten year period, a suitable turbine could be selected to maximise power generation and ensure the investment is paid back within this period. If long term flow variation is not taken into account at the turbine selection and design stage, the installation is at risk of becoming obsolete with future changes in the flow rates impacting power generation.

Ten valves in the Dublin city region and five valves in the Welsh water supply network were investigated. The locations of these valves are indicated in Figure 5.2 (A). All of the Irish valves analysed were located in the Dublin City region. The majority of the Welsh valves analysed were located in south Wales near Cardiff and Newport, with one valve located in North Wales near Flint. Figure 5.2 (B) shows the location of the valves studied in Dublin City in more detail. In practice, current Supervisory Control and Data Acquisition (SCADA) systems can often provide system or region-wide demand data in the form of time series, typically with resolutions of 15 minutes. Both the DCC and WW telemetry data analysed was recorded at a resolution of 15 minutes.

Flow and pressure data was analysed for 8 valves and one reservoir in the Dublin City region over this ten year period. The two Cookstown data points were valves located at the Cookstown Reservoir. Flow data for five WW PRVs was also obtained. Table 5.1 provides an overview of the data analysed for each valve including the power generation potential at each valve based on 2012 average flow rate and pressure drop, and assuming a conservative constant system efficiency of 65%. These valves were all selected for more detailed analysis because they were found to have relatively high power generation potential during the feasibility study presented in Chapter 4. One of the PRVs (V8) listed in Table 5.1 went online in 2006, therefore only seven years of data was available for this valve. Similarly for Wales, water flow rate and pressure data was obtained for five PRVs, as shown in Figure 5.2, with potential for hydropower energy recovery. The amount of data available varied for each site. Eight years of data was obtained for the Rhydyfelin, Risca and Llanishen PRVs. Ten years of data was obtained for Mountpleasant and Pontypool PRVs.

Fig. 5.2: (A) All site locations (top); (B) Dublin city - site locations (bottom)

Valve ID	Name	Data analysed	2012 Power Output (kW)
DCC Valves			
V1	Blackhorse Bridge	Nov 2002 - Dec 2012	79.04
V2	Brunswick St	Nov 2002 - Dec 2012	16.40
V3	Cookstown Res. 1	Nov 2002 - Dec 2012	NA
V4	Cookstown Res. 2	Nov 2002 - Dec 2012	NA
V5	Donnybrook	Nov 2002 - Dec 2012	27.86
V6	Merrion	Nov 2002 - Dec 2012	22.29
V7	Poplar Row	Nov 2002 - Dec 2012	17.34
V8	Rainsford St	Feb 2006 - Dec 2012	24.70
V9	Rialto Bridge	Nov 2002 - Dec 2012	50.35
V10	Slievebloom	Nov 2002 - Dec 2012	28.09
V11	Thomas Court	Nov 2002 - Dec 2012	86.13
WW Valves			
V12	Llanishen west	Apr 2004 - Dec 2012	9.3
V13	Mountpleasant	Mar 2004 - Dec 2012	10.3
V14	Pontypool	Jan 2002 - Dec 2012	15.3
V15	Rhydyfelin	Jan 2002 - Dec 2012	4.90
V16	Risca	Mar 2004 - Dec 2012	31.4

Table. 5.1: Overview of data for long-term flow variation analysis

Average annual flow rate variation for the Dublin city valves is illustrated in Figure 5.3. From this initial exploratory analysis, it was found that the average flow rates for each valve varied year on year from 2002-2012 by as much as 350% in some cases (e.g. Cookstown 30). Both valves V3 and V4 are at Cookstown Reservoir. Valve V4 is on a trunk mains bringing water from the Ballymore Eustace reservoir and treatment works to Cookstown reservoir. Valve V3 however is on one of a number of mains leaving Cookstown Reservoir to feed the Dublin city DMAs.

Figure 5.4 illustrates the change in average power generation if a turbine was installed at valve V7 in 2003 based on the average flow rate in 2002/2003. Using variable flow rates and pressure drops as recorded at this valve over the ten year period along with variable turbine efficiencies, the annual power generation for different turbine types was calculated and plotted. It can be seen that the power generation increases initially and then decreases, increasing again and generating peak power output between 2007 and 2009, followed be a sharp decrease to 2011. The decreased power generation is experienced most significantly by the PAT and the Fixed Propeller turbines, these turbines both operate poorly when flow rates drop below 60% of the design flow. The peak gen-

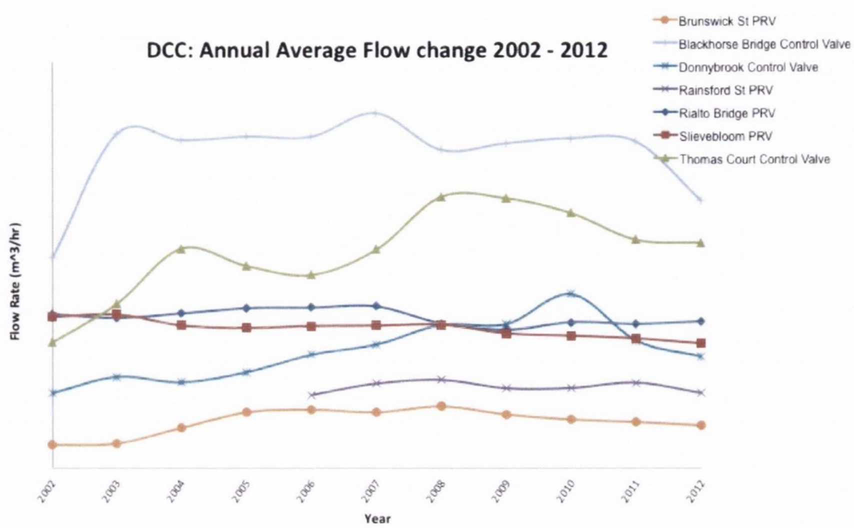


Fig. 5.3: Annual average flow change - Dublin valves

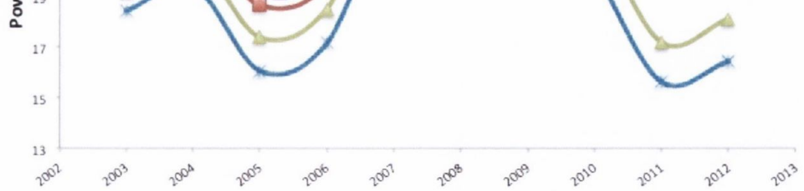


Fig. 5.4: Annual power generation at V7 using variable flow rates, pressures and turbine efficiencies

eration period would coincide with the peak boom years in Ireland, with the following decline mirroring the recessionary years. These relationships will be explored in more detail through the MLR analysis later in this chapter.

This ten year period analysed covers Irelands widely publicised economic boom years followed by its entry into recession and the global financial crisis. The boom period saw unemployment rates reach as low as 2.6% in 2001. Beginning in 2008, Ireland experienced a severe financial crisis resulting in high unemployment and emigration. Furthermore, Ireland's financial crisis was exacerbated as it coincided with the global financial crisis which began in 2007 (Woods and O'Connell, 2012). Prior to this, Ireland experienced a property and credit bubble. 2008 saw the catastrophic burst of this property bubble in Ireland, leading into the recent years of recession. This is illustrated in the unemployment rates for Dublin city, Wales and the UK as shown in Figure 5.5. Wales and the UK as a whole were also affected by the financial crisis, with unemployment increasing sharply between 2008 and 2010, however the recession was not as severe as the Irish recession.

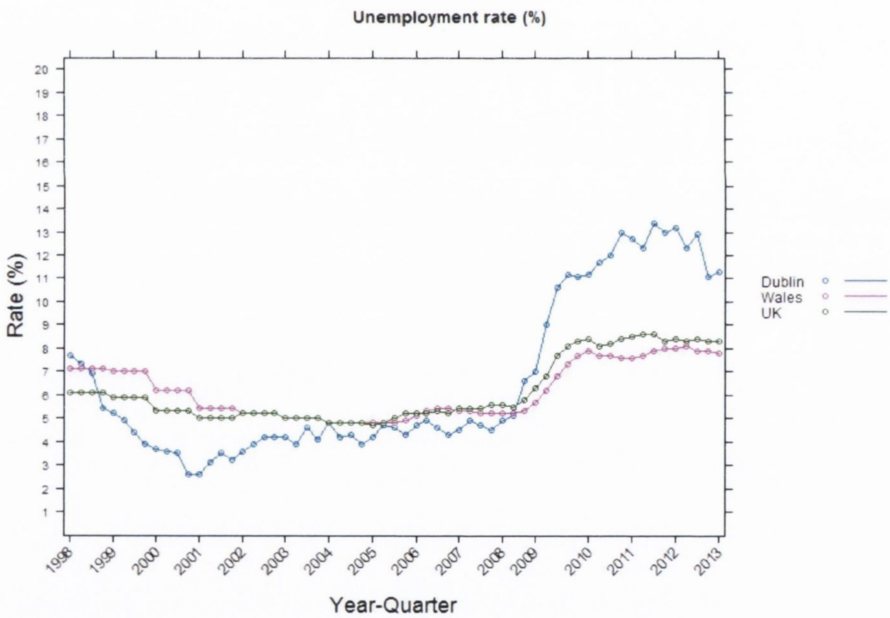
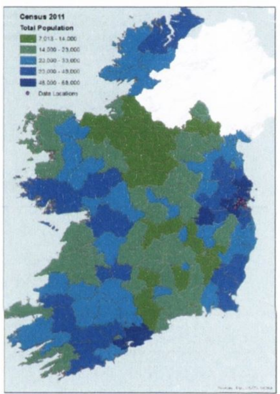
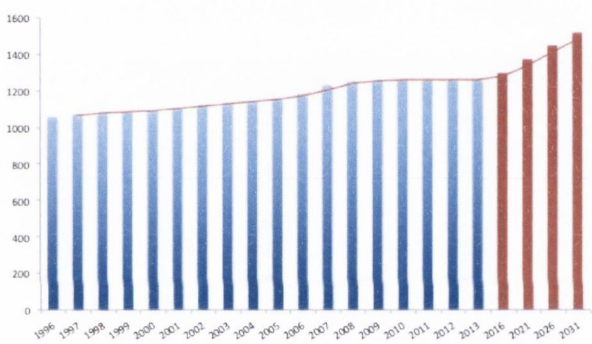


Fig. 5.5: Dublin unemployment rate in comparison with Welsh and UK unemployment rates

Socio-economic data was obtained from the Central Statistic Office (CSO) in Ireland and the Office of National Statistics (ONS) in the UK. Figure 5.6a illustrates the population density in Ireland from the 2011 census, with the Irish site locations for this analysis shown in pink. All of the sites studied are in the Dublin city region, a highly populated area in comparison to the rest of Ireland. Water demand and hence flow rates in water supply mains would be expected to be higher in the more populated areas. Figure 5.6b illustrates the population growth in Dublin since 1996, with future population estimates as published by the CSO also plotted. It can be seen that population has increased steadily since 1996 and is forecast to increase further up to 2030. This projected population increase will increase water demand in the region, increasing flow rates in all mains, and potentially requiring the further development of new water sources to ensure all demands can be met.



(a)



(b)

Fig. 5.6: (a) 2011 Population in Local Electoral Districts; (b) Annual population change in Dublin 1996-2013 (blue), and CSO forecasts (red)

Welsh mid-year population estimates (MYEs) were sourced from the ONS. Population counts from the Census years were rolled forward using birth and death registration data along with estimates of international migration (outside UK) and internal migration (within UK) flows to calculate annual estimates of the usually resident population of each area. The population estimates for 2002-2010 for Wales were then revised to take into account the results of the 2011 Census.

Other socio-economic variables obtained for inclusion in this analysis for both Ireland and Wales were the volume of production indices for the construction industry and the number of houses built. These were identified as indicators of construction and economic growth, potentially influencing water usage. Evident in the Irish data was a period of peak housing construction from 2006 to 2009 (Figure 5.7), at the height of the boom period, when unemployment rates were at low levels of between 4% and 5%. Housing construction dropped suddenly following the financial crisis. Since 2010, the number of houses constructed has been at its lowest level since 1994.

Fig. 5.7: Number of houses constructed in Dublin Citcy Council

With our ever-changing climate and increasing population, water supply worldwide is under stress. As published in the IPCCs fifth assessment report, Europe has been warming faster than the global average of 0.27°C per decade. In northern European countries such as Ireland and the UK, temperatures have risen by 0.48°C per decade (AR5, 2013). Warmer weather will directly impact upon both water usage and water availability. Large scale hydropower stations at reservoirs and dams would experience reduced power generation capacity with increased temperatures due to the increased rate of evaporation at reservoirs.

Furthermore, both warm and cold weather will impact demand patterns, and hence power generation at hydropower turbines installed within distribution mains. Warm weather would result in increased water demand for irrigation, garden hoses, maintenance of local sports grounds etc. Cold weather will also affect water flow rates in pipes, through increased frequency of burst pipes and as a result of users leaving taps running to prevent burst pipes.

The winter of 2010/2011 in the UK was a record breaking cold weather event. December 2010 was the UK's coldest December since the UK Met Office records began in 1910, with a mean temperature of -1°C. The UK mean temperature for winter 2010/2011 as a whole was 2.4°C, making it less cold than winter 2009/10 which was 1.6°C but still the second-coldest winter since 1985/86 with 2.3°C (UK Met Office, 2011). Many of the valves in both Ireland and Wales saw closures during these two cold winters of 2009/2010 and 2010/2011, with very low or no flow for periods during maintenance etc. These periods of closures would have resulted in lost running time of any turbines installed, impacting on the power generation capacity and hence the predicted investment payback.

In order to investigate the impact of climate factors on flow rates at the valves studied, climate data for this ten year period was also obtained. Data from two weather stations in Dublin, one at Casement Aerodrome and one at Dublin Airport were obtained from Met Eireann. The data measured at these stations included the rainfall (mm), dry-bulb temperature (°C), sunshine (hours), the relative humidity and the vapour pressure. Hourly climatic data for Wales was sourced from the UK Met Office for two Welsh weather stations at St. Athan and Hawarden. The data resolution for both the Irish and UK weather stations was hourly.

Water leakage rates from Welsh Water mains were sourced from the Welsh Water Annual Report and Accounts (Welsh Water Dwr Cymru, 2003-2013). This data included both the amount of leakage per km of pipe and the total amount of leakage in megalitres per day. The equivalent water leakage data for DCC mains were sourced directly from DCC. Figure 5.8 illustrated the change in leakage over this period for both DCC and WW. These rates are in megalitres per day. Welsh Water have approximately 26,500km of water mains over which this leakage occurs (Welsh Water, 2014), while DCC have just 2,700km of water mains (Dublin City Council, 2010).

Water leakage is a major challenge for WSPs. Ireland in particular, as mentioned in Chapter 1, looses almost 50% of its treated water. In some Irish local authorities, over 50% of water supplied is unaccounted for. Total system leakage rates in the Irish local authority of DCC as well as leakage rates from all of Welsh Water's infrastructure over the past ten years are shown in Figure 5.8. It is clear from the Welsh Water rates that through targeted strategic infrastructure investment, upgrades and maintenance, their leakage rates have been steadily decreasing. The increase evident both in the Welsh Water data and in the DCC data, between the years 2009/2010 and 2010/2011, was due

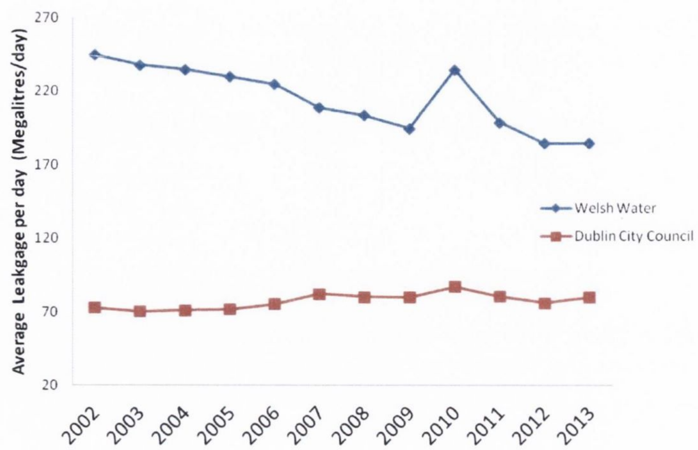


Fig. 5.8: Dublin City Council and Welsh Water: Water supply leakage rates from 2002-2013

to the very cold winter temperatures experienced. These cold temperatures resulted in an increase in burst pipes. However since then, leakage has again decreased in both districts.

Finally, another important factor to consider that has an impact on water usage is the price of water. Ireland did not charge for domestic water usage during the study period, however domestic water charges are planned to be introduced in Quarter 4 of 2014. Irish water, the new national water utility, will begin to charge for water under the terms of Irelands Programme of Assistance with the EU-ECB-IMF. Charging will commence in Quarter 4 of 2014, with customers due to receive their first bills in Quarter 1 of 2015 (Irish Water, 2014). This historic analysis of the effect of water charges on the water flow rates in Wales will therefore be important when considering future water demand for the Ireland network.

The cost of the average household bill charged by all UK water companies is published by Ofwat each year (OFWAT, 2013). The average household bill as charged by Welsh Water is plotted in Figure 5.9. It can be seen here that bills have continuously increased between 2004 and 2010. Prices decreased slightly in 2010/2011, but have since begun to rise again. The influence of these price changes upon the quarterly change in average flow rates at each of these valves will also be investigated in this analysis.

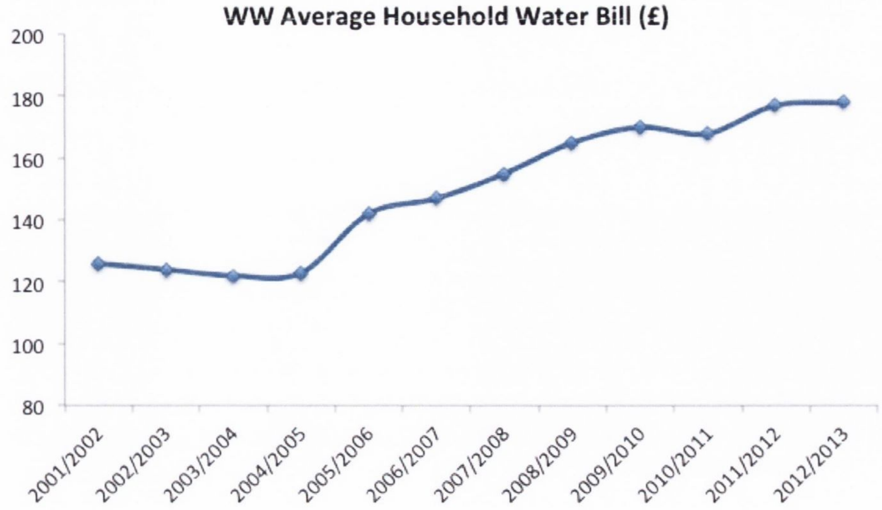


Fig. 5.9: Welsh Water average household water bill (£) from 2001 to 2013

5.4 Methodology

This section describes the varying methods and modelling tools employed for this analysis. All models, both MLR and ANN were applied within the MATLAB environment. Multiple linear regression was initially employed to establish the key influential variables on water flow rate variation. This exploratory analysis allowed for decisions to be made on which of the variables to be included in further regression models, as well as the selection of the timestep duration for further models. Following an initial hourly regression analysis, multiple linear regression was again employed on the ten years of data, this time at a quarterly timestep, to enable the investigation of longer term socio-economic trends on water flow rate variation. ANNs were then employed to develop a model for predicting water flow rates at these valves using the same input data as for the MLR analysis. A direct comparison could then be made between the accuracy of these two prediction methods. Finally, a scenario-based analysis was employed using the two prediction models developed to predict long term flow rates at these valves. Three scenarios were analysed to predict the flow rate at these valves in the years 2020 and 2030.

5.4.1 Hourly Data Investigation

Initial linear regression models were developed using the high resolution hourly flow and climatic data. The aim of these regression models was to investigate the level of correlation between these influential variables on the hourly change in water flow rates. As well as the climatic variables, other variables related to the time of the day, the day of the week, weekday versus weekend and the month were also investigated. These are all variables that would commonly be included in short term water demand prediction (Babel and Shinde, 2011).

A further investigation of diurnal variation of flow rates was then undertaken at each valve. Trends in diurnal variation based on the hour of the day and the day of the week were investigated. Average annual demand patterns were calculated and plotted. The annual changes in these diurnal flow rate patterns were analysed to investigate how these daily water usage patterns varied over the ten year period, in terms of both the magnitude of the average flow rates and also changes in the diurnal flow pattern.

The primary purpose of this initial data exploration, which included the development of hourly regression models, was to enable the selection of appropriate variables and timesteps to employ for the later development of long term predictive models.

5.4.2 Multiple Linear Regression

Following this initial exploratory analysis, MLR was again employed on the ten years of data, this time at a quarterly timestep. Quarterly timesteps were selected because the majority of the socio-economic data available is produced on a quarterly basis. These quarters were defined as Quarter 1: January, February and March; Quarter 2: April, May, June; Quarter 3: July, August, September; and Quarter 4: October, November, December. This longer duration of timestep for analysis enabled the investigation of relationships between water flow rate variation and longer term socio-economic trends such as changes in the rate of unemployment, the regional population, the amount of construction and the annual change in leakage rates from the water supply mains in question. It also allowed for the investigation of long term flow rate changes with changes in climatic variables, such as average temperatures, maximum temperatures and average rainfall.

MLR is commonly employed by WSPs to indirectly forecast water demand. MLR is a common method to determine statistically based relationships and functions between several independent variables (X_1, X_1, ..., X_n) and a dependent variable Y according to Equation 5.4.1. In a regression analysis, water demand relationships are expressed in the form of mathematical equations, showing water demand as a function of one or more independent explanatory variables. MLR is regularly employed by water service providers for water demand forecasting and has been reported on in much water demand forecasting literature (McDonald et al., 2011; Donkor et al., 2014). The general form of a MLR analysis is:

$$Y = f(X_1, X_2, X_3, ..., X_n) \tag{5.4.1}$$

where, Y is the dependent variable, and X_1 to X_n are the predictor (or independent) explanatory variables. The explanatory variables commonly applied to water demand forecasting were discussed in Section 5.2. The independent predictor variables used for the initial MLR model for both the DCC valves and the WW valves are detailed in Table 5.2.

Table. 5.2: Input variables for Regression models for DCC and WW Valves

DCC ID	Input Variable	WW ID	Input Variable
$\mathbf{D_1}$	Quarter	$\mathbf{W_1}$	Quarter
$\mathbf{D_2}$	Unemployment rate (%)	$\mathbf{W_2}$	Welsh regional unemp. rate(%)
$\mathbf{D_3}$	Population Dublin	$\mathbf{W_3}$	Welsh regional population
$\mathbf{D_4}$	Participation rate (%)	$\mathbf{W_4}$	Ave. Household water bill (£)
$\mathbf{D_5}$	Irish Construction Index	$\mathbf{W_5}$	Welsh Construction Index
$\mathbf{D_6}$	House Builds	$\mathbf{W_6}$	House Builds
$\mathbf{D_7}$	Leakage rate (%)	$\mathbf{W_7}$	Leakage rate (Ml/d)
$\mathbf{D_8}$	Ave. Temperature (°C)	$\mathbf{W_8}$	Ave. Temperature (°C)
$\mathbf{D_9}$	Max. Temperature (°C)	$\mathbf{W_9}$	Max. Temperature (°C)
$\mathbf{D_{10}}$	Min. Temperature (°C)	$\mathbf{W_{10}}$	Min. Temperature (°C)
$\mathbf{D_{11}}$	Total Rainfall (mm)	$\mathbf{W_{11}}$	Total Rainfall (mm)
$\mathbf{D_{12}}$	Ave. Rainfall (mm)	$\mathbf{W_{12}}$	Ave. Rainfall (mm)
$\mathbf{D_{13}}$	Ave. Rel. Humidity (%)	$\mathbf{W_{13}}$	Ave. Rel. Humidity (%)
$\mathbf{D_{14}}$	Ave. Vap. Pressure (hPa)	$\mathbf{W_{14}}$	Ave. MSL Pressure (hPa)
$\mathbf{D_{15}}$	Ave. Sunshine (Hrs)	$\mathbf{W_{15}}$	Ave. Cloud base
$\mathbf{D_{16}}$	Total Sunshine (Hrs)	$\mathbf{W_{16}}$	Ave. Global Radiation (kJ/m^2)

Tables 5.3 shows a sample of the quarterly model input data (quarter 4 of each year)

for the Brunswick Street Valve. Similar input data files were made for each of the other valves and are included in Appendix C. Average flow rates for each quarter were used as the dependent variable for analysis. Average climatic data was also used, such as mean temperature, mean rainfall and mean relative humidity. Total rainfall was also used. Quarterly data were available for many of the socio-economic variables, such as unemployment rates, participation rates, the volume of production in the construction industry and the number of houses constructed. However, the leakage rates and population data was available on an annual basis only. For this study, this data was linearly interpolated to a quarterly basis. Linear interpolation was also employed by Babel and Shinde (2011) on socio-economic variables, such as population change, for water demand prediction. This interpolation was included to remove the sudden drop or jump in data at each year change, because it was assumed that in reality these values would gradually vary over the four quarters.

5.4.3 Artificial Neural Networks

The next phase of research saw the application of ANNs for the long term forecasting of flow rate at each valve. ANNs can be applied to generalise non-linear relationships between inputs and output (target) data. The primary influential variables as reported by the previous multiple linear regression analyses were again used as the inputs for the development of these ANN algorithms. The target data was the flow rate at each valve. A direct comparison could then be made between the accuracy of these two prediction methods. Hybrid methodologies, such as the use of both MLR to select the optimal input variables, followed by ANN modelling has been reported on by Babel and Shinde (2011) and Donkor et al. (2014). Hybrid composite forecasts have been reported as more effective than either ANNs or MLR on their own for short-term water demand forecasting (Donkor et al., 2014). ANNs and their performance for medium to long-term forecasts has not been widely researched.

ANNs are applied here for non-linear regression fitting. An ANN is an interconnected group of artificial neurons. Each neuron executes a non-linear computation based on the input values and the resulting value is fed to other neurons. Neurons are usually arranged as a series of interconnected layers. Based on the input data provided to the network, an algorithm (usually back-propagation) is used to iteratively adjust the neuron connection weights in order to improve the predictive performance of the network.

Table. 5.3: Database for Predictors for the Brunswick Street PRV model

Year	Y	D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8	D_9	D_{10}	D_{11}	D_{12}	D_{13}	D_{14}	D_{15}	D_{16}
2002	67.41	4	4.2	1128	62.5	247.4	1172	34.31	7.53	15.5	-1.9	283.1	0.19	86.70	9.14	0.06	87.3
2003	69.30	4	4.1	1139	62.3	272.1	743	33.51	7.48	17.9	-5.1	221.3	0.10	85.74	9.07	0.11	238.3
2004	149.32	4	3.9	1152.7	62.8	313.8	1307	33.65	8.04	15.7	-1.2	205.1	0.09	85.95	9.36	0.10	221.7
2005	174.41	4	4.3	1149.2	64.3	367.4	2064	33.99	8.14	18.9	-3.1	221.5	0.10	87.14	9.65	0.10	230.1
2006	182.26	4	4.3	1207.3	65.6	369.4	2058	34.95	8.72	16.9	-2	271	0.12	85.67	9.84	0.13	287.1
2007	155.41	4	4.5	1241.95	65.8	279.6	2077	36.51	6.86	16.4	-3	185.9	0.08	88.70	9.01	0.12	262.1
2008	179.82	4	7	1255.6	64.8	188.3	1140	35.84	6.86	16.4	-3	185.9	0.08	88.70	9.01	0.12	262.1
2009	148.23	4	11.1	1259.05	63.2	120.4	525	35.94	7.47	17.3	-6.5	303.2	0.14	89.06	9.49	0.10	227.4
2010	130.91	4	13	1260.55	62.4	90.4	274	38.51	5.27	18.4	-11.5	188.9	0.09	89.15	8.33	0.13	280.9
2011	148.25	4	13	1262.2	62.2	85.9	146	36.62	9.19	19.6	-2.1	270.1	0.12	86.53	10.37	0.10	217.8
2012	116.90	4	11.1	1262.65	61.9	81.8	155	35.73	6.81	14.1	-3.1	196.1	0.09	87.16	8.76	0.12	254.1

Where for each quarter (D_1), D_2 is unemployment rate; D_3 is the population; D_4 is the participation rate (%); D_5 is the volume of production in construction index; D_6 is the no. of houses built; D_7 is the DCC leakage rate; D_8 is the ave. temperature
D_9 max temperature; D_{10} min. temperature; D_{11} total rainfall; D_{12} ave. rainfall; D_{13} ave. relative humidity;
D_{14} ave. vapour pressure; D_{15} ave. sunshine; D_{16} total sunshine;

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For this analysis, feed-forward neural networks with one hidden layer of ten neurons, were used. Feed-forward networks can be used for any kind of input to output mapping. A feed-forward network with one hidden layer and enough neurons in the hidden layers, can fit any finite input-output mapping problem Matlab (2014*a*). The Levenberg-Marquardt back-propagation training algorithm was applied to update the weights for the network based on the input and output data supplied. The input data was randomly divided into training data (70%), validation data (15%) and test data sets (15%). The training set was used to teach the network. Training continues as long as the network continues to improve as compared with the validation set. The test set provides a completely independent measure of network accuracy Matlab (2014*a*). Networks were re-trained based on their performance outputs until a satisfactory performance was reported. The trained networks were then saved for re-application on new data to forecast future flow rates.

5.4.4 Model Performance

The accuracy of the performance of both the MLR and ANN models developed in this research were evaluated according to the Root Mean Squared Error (RMSE), the R^2 and the Adjusted R^2 of the models. The coefficient of determination (R^2) provided an indication of the degree of correlation between the observed and predicted values. R^2 values range from 0 to 1, with 1 indicating a perfect explanation of variance, and 0 indicating no statistical correlation. The RMSE provides an indication of the discrepancy between the actual values and the fitted values. A favourable model was considered a model with high R^2 and low RMSE. All parameters considered in this study were tested for their statistical level of significance. A p-value lower than 0.05 was considered statistically significant, as falling within the 95 percentile confidence interval.

Where, for N time periods (observations), Y_t and $\hat{Y}_t$:

$$MSE = \frac{1}{N}\sum_{t=1}^{N}(Y_t - \hat{Y}_t)^2 \tag{5.4.2}$$

$$MSE = \frac{1}{N}\sum_{t=1}^{N}(Y_t - \hat{Y}_t)^2 \tag{5.4.3}$$

The MLR and ANN analyses applied in this research explore relationships between social variables relating to water usage and hence perfect correlations would not be expected. As has been discussed in the critical literature review of Chapter 2, in prior long term water demand forecasting research, models with R^2 values of higher than 0.3 were deemed moderately accurate.

5.4.5 Future Scenarios

Following on from both the MLR and ANN model development, future scenarios for flow rates and hence power generation potential were predicted for three of the valves studied. The characteristic MLR equations as developed were used, based on the key influential variables found for each of the three valves studied.

In order to calculate the future estimated flow rate for these scenarios, forecasts for the related input data were required. Population forecasts as published by the CSO were obtained for the Dublin region. Mid-year population projections were also obtained for Welsh local authorities up to the year 2036 as produced by the Welsh Government Knowledge and Analytical Services. Population projections provide estimates of the size of the future population, and are based on assumptions about births, deaths and migration. These assumptions are based on past trends. Projections can only indicate what may happen should these recent trends continue. Projections done in this way do not make allowances for the effects of local or central government policies on future population levels, distribution and change. The projected population for Dublin is for April of each year, and for Wales is for the 30th of June for each year.

The impact of climate change on Ireland and the UK has been studied by researchers. Sweeney (2001) at NUI Maynooth in Ireland have published findings of a large scale research project on the climate change impacts for Ireland. This study predicted, with the study base year 2000, that average seasonal temperatures across Ireland would increase by between 0.75°C and 1.0°C (Table 5.4) by 2020, part of which has already been experienced over the period since 1990. By the 2050s, Irish temperatures were suggested to increase by between 1.4°C and 1.8°C, with the greatest warming occurring during the autumn. By the 2080s, increases were forecasted to be in the range 2.1°C and 2.7°C.

Furthermore, it has been forecast that winter precipitation is likely to increase marginally by the 2020s, by approximately 3%, with summer reductions of a similar order, approx-

Table. 5.4: Mean temperature increases for each season and time period

Season	DecemberFebruary	MarchMay	JuneAugust	SeptemberOctober
2020	0.7	0.8	0.7	1.0
2050	1.4	1.4	1.5	1.8
2080	2.1	2.0	2.4	2.7

imately 3%; however, reductions of between 10% and 16% have been suggested for regions along the southern and eastern coasts of Ireland.

5.5 Results

The results of this analysis are presented according to the four main research phases as defined previously, beginning with the results of the hourly data investigation, followed by the MLR analysis, the ANN analysis and finally the results of the long term flow scenario forecasts.

5.5.1 Hourly Data Investigation

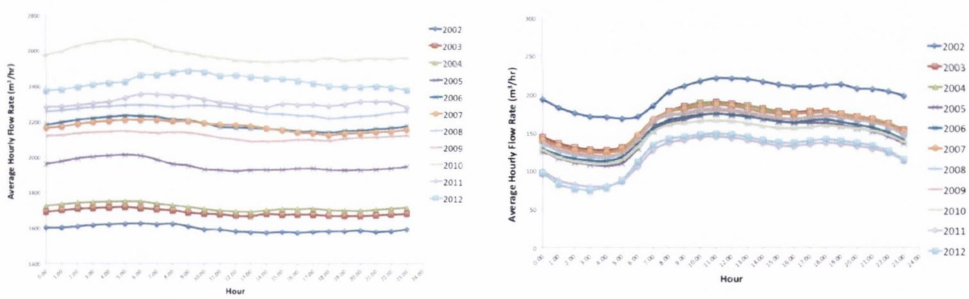
An initial investigation of the correlation between the hourly flow rate data and the hourly climatic data was undertaken. The results of univariate linear regression models between each of these hourly input variables and the hourly flow rate of valves V1 to V5 are detailed in Table 5.5. These regression models reported very low correlations between the hourly flow rates and climatic variables such as the hourly temperature, the amount of rainfall or the relative humidity. The reported R^2 values for each of the individual cases was less than 10%, with some as low as 0.1%. However, correlations between the flow rate change and the hour of the day reported higher levels of correlation. In Table 5.5 it can be seen that though climatic variables such as the amount of sunshine and the temperature explain up to 9% of the variation, the most significant contributor to the overall regression model for all valves was the hour of the day.

An investigation into the change in daily flow rate patterns at these valves was then undertaken. It was found that though the magnitude of the average annual flow rate varied year on year, the daily demand profiles did not vary significantly between each year. Average annual diurnal flow patterns for valves V4 and V7 are plotted in Figures 5.10a

Table. 5.5: Correlation between flow rate and hourly climatic and timestep variables

Valve ID:	**V1**	**V2**	**V3**	**V4**	**V5**
Variable:	R-squared				
Temperature	0.0351	0.0070	0.0066	0.0021	0.0010
Rainfall	0.0001	0.0000	0.0001	0.0001	0.0000
Relative Humidity	0.1100	0.0567	0.0096	0.0022	0.0079
Sunshine	0.0942	0.0294	0.0172	0.0002	0.0164
Vapour Pressure	0.0017	0.0007	0.0018	0.0007	0.0052
Weekday	0.0060	0.0027	0.0035	0.0001	0.0039
Hour	0.2580	0.1030	0.0507	0.0013	0.0325
Month	0.0149	0.0029	0.0007	0.0002	0.0014
Total model:	0.3580	0.1770	0.0702	0.0041	0.0608

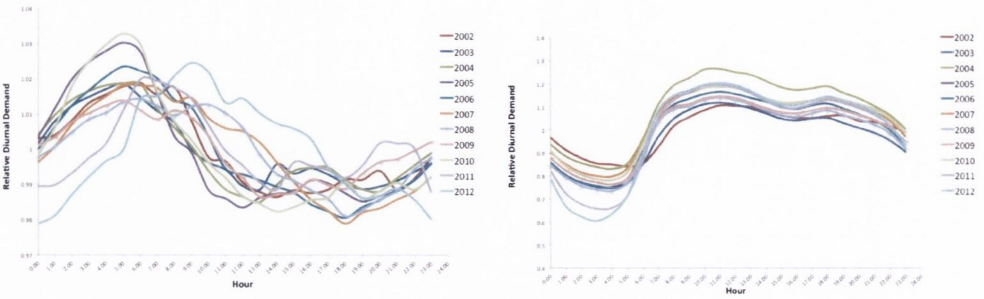
and 5.10b. Here it can be seen that though the magnitude of the average flow curves changes year on year, the hourly diurnal pattern does not change significantly. For example for valve V4, though there has been a large increase in the average flow rate between the years 2002 (dark blue) and 2012 (pale blue), the hourly flow rate pattern has not changed significantly.



(a) (b)

Fig. 5.10: (a) Annual average diurnal flow rate (m^3/hr) at V4 and at (b) V7

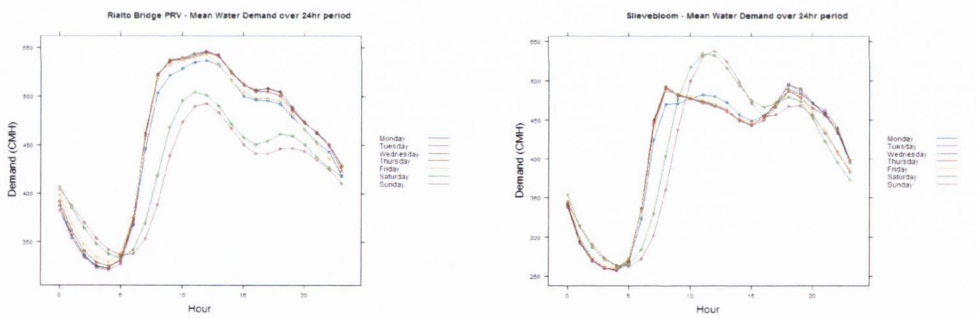
This is further evident in Figures 5.11a and 5.11b. This plot shows the average diurnal demand pattern (Q/Q_0) for each year 2002-2012 at the same valves. For V4 in Figure 5.11a, though the demand pattern for 2012 (pale blue) has shifted to having a peak demand at a slightly later time of 9am, the overall variability between maximum and minimum points has not changed significantly. The flow varied between 98% and 105% of the average flow rates. In Figure 5.11b the diurnal flow trend changes very little year on year, with the flow rate varying by approximately the same amount each year. 2011 (pale purple) and 2012 (pale blue) reported lower minimum night-time flows, down to about 60-65% of the average flow rate, whereas other years the minimum night flow was about 75% of the average flow rate.



(a) (b)

Fig. 5.11: (a) Relative diurnal demand patterns (Q/Q_0) at V4 and at (b) V7

Further analyses of the average demand patterns on each day of the week showed that for many of the valves, different demand patterns were evident on weekdays versus weekend days. This is shown in Figure 5.12 for valves V9 (Rialto) and V10 (Slievebloom).



(a) (b)

Fig. 5.12: Average demand patterns on weekdays versus weekends (a) V9 and (b) V10

Valve V9 shows decreased peak flow rates on weekends, whereas valve V10 shows increased peak flow rates on weekends. This is due to the local demands of the areas. Valve V10 is a largely residential area located further from the city centre than valve V10, while valve V10 is located closer to the city centre, with more businesses and shops in the local area which would see more water usage during the week. Furthermore, it is located near a large hospital with university campus, and there would be many students living in the area who may commute to other parts of Ireland on the weekends, further reducing the water demand on weekends.

From this exploratory analysis of hourly flow rate conditions, it was decided that the long term forecasting model to be developed should predict an average flow rate. A demand profile based on the average demand pattern of the most recent year of data would then be applied to this average flow rate, for the approximation of the diurnal flow conditions into the future.

5.5.2 Multiple Linear Regression

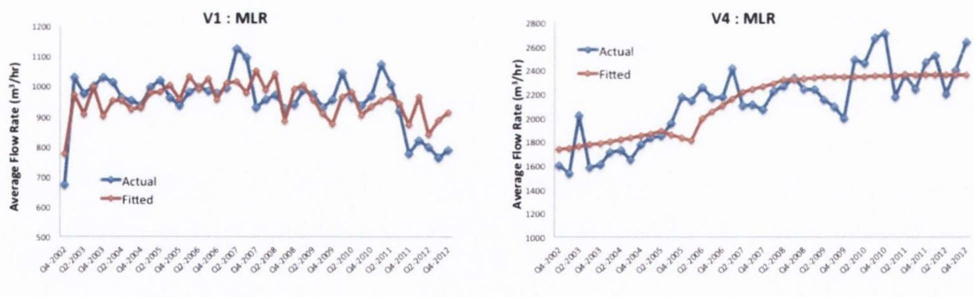
The explanatory variables investigated for both the DCC valves and the WW valves were defined in Table 5.2. A correlation analysis was undertaken between these independent variables and the flow rate at each valve. Backwards stepwise linear regression was then employed to ascertain the best model. When any two variables were highly correlated with each other (greater than 0.85), one of the variables was removed from the model. An example of the correlation matrix of independent explanatory variables for the Donnybrook PRV (V5) is provided in Table 5.7. For example in this matrix, as would be expected, high levels of correlation were found between the maximum, minimum and average temperatures, so just one of these variables was left in the model. The variable with the highest correlation with the flow rate was left in, while the others were removed. Similarly as would be expected, the volume of production in the construction industry was highly correlated with the number of houses built. Backwards stepwise linear regression was employed until a model with a high adjusted R^2 using as few variables as possible was found. For the Donnybrook valve (V5) discussed, the best model found using stepwise linear regression was using the rate of participation in the labour force, the leakage rate and the minimum temperature as predictors.

Table 5.6 details the model data for the best fit models. All of the models presented reported p-values of less than 5%, indicating statistical significance. Some models reported relatively high R^2 values, the Brunswick Street PRV (V2) model for example reported an adjusted R^2 of 66.8%. This model reported highest correlations with the population and the number of houses built in Dublin.

Table. 5.6: Stepwise Multiple Linear Regression Results

Valve ID	R^2	Adjusted R^2	RMSE (m^3/hr)	No. of observations	F-statistic vs Constant model	p-Value
V1	0.416	0.369	71.8	41	8.78	1.58E-04
V2	0.685	0.668	20.1	41	41.2	3.03E-10
V3	0.173	0.152	253	41	8.14	6.89E-03
V4	0.674	0.666	180	41	80.6	4.96E-11
V5	0.602	0.581	54.7	41	28.7	2.56E-08
V6	0.219	0.178	166	41	5.34	0.00903
V7	0.672	0.646	11	41	25.3	4.42E-09
V8	0.424	0.378	26.2	28	9.21	1.01E-03
V9	0.251	0.211	52.6	41	6.36	4.14E-03
V10	0.325	0.308	32	41	18.8	9.89E-05
V11	0.474	0.446	92.6	41	72.1	4.97E-06
V12	0.154	0.129	84.1	36	6.2	0.0178
V13	0.523	0.434	326	33	5.92	0.000811
V14	0.519	0.482	99.6	43	14	2.39E-06
V15	0.31	0.289	2.68	35	14.8	5.12E-4
V16	0.265	0.22	79.2	36	5.93	0.00629

The actual and fitted curves for valve V1 and valve V4 are plotted in Figure 5.13. Though the model of best fit for V1 was found to explain only 36.9% of the variance, it can be seen on the fitted plot that the model has accurately predicted some points and the general trends in terms of peaks and lows are correct. Valve V4 reported the highest adjusted R^2 value of all the MLR models at 66.6%. The MLR model picks up on a broad increasing trend, the population, however it does not as accurately predict the peaks and troughs.



(a) (b)

Fig. 5.13: MLR analysis: Actual values and fitted values for (a) Valve V1 and (b) valve V4

Table. 5.7: Correlation matrix for the Donnybrook valve (V5)

D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8	D_9	D_{10}	D_{11}	D_{12}	D_{13}	D_{14}	D_{15}	D_{16}
1	0.01	0.02	0.07	-0.04	0.07	0.001	0.27	0.18	0.24	0.5	0.5	0.36	0.37	-0.26	-0.25
0.01	1	0.78	-0.5	-0.94	-0.85	0.79	-0.03	0.08	-0.15	0.06	0.005	0.28	0.03	0.06	0.07
0.02	0.78	1	0.06	-0.75	-0.5	0.89	0.0003	0.023	-0.1	0.18	0.075	0.24	0.04	0.1	0.12
0.065	-0.5	0.06	1	0.56	0.77	-0.07	0.18	0.03	0.19	0.22	0.17	-0.12	0.18	0.06	0.06
-0.04	-0.9	-0.75	0.56	1	0.91	-0.77	0.09	0.01	0.18	-0.095	-0.07	-0.31	0.04	-0.005	-0.01
0.07	-0.85	-0.5	0.77	0.9	1	-0.55	0.09	-0.04	0.17	0.002	0.01	-0.17	0.06	-0.1	-0.1
0.001	0.79	0.9	-0.07	-0.77	-0.55	1	-0.06	-0.003	-0.2	0.05	0.001	0.24	-0.014	0.09	0.09
0.28	-0.03	0.0003	0.18	0.09	0.09	-0.06	1	0.9	0.9	0.23	0.16	-0.62	0.98	0.65	0.65
0.18	0.08	0.02	0.03	0.01	-0.04	-0.003	0.91	1	0.73	0.075	0.01	-0.68	0.86	0.79	0.79
0.24	-0.15	-0.1	0.19	0.18	0.2	-0.2	0.9	0.7	1	0.2	0.17	-0.5	0.89	0.46	0.47
0.5	0.06	0.18	0.22	-0.09	0.002	0.05	0.23	0.075	0.2	1	0.96	0.28	0.3	-0.16	-0.16
0.5	0.005	0.075	0.17	-0.07	0.01	0.001	0.16	0.01	0.17	0.96	1	0.3	0.2	-0.23	-0.24
0.36	0.28	0.24	-0.12	-0.3	-0.17	0.24	-0.62	-0.68	-0.53	0.28	0.3	1	-0.46	-0.83	-0.83
0.37	0.03	0.04	0.18	0.04	0.06	-0.01	0.98	0.86	0.89	0.3	0.2	-0.46	1	0.53	0.54
-0.26	0.06	0.1	0.06	-0.005	-0.1	0.09	0.65	0.79	0.46	-0.16	-0.23	-0.8	0.53	1	0.99
-0.25	0.07	0.12	0.06	-0.007	-0.1	0.09	0.65	0.79	0.47	-0.16	-0.24	-0.83	0.54	0.99	1

* Where for each quarter (D_1): D_2 - unemployment rate; D_3 - population; D_4 - participation rate;
D_5 - construction index; D_6 - no. of houses built; D_7 - leakage rate; D_8 - ave. temperature
D_9 - max temperature; D_{10} - min. temperature; D_{11} - total rainfall; D_{12} - ave. rainfall; D_{13} - ave. relative humidity;
D_{14} - ave. vapour pressure; D_{15} - ave. sunshine; D_{16} - total sunshine;

For three of the poorer performing models at V12, V15 and V16, a smaller number of observations were used as model inputs. This may have led to the poorer explanation of variance at these valves. Further plots of the MLR results for each of the 14 valves are included in Appendix C.

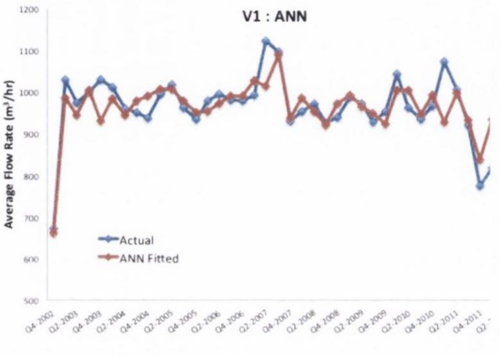
5.5.3 Artificial Neural Network

All of the ANN models out-performed their equivalent MLR models. The superior performance of ANNs in this application can be attributed to their ability to identify non-linear trends and relationships between the input and target data supplied. An overview of the model statistics for each ANN model is provided in Table 5.8. The highest explanation of variance was found for valve V2 which reported an adjusted R^2 of 90.36%.

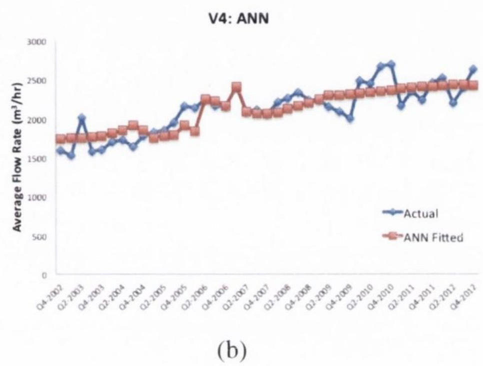
Table. 5.8: ANN - Best Fitted Model Statistics

Valve ID	Name	R^2	Adjusted R^2	RMSE (m^3/hr)
V1	Blackhorse Bridge	0.64	0.62	64.82
V2	Brunswick St	0.91	0.90	14.92
V3	Cookstown A	0.33	0.31	342.46
V4	Cookstown B	0.69	0.68	125.43
V5	Donnybrook	0.88	0.87	44.02
V6	Merrion Gates	0.28	0.24	209.95
V7	Poplar Row	0.78	0.77	7.37
V8	Rainsford St	0.94	0.93	15.94
V9	Rialto Bridge	0.88	0.87	18.63
V10	Slievebloom	0.33	0.31	16.79
V11	Thomas Court	0.71	0.70	103.22
V12	Llanishen	0.24	0.22	63.08
V13	Mountpleasant	0.68	0.62	298.72
V14	Pontypool	0.75	0.73	96.28
V15	Rhydyfelin	0.39	0.37	1.82
V16	Risca	0.58	0.56	19.29

Similar to the MLR model results, low correlations were again found for valves V3 and V12. The actual values and fitted values for V1 and V4 are plotted in Figure 5.14. The ANN models were found to accurately fit the data for these valves, both the general trend and the peaks and troughs. Valve V4 is located on the inlet mains to the Cookstown Reservoir. This reservoir serves a large portion of the Dublin city and South Dublin regions. The increasing flow rate trend at this valve, mirrors the increasing population of the Dublin region over the last ten years.



(a)



(b)

Fig. 5.14: ANN model: Actual values and fitted values for (a) Valve V1 and (b) valve V4

Further plots of the actual and fitted values as forecast by the ANN models are provided in Appendix C.

5.5.4 Future Scenarios

Following on from the development of predictive models based on the application of both MLR and ANN analyses, future climate scenarios were developed for three of the sites investigated. Three sites with the highest R^2 values were selected as these would forecast most accurately based on the correlations found; These were the Brunswick St PRV (V2), the Poplar Row Valve (V7) and the Cookstown B Reservoir valve (V4).

The linear regression equation for the Cookstown B Valve (V4) was found to increase linearly with the Dublin population, with the characteristic equation found to be:

$$y \sim 1 + Population \tag{5.5.1}$$

$$y = -3522.7 + 4.6597 * \text{Population} \tag{5.5.2}$$

Using the CSO population predictions for Ireland as described in Section 5.3, future flow scenarios for this valve were plotted, with the 2012 average diurnal demand pattern applied as illustrated in Figure 5.15.

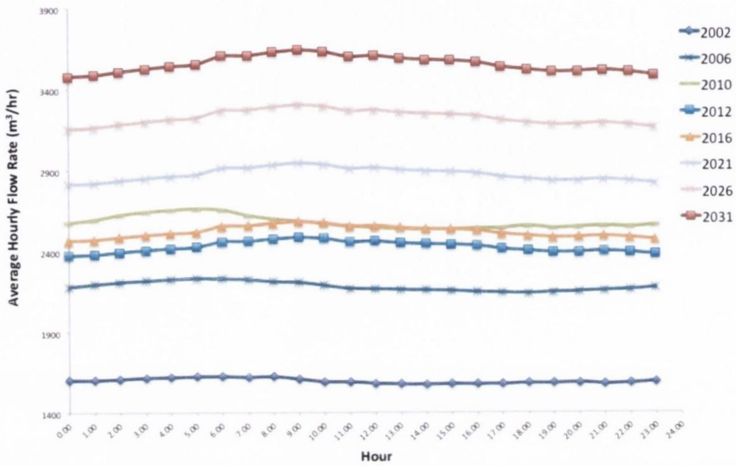


Fig. 5.15: Annual average diurnal flow rates at V4 with 2016, 2021, 2026 and 2031 flow rate forecasts plotted

As can be seen, with the population of Dublin predicted to increase, flow rates at this valve could reach more than double the 2002 average flow rates.

Flow rates at the Brunswick St PRV (V2) were also found to increase with the population, and also to increase with the number of houses built. The characteristic equation was found to be:

$$\text{Ave. Flow} = -572.33 + 0.55424 * \text{Pop} + 0.043139 * \text{Houses} \tag{5.5.3}$$

According to a forecast published by the ESRI (Morgenroth, 2014), due to increased demand for housing in Ireland and a shortage of available housing stock, it is estimated that 18,000 new houses are required to be built in Ireland annually between 2014 and 2020. The majority of these, between 13,000 and 15,000 are estimated to be required for the Dublin region. A 2020 forecast for this valve was estimated based on a peak construction of 15,000 houses per year, therefore 3,750 houses to be built per quarter.

Peak housing construction during the boom period reached 2,077 houses built in quarter 4 of 2007. Together with the CSO forecast population for 2020, the forecast 2020 diurnal flow rate at valve V2 is plotted in Figure 5.16. Another calibration of the regression model was undertaken using actual flow data for 2013 at this valve obtained from DCC. The 2013 quarter 1 average daily flow rate was predicted using the published numbers of houses built in the Dublin City Council region, and the published population estimates for Dublin for 2013. The daily average demand pattern for 2010/2011 was applied to this forecast average flow rate.

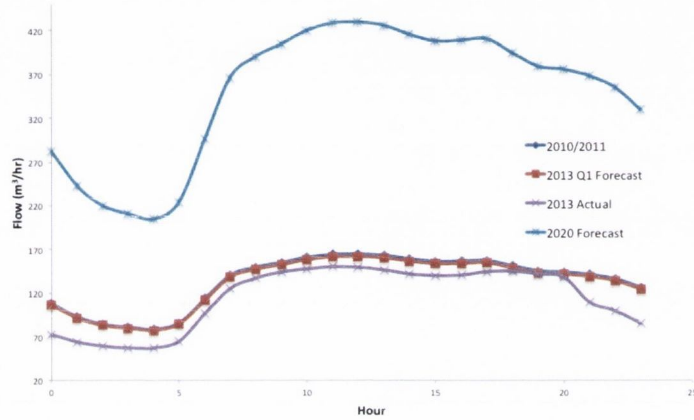


Fig. 5.16: The average 2010/2011 and 2013 diurnal flow rates at V2 with the average MLR forecast diurnal flow rates for 2013 and 2020 plotted

As can be seen in this plot, the regression model developed using the 2002-2012 flow data has allowed for a good approximation of the 2013 quarter 1 flow rate. Though it has slightly over-estimated the flow rates, many of the points plotted indicate good correlation.

Flow rates at the Poplar Row PRV (V7) were found to decrease with the rate of unemployment and with the average temperature, and increase with the amount of leakage. Flow rates at V7 were also found to decrease with increased unemployment in the Dublin region. This could be due to a high level of unemployment in the surrounding areas in that region which could indicate business closures resulting in reduced water demand. The characteristic equation for valve V7 was found to be:

$$\text{Ave. Flow} = 151.55 - \text{Unemp} * 4.9091 + 1.5006 * \text{Leakage} - 1.3233 * \text{Ave. Temp} \tag{5.5.4}$$

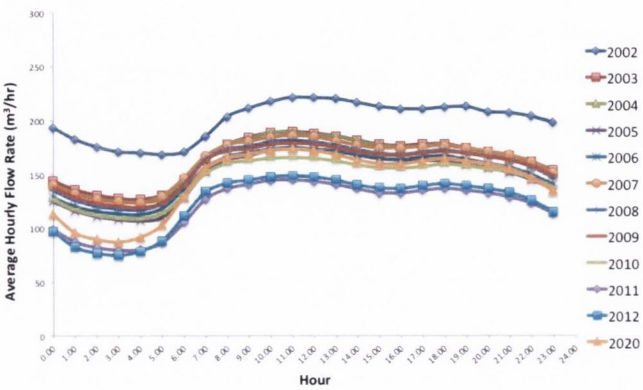


Fig. 5.17: Annual average diurnal flow rates at V7 with 2020 flow rate plotted

Future flow rate at the Poplar Row valve (V7) is plotted in Figure 5.17. The average temperature for 2020 was estimated based on the predictions by Sweeney (2001). This prediction was for average seasonal temperatures across Ireland to increase by between 0.75°C and 1.0°C. 2013 leakage levels for DCC were reported to be 36%, it was assumed that by 2020 these leakage levels would be reduced to 20% through Irish Water's leakage reduction plans and assuming that through the introduction of water metering many of these leaks would be identified and repaired.

5.6 Discussion

The minimum estimated design life of hydropower turbines is in the range of 20-25 years, with some turbines remaining in operation for even longer periods than that. Large fluctuations in flow rates during these design lives, may result in an installed turbine operating at reduced efficiency or becoming unsuitable for its installed location.

From the hourly flow rate analysis, it was found that for some valves, for example valve V4, there was very little variation in the diurnal pattern over the ten years. The average diurnal flow pattern varied from 98% and 105% of the average annual (design) flow rate. This valve however is located at a reservoir and not within distribution mains, therefore would experience less variability. At valve V7 on the distribution network, more variability in the daily demand was found, with daily flow rates varying between 65% and 130% of the average flow.

A summary of the best reported explanatory variables for each of the DCC valves is provided in Table 5.9. For the DCC valves, the rate of unemployment (D_2), the population (D_3), the rate of leakage from the water mains (D_7) and the number of houses built (D_6) were found to be the most common influential variables on long term flow changes. The results of the equivalent quarterly MLR analysis on the Welsh Water valves are summarised in Table 5.10.

Table. 5.9: Summary of Influential Variables for Each DCC Valve

Variables*:	V1	V2	V3	V4	V5	V6	V7	V8	V9	V10	V11	Freq:
D_1												0
D_2							1		1	1	1	**4**
D_3		1		1				1				**3**
D_4	1				1							2
D_5								1	1			2
D_6		1	1			1						**3**
D_7					1	1	1	1				**4**
D_8											1	1
D_9									1			1
D_{10}					1		1					2
D_{11}												0
D_{12}	1											1
D_{13}												0
D_{14}												0
D_{15}												0
D_{16}												0

* Where for each quarter (D_1): D_2 - unemp. rate; D_3 - population; D_4 - part. rate; D_5 - construction index; D_6 - no. of houses built; D_7 - leakage rate; D_8 - ave. temp; D_9 - max. temp; D_{10} - min. temp; D_{11} - total rainfall; D_{12} - ave. rainfall; D_{13} - ave. rel. humidity; D_{14} - ave. vapour pressure; D_{15} - ave. sun.; D_{16} - total sun.;

Of the climatic variables tested, the only significant indicators of flow rate change were temperature, rainfall and the cloud level. The average temperature and the maximum temperature were found to be influential for two DCC valves. This is likely due to the seasonal demands in those areas for the maintenance of gardens, parks and sports grounds. Three of the five WW valves reported significant correlations between the flow rate and the average household water bill. This should be noted for Irish water and the future plans to introduce water metering tariffs.

Table. 5.10: Summary of Influential Variables for Each WW Valve

WW Valves: **Input Variable:**	V12	V13	V14	V15	V16	**Total:**
W_1						0
W_2						0
W_3			1	1	1	**3**
W_4		1	1		1	**3**
W_5						0
W_6		1				1
W_7		1				1
W_8		1				1
W_9	1					1
W_{10}			1			1
W_{11}						0
W_{12}		1				1
W_{13}						0
W_{14}						0
W_{15}						0
W_{16}						0

Where for each quarter (W_1): W_2 - unemployment rate; W_3 - population; W_4 - ave. household water bill; W_5 - construction index; W_6 - no. of houses built; W_7 - leakage rate; W_8 - ave. temp; W_9 - max. temp; W_{10} - min. temp; W_{11} - total rainfall; W_{12} - ave. rainfall; W_{13} - ave. relative humidity; W_{14} - ave. vapour pressure; W_{15} - ave. cloud; W_{16} - ave. global rad;

Valve closures occurred at various stages throughout the ten year period of study, for example the Pontypool PRV experienced highly reduced flows throughout the year 2010. These valve closures, or bypasses, may be due to related upgrades, by-passes or maintenance works being undertaken in the local network. The occurrence of closures at valves is one unknown that is difficult to predict at the feasibility stage of a hydropower project. Though certain upgrade plans may be defined in the scheme of works by the water company, other upgrades may be undertaken on an ad-hoc basis, due to leakage, local construction work or burst pipes. The risk of occurrence of closures should be included in the feasibility study assessment, in particular when investment payback is forecast to take longer periods of time such as 5 to 10 years.

Of the WW valves, one interesting correlation was the influence of water prices on flow rates. For example, at valve V14, the Pontypool valve, the average flow rate was found to be influenced by the water price, the population and the minimum temperature. Increases in price caused a decrease in the average flows. Furthermore increases in the minimum average temperatures resulted in increased average flow rates. This increase in flow rates as a result of low temperatures could be attributed to increased leaks due to frozen pipes causing bursts, or increased flow rates due to people leaving taps running to prevent pipes freezing. The best MLR model for V2 corresponded to a relationship between flow rate (y) such that:

$$y \sim 1 + Price + Population + Min.Temperature \tag{5.6.1}$$

With the MLR characteristic equation predicted for V14 as:

$$y = -94478 - 10.47 * Price + 1.0579 * Population + 12.109 * Min.Temp \tag{5.6.2}$$

The impact of future water price changes was then investigated for V14. The average flow rate in quarter 3 of 2012 was 380m^3/hr. The average household water bill in 2012 was £178. Using the MLR equation detailed in Equation 5.6.2, future changes to this water price were investigated. If water prices increased by 5%, it was estimated that the average flow rate would reduce to 312m^3/hr. If water prices increased by 10%, the average flow rate could further reduce to 219m^3/hr. Conversely, a 5% decrease in water prices, would lead to an increase of the average flow rate to 498m^3/hr. While a 10% price increase caused the future forecast average flow rate to increase to 592m^3/hr.

In reality water is a necessity for households and businesses. While demand will respond to marginal changes in prices, the larger the price hike the more inelastic demand becomes, as users still require a minimum supply regardless of the price. In economics, this is defined as the price elasticity of demand, with water an inelastic, non-substitutable commodity.

Further significant correlations with the minimum temperature and an increased flow rate were found for three of the DCC valves. For two of these three valves, correlations with the leakage rate were also found. This would further increase the likelihood that the increased leakage occurring at these valves when minimum temperatures are experienced is due to an increase in burst pipes during cold periods.

The ANN models were found to more accurately predict the water flow variation based on the input variables. This is most likely due to their ability to identify non-linear trends in the data. This would be especially relevant with climatic variables, as the relationships between climatic variables such as temperature and rainfall with urban water demand may be non-linear (Billings and Jones, 2008).

Though the ANN models outperformed the MLR models as presented in the results tables, ANN models require more input for future forecasts. All of the ANN models were developed using the available data, varying from 6 to 10 years of quarterly inputs. In order to forecast using these ANN networks developed, a full 6 to 10 years of input data is required to be passed to the ANN. To predict flow rates for ten years of data for the Cookstown valve for example, the equivalent ten years of population estimated must be passed to the network. Population forecasts are usually made for every five years, and as such the intermediate years would need to be either linearly interpolated or estimated. These increased assumptions would reduce the accuracy of the ANN forecast. However, for short-term forecasts, ANNs would be more accurate. To forecast for the next five years, for example, the ten years of input data would include the five most recent years of known data along with five years of forecast input data. Moreover, the majority of the models developed using less than the full 10 years of data were found to underperform models developed using the full ten years of data.

Flow rates were forecast for future scenarios, taking future climate change factors, population change and economic changes into account. A comparison of the change in power generation at valves V2 and V7 are detailed in Table 5.11. Both valves were estimated to see power generation increases by 2020, due to increased flow rates. For the V2 case, this flow increase was primarily due to the ESRI forecast increase in housing construction for the Dublin region. For valve V7, flow rates were found to be linked to the leakage rate in DCC mains, and decreases in the level of unemployment in Dublin. With unemployment rates in Ireland predicted to decrease to 7.5% by 2020, average temperatures to increase as forecast by Sweeney (2001) and assuming DCC leakage rates are reduced to 20% by 2020, the average quarterly flow rate at V7 was forecast to increase.

Table. 5.11: Power generation variation from 2003 to 2020 forecast

	2003	2008	2012	2020
V2: Brunswick St PRV	8.24	21.06	16.73	43.45
V7: Poplar Row PRV	15.86	19.7	17.32	20.12

The predicted power generation values for 2020 as presented in Table 5.11 were calculated based on the average pressure drop across the PRVs at V2 and V7. However, the input pressure to these PRVs would also vary with the forecast increase in flow rates. As the flow rate increases, the headloss across the pipes upstream of these PRVs would also increase. According to the Hazen-Williams approximation for headloss, h_f:

$$h_{\mathrm{f}} = 10.67 \frac{Q^{1.85}}{C^{1.85}.D^{4.87}} L \tag{5.6.3}$$

where C is the Hazen-Williams friction coefficient, which indicates the roughness of the interior surface of a pipe (Savić and Banyard, 2011). Assuming the pipe length, roughness and diameter remain the same in 2020, the headloss will increase by $(\Delta Q)^{1.85}$.

This increase in headloss will accumulate along all pipes prior to the V2 an V7 PRV locations. Furthermore, with the increasing headloss due to the larger flow rates, the output pressure specified by the PRV may need to be higher to ensure pressure requirements are met downstream of the PRVs. In order to include this potential reduction in the pressure drop at these PRVs, future power generation estimates were calculated for five different future pressure scenarios: varying from a 10% to a 50% decrease in the pressure drop across the turbine from the 2012 average value. These resulting power outputs are shown in Table 5.12.

Table. 5.12: Forecast power generation in 2020 under varying pressure scenarios

Turbine Pressure Decrease (%):	10%	20%	30%	40%	50%
V2: 2020 Power Output (kW)	39.10	34.76	30.41	26.07	21.72
V7: 2020 Power Output (kW)	18.11	16.1	14.09	12.07	10.06

One limitation of this analysis was that the population statistics employed were for the regional local authority within which each of the valves were located. However, in practice, many of these valves would supply a smaller local area. Water companies would have more accurate data available regarding the number of connections at each DMA, and the breakdown between domestic and industrial users. This data could be used to more accurately model the population supplied and hence the demand predicted through each valve.

5.7 Summary

Both MLR and ANN analyses were applied and provided methods of forecasting average quarterly flow rates at valves within water supply networks. MLR analysis provided an indication of the level of correlation between water flow rates and the independent variables tested. The most significant relationships, common to both the Irish and Welsh data, was the relationship between water flow rates and the local population. Water price was shown to have a significant influence on flow rates at 3 of the 4 WW valves studied. It is recommended that MLR be used during feasibility studies of new hydropower projects planned within distribution networks. MLR has been shown to provide an indication of long term changes in flow rates, due to economic and climatic changes. The MLR models can be used to forecast future average flow rates at potential hydropower locations. This forecasted average flow rate combined with the application of the characteristic average diurnal water demand pattern for that valve, will allow for accurate forecasting of flow rates and hydropower generation capacities into the future. A number of different future scenarios could be tested as part of the sensitivity analysis, such as large population growth or large increases or decreases in unemployment.

For all future scenarios modelled, flow rates were predicted to increase by 2020, largely due to the forecast increase in the populations of both Ireland and Wales. Growth in flow rates would increase the hydropower generation capacity at these sites. However, with large increases in water demand, new water resources may need to be developed, which may render certain parts of the network obsolete. This risk should also be noted at the feasibility stage of any hydropower project. Overall, MLR and ANN analyses were found to be good methods for approximation of flow rates at potential hydropower locations within WSNs. MLR is the most accessible and requires less assumptions for long term forecasting and is therefore recommended as the preferred option for long term flow rate forecasts (10 years plus).

CHAPTER 6

Optimisation

6.1 Introduction

The previous chapters discuss the potential for hydropower energy recovery at existing infrastructure within WSNs, such as at pressure reducing valves, reservoirs and break-pressure tanks. This Chapter however is focused on the selection of a new point to install a turbine within a WSN. Optimisation techniques are applied and tested for the identification of an optimum point to install a turbine within a distribution network, given the network layout and its water demands.

Optimisation is a process of maximising or minimising a desired objective function while ensuring some required constraints are met. It is widely used in the design and decision making process of many disciplines and businesses, such as in manufacturing processes to reduce materials used, minimise energy requirements etc. Optimisation methods can be employed by water service providers in the feasibility and design stage to assist with the decision making process. Examples of the application of optimisation models to water supply network design include the choice of the least cost combination of pipes or to minimise energy usage by pumps.

In this chapter, an option for the application of optimisation techniques to select optimal locations to install new pressure reducing hydropower turbines in a water supply networks is presented. A description of the formulation of the optimisation problem to be solved is provided along with an overview of the hydraulic theory, the optimisation methods and solvers employed for these analyses. Three optimisation methods were tested including linear programming techniques and a genetic algorithm.

Optimal design of water supply networks has been widely researched, however new algorithms continue to be developed, tested and reported on. Traditional mathematical optimisation methods such as linear programming have been used to optimise the installation of pressure control for leakage reduction (Jowitt and Xu, 1990; Sterling and Bargiela, 1984), optimise pipe diameters in a WSN (Alperovits and Shamir, 1977) and to optimise pumping schedules to refill reservoirs for minimised energy usage and costs (Jowitt and Germanopoulos, 1992). A critical review of key research in this field was presented in Chapter 2.

This chapter investigates the application of optimisation techniques, both traditional mathematical and also heuristics based approaches, to find the optimal location to install hydropower turbines within water supply networks as both an energy recovery and pressure management measure.

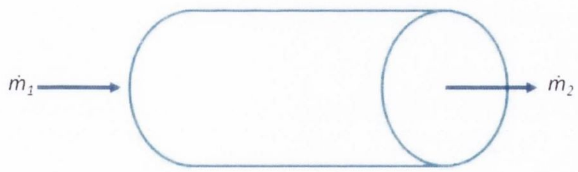
6.2 Principles of Fluid Flow

The optimisation model presented in this chapter was applied to a water supply network, and as such required computational representation of fluid flow in pipes. In this section the hydraulic theory required for the formulation and implementation of this optimisation model is presented.

6.2.1 Conservation of Mass

The law of conservation of mass states that mass can neither be created nor destroyed. Applying this law to a control volume, such as a section of a pressurised pipe, where the mass in the pipe section does not change:

Assuming that water is incompressible, the law of the conservation of mass can be ap-



plied to these volumes or discharges, hence,

$$Q_1 = Q_2 \quad \text{or} \quad v_1 A_1 = v_2 A_2 \tag{6.2.1}$$

Where Q is the volumetric flow rate (m^3/s), v is the velocity (m/s) and A is the pipe section area (m^2). This equation is also referred to as the *continuity equation*.

6.2.2 Conservation of Energy

Water flowing in water supply mains typically comprises of three main forms of energy: potential, pressure and kinetic energy.

$$z + \frac{p}{\rho g} + \frac{v^2}{2g} \tag{6.2.2}$$

In water distribution systems, water flow velocities rarely exceed 1-2m/s, hence velocity heads are small, negligible in comparison with pressure heads. For this reason, the velocity head is often ignored for optimisation modelling to simplify the process (Savić and Banyard, 2011; Bragalli et al., 2012).

6.2.3 Head Loss

Pressure head will be lost along a pipe network through both frictional head loss and local head loss. Local head loss describes pressure head lost due to bends in pipes, or the presence of pipe fittings or valves. Frictional head loss will occur along a pipe, due to friction on the pipe surface, dependent on the pipe material, roughness, diameter, the fluid velocity, type of fluid in the pipe, etc. Key research and theory development for the analysis of fluid flow in pipes is summarised in Table 6.1.

Table. 6.1: Pipe flow theory development Chadwick et al. (2004)

Date	Name	Contribution
1839-41	Hagen and Poiseuille	Laminar flow equation
1850	Darcy and Weisbach	Turbulent flow equation
1884	Reynolds	Distinction between laminar and turbulent flow - Reynolds' Number
1913	Blasius	Friction factor equation for smooth pipes
1914	Stanton and Pannell	Experimental values of the friction factor for smooth pipes
1930	Nikuradse	Experimental values of the friction factor for artificially rough pipes
1930s	Prandtl and von Karman	Equations for rough and smooth friction factors
1937-39	Colebrook and White	Experimental values for the friction factor for commercial pipes and the transition formula
1944	Moody	The Moody diagram for commercial pipes
1958	Ackers	The Hydraulics Research Stations Charts and Tables for the design of pipes and channels
1975	Barr	Direct solution of the Colebrook-White equation

There are a number of equations that have been developed to describe frictional head loss along a pipe. Early experimental work carried out by Hagen and Poiseuille between 1839 and 1841 lead to the development of an approximation for head loss for laminar pipe flow. This equation for head loss h_f is:

$$h_{\mathrm{f}} = \frac{32\mu LV}{\rho g D^2} \tag{6.2.3}$$

Reynolds' experiments in 1884 demonstrated that there were two main types of flow, laminar and turbulent. Through further experimental testing, he found a distinction between frictional head loss for laminar flow and for turbulent flow. The frictional headloss in a pipe with laminar flow was found to be proportional to the velocity, while for turbulent flow it was found to be proportional to the square of the velocity (Chadwick et al., 2004).

The Darcy-Weisbach equation developed in 1850 relates frictional headloss along a pipe to the average velocity of the fluid flow:

$$h_{\mathrm{f}} = \frac{\lambda LV^2}{2gD} \tag{6.2.4}$$

where λ is the non-dimensional friction factor. For turbulent flow (Re > 4000) the Colebrook-White formula can be used to solve for λ (Savić and Banyard, 2011). This formula relates the friction factor λ to k the roughness coefficient, and the Reynolds number, Re:

$$\frac{1}{\sqrt{\lambda}} = -2log\left(\frac{k}{3.7D} + \frac{2.51}{Re\sqrt{\lambda}}\right) \tag{6.2.5}$$

The most accurate head loss formula is a combination of the Darcy Weisbach and the Colebrook-White equation. However, these equations can be expensive computationally. For this reason, the empirical formula developed by Hazen-Williams is more commonly applied for optimisation models and is widely used by water supply engineers. This equation also describes head loss across pipes, and is less complicated and expensive to model computationally.

$$h_f = 10.67\frac{Q^{1.85}}{C^{1.85}.D^{4.87}}L \tag{6.2.6}$$

where C is the Hazen-Williams fricton coefficient, which indicates the roughness of the interior surface of a pipe (Savić and Banyard, 2011).

6.3 Pressure Regulation

Pressure regulation is a top priority of WSPs, both to reduce leakage and also to reduce excess energy used to pump water. The selection of minimum pressure criteria within district metered areas (DMAs) is usually based on the demand requirements of the area, the elevation in the area and the pressure limits of the pipes. Minimum pressure requirements for water at water taps are specified by regulators. In the UK, water pressure at water taps in all homes and businesses, must be at a minimum required pressure of 7m as defined by OFWAT the UK water supply regulators (OFWAT, 2014). Pressure levels are usually managed through the installation of control valves, PRVs or BPTs as discussed in Chapter 1 and Chapter 2.

In previous optimisation research applied to the optimal location and setting of PRVs, mimimum pressure constraints were assigned at all nodes, or at certain pressure reference nodes in the networks analysed.

6.4 Mathematical Formulation

The optimisation model was formulated such that it output the same results that a hydraulic solver such as EPANET would output: the flow rates in each pipe, the pressure heads at each node, over a fixed time period. This allowed the optimisation results to be verified hydraulically with a corresponding EPANET hydraulic analysis. The other key output of the optimisation model is the optimum location for installation of a hydropower or multiple hydropower turbines. This turbine could then be represented in EPANET as a fixed output PRV, and the hydraulics could be updated for comparison.

6.4.1 Objective Function

The choice of objective function was the primary decision made prior to the optimisation model development. A number of potential options were considered. Firstly, the option to minimise the pressure throughout the network, similar to an optimal location of a PRV problem was considered. This could be achieved by minimising the average service pressures across all pipes (links). This objective however, should result in the same optimal locations as a PRV problem. Another objective considered was to maximise the power output generated in the network. This objective would be dependent on both the pressure and the flow rate at potential locations for turbine installation, which could lead to the selection of different optimal points than the pressure minimisation objective. These locations would be optimal hydropower locations as opposed to optimal pressure reduction locations. This second objective was therefore selected, as it would seek to generate the most electricity as the primary objective as opposed to pressure reduction, whilst it would also perform the pressure management task by reducing the network pressures to meet the desired pressure constraints.

Further potential objective functions were considered for optimisation, but have not been included in this analysis. These included the minimisation of the total costs. This would require more detailed information on the costs involved at each different potential installation point, such as the distance from the electric grid, site access constraints, pipe diameter and material. The net cost-benefit would be minimised, which would comprise of the total estimated installation costs minus the annual revenue generated by the installed hydropower turbine over a fixed number of years (e.g. 10 years). This could result in a different optimal location to install than the purely maximised power generation objective. It could also be applied following the selection of two or more optimal locations for maximised power generation, to select the most cost-effective option of them. Another objective similar to this that was considered was the minimisation of the net payback period. Similar input data would be required, however the objective function would be formulated differently.

The primary objective function as defined for this model was to maximise the total net power generated when hydropower turbines were installed in a water supply network in order to reduce network pressures to specified target pressure levels.

Through the introduction of an additional binary variable, the total number of turbines to install could be constrained. The optimisation model was then designed to select optimal locations to install turbines such that power generation is maximised. The power formula is a function of both water pressure and flow rate:

$$\sum_{t=1}^{T} P_{\text{output}} = \rho g \sum_{t=1}^{T} Q_{\text{i,j}} D_{\text{i,j}} e_0 \tag{6.4.1}$$

Where T is the number of turbines, $Q_{i,j}$ is the flow rate across that pipe length i,j, $D_{i,j}$ is the head drop across the turbine, e_0 is the turbine efficiency which would vary dependent on the flow rates.

6.4.2 Decision Variables

The decision (or design) variables are the unknowns that the optimisation model is required to calculate. For this optimisation problem the decision variables were defined as: $Q_{i,j}$, P_n and $D_{i,j}$. Where $Q_{i,j}$ was the flow in each link (pipe), k, between nodes i and j, P_n is the total hydraulic head at each node n and $D_{i,j}$ is the optimal pressure drop across an installed turbine along link i,j. For all links k = 1,.... K and for all nodes n = 1..., N. Where K is the total number of links and N is the total number of nodes in the WSN.

For a WSN with known layout and demands, as described above, the optimal WSN design for hydropower turbine inclusion is to position a turbine at a location where the most power can be generated. Certain constraints must also be met, such as maintaining adequate service water pressures. This optimal design is also subject to the following constraints:

6.4.3 Equality Constraints

Equality constraints were also defined for this formulation. These constrained certain decision variables to be equal to known network data, such as nodal demands. There are a number of equality constraints required in this analysis. For each node, the flow into and out of each node was represented using Equation 6.4.2. This is in accordance with the continuity equation as described in Section 6.2.1. The sum of the flows into each node must equal the sum of the flows out of that node. There are N equations, to represent the flow in and out of each node $n = 0, 1, \ldots N$.

$$\sum Q_{\mathrm{in,n}} - \sum Q_{\mathrm{out,n}} = Demand_{\mathrm{n}} \tag{6.4.2}$$

The hydraulic head as described in Section 6.2.2, is the total energy per unit weight of the water, and is expressed in terms of height. The head loss between nodes i and j is:

$$H_{\mathrm{i}} - H_{\mathrm{j}} = h_{\mathrm{i,j}}. \tag{6.4.3}$$

where H_i and H_j are the hydraulic heads at node i and j respectively, and $h_{i,j}$ is the head loss between nodes i and j.

According to Bernouillis equation the hydraulic head is the sum of the pressure head, elevation head and velocity head. The velocity head (kinetic energy) in water distribution mains can be ignored as it is negligible when compared to the elevation and pressure head (Bragalli et al., 2012). The hydraulic head is then therefore a combination of the pressure head p_i and the elevation head e_i.

$$(p_{\mathrm{i}} + e_{\mathrm{i}}) - (p_{\mathrm{j}} + e_{\mathrm{j}}) - h_{\mathrm{i,j}} = 0 \tag{6.4.4}$$

With the addition of a turbine at a node the equation then becomes:

$$(p_{\mathrm{i}} + e_{\mathrm{i}}) - (p_{\mathrm{j}} + e_{\mathrm{j}}) - D_{\mathrm{i,j}} - h_{\mathrm{i,j}} = 0 \tag{6.4.5}$$

Where $D_{i,j}$ is the pressure drop across a turbine installed along the link or pipe length i,j, for all pipes, $k = 1, \ldots K$, in the network. To account for the flow direction between nodes, the sign of the flow rate was included in this conservation of energy constraint, according to:

$$Sign(Q_{i,j}) * ((p_i + e_i) - (p_j + e_j) - D_{i,j}) - h_{i,j} = 0 \tag{6.4.6}$$

6.4.4 Inequality Constraints

Inequality constraints are applied to constrain certain decision variables to be less than or greater than specified values. The primary inequality constraints for this formulation related to the setting of maximum and minimum pressure limits for each node n (i.e. no negative pressure):

$$P_n > 0 \tag{6.4.7}$$

As part of a pressure management strategy, the minimum required service pressure and maximum allowable pressure should be maintained at each node $n = 1,...N$.

$$P_{min} < P_n < P_{max} \tag{6.4.8}$$

A final constraint was added to limit the total number of turbines, T to install in the WSN.

$$\sum_{t=1}^{T} T < Max_{turbines} \tag{6.4.9}$$

The addition of this integer decision variable would require the use of an optimisation solver capable of handling both continuous and integer variables, such as a mixed integer non-linear programming (MINLP) solver.

6.5 Optimisation methods

In this section, a description of the three different optimisation methods that were applied in this research are presented. First of all, the chosen objective function to be maximised as discussed in Section 6.4.1 is a non-linear, non-convex function. Convex functions such as $f(x) = x^2$ (Figure 6.1) and $f(x) = e^x$ have one global optimal solution. Non-convex functions can have many locally optimal solutions. Ensuring a solution is a global optimum can be achieved through careful formulation of the problem, through the tightening of constraints and bounds on the decision variables and through a trial and error process to find a good initial start point, x^0, for the model.

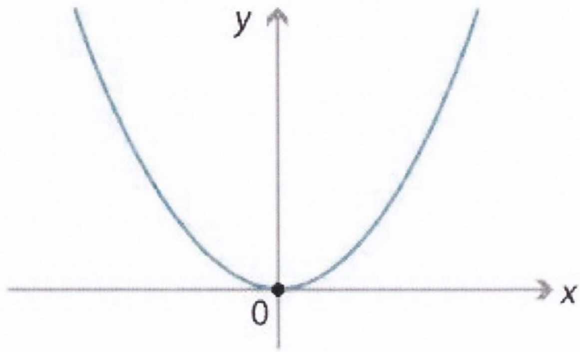


Fig. 6.1: Graph of $f(x) = x^2$

6.5.1 Non-linear Programming

Constrained non-linear programming is a method of minimising or maximising a given objective function while meeting some defined constraints. The minimisation function would be structured as follows:

$$
\begin{aligned}
\text{minimise} \quad & f(x) \\
\text{subject to} \quad & A.x \leq b \\
\text{and} \quad & Aeq.x = beq \\
\text{and} \quad & x^{\mathrm{L}} \leq x \leq x^{\mathrm{U}} \\
\text{and} \quad & c(x) \leq 0, \\
\text{and} \quad & ceq(x) = 0
\end{aligned}
\tag{6.5.1}
$$

where $x = (x_1, x_2, \ldots, x_n)$ is a column vector of n real-valued design variables. f is the objective function, $A.x$ is a matrix of n inequality constraints, $Aeq.x$ is a matrix of m equality constraints, $c(x)$ is a matrix of non-linear inequality constraints and $ceq(x)$ is a matrix of non-linear equality constraints. Notation such as x^0 for the starting point and x^* for the optimum point are generally used. Furthermore, to change the objective function from a minimisation to a maximisation, the maximisation of *f(x)* is equal to the minimisation of *-f(x)* (Belegundu and Chandrupatla, 2011).

6.5.2 Mixed Integer Non-linear Programming

The addition of a constraint on the number of turbines to install complicates the model slightly. The selection of the location to install a turbine requires the addition of a binary decision variable (1/0), for whether to install a turbine at each node or not. The other decision variables, flow rate and pressures, are continuous variables.

$$\begin{aligned} \text{minimise} \quad & f(x) \\ \text{subject to} \quad & A.x \leq b \\ \text{and} \quad & Aeq.x = beq \\ \text{and} \quad & x^{\mathrm{L}} \leq x \leq x^{\mathrm{U}} \\ \text{and} \quad & c(x) \leq 0, \\ \text{and} \quad & ceq(x) = 0 \\ \text{where} \quad & x_{\mathrm{a}} \in \mathrm{Z} \\ \text{and} \quad & x_{\mathrm{b}} \in [0,1] \end{aligned} \tag{6.5.2}$$

Where the decision vector x is made up of x_a continuous variables and x_b binary variables $\in [0,1]$.

6.5.3 Genetic Algorithm

A GA is an optimisation algorithm that uses a search process based on natural evolution theory. In recent years, GAs have been identified as good alternatives to classical deterministic optimisation approaches. A GA begins with a randomly generated initial population and applies three operators, the selection, crossover and mutation operators to find the optimal model. The application of GAs for optimal design of WSNs has been described in the literature review of Chapter 2. One drawback with GAs is that they cannot guarantee a global optimum. The solver may stop when a 'better' solution is found in comparison to the previous solutions, however this solution may not be an optimal solution. The primary drawback with GAs however lies in the high computational requirements needed for a solution to reach optimal or near-optimal.

The GA provided by the Matlab Optimisation Toolbox was employed in this analysis. This algorithm begins by creating a random initial population, it then creates a sequence of new populations. At each step, the algorithm uses the individuals in the current generation to create the next population. To create the new population, the algorithm performs the following steps (Matlab, 2014*b*):

1. Scores each member of the current population by computing its fitness value.
2. Scales the raw fitness scores to convert them into a more usable range of values.
3. Selects members, called parents, based on their fitness.
4. Some of the individuals in the current population that have higher fitness values are retained and passed to the next population. They then produce children from the parents. Children are produced either by making random changes to a single parent-mutation or by combining the vector entries of a pair of parents-crossover. Members with weaker fitness are discarded.
5. Replaces the current population with the children to form the next generation.

The algorithm stops when one of the stopping criteria is met. Stopping criteria can be specified within the Matlab file and passed on to the optimisation solver. Stopping criteria include the specification of maximum number of generations the algorithm should undertake, or a maximum time limit for the algorithm to run, or setting a fitness limit where the algorithm stops when the value of the fitness function for the best point in the current population is less than or equal to this fitness limit, amongst others. The minimisation function of a GA, similar to the previous approaches, is structured as follows:

$$\begin{aligned} \text{minimise} \quad & f(x) \\ \text{such that} \quad & A.x \leq b \\ \text{and} \quad & Aeq.x = beq \\ \text{and} \quad & x^{\mathrm{L}} \leq x \leq x^{\mathrm{U}} \\ \text{and} \quad & c(x) \leq 0, \\ \text{and} \quad & ceq(x) = 0 \end{aligned} \tag{6.5.3}$$

Where $A.x \leq b$ are the linear inequality constraints, $Aeq.x = 0$ are the linear equality constraints, x^L and x^U are the lower and upper bounds on x, $c(x)$ are the non-linear inequality constraints, $ceq(x)$ are the linear equality constraints.

6.6 Optimisation Solvers

In this section, the methods of optimisation employed in this analysis are described. Three optimisation methods and hence solvers were tested. The first was using a non-linear programming (NLP) solver. The second was using a genetic algorithm (GA). The third method was using a mixed integer non-linear programming (MINLP) solver. All of these optimisation problems were formulated and implemented in Matlab. The NLP solver and the GA solver from the Matlab Toolbox were applied. An open-source MINLP solver was executed in Matlab via the Opti Toolbox (Currie and Wilson, 2012).

As the objective function to be maximised is non-linear, initially the problem was solved using the NLP solver Fmincon from the Matlab Optimisation Toolbox (version 8.2). Fmincon finds a minimum (or maximum) of a constrained non-linear multi-variable function. This solver will only allow the inclusion of continuous decision variables and so the final constraint (Equation 6.4.9) on the number of turbines to install could not be applied.

The GA solver provided in the Matlab optimisation toolbox does not permit the use of both equality constraints and integer decision variables, therefore a problem formulation similar to that developed for the NLP analysis was employed, assuming a turbine was installed at all nodes.

Due to the non-linear nature of the objective function and the presence of both continuous and integer decision variables, MINLP was selected as the most suitable mathematical programming method to apply to this problem. As has been applied in recent research, (Eck and Mevissen, 2012; Bragalli et al., 2012), the BONMIN (Version 1.7.4) solver (Basic Open-source Non-linear Mixed Integer programming) was employed for this analysis. This is an open source code for solving general MINLP problems. The NLP branch-and-bound algorithm was selected (BONMIN B-BB). This is a simple branch-and-bound (BB) algorithm based on solving a continuous non-linear program at each node of the search tree and branching on variables (Bonami et al, 2008). The different methods that BONMIN implements are exact algorithms when the objective function and the constraint function are convex, but are only heuristics for a non-convex problem as in this highly non-linear case.

6.7 Water Supply Network Model

A network model is a computational model of a water distribution network. These are used in the planning, engineering, operations and management of water utilities (Mays, 2000) and are crucial for the smooth running of an urban water network. Many commercial software packages exist. A network modelling package commonly used by local authorities, in Ireland and abroad, is the freely available US Environmental Protection Agency Network (EPANET) software (Rossman, 2000).

A theoretical network model of the WSNs studied was created using EPANET. This software performs extended-period simulations of hydraulic and water quality behaviour within pressurised pipe systems. The pipe network to be analysed would consist of pipes, nodes, pumps, valves, storage tanks and reservoirs. EPANET then tracks the flow of water in each pipe, pressure at each node, height of water in each tank and also the concentration of different chemical species throughout the network during each simulation. The solver employs the gradient method to hydraulically balance the network. The EPANET options for approximation of headloss are either the Hazen-Williams or the Darcy-Weisback approach. The Hazen-Williams approximation was selected for all of the EPANET hydraulic analysis.

Results from the EPANET hydraulic analyses were used for comparison with the optimisation output to check the accuracy of the optimisation algorithm developed. In order to model the presence of a turbine in the network, a PRV with a fixed output was installed in the EPANET hydraulic model.

6.8 Case Studies

The discussed optimisation model was applied to a number of sample WSNs. Initially it was trialled on a theoretical 5-Node WSN, then it was applied to a benchmark 2-Loop WSN as reported on in literature (Alperovits and Shamir, 1977), and finally it was applied to the benchmark 25-Node Network as discussed in Chapter 2 and reported on by many for optimal pressure management and to find optimal locations of PRVs (Sterling and Bargiela, 1984; Eck and Mevissen, 2012; Giugni et al., 2014).

6.8.1 5-Node Water Supply Network

The first network to be analysed was a theoretical 5-Node WSN shown in Figure 6.2. This network consisted of five pipes, four demand nodes and one reservoir of constant head. Further details of the pipes and junction input data are provided in Table 6.2. The pressure bounds were constrained to be within a minimum set pressure of 15m and a maximum pressure of 35m at all nodes. This small simple network, with one source and one loop, was used initially to test a number of constraints and optimisation options. Three methods were applied to optimise this network, NLP, a GA and MINLP. Two head loss approximations were also tested, the Hagen-Poiseuille and the Hazen-Williams for comparison.

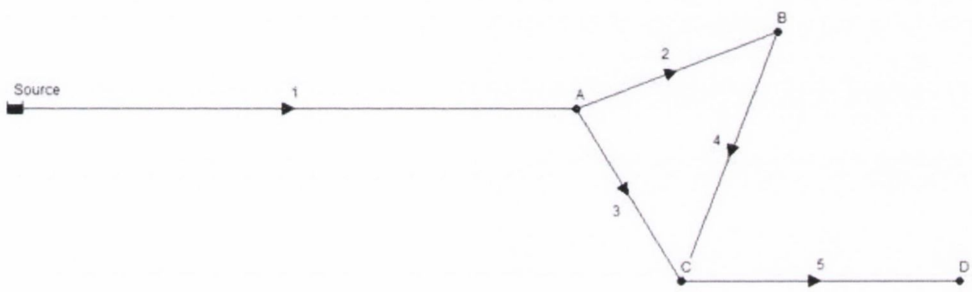


Fig. 6.2: 5 Node WSN: Network layout

Table. 6.2: 5-Node Theoretical Network: Input data

Pipes					Junctions		
ID	Node1	Node2	Length (m)	Diameter (mm)	ID	Elevation (m)	Demand (m^3/hr)
1	Source	A	1000	200	A	12	41
2	A	B	800	150	B	8	34
3	A	C	1200	200	C	9	55
4	B	C	1000	150	D	6	23
5	C	D	1000	150			

6.8.2 2-Loop Water Supply Network

The second case WSN the optimisation model was applied to was a sample 2-Loop WSN as has been tested and reported on in literature. The 2-Loop WSN consisted of a reservoir and six junctions (nodes) connected by eight pipes (links). Figure 6.3 shows a schematic of this gravity fed network. Table 6.3 provides further details of the network input data, such as pipe lengths, diameters and water demand at each node. Pipe diameters were assumed from the results of the Alperovits and Shamir (1977) optimisation model, which was to select the least cost set of pipe diameters that would meet network demands and pressure constraints. The source (Res) was assumed to have a constant fixed head of 210m. This network data was input to both EPANET and to the optimisation models.

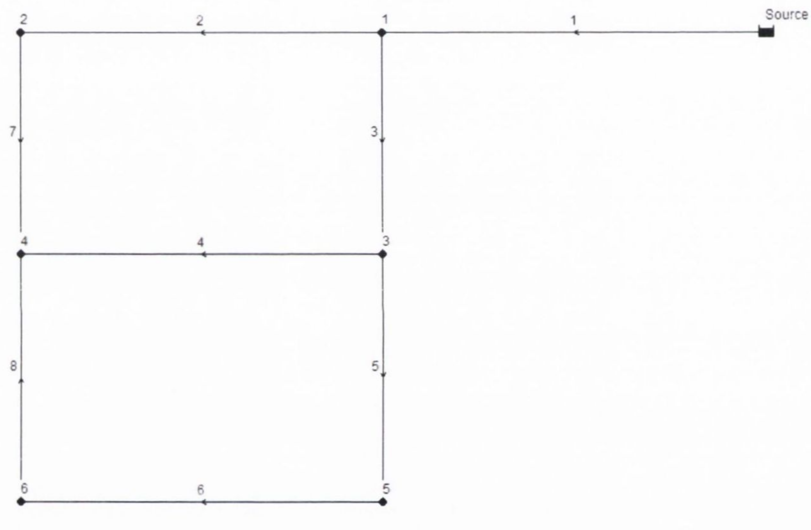


Fig. 6.3: 2-Loop WSN layout

The pressure at each node of the 2-Loop network was constrained to be less than 35m. The lower bound on the nodal pressure was fixed at 15m to ensure a sufficient service pressure at each node.

Table. 6.3: 2-Loop Benchmark Network: Pipe and Junction data

Pipes					Junctions		
ID	Node1	Node2	Length (m)	Diameter (mm)	ID	Elevation (m)	Demand (m^3/hr)
1	Res	A	1000	457.2	A	150	100
2	A	B	1000	203.2	B	160	100
3	A	C	1000	457.2	C	155	120
4	C	D	1000	152.4	D	150	270
5	C	E	1000	406.4	E	165	330
6	E	F	1000	254	F	150	200
7	B	D	1000	152.4			
8	F	D	1000	152.4			

6.8.3 25-Node Benchmark Water Supply Network

Following the analyses of the two small WSNs described previously, the benchmark 25-Node network (Sterling and Bargiela, 1984; Nicolini and Zovatto, 2009; Eck and Mevissen, 2012; Giugni et al., 2014) was then analysed. This network layout is presented in Figure 6.4. Further network details, such as junction demands, pipe lengths, diameters, and Hazen-Williams friction coefficients are presented in Appendix D. Due to the poor performance of the Hagen-Poiseuille headloss approximation on the analysis of the 5-Node network, it was decided to use the more accurate Hazen-Williams headloss approximation for the more complex 25-Node WSN. It was also decided to solve the optimisation problem using a combination of NLP and MINLP. The GA option was disregarded due to its longer solving time, combined with its less accurate approximations of flow rates as reported for the 5-Node WSN.

In previous research, using this benchmark network, a number of different pressure reduction strategies have been proposed. The earliest appearance of this network was by Sterling and Bargiela (1984). This research proposed the installation of three control valves on pipes 11, 21 and 29. Upper pressure bounds were fixed for nodes 6, 13, 18 and 22, which are the nodes with the highest ground levels. The minimum service pressure to be maintained at these nodes was fixed at 30m. Jowitt and Xu (1990) selected the same control valve locations as Sterling and Bargiela (1984). The pressure reference nodes selected were different however, these were at nodes 13, 19, 21 and 22. Again, the minimum acceptable pressure for these reference nodes was fixed at 30m above ground level. In the Giugni et al. (2014) analysis, a minimum pressure of 25m was specified for all nodes. A summary of research using this benchmark 25 Node network, from 1984 to today, is presented in Table 6.4.

Another objective that has been optimised in previous research for this 25-Node network was the minimisation of the total volume of leakage in the network by optimising valve locations and valve controls. This volume of leakage was calculated according to:

$$\sum_{t=1}^{N} QS_{\mathrm{ij}} = CL_{\mathrm{i,j}}.L_{\mathrm{i,j}}.(P_{\mathrm{i,j}})^{1.18} \tag{6.8.1}$$

Where QS is the water leakage volume occurring in pipe element of length $L_{i,j}$ spanning nodes i and j; $CL_{i,j}$ is a coefficient that relates the leakage per unit length of the pipe to service pressure and depends on the system characteristics (e.g. the age and deterioration of the pipe and the soil properties etc). $P_{\mathrm{i,j}}$ is the average service pressure across the pipe which can be approximated by the average pressure at each of the nodes i, j. In previous research for this network, the coefficients $CL_{\mathrm{i,j}}$ relating the leakage to the service pressure for all pipes was assumed as 10^{-5}, leading to 16% leakage under controlled conditions (Jowitt and Xu, 1990; Nicolini and Zovatto, 2009; Giugni et al., 2014).

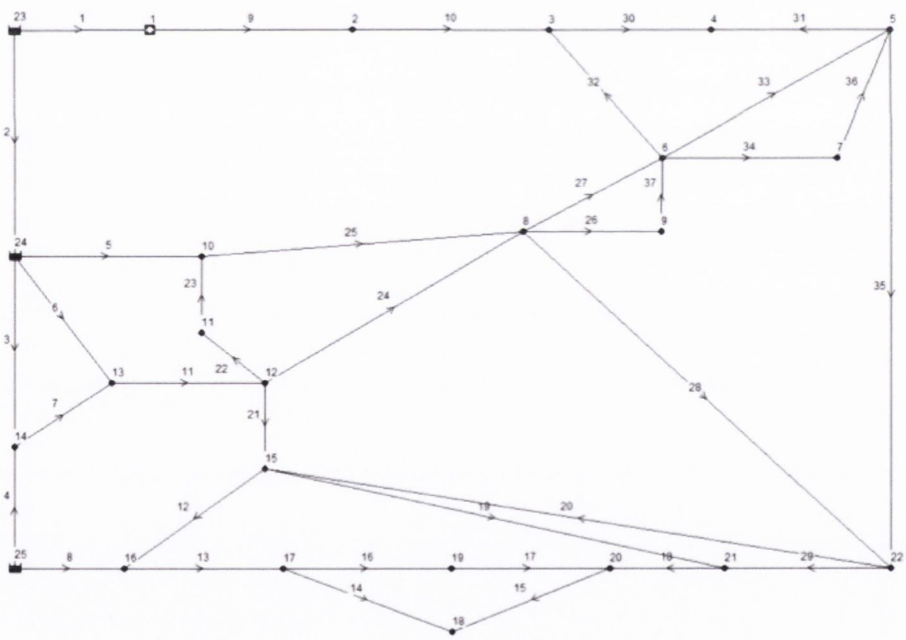


Fig. 6.4: Benchmark 25-Node network layout (Sterling and Bargiela, 1984; Nicolini and Zovatto, 2009; Eck and Mevissen, 2012)

Table. 6.4: Benchmark 25-Node WSN: Results from previous analyses

Author	Objective	Technique	Optimal PRV/Turbine Locations
Sterling and Bargiela (1984)	Optimise valve settings in order to minimise pressures for leakage reduction	Linear programming - sparse revised Simplex method	assumed location of control valves at **Pipes 11, 21, 29**
Jowitt and Xu (1990)	Minimise total system leakage	Linear programming	**Pipes 11, 21, 29** (assumed prior to solving, as this optimisation was to find optimal settings of PRVs given their location)
Araujo et al. (2006)	Minimise total no. of PRVs to install and minimise network pressures	Genetic algorithm	For 2 PRVs: **Pipes 1 and 11;** For 3 PRVS: **Pipes 11, 21, 29** (These were selected based on Sterling and Bargiela (1984); Jowitt and Xu (1990), et al.)
Nicolini and Zovatto (2009)	Minimise total no. of PRVs and to minimise the total system leakage	Multi-objective genetic algorithm	1 PRV: **Pipe 11**; 2 PRVs: **Pipes 1 and 11**; 3 PRVs: **Pipes 1, 11, 20**
Fontana et al. (2012)	Minimise total system leakage, optimal location and setting of PRVs	Multi-objective genetic algorithm	**Pipes 1, 11, 20**
Eck and Mevissen (2012)	Minimise the sum of the nodal pressures	MINLP	**Pipes 1, 5, 11**
Giugni et al. (2014)	Minimise nodal pressures	Genetic algorithm	1 PRV **Pipe 11**; 2 PRVs **Pipes 1, 11**; 3 PRVs **Pipes 1, 11, 21**;
Giugni et al. (2014)	Maximise power generation	Genetic algorithm	1 turbine: **Pipe 11**; 2 turbines: **Pipes 5 and 11**; 3 turbines: **Pipes 1, 5, 11**

This benchmark 25-Node network is fed from three sources, at nodes 23, 24 and 25. To simplify this analysis, the head was assumed as constant at each of these source nodes, as has been assumed in previous work (Giugni et al., 2014). In early research, the location to install three PRVs was assumed, then the optimisation algorithms were applied to optimise the settings of these valves. The location assumed for installation of these was at pipes 11, 21 and 29 (Sterling and Bargiela, 1984; Jowitt and Xu, 1990; Araujo et al., 2006). Later research, however, to find optimal locations to install PRVs has found other pipes more efficient at minimising pressures and leakage. These locations have been found on pipes 1,11,20 Nicolini and Zovatto (2009), pipes 1,5,11 Eck and Mevissen (2012) and pipes 1,11,21 Giugni et al. (2014). Optimal locations for turbine installation were found to be at pipes 1,5,11 by Giugni et al. (2014) using a GA.

6.9 Results

The results of each optimisation model are presented in the following tables. The running data including the number of variables, constraints and the solving time are summarised in Table 6.5. All of these computational analyses were performed on a machine with 4GB of RAM, a 32-bit operating system, a 2.83 GHz processor speed running Microsoft Windows 7.

Table. 6.5: Running Data, using Hazen-Williams Head Loss Approximation

Network	Model	Continuous Variables	Binary Variables	Constraints	Solution Time (s)
5-Node	NLP	14	0	13	0.56
	GA	14	0	13	341.31
	MINLP	14	5	19	0.92
2-Loop	NLP	22	0	20	1.06
	MINLP	22	8	110	25.72
25-Node	NLP	96	0	229	4.9 (min.) - 44.3 (max.)
	MINLP	96	37	120	61.06 (min.) - 144.59 (max.)

6.9.1 5-Node WSN Results

The results of an initial investigation on the use of two different headloss approximations, the Hagen-Poiseuille (H-P) and the Hazen-Williams (H-W) approximations, are presented in Table 6.6. Solution time was much quicker when using the linear H-P headloss approximation, especially for the GA solution. Solving time for the GA was found in 9.75 seconds using the H-P approximation as opposed to 341 seconds using the non-linear H-W headloss approximation. However, it was found that the reported flow rates using the H-P headloss approximation were much less consistent with the flow rates as reported by an EPANET hydraulic analysis. The relative error between these was as high as 117% for Link 4. While the resulting flow rates as reported by the optimisation model using the H-W headloss approximation were very consistent with the EPANET results, with errors ranging from -0.036% - + 0.003%.

Table. 6.6: 5-Node WSN: Flow Rate Results Comparison

Network	Link	H-P* Flow Rate (m^3/hr)	H-W** Flow Rate (m^3/hr)	EPANET Flow Rate (m^3/hr)	H-P* % Error	H-W** % Error
5 Node	1	153	153	153	0	0
5 Node	2	52	40.69	40.46	- 28.5	- 0.006
5 Node	3	60	71.3	71.54	+ 16	+ 0.003
5 Node	4	18	6.69	6.46	- 117	- 0.036
5 Node	5	23	23	23	0	0

* H-P = Hagen-Poiseuille head loss approximation
** H-W = Hazen-Williams head loss approximation

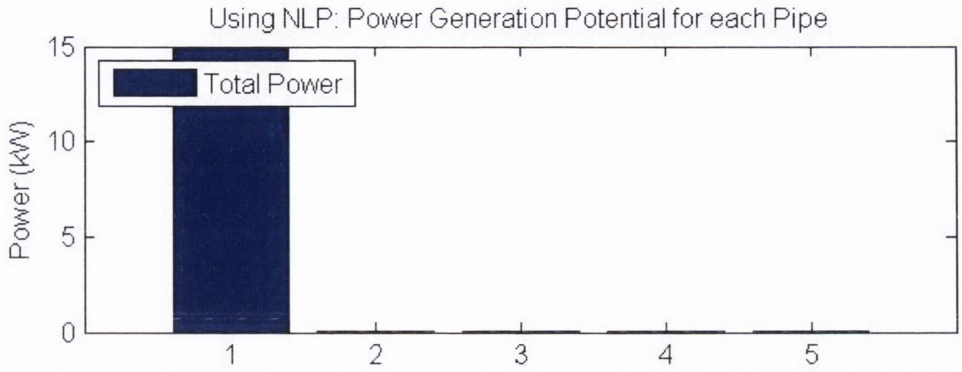
The results for the 5 Node theoretical WSN are detailed in Table 6.7. For the initial run, using the NLP solver, the MATLAB Fmincon solver, a turbine was set to be installed at all nodes. The optimal arrangement found link 1 to be the location with the most power generation potential at 14.99kW, with the other three locations all generated less than 0.00001 kWs. Despite the inability of the NLP solver to select just one location to install a turbine, it could be clearly identified which of the five pipes was the optimal point to install a turbine on for maximised power generation.

Table. 6.7: 5-Node WSN: Results Overview

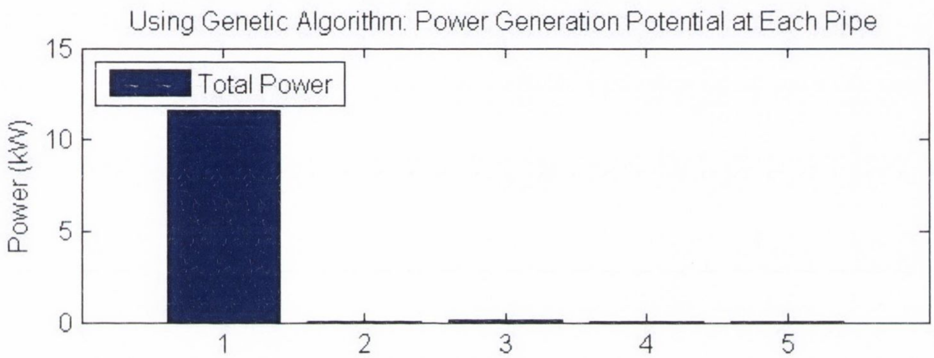
NLP				MINLP			GA		
Link	Flow (m^3/hr)	Head Drop (m)	Power (kW)	Flow (m^3/hr)	Head Drop (m)	Power (kW)	Flow (m^3/hr)	Head Drop (m)	Power (kW)
1	153	55.34	14.99	153	55.34	14.99	152.99	42.66	11.56
2	40.69	2.4E-11	0.0000	40.69	0		35.33	0.46	0.029
3	71.3	6.1E-12	0.0000	71.31	0		76.67	0.86	0.12
4	6.69	4.7E-12	0.0000	6.69	0		1.33	2.22	0.005
5	23	1.5E-12	0.0000	23	0		22.99	0.54	0.022
		Total:	14.99		**Total:**	14.99		**Total:**	11.74

The MINLP model was then applied and constrained such that only one turbine could be installed. The optimum link selected by the algorithm was at link 1, which again had a power generation capacity of 14.99kW. The GA solution, similar to the NLP model, found Link 1 to be the node with the most power generation potential, however it solved at a lower point at 11.56kW, and not at the optimum point. The total power generation potential at all nodes as found by the GA was 11.74kW. While the output variables of the GA solution were all within the problem bounds and constraints, it did not find an absolute optimum point.

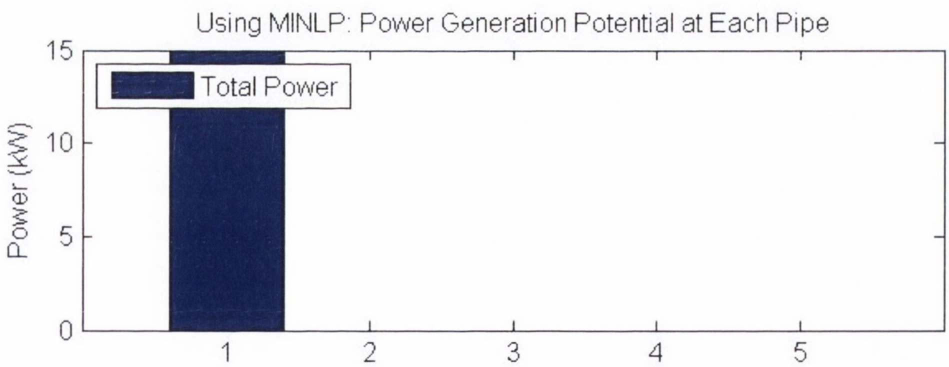
The output screens from each of these models are shown in Figure 6.5. For the 5-node network, NLP was found to be an efficient method to find an optimal point to install a turbine, despite not having the capability to model binary variables for the location choice. The solving time was the quickest at 0.56 seconds, and the result output was very close to the output of the MINLP model. The MINLP also found an optimal solution quickly in 0.92 seconds. It was decided to proceed with NLP and MINLP for the following cases studies, due to their accurate comparability with the EPANET hydraulics and also their quick solving time.



(a)



(b)



(c)

Fig. 6.5: Optimisation Model Output: 5-Node (a) NLP (b) GA (c) MINLP

6.9.2 2-Loop WSN Results

The results for the 2-Loop WSN using NLP are detailed in Table 6.8, including the pressure head at each node and across turbines as well as the estimated power output potentials for the two-loop network. For the initial run, using the Matlab NLP solver Fmincon, a turbine was set to be installed on all links. The optimised arrangement showed Link 1 to have the highest power generation potential at 37.73 kW. The NLP optimal set-up found negligible power generation on the other pipes in the network.

Table. 6.8: 2 Loop WSN: Results of NLP Optimisation

NLP				
Link	Flow (m^3/hr)	Nodal Head (m)	Head Drop (m)	Power (kW)
1	1120	40.88	12.36	24.52
2	185.99	17.26	1.03	0.34
3	834	31.98	0.00	0.00
4	117.87	15	0.00	0.00
5	596.13	18.25	0.00	0.00
6	266.13	15	0.00	0.00
7	85.99	NA	0.00	0.00
8	66.13	NA	2.46	0.29
			Total:	25.15

Using the MINLP model where the model was constrained such that only one turbine could be installed, a similar result was obtained for Link 1. Like the NLP model, Link 1 was found to have the most power generation potential, with an estimated power output of 36.8kW. The results for the 2 Loop WSN using MINLP are detailed in Table 6.9.

These results have shown that by selecting a suitable location to install a turbine within a water supply network, overall network pressures can be minimised to within specified pressure bounds. This reduction in pressure reduces the intensity of leakage and the frequency of burst pipes.

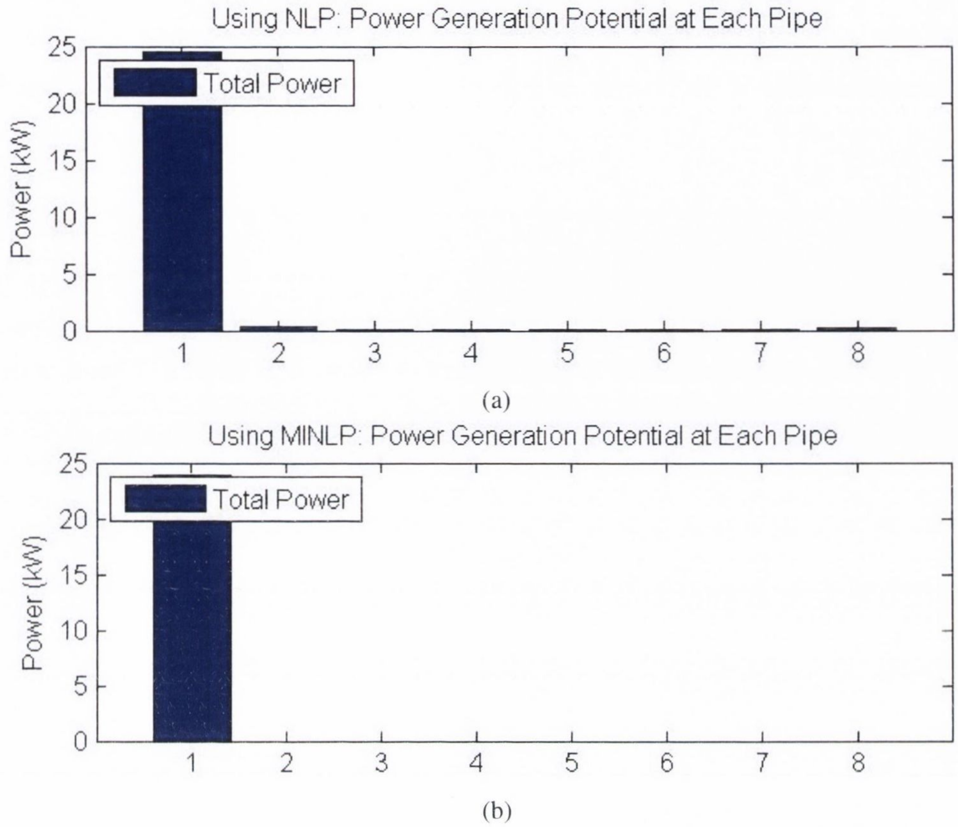


Fig. 6.6: Model Output: (a) 2-Loop NLP (b) 2-Loop MINLP

Table. 6.9: 2-Loop WSN: Results of MINLP Optimisation

MINLP					
Link	Flow (m^3/hr)	Nodal Head (m)	Install Turbine	Head Drop (m)	Power
1	1120	41.19	1	12.06	23.92
2	185.44	18.66	0	0	
3	834.56	32.37	0	0	
4	114.56	16.55	0	0	
5	600.37	18.5	0	0	
6	270.37	15	0	0	
7	85.44	NA	0	0	
8	70.37	NA	0	0	
				Total:	23.92

6.9.3 25-Node WSN Results

The results of the application of this optimisation formulation to the benchmark 25-Node WSN are presented in the following sections. The 25-Node WSN was optimised using both NLP and MINLP solvers.

6.9.3.1 NLP

The NLP optimisation algorithm was applied to the 25-Node WSN as described previously, and with further network details provided in Appendix D. This algorithm assumed there was a turbine installed on all links, through the inclusion of a pressure drop to be added along all links. The first analysis employed the NLP optimisation formulation and assumed a turbine to be installed on all links. An optimal solution was found in 44.3 seconds, with the results shown in Figure 6.7. From this, it can be seen that the link with the largest power generation potential was Link 11. Discounting all links with power generation potential of less than 0.1kW, the solution that would maximise the total power generation in the network would be to install turbines at links 1, 11, 24 and 25. In practice however, an optimal solution with less installed turbines would be preferable to reduce the initial investment costs.

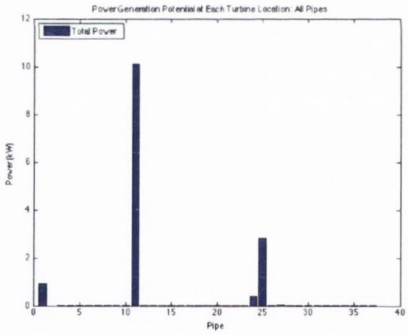


Fig. 6.7: 25 Node WSN: NLP no constraint on total pressure

A new constraint was then added in order to further constrain the total amount of pressure to be removed from the network. This linear inequality constraint was added to reduce the solution space and improve solving time. The additional constraint limited the total pressure drop removed from all pipes to be less than a certain value. With the maximum total pressure drop constrained to be less than 30m in total, an optimal solution was found in 24.51 seconds, with the output file as shown in Figure 6.8. This found the most power generation to be at links 1, 5 and 11, with all links with power generation potential of less than 0.1kW disregarded.

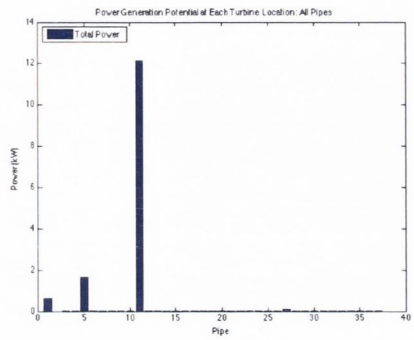


Fig. 6.8: 25 Node WSN: Total pressure drop less than 30m

The pressure drop was then constrained to be less than 25m. An optimal solution was found in 24.97 seconds and the resulting power generation for this model is shown in Figure 6.9. This reported the optimal links for power generation to be links 5 and 11. From all of these analyses, it was clear that Pipe 11 was the location with the most power generation potential, whilst also meeting the pressure requirements of the network. Other locations with additional power generation capacity were found at pipes 1, 5, 24 and 25.

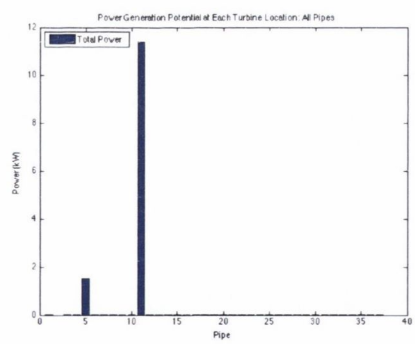


Fig. 6.9: 25 Node WSN: NLP total pressure removed <25m

6.9.3.2 MINLP

An MINLP formulation of this optimisation problem was then implemented in Matlab and solved using the BONMIN solver. The ability to include integer decision variables allows the MINLP formulation to select the optimal locations to install turbines. For the installation of one turbine, pipe 11 was again found to be the optimal location, providing a generation capacity of 6.57kW. The total number of turbines to install was then fixed at three. The results of this model are shown in Figure 6.10. An optimal solution was found in 61.06 seconds. The optimal locations for three turbines to be installed were found to be at links 1, 5, and 11. The total power generation capacity of this combination was found to be 16.43kW at average network water demand and assuming a turbine efficiency of 100%. Excess pressure that could be used to drive hydropower turbines was found to be 12.93m, 12.8m and 12.68m at pipes 1, 5 and 11 respectively. The majority of the power generation capacity was found to be at link 11, at 11.59kW, with a further 2.7kW available at link 5, and 2.14kW available at link 1. Through the installation of these three hydropower turbines, the overall average network pressure was reduced to 28.8m.

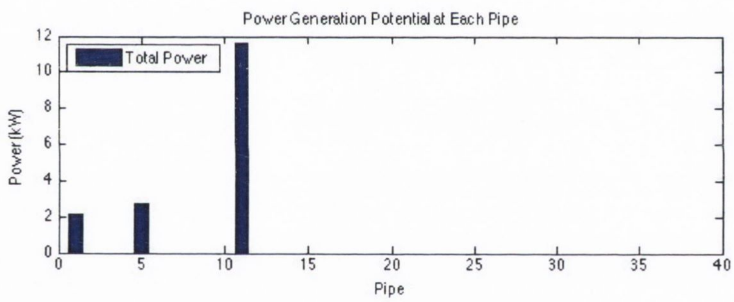


Fig. 6.10: MINLP Results: 3 Turbines

6.9.3.3 Optimal Turbine Operation

An analysis of the optimal operation of one turbine installed at link 11 was then undertaken. The results of this analysis were also compared with the output of an EPANET hydraulic analysis. This optimisation model was solved in 4.9 seconds. It was found that at pipe 11, there was 14.06m of excess pressure head which could be used to drive a turbine. This would result in the total network pressures reducing to an average of 30.04m. A comparison was then made with the results of the MINLP optimisation model and an equivalent EPANET hydraulic analysis. The presence of a turbine was represented by a PRV in EPANET. The PRV output pressure was fixed at the same output pressure as reported by the MINLP optimisation algorithm. This network layout is shown in Figure 6.11 with the PRV shown at node 26.

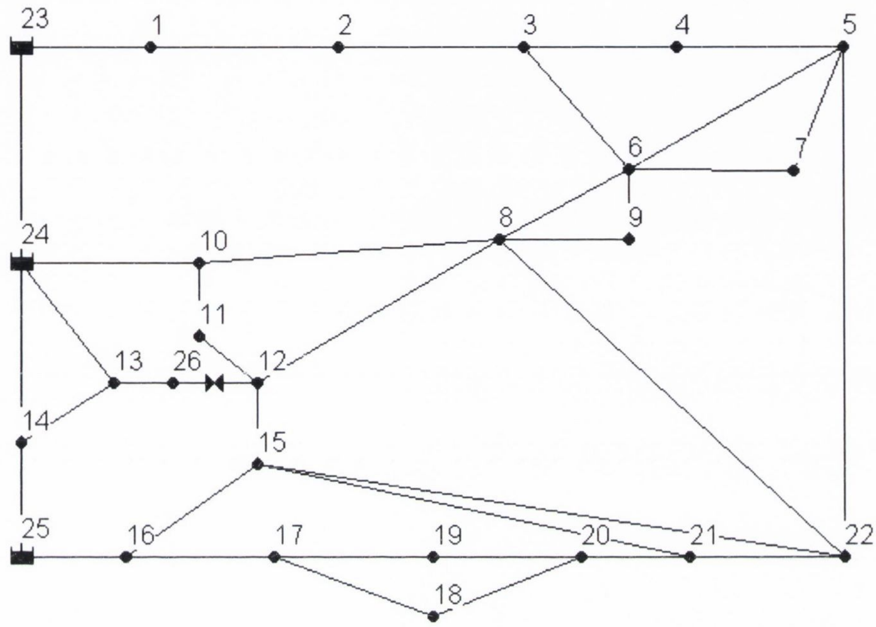


Fig. 6.11: EPANET calibration network with PRV to represent pressure reduction across a turbine

The comparison of the EPANET hydraulics and the hydraulics computed by the optimisation algorithm are detailed in Table 6.10. Both the flows and pressures reported by the MINLP optimisation solution were very consistent with the results of the EPANET hydraulic analysis. The largest relative error recorded between the flow rates was 1.964% and occurred at Link 22, representing a 0.01 l/s difference in flow. The largest relative errors between the flow rates were all found for the pipes with the lowest flow rates of less than 2 l/s. For all pipes with flow rates greater than 2 l/s (81% of the pipes), the relative errors were all less than 0.1%. The resulting pressures from the MINLP model were also very consistent with the comparable EPANET hydraulics results, with the largest reported relative error of 0.022% at Node 2 and Node 17, representing a difference of 0.01m.

Table. 6.10: Comparison of NLP output with EPANET output for pressure nodes

Link ID	NLP Flow (l/s)	EPANET Flow (l/s)	Relative Error (%)	Node ID	NLP Pressure (m)	EPANET Pressure (m)	Relative Error (%)
1	20.21	20.21	0.018	1	37.96	37.96	0.012
2	0.00	0	0.00	2	37.90	37.89	0.022
3	1.50	1.5	-0.073	3	27.05	27.05	0.013
4	24.29	24.31	0.076	4	28.74	28.74	0.002
5	63.61	63.58	-0.047	5	26.74	26.74	0.005
6	26.88	26.9	0.078	6	26.03	26.02	0.02
7	20.79	20.81	-0.095	7	26.37	26.37	0.005
8	13.51	13.5	-0.056	8	27.32	27.32	0.013
9	15.21	15.21	-0.024	9	27.15	27.15	-0.016
10	5.21	5.21	-0.069	10	26.91	26.91	0.001
11	47.67	47.71	-0.085	11	29.58	29.58	0.001
12	4.93	4.93	-0.041	12	26.58	26.58	0.005
13	8.58	8.58	0.053	13	32.86	32.86	-0.014
14	2.89	2.89	0.042	14	35.96	35.96	0.004
15	2.11	2.11	0.058	15	31.84	31.84	0.009
16	5.69	5.69	0.058	16	29.93	29.93	0.007
17	0.69	0.69	0.476	17	32.89	32.88	0.022
18	1.42	1.42	0.317	18	31.72	31.72	0.005
19	0.37	0.38	1.353	19	29.77	29.77	-0.003
20	3.23	3.23	-0.09	20	32.74	32.74	-0.002
21	12.22	12.22	-0.035	21	29.83	29.83	0.012
22	0.46	0.47	1.964	22	25.00	25	0.000
23	9.54	9.53	0.097				
24	34.99	35.02	0.078				
25	49.07	49.05	-0.042				
26	7.62	7.62	0.043				
27	35.51	35.51	0.013				
28	20.94	20.94	-0.006				
29	1.05	1.05	-0.034				
30	4.61	4.61	-0.05				
31	0.39	0.39	-0.591				
32	0.60	0.6	-0.219				
33	5.96	5.96	0.057				
34	27.77	27.77	0.012				
35	3.34	3.34	-0.134				
36	27.77	27.77	0.012				
37	7.62	7.62	0.043				

A further analysis was then undertaken of the operation of an installed turbine at pipe 11 over a diurnal water demand profile. The demand pattern as reported on for this network in the literature was applied, as shown in Figure 6.12. The turbine was fixed to be installed at Pipe 11. Optimal operation of the hydropower turbine, i.e. the pressure output required at the turbine for each period was solved using the optimisation algorithm. The optimal turbine to install was investigated based on different turbine efficiency profiles as shown in Figure 6.12. An efficiency curve for a pump-as-turbine (PAT) was also included as an option. PATs, as discussed in previous chapters, have been noted as a more cost-effective solution in comparison to traditional micro-hydropower turbines on the market.

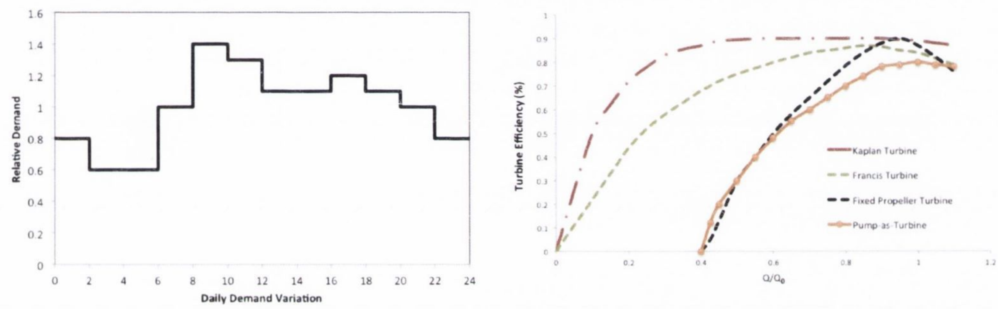


Fig. 6.12: (L) Diurnal demand pattern; (R) Turbine efficiency curves

It was found that power generation potential was reduced over night-time periods, when demand was lowest. Optimal power generation was found for the Kaplan turbine, which operated most efficiently over the variable flow rates. The power generation for each turbine type is shown in Figure 6.13. The PAT and Fixed Propeller turbines both were unable to efficiently generate any power during low flow night-time periods. During these periods, the available pressure head to drive a turbine was reduced, due to the need to maintain the minimum service pressure of 25m. The PAT was shown to perform as well as the Francis turbine for higher flow rates. Assuming 100% turbine efficiency, as has been assumed in previous work (Giugni et al., 2014), was shown to over-estimate the power generated, in particular at higher flow rates, for example between 8am and 10am.

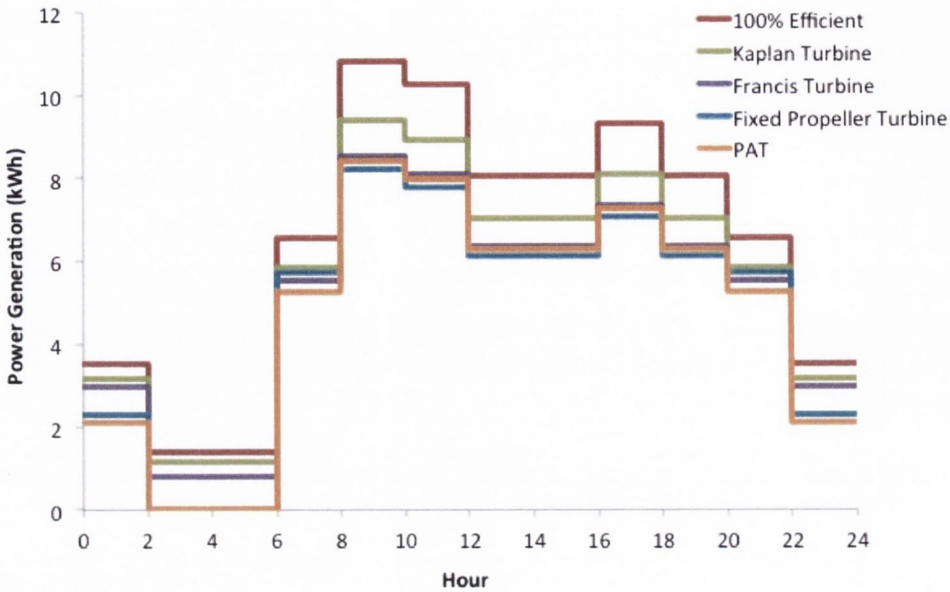


Fig. 6.13: Turbine selection for turbine installation at Link 11

6.10 Discussion

The potential for the application of optimisation algorithms to choose optimal points to install hydropower turbines has been demonstrated. The selected objective function was to maximise the total power generation of a given WSN. This objective was dependent on the flow rates and pressures in each pipe. The developed algorithm maximises this power generation while maintaining required set service pressures in the network. The optimisation formulation installs turbines within the network and maximises the power generation at these turbines by reducing pressures to within minimum service requirements. The result is a reduction in network service pressures, reducing the intensity of leakage and likelihood of burst pipes, whilst also generating much needed electricity for WSPs.

The results of the application of the optimisation models discussed have also enabled a comparison of the use of different head loss approximations. As demonstrated on the 5 node network, the Hagen-Poiseuille headloss approximation enabled a faster solution time of the problem, due to the absence of non-linear constraints. The headloss equation could be modelled using less computationally expensive, linear equality constraints. However, the resulting flow rates as computed by the model were less accurate. The highest relative error was found on Link 4 at -117%. The resulting flow rates for each link as output by the NLP model using the Hazen-Williams approximation for headloss were very consistent with the corresponding EPANET outputs. These relative errors ranged between -0.036% and +0.003%. Despite resulting in less accurate flow rates and pressures, both models found the same pipe in the network to be the optimal point to install a turbine. For small networks like this one, the optimal location to install a turbine would be relatively easy to discern. However, for more complex networks, with many links, nodes and loops, accurate modelling of head loss and hence energy transfer through the network, would need to be as accurate as possible. It was therefore decided that the Hazen-Williams headloss approximation would be used for the following more complex networks.

It has been shown that for small WSNs, as in the 5-Node and the 2-Loop analyses, i.e. cases with less than ten nodes, and less than two loops, a NLP algorithm assuming a turbine is installed at all pipes has been shown to accurately locate the pipe with the most potential for hydropower generation. With this point selected, a further NLP analysis can be run to find the optimal settings for a turbine assuming only one turbine is installed at that pipe. However, for more complicated networks with multiple loops, connections and interconnections, it may not be clear from a NLP solution alone as to which link has the most power generation potential.

This hypothesis was then tested on the more complex 25-Node benchmark WSN. NLP was found to provide a good approximation of the optimal locations for hydropower energy recovery. A test constraint on the total pressure drop to be removed from the network improved the ability to identify the pipes with the most power generation potential. For the installation of three turbines, there were a number of potential optimal combinations. Through trial and error, these combinations were tested. The results of these analyses are presented in Table 6.11. The optimal solution was found to be to install turbines on pipes 1, 5 and 11. Also detailed in Table 6.11 are the average network nodal pressures following installation of each combination of turbines. Installing turbines on links 1, 5 and 11 also resulted in the largest decrease in the average nodal network pressures, at 28.8m. Prior to any turbine installation the average nodal pressure in the network was 40.86m. The peak power generation points identified during the NLP solution were then applied as initial start points for the MINLP solution.

Through comparison with the EPANET hydraulic analysis output, the MINLP optimisation model developed was shown to provide very accurate approximations of the flow rates and pressures in the network, with the largest relative error reported as 1.96%.

Table. 6.11: Optimal power generation - 3 turbines

3 Turbine Combination	Total Power Generation (kW)	Solution time (s)	Average Network Nodal Pressure (m)	Max. Nodal Pressure (m)	Min. Nodal Pressure (m)
Links 5, 11, 24	14.02	8.28	29.99	37.96	25
Links 1, 5, 11	16.43	10.87	28.8	35.88	25
Links 11, 5, 8	14.96	14.26	30.04	37.96	25
Links 11, 24, 25	10.12	7.01	31.13	37.96	25

Through the application of this optimisation algorithm to the benchmark 25-Node network, a direct comparison between the results of this optimisation formulation with other research results for PRV location models could be made. A summary of the results of where the optimum locations for installation of PRVs as found in literature were presented previously in Table 6.4. The results of this analysis agree with some previous results. For the optimal location of one PRV, pipe 11 was found to have the greatest hydropower potential, as has been found by Nicolini and Zovatto (2009) for optimal PRV location, and by Giugni et al. (2014) for both optimal PRV and hydropower turbine location. For the optimal installation of two turbines, a different optimal solution to Nicolini and Zovatto (2009) was found, at pipes 5 and 11. This optimal solution again agrees with the results of Giugni et al. (2014) for optimal installation of hydropower turbines. For the optimal installation of three turbines, the optimal arrangement reported using MINLP was to install turbines on links 1, 5, and 11. This was different to the results of many previous research for optimal location of PRVs. However, it was the same combination as was reported by Eck and Mevissen (2012) for optimal PRV location using a MINLP solver. It was also the same optimal combination as reported by Giugni et al. (2014) for installation of three turbines to maximise power generation.

The Giugni et al. (2014) optimal solution was found using a GA. This optimal solution was reported to generate a maximum power capacity of 15.9kW at average demand at 100% efficiency. The optimisation algorithm developed and applied in this thesis reported a higher total power generation capacity of 16.43kW for the maximisation of the net power generation in this network. The MINLP approach applied in this research was found to provide an improved result than that of the GA applied by Giugni et al. (2014). Though the 0.53kW increase in power capacity represents a small increase in total power generation for this small benchmark 25-Node network, for a larger more complex WSN this increased power generation would be expected to be more significant.

However, though the total power generation potential of three installed turbines was found to be 16.43kW, the turbine at pipe 11 was found to contribute the most to this total power output. The turbine at pipe 11 was found to generate 11.59kW, while the turbine at pipe 5 generated 2.7kW and at pipe 1 generated 2.14kW. This was assuming a 100% turbine efficiency. Applying different turbine efficiency curves, the total power generated between these three turbines varied from 14.24kW for a Kaplan turbine to 10.73kW for a PAT. A cost-benefit analysis of these turbine installations would be required to decide upon the economic viability of their installation. As has been discussed in the economic feasibility study presented in Chapter 4, for a hydropower energy project in a WSN to be deemed feasible, investment payback is generally required to be achieved within ten years. The smaller the power generation potential, the longer it would take to cover the initial financial investment required for the installation. These initial costs may vary widely depending on the site specifications, proximity to the grid, access to site and the amount of civil works required. Furthermore, the presence or eligibility for a green incentive or a REFIT tariff could improve the financial case.

In order to discount power generation at nodes with very small potential excess pressure, the lower bound on the turbine pressure drop could be increased to a specified minimum level. For this analysis, the lower bound on the pressure drop was fixed at zero, which allowed for sites with very low head to be included. These low head sites were included because the combination of a pipe with high flow rates and low head may still lead to maximised power generation potential. However in practice, it could be effective to introduce a higher minimum turbine head value, to discount sites with low head. The developed algorithm allows this distinction to be made for future analyses.

As well as reducing average service pressures in the WSN, the installed turbines would provide a valuable additional source of electricity for the WSPs. The cost of electricity per kW for industrial users in Ireland in 2013 was reported to be 13.31c per kWh (Eurostat, 2014). For the 5 Node WSN, if a turbine were installed at Link 1 generating 14.99kW and the electricity generated was used directly by the water supply provider, this would result in an annual electricity saving of €17,477.68. For the 2 Loop WSN, if a turbine were installed at Link 1 generating 23.92kW as estimated by the MINLP model, and the electricity generated was used directly by the WSP, this would result in an annual electricity saving of €27,895.06 for the WSP.

The optimal MINLP solution for the installation of three turbines was found in under 150 seconds. However, a quicker solution (in 61.06 seconds) could be found using a modified initial start point, x_0, starting with the locations identified following the NLP analyses as initial locations for turbines.

For the 25-Node benchmark network, as discussed previously, the majority of the power generation potential was found at Link 11. If one turbine were installed here, an average 6.57kW of electricity could be generated, which would lead to an annual electricity saving for the WSP of €7,660.33. If three turbines were installed at the optimum reported locations, a total of 16.43kW could be generated, resulting in an increased annual electricity saving of €19,156.66. However, as discussed previously, the initial installation costs would also increase. A further more detailed, site-specific cost-benefit analysis would be required to decide whether the option to install three turbines would be cost-effective in the medium-long term.

The installation of additional turbines on pipes 1 and 5, resulted in the power generation capacity at pipe 11 to increase from 6.57kW to 11.59kW. Instead of installing additional turbines on pipes 1 and 5, PRVs could be installed which would enable further control of network pressures allowing for increased power generation on pipe 11. PRVs would be less expensive than turbines to install. This would reduce the total initial investment costs of installing and connecting three turbines, and would increase the generation capacity of one installed turbine at pipe 11 by 5.02kW.

These models were all tested on relatively small benchmark networks, however in practice network sizes could be much larger and more complex. Further research is required into the application of optimisation techniques to large-scale real WSN scenarios.

6.11 Summary

The focus of this Chapter was to investigate the application of optimisation techniques for the optimal location of hydropower turbines within water supply networks for pressure reduction. Using both mathematical optimisation and evolutionary optimisation algorithms, optimal solutions were found. However in the case of GAs and MINLPs, a global optimum cannot be guaranteed. For the smaller networks, both the 5-Node

WSN and the 2-Loop WSN however, the optimal locations to install a turbine using a GA matched that of the NLP and the MINLP models. However, the maximised power generation using the NLP and MINLP models was found to be higher than the maximised potential using the GA.

The results of the initial analysis show that optimisation algorithms can be used as a decision aid in the design process for optimal installation of hydropower turbines in water supply networks. Further research is recommended on the application of this optimisation model on a larger water supply network and real-world case studies. Further research is also recommended on the development of a similar optimisation formulation but for a different objective, to minimise the total project costs, or to minimise the investment payback period. These objectives are particularly relevant when the estimated power generation potential is small.

CHAPTER 7

Implementation

7.1 Introduction

As well as investigating the technical issues related to the identification and exploitation of micro-hydropower (MHP) energy recovery in WSNs, the author also explored the related organisational and implementation aspects. As discussed in Chapter 2, there is a growing body of literature reporting the potential for MHP in water distribution networks. However, there has been little uptake within the water industry as yet. Some of the technical reasons for this were previously discussed, including the effects of flow rate variation, and the necessity to regulate the pressure at the turbine outlet. Other issues identified in literature that may be preventing the uptake of this technology by industry include cost and policy issues. For an MHP project in a WSN to be a success, several different organisations are required to collaborate. Efficient and timely completion and implementation of a project may depend on a number of factors. Though there are few examples of installed turbines on water distribution mains, there are examples of turbines installed further upstream in the network, at water treatment works and reservoirs. To gain further insight into the key factors affecting successful project implementation, a set of case studies were developed for two previous completed hydropower projects on water supply infrastructure at points further up in the network. In this chapter these two case studies are presented, along with a cross-case comparison and implications for future projects.

7.1.1 Research Question

The overall research question explored in this thesis is: *what is the potential for hydropower energy recovery from the water supply network?* In order to investigate this potential, the following sub-questions will be addressed: what is the feasibility for energy recovery in the water supply network, how do uncertainties and variations affect feasibility, how can a water supply network design be optimised for energy recovery and how can energy recovery projects be implemented and replicated in practice from organisational and management perspectives?

This chapter relates to my final research sub-question. How can energy recovery projects be implemented and replicated in practice from organisational and management perspectives?

7.1.2 Research Model

The overall research question discussed previously is centred on the implementation of hydropower energy recovery turbines in water supply networks. Though there are some installations of this type already in practice, they are not yet widespread. To investigate how best to implement MHP in practice, a comparative case analysis of previous installations has therefore been selected as the most suitable methodological approach to address this question. Through these case studies, the organisational, management and regulatory issues associated with the implementation of MHP projects in the water industry were investigated. The framework underpinning this case analysis is shown in Figure 7.1.

In order to approach the research question, some background theory and literature on how projects are managed was required. Key papers from operations management literature, as well as from water supply management literature were consulted, as presented in Section 2.7. This literature provided frameworks for describing and understanding the outcomes of the case research undertaken.

It was decided to develop two case studies of existing hydropower installations on the

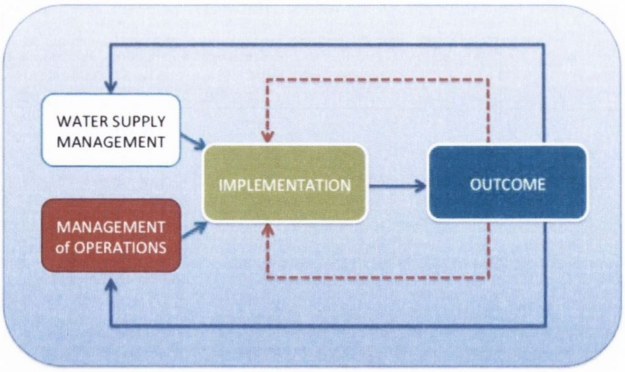


Fig. 7.1: Chapter 7: Research Model

water supply infrastructure. The selection of two cases allows for increased insights through a cross-case comparison. From the technical perspective, it was decided that two cases of the same technical type would be analysed. This would allow for a like-for-like comparison on the technical parameters. However, to gain further insight into the management and implementation of these projects, cases from different water supply network jurisdictions, and also from different organisational management types, were selected. Worldwide, water supply operation varies from completely privately operated to publically operated. The structures of these organisations may influence the adoption of new technology and also the way in which projects are deployed from an operations management perspective. These potential differences or similarities will be explored through this cross case comparison. In Ireland, water supply is predominantly publically operated, however in England and Wales, water supply is privately operated.

The choice of site in Ireland was limited as there are only a few examples to choose from. Currently, there are two installed hydropower turbines at water treatment works, one at Vartry Reservoir Co.Wicklow, and another at a treatment works in Co. Kerry. Dublin City Council (DCC), who are responsible for the Vartry Reservoir and waterworks, are members of the Hydro-BPT research project steering committee, so therefore the DCC Vartry turbine was selected for analysis due to ease of access to data.

The second site selected was a Welsh Water site. In contrast to Ireland and other countries such as the United States, all water supply in England and Wales is privately operated. As with DCC, Welsh Water (WW) is also a member of the Hydro-BPT steering committee.

7.2 Methodology

Case research methodology was employed for this analysis. Semi-structured interviews were carried out with key players involved in the two projects selected and summarised in Table 7.1. Follow up informal interviews were also undertaken. All relevant publically available documents relating to these projects were gathered and analysed as well as some data and documents obtained from the interviewees. Interview protocol documents were prepared in advance of the two primary semi-structured interviews. These protocols outline the topics to be covered in the interviews, the questions to be asked and a list of required further data and documents. Prior to the interviews, an email was sent outlining the topics to be discussed. Copies of these interview protocol documents have been included in Appendix D. For the Vartry hydropower project, the key informant was identified to be the Vartry Waterworks Chief Engineer (CE). For the PenyCefn hydropower project, the key informant was more difficult to ascertain. The primary interview was undertaken on site with the turbine suppliers during their biannual maintenance checks. Another more informal interview was then undertaken with the chief operator of the water treatment plant. Interviews for each case study were performed on site, and ranged from between one and two hours in duration and included site visits to the turbines and water treatment works.

Table. 7.1: Case study outline

Client	Location	Nature of Organisation	Location of Turbine	Site Type
Dublin City Council	Vartry Waterworks	Public body	Water treatment works	Retrofit
Welsh Water	PenyCefn Waterworks	Private company	Water treatment works	New build

7.2.1 Organisations

The organisational structure underpinning the two water supply organisations involved in these hydropower projects differ in a number of ways, mainly due to the nature of the organisations, with DCC a public body and WW a private company.

7.2.1.1 Ireland

The water industry in Ireland, at the time of the Vartry hydropower installation, was operated by 34 local authorities throughout the country. Each local authority was in charge of the water infrastructure in their own county including sourcing the water, its treatment process and distribution. Collectively they supplied water to approximately 1.1 million households connected to public mains. Water services cost over €1.2 billion to run in 2010, with operational costs amounting to €715 million and a further €500 million covering capital costs. Currently only non-domestic water tariffs are implemented in Ireland collecting €200 million in water charges, meaning the €1 billion shortfall is largely State funded.

Water supply in Ireland is operated and legislated by the Department of the Environment, which is headed by the Minister for the Environment, Community and Local Government. The department of the Environment draws up legislation regarding the water industry and other environmental services under directives implemented by the EU. New infrastructure projects are generally put out to tender by project. The budget for any water or wastewater project is sought under the Department of the Environment. The funding is distributed to the local authorities through two bodies, the Water Services investment Programme (WSIP) which covers all major water and sewage schemes costing over €1 million, and Rural Water Programme (RWP) which covers smaller projects within the county councils (O'Brien and Shine, 2013).

In April 2012, the Irish Government announced reforms to the water industry in Ireland which included creating a new water utility company called Irish Water, a new funding model which included raising finance internationally and through water charges and the appointment of a regulator for the Irish Water Industry. Over the following 5 years the Irish Water Programme was to go through a number of phases to gradually transfer responsibilities of the water service from the local authorities to Irish Water by 2017.

7.2.1.2 Wales

Dwr Cymru Welsh Water (WW) provides water and wastewater services to the majority of Wales, its service area extends from the Southeast of the country up to the Northwest. WW has gone through many different stages throughout its lifetime. Originally Welsh water was supplied by many publically owned water authorities which were privatised by the Welsh National Development Authority in 1973. With surplus cash from market stocks it began investing in many areas including the leisure and energy industry. In 2000, the company was split and the water sector was sold to Glas Cymru, a company set up purely for the purposes of taking control of WW. This was as a result of the company experiencing financial difficulties and under-investment with respect to their water services. WW today is a non-profit company with steady and predictable annual revenue streams. Since its rebirth it has progressively grown and continues to expand, redeveloping its infrastructure from the ground up (O'Brien and Shine, 2013).

WW are a major energy user, they are in the top ten energy consumers in Wales (*Environment and Sustainability Committee: Inquiry into Energy Policy and Planning in Wales*, 2011). In 2010, WW spent £34 million on gas and electricity. However, WW are proactively developing renewable energy resources both to reduce energy bills and also to reduce their carbon footprint. WW have agreed to reduce their carbon emissions from 2007 levels by 25% by 2015, and to further reduce emissions by half by 2035 (Welsh Water, 2007).

WW have a designated Energy Division who oversee the development of energy-related projects. There are three teams which operate within this division; Commercial Energy, Marketing, Energy and Operations, and Innovation and Energy. The three divisions work in a cross functional, collaborative manner. Each team has a manager whose role is to track the progress and current direction of the team. The department comprises 15 staff members and despite its size, it covers a range different areas including energy costing and purchasing, pursuit of new renewable sources and efficiencies in all areas of water and wastewater sites.

The main concern of WW when evaluating potential new energy projects, is the project investment payback period. Projects considered feasible would have a maximum payback period of 10 years with the optimum being less than 5 years. Large scale projects like hydropower construction have large initial costs but are offset by a payback time which can be relatively short in some cases. Other projects, such as a pump upgrade, will also concern not just energy costs but repairs or replacement if the machine is coming to the end of its life span (O'Brien and Shine, 2013).

The Energy Division contracts a lot of the work involved in the projects to third party private companies. In particular, the division procured a hydro framework via the Official Journal of the European Union (OJEU) tender process. Four companies were successful in this process and they provide feasibility study services and turnkey solutions to specific hydropower projects. In their Procurement Plan (2006), WW state that they have sought, wherever possible, to build on the experience and successes of their previous partnerships in order to produce further improvements in service delivery performance and efficiency.

7.2.1.3 Summary

DCC and WW have differences and similarities in how they organise themselves and in how they approach innovation and development within their water and wastewater infrastructure. Through this cross-case analysis of the two hydropower projects, these differences and similarities will be explored in greater detail. Each of the case studies is presented in a similar format, beginning with an introduction and background, followed by a description of the key project stages for each, an analysis and discussion of each project individually and finally, a cross-case analysis and conclusions.

7.3 Case Study 1: Vartry Waterworks

Vartry reservoir and waterworks supplies 75 million litres of clean water per day to parts of Dublin and Wicklow. The total water storage capacity is 16,900 million litres which is equivalent to 200 days supply at average plant output. The reservoir, located in scenic Roundwood, Co.Wicklow (Figure 7.2), originally dates back to the 1860s. The construction of Vartry reservoir which was completed in 1868 provided Dublin with its first water treatment works. It remained Dublins principal water supply until the development of the Liffey Scheme at Bohernabreena in 1944. It was constructed by building an earthen dam across the Vartry River valley. In the 1920s additional storage was added with the construction of an upper reservoir which discharges to the original lower reservoir (Corcoran, 2005). The water discharges from the reservoir via a draw-off tower and valve house to a stilling basin below, where previously the excess energy was dissipated. From the stilling basin the raw water then flows by canals through slow sand filters. Filtration is then followed by chemical disinfection and pH adjustment. The reservoirs with their fine crafted stone draw-off towers are a popular local scenic attraction.

Fig. 7.2: Draw-off tower at the Lower Vartry Reservoir (L); Vartry Reservoir (R)

Since November 2008, Vartry waterworks have been generating electricity through the installation of a hydropower turbine between the valve-house and the stilling basin. This recovers the energy that was previously dissipated by a throttling valve and in the stilling basin. However, this was not the first hydropower turbine to be installed at Vartry. Vartry has a long history of hydropower. The previous turbines installed used part of the incoming flow from the lower reservoir to generate electricity. The more recent turbine installed was a Pelton wheel turbine in the 1940s.

The new turbine, a Kaplan turbine, was manufactured by NHT Engineering Ltd. Figure 7.3 below shows the old Pelton turbine and the current turbine at Vartry. This new turbine was deemed to meet the site specifications, such as the water pressure and flow rate ranges and was easy to install, maintain and operate. Water supply takes precedence over turbine operation, therefore, in parallel with the turbine, an automatic gravity operated bypass was required to ensure that flow to the plant could not be interrupted should any issues arise with the turbine. The turbine generates 90kW of power, which would be enough to power between 20-30 houses per year. This turbine installation generates enough electricity to power the treatment works, with a significant amount of excess power then sold to the grid.

Fig. 7.3: Pelton turbine at Vartry Reservoir (L); Axial flow turbine installed in 2007 (R)

7.4 Vartry Hydropower Project Stages

This project, from its initial conception to completion can be divided into six main stages: Feasibility, Detailed Design, Planning, Construction, Commissioning and Grid Connection, as shown in Figure 7.4 below. The initial feasibility study was undertaken in March 2002, and the hydropower turbine was online and generating in November 2008, giving a total project timeline of over 6.5 years.

Fig. 7.4: Vartry Hydropower Project: Stages

7.4.1 Feasibility Stage

In 2002, the CE at Vartry attempted to initiate the hydropower project. In March 2002, an invitation to tender for the feasibility study was launched. Following an invitation to tender to specialist firms, three quotes were received and assessed. Energy Control Systems gave a reasonable price and ultimately found the project to be feasible and they were appointed in April 2002. Energy Control Systems had also previously completed a small hydro project on the River Liffey at Sallins. The feasibility study found that the project was technically and financially viable. A cross-flow turbine was recommended as the best choice of turbine to install. The estimated rated power of the proposed project was 150 kW and the estimated average annual generation was approximately 700,000kWh, the estimated forecast investment payback period was 8.3 years. Following the feasibility study, Energy Control Systems decided to step back as they wanted to tender for the control systems. The design consultants were later confused that Energy Control Systems were not involved.

A further feasibility study was also undertaken for the installation of another hydropower turbine at the upper reservoir. However the investment payback period for this was estimated at between 18 and 20 years and was therefore deemed not feasible. This reservoir is only used during peak water usage, so would not have as consistent a potential for energy recovery as at the main reservoir. However, the CE does intend to re-visit this as an option in the future.

7.4.2 Detailed Design

Following public tender in 2004, Fingleton White and Company were selected to design and supervise the construction of the project. They had previous experience working on other smaller energy recovery schemes, <1MW. They were interviewed in June 2003 and appointed in December 2003. The estimated consulting costs were about 50% higher than DCC's early estimates; this increase was based on the complexity of the project.

Site investigation revealed that the complicated nature of the intake and the condition of the old pipes resulted in an additional headloss of 4m, reducing the rated power output to 20% less than originally estimated. This, combined with an ESB export limit of 60 kW (greater would require a substantial grid upgrade fee), resulted in a decision to seek tenders for a 90 kW turbine to supply an estimated average output of 75kW.

The physical tests for the feasibility study consisted of tapping into the pipes at various points to take readings. There was no inflow meter at the reservoir, therefore there was some guess work involved. From these tests they learned for future projects that it is necessary to investigate the intakes thoroughly. The original pipes dating from the 1860s consisted of many turns, expansions and contractions and so there were large head losses across them. The estimated calculated head loss was approximately 2.6m across these pipes, however the actual headloss turned out to be higher at approximately 4m.

Fingleton White also estimated project payback period to be 8.6 years, similar to the initial feasibility study. One problem with the initial feasibility study was that the cost for the consultants was not included, so, in reality, they did not know what to expect.

It was necessary for the water turbine to meet the following specifications (Fleming, 2012):

1. Operating heads over the range of 7 - 16 m.
2. Operating flows of 580 - 1,200 l/s.
3. Must not generate dangerous water hammer effects.
4. Able to safely withstand full runaway speed indefinitely.
5. Easy to install, operate, and maintain.

In April 2005 DCC went to tender for the turbine supplier. The consultant went ahead and negotiated with six turbine manufacturers and presented the best technical option in anticipation of a public tender. They recommended the Ossberger turbine as the most suitable. This proposed design would take 80% of the flow through a crossflow turbine. However the actual turbine selection still had to go to public tender. When it went to public tender, two companies responded, Sink and NHT, Ossberger did not.

NHT suggested that a Kaplan turbine would better suit the flow conditions. Pelton and turgo turbines were discounted as the pressure head range is too low. However, both the head and flow ranges were well within those of Kaplan and Cross-flow water turbines. Sink, a Czech company, proposed installing a crossflow turbine. However, they had had major problems with their bearings in two previous projects, within 1-2 years after installation.

Following public tender, a Kaplan turbine manufactured by NHT Engineering Ltd was selected for the project. The turbine selected has water-lubricated bearings to remove the risk of oil/grease contamination of the water. It also regulates the inflow to any required pre-set level by means of hydraulically operated guide vanes linked to a flow meter.

In addition, because water supply at Vartry at all times takes precedence over turbine operations, an automatic gravity operated bypass was required to ensure that flow to the plant could not be interrupted if the turbine stopped. Because water supply is the number one priority at Vartry, the flow rate is the variable that must be regulated, the power generated is secondary. A bypass is also in place in case of any electrical faults, maintenance works or other issues with the turbine. The bypass set up consists of two butterfly valves and counter weights. When the power shuts down, the turbine switches off. Oil dampened hydraulics prevent the valves from slamming. Since installation, the experience has been that, for a variety of normal operating reasons, the electricity shuts off approximately once or twice a month. There is a stand-by diesel generator for when this occurs. The turbine will automatically attempt to restart over the course of 3 hours from the electricity short, first after 30 minutes, then again after an hour then again after 90mins.

7.4.3 Planning

The treatment plant is located in an Area of Outstanding Natural Beauty and is also highly visible from the public road. The new installation was carefully designed to blend with the existing plant layout and buildings and incorporates a request of Wicklow County Councils planners that the ridge-line of the new turbine house be along same axis as the existing generator house.

Planning permission was sought from Wicklow County Council in August 2004, and was granted in January 2005. The consultants had recommended not applying for planning permission as they felt it was not necessary. However to show good compliance, respect and good initiative they did apply for planning permission. There was one design change required, to move the turbine building slightly so that it could not be seen from above. Once that change was put in, planning was approved and there were no further objections.

7.4.4 Construction

Construction began in April 2007. One issue encountered during this stage was the difficulty in dealing with the large old intake pipes. Due of the unique nature of these pipes, it was decided that DCC would do the pipework themselves as they had in-house expertise in dealing with them. It was felt that it would be difficult to find this expertise elsewhere. This intake pipework turned out to be very difficult to manage and ended up costing 3.2% of the total project cost. Finally, due to the fact that this work was completed by in-house DCC employees, it was not eligible to be claimed for on the SEAI grant.

Fig. 7.5: Vartry waterworks turbine installation (L); Turbine intake (R).

7.4.5 Commissioning

When the project initially went out to tender for the sale of electricity to the grid, it received no realistic offers. This was in 2007, at the height of the Celtic Tiger boom period when perhaps, people were not interested in such small projects. Another key decision at this stage was to accept the ESB export limit of 60 kW, this meant that they could avoid paying for a grid upgrade, which is required for larger power exports, saving them additional costs. However it limited the amount of electricity that could be sold to the grid.

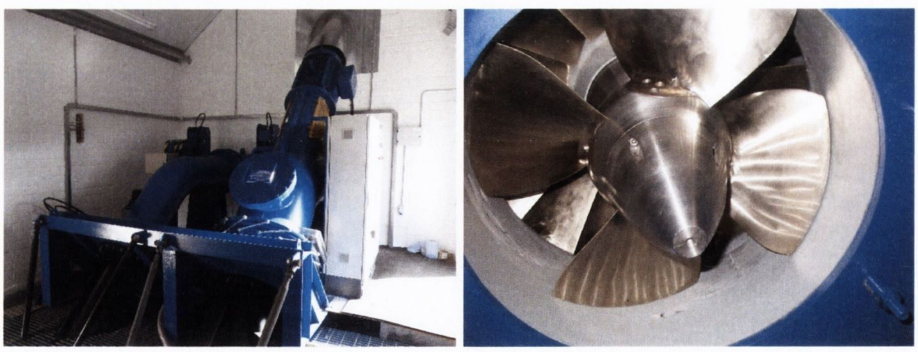


Fig. 7.6: Vartry waterworks turbine and bypass valve (L); Kaplan turbine propeller and wicket gate (R).

7.4.6 Grid Connection

From November 2007 to November 2008 the turbine was connected with the electrics up and running. However, with no sale agreement in place, this electricity was supplied to the grid for free. They went out to tender for the sale of electricity to the grid again one year later and received two prices this time from Bord Gais and Airtricity. Bord Gais had no maintenance cost whereas Airtricity did. By November 2008, the sales agreement was in place with Bord Gais and the fifteen year REFIT scheme kicked in. This REFIT price is fixed for 15 years and is linked to the Consumer Price Index (CPI).

7.5 Discussion

As DCC did not have previous experience with a hydropower project of this scale, there were a number of unexpected hurdles to overcome. The numerous permits and legislative requirements presented a significant barrier. Applications were complex and numerous and caused significant delays. Another issue that caused delays and incurred additional costs related to the grid connection process. When they first went out to tender to find an electricity buyer, there was no interest expressed. This lead to a year long delay causing loss of potential revenue as well as incurring additional costs and time lost having to re-apply for permits the following year. Also, due to this delay, they did not receive the full SEAI grant that had already been approved. One issue to bear in mind when working on a retrofit project on old water supply mains, was that the intricate pipework was very much specialist work.

It was noted that it was better value and more cost saving to use the electricity generated on site where possible. It cost 14c per kWh to buy electricity and they are being paid 8c per kWh through the REFIT scheme to generate. Therefore the more economic option would be to use the electricity on site.

Finally, the overall project costs were a negative factor preventing the installation of similar schemes by other smaller local authorities, in particular the consultants costs. The breakdown of the overall project costs, as percentages of the total project cost, are presented in Table 7.2. The turbine cost was found to contribute to approximately 35% of the total installation costs. This is slightly higher than the estimated 30% that has been suggested and assumed in previous research for estimating hydropower project costs (Ogayar et al., 2009; Giugni et al., 2009).

There was a need for capital investment up front for this project, which many other Irish local authorities would not have readily available. However with more organised scheduling of payments this problem could potentially be avoided for future projects. There is also an opportunity for economies of scope through sharing of expertise and consultant costs over multiple projects. This opportunity is realistic in the context of the emergence of one water authority, Irish Water.

Design fees were much higher than originally anticipated. This was primarily due to the complexity of the project due to having to retrofit the turbine into an existing treatment plant. The long delay in on-site commencement also meant that various approvals/licenses/sales agreements (many complex) had to be re-applied for and this resulted in considerable extra cost.

Table. 7.2: Vartry Hydropower Project Costs

Vartry Hydro Project Costs (incl. VAT)	
Work Description	**Cost (% of Total)**
Civils	35.10
Turbine manufacture & installation	34.39
Electrical Services	6.94
Intake Pipework	3.20
ESB Connection/meter	0.51
Fit Intake Screens	0.26
Manufacture Intake Screens	0.35
Planning App fee	0.03
G10 Test (grid connection)	0.17
Design and Construction Supervision	17.72
July 2002 Feasibility Study	1.33
SEI Grant	-17.28

Though setbacks and delays were encountered along the way, the project was a successfully completed. The project success is investigated further later in the case analysis under different measures for success. The Vartry experience offers us valuable insights into how future small scale hydropower developments should be implemented, especially in an Irish context.

7.6 Case Study 2: PenyCefn Water Treatment Works

Pen y Cefn Water Treatment Works is located in Snowdonia and serves the nearby town of Dolgellau. Dolgellau is a small town in Gwynedd in north-west Wales, with a population of 2,678. An upgrade to the previous treatment works at PenyCefn was completed in May 2011. Along with this upgrade a hydropower turbine was installed. The treatment plant is gravity fed from the nearby Llyn Cynwch reservoir with raw water at a pressure of up to 10 Bar. The raw water is fed into an open dissolved oxygen flotation (DAF) tank so this pressure is not required for the process and would have previously been removed through a pressure reduction valve (PRV). As the end of the pipe into the DAF plant is elevated to a height of about 5 metres, normally a turbine would have had to be mounted at this height which would have proved challenging and expensive (Zeropex, 2014).

Welsh Water selected a Zeropex Difgen turbine to be sited at ground level. It generates electricity while maintaining the required minimum pressure at its outlet, allowing water to enter the elevated tank. The turbine also controls the raw water flow into the treatment process by controlling its flow rate to match the desired flow. The turbine is equipped with a load bank, which allows the it to continue running if the electricity supply from the grid fails which allows a controlled shutdown with no hydraulic issues resulting.

Key facts:

- Difgen Model: DG13-14
- Differential Pressure: 9 - 10.5bar
- Flow Range: 10 to 30l/s
- Power Output: 8-17kW (generator rated at 37kW)
- Annual Revenue: 29,000 GBP
- Payback Period: 2.8 years (for turbine only)

7.7 PenyCefn Hydropower Project Stages

This project, from its initial conception to completion can be again divided into stages similar to the Vartry Project: Feasibility, Detailed Design, Construction and Commissioning, as shown in Figure 7.7. The planning and grid connection phases that were included for the Vartry case, do not apply for PenyCefn. The turbine at PenyCefn was installed in tandem with an upgrade to the water treatment facility. Planning permission was required for the construction of the new water treatment works, however planning was not required for the additional installation of a turbine and, therefore falls out of the scope of this case study. The electricity generated by this hydropower turbine will all be used on-site at the treatment works, therefore the grid connection stage also does not apply. The initial feasibility study was undertaken in July 2010, and the completed installed hydropower turbine was online and generating by September 2011, giving a total project timeline of approximately 1.25 years.

Fig. 7.7: Vartry Hydropower Project: Stages

7.7.1 Feasibility Study

The Welsh Water Energy team undertook an in-house infrastructure wide analysis of potential hydropower sites. The initial analysis highlighted over approximately 100 potential sites. This list was narrowed down to 10-20 sites with potential, including PenyCefn, to be developed during the 2010-2015 strategic Asset Management Plan (AMP5) period. A more accurate feasibility study was then undertaken of these sites by an appointed consultant. In the case of this project, the consultant selected for the Water Treatment upgrade was also used.

A risk assessment was undertaken as part of this feasibility study. The primary risk identified in this assessment was whether the project would be eligible for claiming a renewable energy feed-in tariff (REFIT). Other risks highlighted were that the Zeropex turbine is not widely proven within the UK water industry and also the decibel rating of the turbine was found to be just borderline acceptable for industrial installations. It was decided that an as-installed assessment may be required to determine if a further acoustic enclosure would be required.

Once the turbine supplier was selected, they underwent their own internal feasibility study. The supplier undertook an initial calculation from provided data, followed by a site visit to check site conditions.

7.7.2 Detailed Design

Black and Veatch were appointed as the consultants for both the water treatment works upgrade and the hydropower installation. The PenyCefn hydropower turbine was installed in tandem with an upgrade to the existing water treatment facility. This meant that the new installation did not require the construction of additional turbine housing. A space was left in the new water treatment works during construction for the installation of the selected turbine. However, it was decided to install a turbine after the planning permission and design of the works, so there were some lifting restrictions due to the ceiling height in the building.

7.7.3 Construction

Major replacement works to the existing Pen y Cefn Water Treatment Works were required to meet AMP5 water quality drivers. The treatment works was required to triple its capacity in providing potable water to the Dolgellau area from 30,000 litres to 90,000 litres per day while also providing a new Ultraviolet treatment facility. Black and Veatch were appointed the consultants for this upgrade, with construction provided by Dawnus. Construction began in 2010 and was completed in May 2011. A space was left in for the installation of a turbine during the construction of the treatment works. Figure 7.8 and Figure 7.9 show images of the construction of the treatment works upgrade.

Fig. 7.8: PenyCefn Treatment Works Construction

Dawnus were commissioned to undertake both civil engineering and building activities. Civil engineering work consisted of 4,000m^3 of earthworks, 900m^3 of concrete, 190 tonnes reinforcement, 1,500m^{2}of formwork and 500m of ductile pipe-work. Building work consisted of the installation of a steel portal frame, block-work and masonry, composite cladding and building finishes. The completed works were then landscaped and planted with trees and bushes of local provenance.

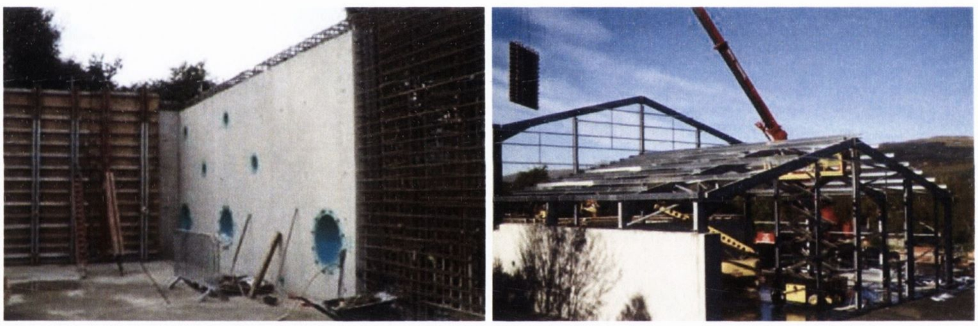


Fig. 7.9: PenyCefn Treatment Works Construction

7.7.4 Commissioning

The turbine has been up and running and generating since September 2011. The electricity is used on-site at the water treatment works. However, Llyn Cynwch is subject to a winter refill protocol. During winter periods, Llyn Cynwch is occasionally refilled from a low level river intake to supplement the natural catchment during dry weather spells. This is currently a grey area with OFGEM/MCS and WW are in negotiations to figure out a means to qualify for a REFIT. To guarantee the REFIT, the low level refill pumping station would need to be abandoned.

7.7.5 Discussion

This project was successfully completed, from initial feasibility to online and generating, in a total timeframe of approximately 1.25 years. Though the detailed feasibility study by Black and Veatch was completed in July 2010, the site would have been identified as having potential by the in-house Welsh Water Energy Bureau prior to that. This strategic site selection process successfully sped up implementation. Though the potential issue regarding the eligibility to obtain a REFIT was flagged at the feasibility stage, it was decided to go ahead with the project with the hope that it would become possible to claim a REFIT at a later stage. This decision has meant that at the moment, the turbine may be on course for an estimated 22 year payback as they have not yet been able to obtain a REFIT.

Despite the power generation capacity of this plant being relatively small (<10kW) and the payback period without REFIT being relatively long (estimated at approximately 22 years), the project was successfully completed in a short space of time and with very few problems encountered along the way. This is largely due to the supportive organisational structures and policies within Welsh Water. The procurement and tendering protocols, as well as the in-house site selection process by the Energy team, streamlined project implementation.

7.8 Cross Case Comparison

The two cases discussed differed in many ways, but both were successfully implemented. A summary of the two cases is provided in Table 7.3. The success of each of these two cases was compared based on the project outcomes using Nystroms success outcomes as a framework (Nystrom, 1985). The three outcomes, as discussed earlier, defined by Nystrom are the technological, competitive and financial outcomes.

Table. 7.3: Case study summary table

	Vartry Waterworks	PenyCefn Waterworks
Location	Vartry Reservoir, Roundwood, Co.Wicklow, Ireland	Dolgellau, Snowdonia, Wales
Client	Dublin City Council	Welsh Water
Nature of Organisation	Public company	Private company
Location of Turbine	Inflow to water treatment works	Inflow to water treatment works
Site Type	Retrofit	Retrofit with new treatment works build
Rated Power Output	78kW	8-17kW
Investment Payback Period	On track for 8 year payback period	Payback without REFIT was estimated at 22 years, if REFIT is obtained payback could be reduced to approx. 7 years
Driver for Install	Largely personal project driven by Vartry engineer-in-chief	Energy team targets as defined for the AMP5 Investment period
Technological outcome	The retrofit aspects of this project required handling large, old and intricate pipework which added to complexity.	The turbine technology in this case is unique as it regulates the output pressure.
Competitive outcome	Though there exist other similar sites within DCCs water supply district, no other hydropower projects have been implemented	Welsh Water have since installed further turbines of this type at other sites
Financial outcome	This project is on track for 8 year payback and successfully obtained an SEAI grant and REFIT tariff	Without REFIT, estimated 22 year payback; 7 year payback with REFIT.

7.8.1 Technological Outcome

The technological outcome, as defined by Nystrom, is a rating of the technological innovation or uniqueness of the project. Both of these projects were innovative in approaches, as the use of hydropower within water supply networks is an innovative solution to pressure reduction. However, the uniqueness of these two projects can be viewed from another perspective, with regards to the type of solution required. The installation at Vartry was similar to that of any smaller scale hydropower installation. Some complexities did arise however, due to the old pipework on site to which the turbine had to be retrofitted to. However, at PenyCefn, the turbine was installed in tandem with the new treatment works, so retrofitting was not a problem. The complexity of the design solution at PenyCefn was more technologically innovative because the turbine installed was required to maintain a constant pressure at the outlet to allow the water to reach the tank above. This would be similar to the installation of a turbine in place of a PRV, as has been discussed in the previous Chapters of this thesis.

7.8.2 Competitive Outcome

The competitive outcome is described by Nystrom (1985) as the interchangeability of the product with competing products already on the market. The Vartry hydropower turbine, as described previously, was installed at a reservoir between the reservoir and the treatment works. This set up is not unique, and therefore the technology could be applied to other reservoirs. As discussed in Chapter 4, there is similar power generation potential at other reservoirs managed by DCC, however, DCC have not installed any further hydropower turbines. DCC did not have a strategic 'system wide' plan, or long-term vision, to install further hydropower turbines. WW however, since the completion of the PenyCefn treatment works in 2011, have gone on to install a number of other turbines at treatment works within their water supply district. In terms of the competitive outcome and the replicability of these installations, the WW project has been more successful than the DCC project.

7.8.3 Financial Outcome

Nystrom (1985) defined the financial outcome as a measure of the profitability of the product over its life cycle. The turbine installation at Vartry was more successful financially than the PenyCefn hydropower project. To measure the financial success of each project, a comparison was made of the forecasted investment payback periods. The financial case is improved for Vartry due to the fact that the power generation at Vartry is higher (78kW) than at Penycefn (8-17kW). It was also due to the fact that the specialised turbine selected to install at PenyCefn was expensive, despite the low generation capacity. The power generated at Vartry is enough to cover all energy usage at the water treatment works with the excess sold to the grid. In terms of investment payback, the Vartry hydropower turbine is on target for an 8 year investment payback. At the moment, the PenyCefn hydropower turbine has yet to receive accreditation to allow it to qualify as a renewable energy source. It is therefore still on course for a 22.5 year investment payback period. However, WW are still pursuing accreditation which may be achieved in the future.

7.8.4 Stakeholder Involvement

For both of these new hydropower projects implemented within the water industry, there was a large network of stakeholders involved that were required to collaborate and integrate, as illustrated in Figure 7.10. The management of these different stakeholders differed for each of the two case studies presented.

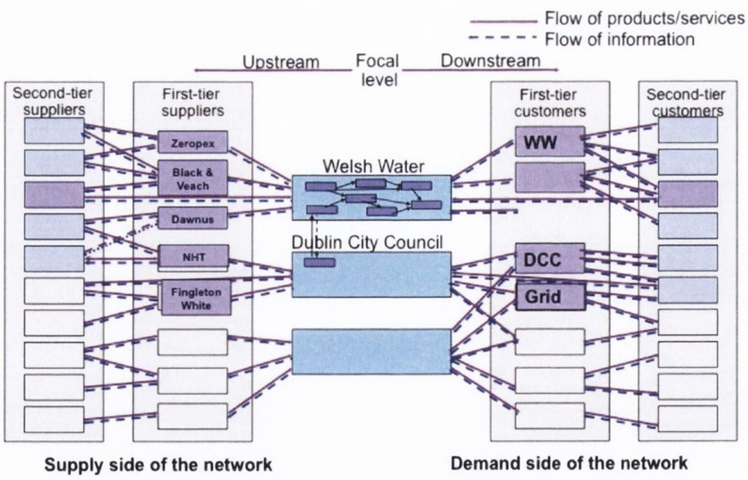
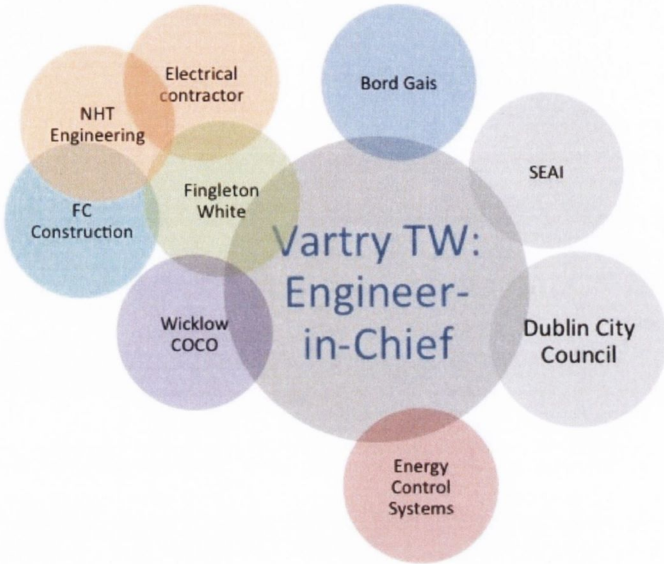


Fig. 7.10: Water supply projects consist of many interconnecting networks

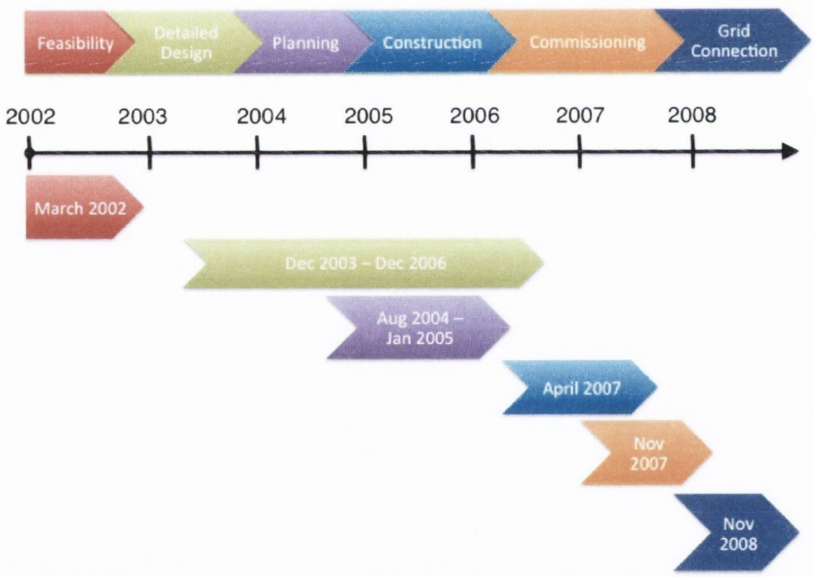
Figure 7.11 illustrates the stakeholders involved at each project stage for the Vartry case study. The colour of each stakeholder bubble in the organisation map illustrates the stage during which each stakeholder was active during. The timeline follows the same colour code and provides an overview of the duration of each of these stages. The diameter of each bubble illustrates the relative impact each stakeholder had on the project. For the Vartry project, the key stakeholder was found to be the CE on site at the treatment works. DCC as an organisation, though the owners and operators of the plant, played a limited role in the project. The PenyCefn project, in contrast, was well supported by Welsh Water, the owners and operators of the plant. Welsh Water, in organisational context, were actively supportive, with many supportive institutional structures and policies in place to streamline implementation.

Figure 7.12 illustrates the stakeholders involved at each project stage and the related timeline for the PenyCefn case study. The construction stage for PenyCefn was shorter than for the Vartry hydropower project because it only involved the turbine delivery, installation and set-up on site. This was primarily as a result of installing this turbine in tandem with the treatment works upgrade. At Vartry however, construction required the handling of intricate, old pipework from the reservoir, the retro-fitting of the turbine to this pipework, and the construction of a new turbine house. Furthermore, planning permission was sought for the Vartry turbine installation which was not required for the PenyCefn installation. It should be noted that while delays were not present for WW on this project, other WW hydropower sites have experienced significant delays due to legislative and planning requirements.

Another key difference between WW and DCC that led to delays for the DCC project, yet accelerated the implementation of the WW project, was the tender protocol of the two organisations. With the Vartry project, a number of tender calls were required during the project, to select, variously, a company to undertake the feasibility study, the consultants, the turbine suppliers and the contractors. However for WW, the tender process was very different. Consultants and contractors are approved as the suppliers for each AMP Investment Period. This approves them for a period of 5-15 years. Contractors are approved as Tier 1 or Tier 2 etc. A tender call is released, and tenders are awarded to a list of approved suppliers and consultants prior to each investment period. Black and Veatch were one of the approved consultants for this period in the north Wales region and therefore were automatically selected for both feasibility and consulting work. This resulted in the process running smoother and more quickly, which is evident in the timeline. The length of time spent on the detailed design stage of the Vartry project included time spent on each individual tender call.

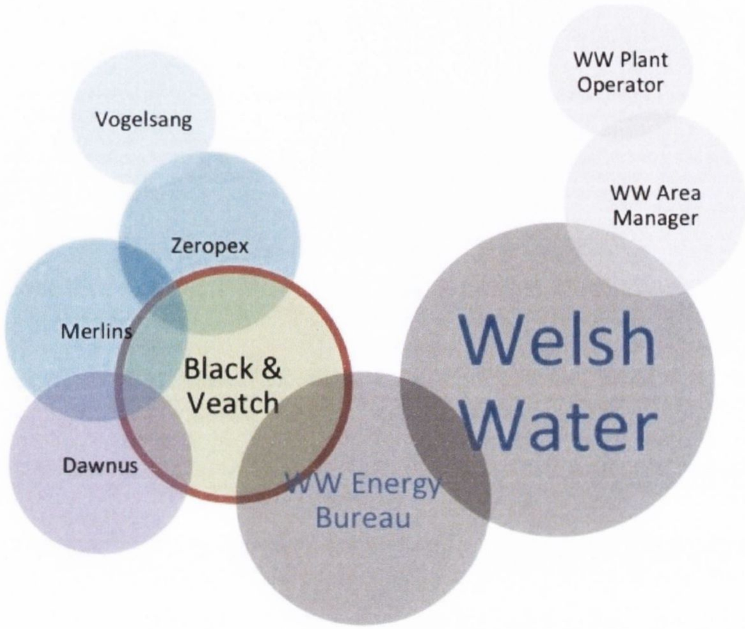


(a)

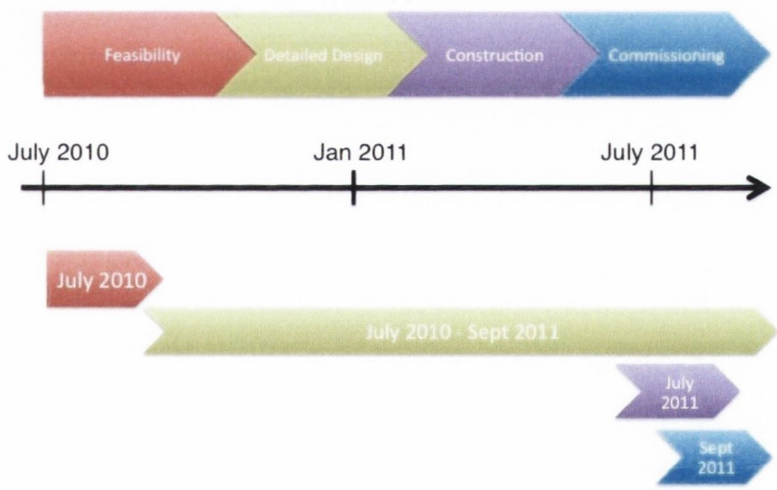


(b)

Fig. 7.11: (a) Vartry organisation network and (b) Vartry project timeline



(a)



(b)

Fig. 7.12: (a) PenyCefn organisation network and (b) PenyCefn project timeline

The timing and stakeholders active at each stage for both the Vartry and PenyCefn projects are summarised in Figure 7.13. From these diagrams, it can be seen that the interaction and integration during each of these projects differed. From the Vartry project diagram, it can be seen that different stakeholders or players became involved separately and one-after-the-other in a relay-race form of integration as described by Gehani (1992). For example, the initial feasibility study was carried out by one company, who then passed the baton to the consultants to complete the detailed design. DCC on this diagram is representative of the CE at Vartry, and project 'champion' as discussed previously. Meetings between the CE and other players were largely bilateral, with the consultants meeting separately with the turbine supplier and contractors.

At PenyCefn however the interaction differed slightly. Firstly, the number of players involved was fewer. Also the same company undertook the feasibility study, detailed design and the project management. The interaction evident on this project was more consistent with a parallel or rugby styled approach as described by Gehani (1992). Different players integrated and interacted over a number of stages. This enabled the opportunity to have ongoing feedback and interaction throughout each phase.

	Feasibility	Detailed Design	Planning	Construction	Commiss-ioning	Grid Connection	Total:
Dublin City Council	X	X	X	X	X	X	6
ECS	X						1
		X	X	X	X	X	5
			X				1
FC				X			1
nht					X		1
Elec					X	X	2
Total:	2	2	3	3	4	3	

(a)

	Feasibility	Detailed Design	Construction	Commissioning	Total:
Dŵr Cymru Welsh Water	X	X	X	X	4
BLACK & VEATCH	X	X	X	X	4
dawnus			X		1
zeropex	X	X	X	X	4
Merlins			X	X	2
Total:	3	3	5	4	

(b)

Fig. 7.13: (a) Vartry: Actors versus Stage; (b) PenyCefn: Actors versus Stage

Figure 7.14 illustrates the range of different players that were involved in the Vartry project, both within DCC and on site. Together these players interacted with each other to design, develop and implement the new hydropower project. Understanding the role of each player involved in this design process, referred to by Verganti (2008) as the design discourse, can lead to increased learning and improved solutions for project implementation. A similar design discourse was present in the PenyCefn project, however the presence of a designated Energy Team within WW would facilitate learning and knowledge from the project to be retained and re-applied for further projects.

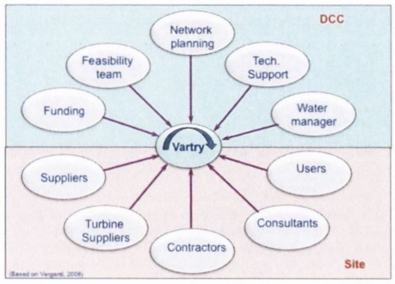


Fig. 7.14: Design Discourse in DCC (adapted from Verganti (2008))

7.8.5 Barriers and Enablers

For each case study, the presence of any of the twelve barriers to sustainable urban water management (SUWM), as defined by Brown and Farrelly (2009) and discussed in Chapter 2, was investigated. The results of this investigation are summarised in Table 7.4.

From this analysis, it can be seen that the Vartry project had many barriers to overcome, including the institutional framework within DCC, and the limited organisational commitment. This was also consistent with one of the key components Jalba (2014) identified that may be deficient between water utilities and other organisations, the presence of a supportive regulatory environment. However, despite these barriers, the project was implemented successfully. This was largely due to the personal interest and leadership skills of the CE on-site at the water treatment works. The CE can be seen to have acted as an emergent leader or project champion as discussed previously in the critical literature review in Chapter 2. The role of the CE in the completion of this project demonstrated the need for an emergent leader, or project champion, on an innovative project like this.

The barriers mentioned identified during the Vartry case were largely socio-institutional rather than technical. This was consistent with the theory presented by Brown and Farrelly (2009) and Taylor (2009), that within an environment including numerous socio-institutional barriers, emergent leaders may come to the fore to act as change agents.

Table. 7.4: Barriers to implementation

Barrier to Implementation (Brown and Farrelly, 2009)	**DCC Vartry Project**	**WW PenyCefn Project**
Uncoordinated institutional framework	✓	
Limited community engagement, empowerment and participation		
Limits of regulatory framework	✓	✓
Insufficient resources (capital and human)		
Unclear, fragmented roles and responsibilities		
Poor organisational commitment	✓	
Lack of information, knowledge and understanding in applying		
Integrated, adaptive forms of management		
Poor communication	✓	
No long-term vision, strategy	✓	
Technocratic path dependencies		
Little or no monitoring and evaluation		
Lack of political and public will		
TOTAL:	**5**	**1**

In the case of the PenyCefn hydropower project however, the installation was much more deliberate, following strategic investment into the planned development of a number of energy recovery projects during that AMP5 investment period (2010-2015). A strategic long-term vision was in place, this hydropower project was one of many to be developed as part of a strategic and coordinated investment plan. There were other drivers, such as the Water UK agreement whereby all water authorities in the UK have agreed to meet a voluntary target of reducing their carbon emissions by 20% by 2020. Also WW have set their own targets to reduce emissions by 25% of 2007 levels by 2015, and by half by 2035 (*Environment and Sustainability Committee: Inquiry into Energy Policy and Planning in Wales*, 2011). The presence of a dedicated department within Welsh Water, dealing solely with the development of energy related projects also demonstrates the level of institutional and managerial support provided by WW. With these institutional frameworks and supports in place, the necessity for a project champion to emerge was eliminated.

7.8.6 Policy and Incentives

One key decision, clear from both cases, in the early stages of project is whether to use electricity on site or sell to grid. If selling to grid, there is a need to engage with potential buyers as early as possible. Whether using the electricity on site or selling to grid, early clarification as to whether the scheme would qualify for a REFIT tariff is also essential. With the WW case, this issue was flagged at the feasibility stage, and WW approached the accreditation department for qualification for renewable energy feed-in-tariffs to discuss solutions. Vartry however, did not engage early with electricity buyers, with engagement beginning only when the turbine was ready to run. Evident in these case studies was the need to engage with potential buyers as early in the process as possible. Failure to do this for the Vartry case led to a year long delay in securing a buyer, leading to the loss of a year's revenue. Figure 7.15 illustrates the impact of earlier management focus in relation to this issue of securing an electricity buyer. In this figure, the actual Vartry project management activity to secure an electricity buyer is illustrated in red, while the blue area represents a more optimal management activity profile. The earlier the engagement to find a buyer begins, the more ability and time there is available to resolve the issue. Later engagement would lead to project delays, resulting in loss of revenue.

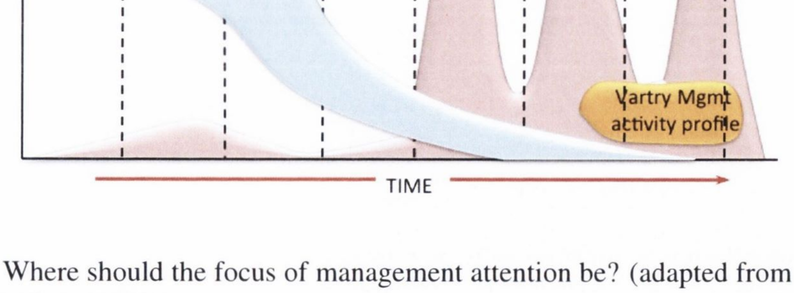


Fig. 7.15: Where should the focus of management attention be? (adapted from Wheelwright and Clark (1992))

7.9 Planning Permission

In the Vartry case, planning permission was sought despite the consultants recommending that it was not necessary. There was no requirement for them to apply for planning, however as a gesture to ensure no local issues with the project, planning permission was sought. This decision added five months to the project timeline and also increased the overall project costs. Though it is important to engage with the local community on planned new developments, the need to do this should be . The Penycefn hydropower project, on the other hand, did not require any specific planning permission as the turbine was housed within the new treatment works. This was a major benefit of installing a hydropower turbine concurrently with upgrades to water supply infrastructure, saving both costs and time.

7.10 Summary

The main conclusions drawn following the case analyses were that in the absence of a supportive socio-institutional framework, the presence of an emergent leader or project champion, can aid in project implementation. However, the presence of supportive institutional frameworks, mechanisms and a dedicated energy team, as has been developed and nurtured within Welsh Water, has been shown to be an effective method of streamlining project implementation. It also provided a mechanism for learning through the installation of a number of turbines within each investment period.

The most influential risk to project viability and profitability lies in the ability of obtaining a REFIT. This was not a major risk for the Vartry project, due to the higher power output available and hence the increased revenue generation ability at the site. However for smaller scale power generation sites, such as the 8-17kW PenyCefn project, and likewise with many of the potential sites identified in the DCC and WW networks as discussed in Chapter 4, the ability of obtaining a REFIT could be the deciding factor as to whether to proceed with an installation or not.

Following the case description and case analyses, a modified version of Coopers Stage-Gate model was developed, with future micro-hydropower (MHP) projects in mind, as shown in Figure 7.16.

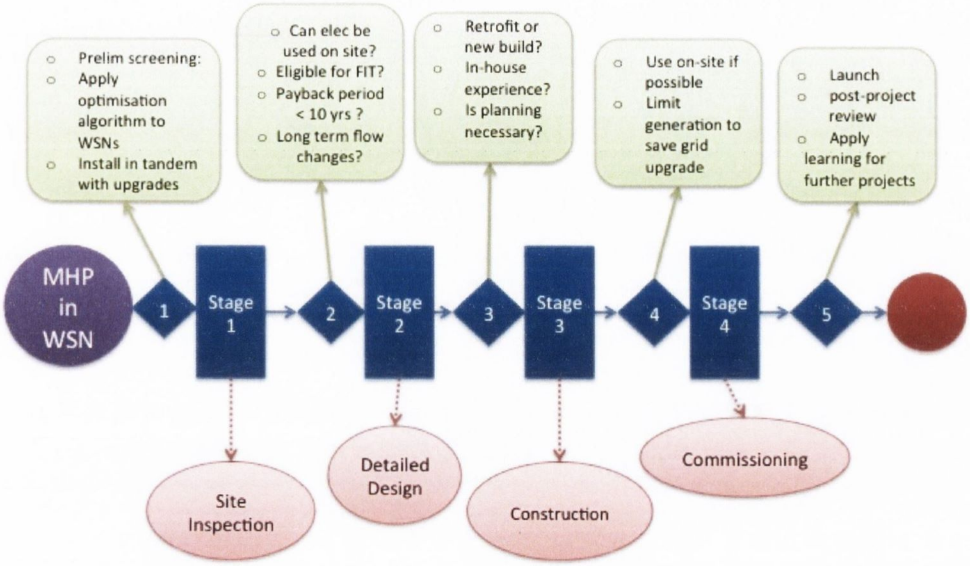


Fig. 7.16: Chapter 7 Updated Framework

The first gate was focused on the identification of hydropower opportunities within the WSN to further investigate. Firstly, existing locations in the WSN of interest should be considered. These include PRVs, BPTs and inlets or outlets to tanks, reservoirs, treatment works or at compensation flows. Furthermore, an optimisation algorithm such as that developed and presented in Chapter 6 could be applied to the WSN for identification of any new locations with hydropower generation potential. Planned expansion and upgrade works should be investigated for the possibility of including hydropower installations alongside these construction works. An initial calculation of the estimated power output at these sites should be evaluated, assuming average flow rates, pressure drops and a conservative system efficiency of 65%. At this stage, it is recommended that sites with estimated power outputs of less than 1kW should be discounted.

Once a list of potentially feasible sites has been identified, the next stage is to investigate the flow and pressure conditions present in more detail and to visit the site. Investment payback periods should be calculated with consideration paid to flow rate and pressure variation, turbine selection, and projected future changes in flow rates. If investment payback can be achieved within ten years then the project should progress to Stage 2, detailed design. Furthermore, sites with no pre-pumping that are eligible for REFITs should be prioritised for development.

Stage three was defined as the detailed design stage, followed by Stage 4, construction. Key decisions at these stages include whether or not planning permission is necessary, whether the electricity can be used on site, and whether it would be more economic to reduce the overall power generation capacity in order to save on the grid upgrade fees. Another key factor to consider prior to the detailed design and construction phases relates to the contractors and consultants involved. Where possible, contractors and consultants who have worked on previous MHP projects with that water company should be reappointed to retain and re-apply any prior learning. Following project commissioning and launch, a post-project review process should be undertaken with all key players to document any further lessons learned for future MHP projects.

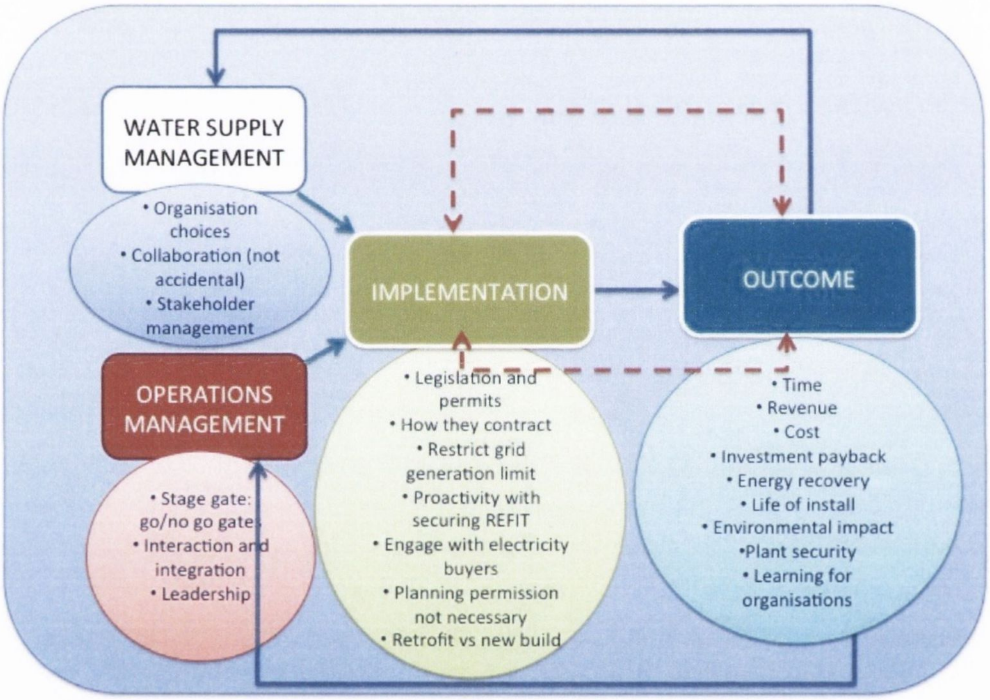


Fig. 7.17: Chapter 7 Updated Research model

Through the development of these two case studies, and through the cross case analysis presented, the research model presented in this chapter has been updated as shown in Figure 7.17. The key findings from the literature consulted and the main conclusions drawn following the cross-case analysis are summarised beneath each heading. The theory and frameworks presented from both water supply management and operations management literature can be incorporated into the implementation approach. The MHP Stage-Gate framework for management of operations could be applied to accelerate project implementation. The need for project leadership was also highlighted. This leadership can either be from one key player involved in the project or otherwise can be organisational leadership through the development of supportive institutional policies and frameworks within the water company.

Other conclusions drawn following the cross-case analysis that should be considered at the implementation phase include the need to be proactive with securing a REFIT and an electricity buyer. This could improve both the cost and the time outcome of a project. These recommendations at the implementation phase will impact on project outcomes, listed as shown in Figure 7.17. Stemming from these outcomes, further lessons can be learned which can then feedback into the implementation of future projects. These outcomes can also feedback into the development of future theory and frameworks to be used in water supply management and operations management. Otherwise, they can supplement previously presented theories, such as was evident in this research, with the need for an emergent leader to drive a project through to completion in the presence of socio-institutional barriers.

The outcomes and benefits of the implementation of MHP in WSNs are outlined. Time and cost effective implementation of innovative hydropower energy recovery schemes results in increased revenue generated, a reduction in the carbon footprint of the water industry and further improvements in the energy security of the water industry.

CHAPTER 8

Discussion

8.1 Introduction

This Chapter summarises the key results presented in this thesis, discussed under the four core themes as illustrated in the thesis research model presented in Figure 8.1: Feasibility, risk analysis, optimisation and implementation.

8.1.1 Feasibility

The focus of this chapter was to present the results of an analysis of the potential for micro-hydropower (MHP) energy recovery in the water supply networks of Ireland and Wales. It was clear from this feasibility study that energy recovery potential exists within water supply networks, however many factors must be considered. Key influential factors relate to the diurnal flow rate variation, suitable turbine selection and variable turbine operating efficiencies.

The results of the feasibility study of 174 potential sites, show that there is potential

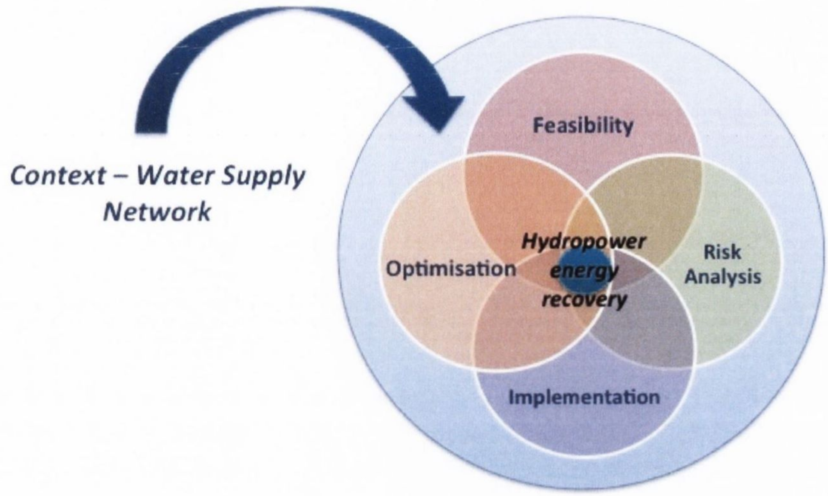


Fig. 8.1: Thesis research model

for MHP energy recovery in the water supply networks of Ireland and Wales. Some of the larger capacity sites were shown to have an estimated power output of over 100kW. An overview of these estimated power outputs are summarised in Table 8.1 below. In total, 1.3 MW of electricity could be generated from these 174 sites. Assuming all this electricity is used by the WSPs, this could lead to a saving of €1.5 million annually (assuming Irish industrial electricity price). In Ireland, annual operating costs of the water industry have been reported as costing €600 million. This €1.5 million saving is small but not insignificant, and along with other energy efficiency measures, such as optimised pump scheduling and anaerobic digestion, could play an important role in the development of more sustainable water supply networks.

Table. 8.1: Overview of Estimated Power Outputs

Estimated Power Output	Number of Sites	% of Sites
<5 kW	122	70.11
5 - 10 kW	26	14.94
10 - 20 kW	14	8.05
20 - 50 kW	8	4.6
50 - 100 kW	2	1.15
>100 kW	2	1.15
Total	**174**	100

Increased power generation potential was found at sites within large cities, where flow rates would be higher due to the increased local water demand. With the current global trend of increased population growth in urban areas, it is expected that water demand and flow rates in urban water mains will also grow, further increasing the total power generation capacity.

The majority of the sites investigated (70%) garnered power generation capacities of less than 5kW. These sites would be expensive to install with current MHP technology, however with the potential application of PATs at these sites the economic case was found to be greatly improved. There is ongoing research in the application of PATs to operate as PRVs. Technical solutions exist however they have not yet reached widespread market application. Another opportunity to improve the economic success of small scale capacity sites was found to be using electricity generated locally rather than connecting to the grid. Using electricity on site would improve the economic benefits by reducing large overheads associated with cabling costs and grid connection fees.

The types of existing infrastructure locations for MHP energy recovery that were investigated were typical of WSNs worldwide, and therefore the findings presented in this study would find application in WSNs elsewhere. Furthermore, the focus of this research was on water supply network infrastructure only, however MHP energy recovery would also have application in other piped water systems such as irrigation networks, wastewater and drainage systems or water-intensive industries. Due to the fact that Ireland and Wales have climates with heavy rainfall, large-scale piped irrigation networks are not necessary. However, in countries with arid or Mediterranean climates, large scale piped irrigation systems are essential. These systems would present further opportunities for the application of MHP. MHP could prove to be of significant economic value in these networks.

Turbine selection and turbine efficiency were shown to have a significant effect on the amount of power generated. Assuming constant turbine efficiency, along with average flow rates and pressures led to overestimated power capacities for many sites. Some cases led to highly overestimated generation in the region of 36-56%. This would be of major significance when estimating the expected investment payback period for new projects. In particular for cases where investment payback is estimated to be achieved after 8-10 years, because flow rates could vary considerably over these longer periods.

From the investment payback analysis it was found that the presence of renewable energy incentives such as FIT schemes had a major effect on the financial viability of MHP projects in WSNs. For the majority of the cases investigated, the most economic option, where possible, was to use the electricity generated on site. However, this would also vary across different countries depending on the electricity prices and REFIT tariffs.

The turbine efficiency and selection analysis highlighted the need to consider variable turbine efficiencies during the feasibility stage of any new hydropower project. The impact of long term changes in water demand on MHP potential was highlighted. The influence of population changes and climate change were also highlighted as areas requiring further investigation. Further research in this area would strengthen the investment case for new MHP projects, through the thorough analysis of longer term, more detailed data.

8.1.2 Risk Analysis

A long term study of flow variation at ten potential MHP locations within both Irish and Welsh WSNs was then undertaken. Climate and socio-economic data were investigated for correlations with long term changes in flow rates at these valves. Both multi-variate linear regression (MLR) and Artificial Neural Networks (ANNs) were applied to develop predictive models for future climate and socio-economic scenarios.

Initial MLR analyses were applied to investigate the key influential variables impacting long term flow changes at these valves. The ANN models were found to out-perform the MLR models in their explanation of the variation in flow data. This is likely due to the ability of ANNs to identify non-linear trends in the input model data. The primary limitation of ANNs for future flow rate prediction is that they require the same amount of input data as supplied to make the model. This would mean for the ANN models developed based on ten years of quarterly economic and climate data, a 2020 forecast would require ten years of future estimated economic and climate data (i.e. for 2010-2020) as model inputs in order to predict the equivalent flow rates. However to forecast the flow rate in 2020 using MLR, future climate and economic data for 2020 alone would be required. MLR therefore requires less assumptions and less input data. For shorter term predictions (1-2 years), ANNs would be recommended however, because the input data for the first 8-9 years would be known.

The most significant relationship, common to both the Irish and Welsh valves, were between water flow rates and the local population. For the Welsh valves investigated, the most frequently reported influential factors on long term flow variation were found to be the population change and the change in the reported leakage rates from those WSNs. For the Irish valves, changes in the population, rate of unemployment, construction activity and the amount of water leakage were the most frequently reported influential variables. For Wales, the price of water was also included as an influential factor.

A sensitivity analysis of the regression results for the Welsh Water valve V14, demonstrated the effect that increases and decreases in the water price could have on the estimated average flow rates at that valve. A price increase of 5% was estimated to reduce the average flow rate by almost 20%, while a price decrease of 5% could lead to an increase in flow rates of 30%. This assumed that no change in population or temperatures took place.

Both the MLR and ANN analyses provided methods of forecasting average quarterly flow rates at valves within water supply networks. MLR analysis provided an indication of the level of correlation between water flow rates and the independent variables tested. The most significant relationships, common to both the Irish and Welsh data, was the relationship between water flow rates and the local population. Water price was shown to have a significant influence on flow rates at 3 of the 4 WW valves studied. MLR provides an indication of long term changes in flow rates, due to economic and climatic changes. The MLR models can be used to forecast future average flow rates at potential hydropower locations. This forecasted average flow rate combined with the application of the characteristic average diurnal water demand pattern for each valve, allows for accurate forecasting of flow rates and hydropower generation capacities into the future.

Future scenarios were then modelled for valves based on published forecasts for future population, unemployment rates and climate factors to predict average quarterly flow rates for the years 2020 and 2030. For all future scenarios modelled, flow rates were predicted to increase by 2020, largely due to the forecasted increasing populations of both Ireland and Wales. These increased flow rates would increase the hydropower generation capacity at these sites. However, with large increases in water demand, new water resources may need to be developed, which may render certain parts of the network obsolete. This risk should also be noted at the feasibility stage of any hydropower project. Overall, MLR and ANN analyses were found to be good methods for approximation of flow rates at potential hydropower locations within WSNs. MLR is the most accessible method and requires less assumptions for long term forecasting and is therefore recommended as the preferred option for long term flow rate forecasts (10 years plus).

It is important to remember for all future scenarios predicted that the future is unknown and unknowable. The models applied can predict estimates for future scenarios based on the analysis of previous trends, however factors of error must always be included, and a variety of different scenarios planned for.

8.1.3 Optimisation

An optimisation algorithm was developed to select optimal locations in WSNs for the installation of hydropower turbines. Three optimisation techniques were tested to find optimal solutions, two mathematical techniques and also a genetic algorithm (GA). The mathematical approaches tested included non-linear programming (NLP) and mixed integer non-linear programming (MINLP). The non-convex nature of the objective function and the constraints for this formulation meant that a global optimum could not be guaranteed. However, through tightening of constraints and improvements in the initial start point, a number of local optimum points can be found. The global optimum solution could then be deciphered as the optimal solution of this set of local optimum solutions. GAs, though popular due to their ease of use, cannot guarantee a global optimum being found. They may stop and report an optimal solution as the solution found is 'better' than the previous solutions.

The three optimisation techniques were initially tested on a small 5-node WSN. The GA solver resulted in the least accurate solution, and also took the longest time to solve. It stopped solving and reported an optimal solution at a total power generation capacity of 11.74kW, while both the NLP and MINLP solutions reported optimal solutions of 14.99kW. The reported flow rates and pressure heads as computed by the GA were also less accurate than the equivalent results of the NLP and MINLP models when compared to the results of an EPANET hydraulic analysis. It was therefore decided to proceed with the NLP and MINLP techniques for solving the next two more complex WSNs.

MINLP was found to be the most suitable approach for the optimisation of the case WSNs presented. This is primarily due to the ability to model integer variables, which is required when choosing an exact location to install a turbine. However NLP was shown to be computationally inexpensive with solutions found for the 25-Node network in a maximum time of 44.3 seconds. For the larger 25-Node Network, the NLP solutions presented were also useful for investigating where the locations with the most power generation potential were in the network. These locations could then be tested as the initial solution for the MINLP models, improving the solving time to find the optimal MINLP solution. The optimal MINLP solution for the installation of three turbines was found in under 150 seconds. However, a quicker solution (in 61.06 seconds) could be found using a modified initial start point, x_0, using the locations identified following the NLP analyses as initial locations for turbines.

A comparison with the results of the 25 Node WSN model and an equivalent EPANET hydraulic analysis was found to be very favourable. The largest relative errors between the pressure heads was found to be 0.02% (approximately 0.01m). The largest relative error between the flow rates was found to be 1.96% (approximately 0.01 l/s).

MINLP was found to be a suitable optimisation technique for solving this objective. It was also found to be less computationally expensive than the GA solution was for the 5 Node network. An optimal MINLP solution for the installation of three turbines on the 25 Node Network was found in under 150 seconds. Furthermore, a quicker solution (in 61.06 seconds) could be found using a modified initial start point, x_0, using the locations identified following the NLP analyses as initial locations for turbines to be installed. The GA solution for the 5 Node network was found in 341.31 seconds, more than double the solution time for the MINLP solution of the larger 25 Node network.

The hydropower optimisation formulation as presented in this thesis can be applied by WSPs to identify new points to install hydropower turbines in WSNs. It could be considered as another option for pressure management in WSNs during planned improvements or upgrades of networks.

8.1.4 Implementation

The potential for hydropower energy recovery within WSNs is established and has been reported on in the literature. Further available potential within the Irish and

Welsh WSNs was discussed following the feasibility study in Chapter 4. However, few projects have been implemented in practice, particularly at PRV locations. Though technical solutions exist, potential organisational, management and regulatory issues could be preventing widespread implementation. To investigate the organisational, management and regulatory aspects involved in the installation of MHP turbines in practice, two case studies were developed of completed hydropower projects installed on water supply infrastructure.

There is a need for a more integrated approach to the management of both energy and water. This was highlighted by the UN during this year's World Water Day, which aimed to increase awareness of the interdependencies and linkages between these two essential resources. Further aims of this campaign included the facilitation of the development of cross-cutting frameworks in order to lead the way to energy security and sustainable water use. The implementation of hydropower projects on water supply infrastructure require just such a cross-cutting framework. As has been reported in Chapter 2, the water industry is a large energy user, estimated to be responsible for 8% of total global energy usage (United Nations, 2014*b*). There is an urgent need to reduce carbon emissions in order to mitigate climate change impacts and meet the legally binding agreements of the Kyoto protocol. Clearly evident in both of the case studies presented was that current legislation is not conducive to the installation of hydropower turbines in WSNs.

Hydropower legislation in both Ireland and Wales is aimed at either run-of-river schemes or at large scale hydropower schemes. For the Vartry case, the numerous applications and permits required led to significant delays in the project implementation. Furthermore, a planning application was made which led to another delay. WW have flagged similar issues with current Welsh legislation. Many delays have occurred at WW hydropower sites due to the requirements to undertake large-scale environmental impact assessments, apply for abstraction licenses and planning permission. WW presented these concerns to the Welsh Government in September 2011, requesting that amendments be made to the current regulatory requirements. It was argued that the requirements for micro-scale hydropower sites should not be the same as for large-scale hydropower and pumped-storarge stations (*Environment and Sustainability Committee: Inquiry into Energy Policy and Planning in Wales*, 2011). Moreover, it was noted that one of their main frustrations was the length of time it can take for all consents to be approved and obthe impacts of climate change.

During the case studies undertaken, further risks were identified in relation to both selling electricity to the grid and also the ability to obtain a REFIT. For the WW case, the most influential risk to project viability and profitability was found to be the ability of obtaining a REFIT. This was not a major risk for the Vartry project, due to the higher power output available and hence the increased revenue generation ability at the site. However it was a major risk for smaller scale power generation sites, such as the 8-17kW PenyCefn project, and likewise with the majority of the potential sites identified in the DCC and WW networks discussed in Chapter 4. The ability of obtaining a REFIT could be the deciding factor on whether to proceed with an installation or not.

A number of conclusions were drawn following these case analyses and the cross-case comparison. It has been shown that in the absence of a supportive socio-institutional framework, the presence of an emergent leader or project champion can aid in project implementation. However, the presence of supportive institutional frameworks, mechanisms and a dedicated energy team, as has been developed and nurtured within Welsh Water, has been demonstrated to be effective in streamlining project implementation. It also provides a mechanism for learning through the installation of a number of turbines within each investment period, and through maintaining contracts with consultants, contractors and suppliers over minimum periods of 5 years.

8.2 Summary

In this chapter, the key results and findings presented in this thesis were discussed. In summary, the feasibility study has demonstrated the overall power generation potential for hydropower in WSNs with a focus on the WSNs of Ireland and Wales. Flow rate variation, turbine selection and turbine cost, have been shown to be key factors when determining the investment payback periods of potential projects. Through an analysis of ten years of long term flow rate data, a methodology for future forecasting of flow rates at potential hydropower locations was presented. An optimisation algorithm was developed which could be used by WSPs to select new locations for hydropower energy recovery in a given WSN. Finally, key issues influencing the smooth implementation of MHP technology in practice as identified during the case studies presented in Chapter 7 were discussed. The four core themes as presented and discussed in this research; Feasibility, Risk Analysis, Optimisation and Implementation, provide a holistic response to the overarching research question addressed in this thesis: *What is the potential for hydropower energy recovery from the water supply network?*

CHAPTER 9

Conclusions

9.1 Introduction

This Chapter presents a summary of the key conclusions, highlights the primary contribution to knowledge and outlines any resulting policy implications, recommendations and areas requiring further research. As outlined in Chapter 1, the overall research question addressed in this thesis was: *what is the potential for hydropower energy recovery from water supply networks?* In order to investigate this potential, the following sub-questions were addressed: what is the feasibility for energy recovery in the water supply network (WSN), how do uncertainties and variations affect feasibility, how can a WSN design be optimised for energy recovery and how can energy recovery projects be implemented and replicated in practice from organisational and operations management perspectives?

9.2 Contribution to Knowledge

Th primary contributions of this thesis were to the research and practice of hydropower energy recovery in WSNs. These contributions include:

1. A critical literature review of the following streams of research;
   i Hydropower energy recovery in the water supply network
   ii Flow rate variation and forecasting in water supply networks
   iii Optimisation and its application to water supply network design
   iv Project implementation in the context of the water industry
2. Analysis of the potential for hydropower energy recovery at existing locations in the WSNs of Ireland and Wales
3. An application of multiple linear regression (MLR) and artificial neural network (ANN) forecasting techniques for the long term prediction of water flow rates at potential hydropower energy recovery locations within WSNs
4. The development and application of an optimisation algorithm for selection of new locations to install hydropower turbines in WSNs for maximised power generation
5. An analysis of organisational, management and regulatory issues associated with the implementation of hydropower energy recovery projects in WSNs

9.3 Research Findings

This thesis research has resulted in a number of key findings. One key finding was that significant untapped potential exists for the recovery of hydropower energy in the WSNs of Ireland and Wales. The largest untapped resource, in terms of the number of potential sites and the overall combined generation capacity was found to be at pressure reducing valves. The total hydropower generation resource available at the Irish and Welsh sites investigated was found to be over 1.3MW. If all of this electricity were used by the WSPs, this would result in a total annual saving of approximately €1.5 million. Flow rate variation and hence suitable turbine selection was found to greatly impact upon the estimated power generation capacity at potential sites.

The long term study of flow rate variation at potential locations for MHP energy recovery confirmed that population was the key determinant of these long term flow changes. Furthermore, the impact that changes in water price would have on flow rates was demonstrated at the Welsh Water valves. A 5% increase in the average household bill was found to reduce water flow rates by almost 20%. This is an important finding, especially for Irish Water and the planned introduction of water charging in Ireland for 2014/2015. The incoming addition of water charges in Ireland could also result in flow rate changes at potential hydropower locations.

Projections for future population growth were found to strongly influence future flow rates. This is likely to result in increased flow rates, especially in urban areas, which could increase the power generation potential. This may require turbines to be selected which can operate efficiently at higher flow rates than the current in-situ flow rates.

Both ANNs and MLR models were applied for the forecasting of future flow rates at potential hydropower locations within WSNs. Forecasting techniques have previously been applied to the estimation of system wide water demand and also to the estimation of water usage by the end user. However, to the author's best knowledge, forecasting techniques have not yet been applied to estimate flowrate at points within water supply mains. This analysis found that ANNs more accurately explained the variations in flow present in the ten year input data. However long term future forecasting with ANNs requires more assumptions and more input data, hence MLR was found to be a more appropriate method for long term flow rate forecasting. However, it is recommended that ANNs be employed for shorter term forecasts (1-2 years).

Another key contribution of this research was the development of an optimisation model for the identification of new locations within WSNs where power generation potential exists. This optimisation algorithm takes in typical input information for a WSN hydraulic analysis, such as pipe lengths, diameters, nodal elevations and demands. Hydraulic constraints are applied to ensure the conservation of energy and mass was adhered to throughout the network. Minimum and maximum pressure limits can be specified for each node. The option to install a turbine was modelled through the addition of an extra decision variable to represent excess available pressure to drive a hydropower turbine. This also reduces the overall network pressures closer to the minimum required pressure limits, which results in a reduction in the amount of leakage from the network. Reducing the overall network pressures would reduce both the intensity of any leaks, and would also reduce the likelihood of pipe bursts occurring. The optimisation algorithm formulated was solved using a genetic algorithm, non-linear programming and mixed integer non-linear programming (MINLP). MINLP was found to be the most suitable method for this application and has not previously been adopted for the optimal location of hydropower turbines in WSNs.

Through the case research undertaken and presented in Chapter 7, key risks involved in the implementation of hydropower projects in WSNs were identified. Regulatory requirements involving applications and consents were numerous and caused delays to the Vartry project. The ability to obtain a REFIT had a major impact on the financial viability of the PenyCefn project, doubling the estimated investment payback period. Furthermore, the Vartry project suffered further delays due to difficulties finding a buyer for the electricity.

Following the case analyses and critical literature review, an updated Stage-Gate model (Cooper, 2008) was developed for the acceleration of the implementation of future micro-hydropower (MHP) energy recovery projects on WSN as shown in Figure 9.1. This presents the key activities required at each stage, and the go/kill criteria for each gate, in order to accelerate suitable projects and filter out unsuitable projects as early as possible. This model combines conclusions drawn from the four core themes of this research.

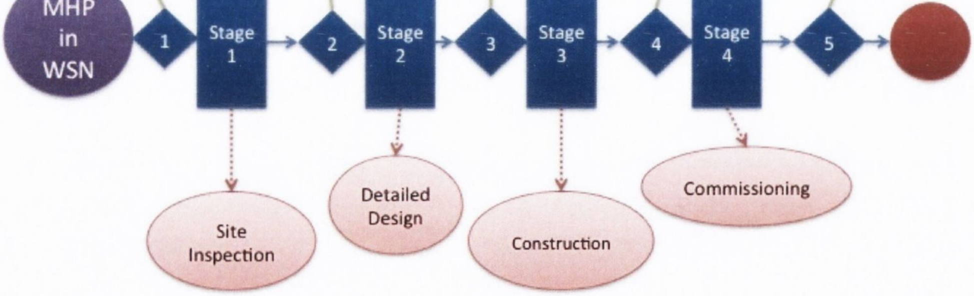


Fig. 9.1: Chapter 7 Updated Framework

Finally, this work has demonstrated the value of undertaking cross-disciplinary research. Though innovative technical solutions may exist, implementation of these solutions in practice requires consideration of the organisations involved, understanding of the regulatory environment present and the management of the necessary operations.

9.4 Impact of Research

This research has been disseminated in relevant water supply platforms, both journal and conference papers, as referenced in this thesis, including:

1. **Corcoran, L.**, Coughlan, P., McNabola, A. (2013). Energy Recovery Potential using Micro Hydro Power in Water Supply Networks in the UK & Ireland. *Water Science & Technology: Water Supply*, Vol 13 No 2 p552560.

2. **Corcoran, L.**, Coughlan, P., McNabola, A. (2014). Innovation and the Water Industry: Case studies of innovation, education and collaboration. *TCD Journal of Postgraduate Research*, Vol 12 p8-23.

3. McNabola, A., Coughlan, C., **Corcoran, L.**, Power, C., Williams, A.P., Harris, I.M., Gallagher, J., Styles, D. (2013). Energy Recovery in the Water Industry using Micro-hydropower: An Opportunity to Improve Sustainability. *Water Policy*, Vol 16 p168-183.

4. **Corcoran, L.**, McNabola, A., Coughlan P. (2014). Optimisation of Water Supply Networks for Combined Leakage Reduction and Hydropower Generation, International Water Association (IWA): World Water Congress, Lisbon, Portugal, Sept 22-26 2014.

5. **Corcoran, L.**, Coughlan, P., McNabola, A. (2012). Energy Recovery Potential within Water Supply Networks of Ireland using Micro-Hydropower Turbines: Analysis of Case Studies, The IWA 9th International Symposium on Water Supply Technology, Yokohama, Japan, 20-22nd Nov 2012.

6. **Corcoran, L.**, McNabola, A., Coughlan P. (2012). Energy Recovery Potential of the Dublin Region Water Supply Network, IWA: World Congress on Water, Climate and Energy, Dublin, Ireland, May 13-18th 2012.

The practical contribution of this research was enhanced by the collaborative nature of the Hydro-BPT research project. The annual steering committee meetings provided the opportunity to get valuable feedback from key industry stakeholders, including water supply professionals, local county council representatives, turbine suppliers and consultants. It also presented opportunities to gain industry contacts for data collection for analysis purposes.

9.5 Policy Implications and Recommendations

A number of key recommendations can be made for the installation of future MHP projects in WSNs as a result of the research findings presented. Firstly, following the results of the feasibility study, it is recommended that, where possible, the most economic option is to use the electricity generated on site. However, using electricity on site may not always be an option depending on the site location. Furthermore, it is recommended that variable turbine efficiencies be considered during future feasibility calculations for more accurate estimation of investment payback periods. This is of particular relevance when installing PATs, because PATs operated inefficiently at low flow conditions.

Following the long term flow variation analysis in Chapter 5, it is recommended that WSPs consider future variations to flow rates at potential hydropower sites during feasibility studies. Projected future population growth was found to strongly influence future flow rates. This is likely to result in increased flow rates, especially in urban areas, which would result in increased power generation potential at hydropower sites. This may require that turbines be selected which can operate efficiently at higher flow rates than the current flow rates.

The key policy recommendations for the implementation of MHP projects in WSNs relate to the development of a supportive regulatory environment, both within water companies and also within the MHP incentives and energy utilities context. Firstly, it is recommended that Irish Water implement similar internal supportive mechanisms to Welsh Water. This would include the development of a designated Energy team with the primary objective of proactively developing new renewable energy resources. It is also recommended that a procurement protocol be adopted whereby tenders are awarded to contractors and consultants over longer periods (i.e. 5 years as in WW) to maintain relationships and increase and retain learning wherever possible.

As highlighted in Chapter 7: *Implementation*, current regulatory frameworks in both Ireland and Wales are not suited to the installation of MHP turbines within WSNs. These regulations and frameworks are directed at either run-of-river schemes and require abstraction licenses from the local fisheries boards. Otherwise these regulations are directed at large scale hydropower and pumped storage stations, requiring large scale EIAs to be undertaken, despite the micro-scale of the power generation. Another policy recommendation relating to the regulatory environment of MHP development is that suitable dedicated legislation and frameworks be developed for the exploitation of MHP in the water industry. These frameworks should reflect both the size of these hydropower projects, and the proportional impact they have on the environment and surroundings. Furthermore, it is recommended that these frameworks and regulations be formulated with input from all relevant stakeholders, including water company representatives, government officials, hydropower experts, local authority representatives, electricity regulators and water regulators.

Further policy implications of this research relate to the current REFIT regulations. As highlighted following the case analysis presented, the ability to obtain a REFIT has a major impact on project viability. Currently in the UK, if water has been pumped previously in the network, a hydropower project is not eligible to claim a REFIT. This was included in the REFIT qualification regulations to disallow pumped storage hydropower stations from claiming REFIT tariffs. However, regulators are currently rigidly upholding this law for small scale hydropower energy recovery when even light seasonal pumping is present. One example of this was for a 15kW WW turbine installation on a gravity-fed network. It has been deemed ineligible due to the presence of seasonal drought pumps which have operated for just 200 hours in the last 12 years (Millington, 2014). It is recommended that specified energy recovery FIT rates (ER-FITs) be implemented. The primary aim of the feed-in tariff scheme is to increase the generation from renewable energy resources, reducing dependency on fossil fuels resulting in a reduction in the production of GHG emissions. The installation of MHP turbines in WSNs would further contribute to these aims, helping Ireland and the UK to meet their legally binding EU emissions targets. As discussed in Chapter 4, the UK REFIT tariff scheme which applies increased tariffs for the smallest generation sites was shown to be the most incentivising option of the current European REFIT tariffs. It is recommended that a sliding tariff scale similar to this UK REFIT scale be adopted for the new ER-FIT.

The Vartry hydropower project suffered a year long delay in finding an electricity buyer. MHP is not an attractive option for energy utilities to purchase, due to the small power generation capacity and the variability of the power generated throughout the day. In order to increase the attractiveness of MHP, it is recommended that governments set specified generation targets for all utilities to ensure they purchase a specified amount of electricity from new renewable energy resources each year. If utilities do not meet these targets, penalty fees could be paid to either the Sustainable Energy Authority of Ireland or the Environment Agency in the UK. These penalty fees could then be reinvested to provide financial aid for the installation of further new MHP projects.

Other recommendations for policy relate to the procurement policies of water companies. Often the tender process will insist that the tender of the least cost be selected. However, when considering a number of project options for further development, the related amount of energy required for each option should also be considered. This would encourage water companies to not always approve the least cost option, but the least energy intensive option. This would result in higher initial costs, but in the long term these costs would be negated by the reduction in the cost of electricity required. Implementation of a policy like this would lead to a further reduction in the total energy usage by the water industry.

9.6 Critical Assessment and Areas for Future Research

Following this thesis research, a number of areas for future research were identified. The optimisation algorithm developed selects optimal points for hydropower energy recovery based on maximising the power generation. These locations however may not be the most economic locations to install turbines. It is recommended that further research be undertaken in the development of an optimisation model to select optimal points based on the minimisation of the total project cost, or minimisation of the investment payback period.

In order to improve the usability of the optimisation algorithm developed for water service professionals, it is recommended that a graphical user interface be designed. Further additions to the optimisation algorithm should also be made so that it can accurately hydraulically model the presence of a range of different types of pumps and valves.

Further research should also be undertaken into the development of the recommended policies and incentive schemes discussed in Section 9.5. Future changes to government policy in relation to environmental taxes such as carbon taxes, carbon limits for both industry and consumers and GHG emissions targets should also be investigated. Changes in these policies could significantly increase or decrease the attractiveness of the future development of MHP energy recovery schemes.

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Appendix A: Published Research

Journal Publications:

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Corcoran, L., Coughlan, P., McNabola, A. (2014). Innovation and the Water Industry: Case studies of innovation, education and collaboration. TCD Journal of Postgraduate Research, Vol 12.

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Corcoran, L., McNabola, A., Coughlan P. (2014). Optimisation of Water Supply Networks for Combined Leakage Reduction and Hydropower Generation, International Water Association (IWA): World Water Congress, Lisbon, Portugal, Sept 22-26 2014.

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Appendix B: Chapter 4 Further Details

Conversion rates to CO_2e from UK Carbon Trust:

Energy source	Units	Kg CO_2e per unit
Grid electricity	kWh	0.5246

Summary of Irish energy recovery site conditions

Name	Head (m)	Flow (l/s)	Power (kW)	Annual Power (MWh)	Annual CO_2 emissions savings (tonnes)
		Dublin City Valve Data			
		Control Valves			
Blackhorse Bridge	56.00	276.00	98.56	864.00	453.25
Stillorgan Rd	41.70	126.30	33.58	294.00	154.23
Merrion Gates	19.50	268.60	33.40	292.00	153.18
Thomas Court	73.50	452.90	212.26	1859.00	975.23
		PRVs North City Centre			
Bayside Boulevard	27.80	97.50	17.28	151.40	79.42
Bond Road	27.30	4.00	0.70	6.20	3.25
Brunswick St.	71.60	37.30	17.03	149.30	78.32
Cardiffsbridge Rd	15.10	10.40	1.00	8.80	4.62
Glenhill	20.00	2.00	0.26	2.20	1.15
Homefarm	31.00	38.20	7.55	66.00	34.62
St. Mobhi Rd.	12.90	115.50	9.50	83.40	43.75
North Docks	24.30	5.20	0.81	7.00	3.67
Poplar Row	76.10	35.90	17.42	152.70	80.11
Rathlin Rd.	26.00	66.10	10.96	96.00	50.36
The Rise	22.90	2.80	0.41	3.50	1.84
Sheriff St.	29.30	9.00	1.68	14.70	7.71
Spencer Dock	1.70	4.50	0.05	0.40	0.21
Wellmount Rd.	9.90	16.90	1.07	9.40	4.93
		PRVs South City Centre			
Anglesea Rd	28.90	44.10	8.13	71.10	37.30
Bellvue	67.90	18.90	8.18	71.60	37.56
Belmont Avenue	22.20	16.20	2.29	20.10	10.54
Beaver Row	28.80	6.10	1.12	9.90	5.19
Longmile Rd.	38.40	38.70	9.48	82.90	43.49
Mespil Rd.	34.60	5.10	1.13	9.80	5.14
Nutley Park	1.40	11.70	0.10	0.90	0.47
Rainsford St.	63.00	69.80	28.04	245.70	128.89
Ringsend Park	16.00	29.70	3.03	26.60	13.95
Rialto Bridge	62.00	118.80	46.97	411.60	215.93
Slievebloom Park	39.60	106.60	26.92	236.00	123.81
Strand Rd, Merrion	27.10	10.90	1.88	16.50	8.66

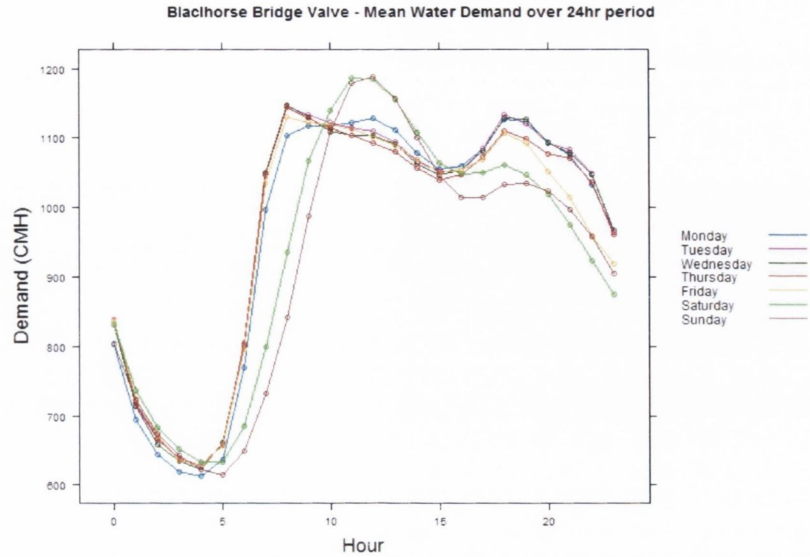
Summary of Irish energy recovery site conditions (continued)

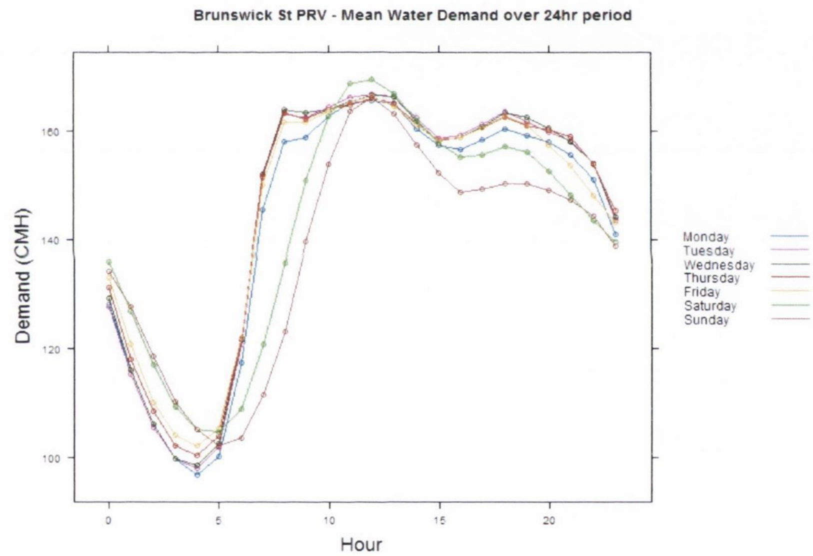
Name	Head (m)	Flow (l/s)	Power (kW)	Annual Power (MWh)	Annual CO_2 emissions savings (tonnes)
		Tanks / Reservoirs Kildare			
Allen	30.58	50.93	9.93	86.99	45.64
Ballycaghan	61.16	13.89	5.42	47.45	24.89
Ballygoran	20.39	104.17	13.54	118.63	62.23
Bishopshill	20.39	1.16	0.15	1.32	0.69
Boardleas	20.39	0.69	0.09	0.79	0.41
Castlefarm	20.39	11.57	1.50	13.18	6.91
Castlewarden	20.39	104.17	13.54	118.63	62.23
Dowdenstown	10.19	54.40	3.54	30.97	16.25
Hillwood	50.97	30.09	9.78	85.67	44.94
Old Kilcullen	20.39	231.48	30.09	263.61	138.29
Redhills	20.39	34.72	4.51	39.54	20.74
		Tanks / Reservoirs Dublin			
Stillorgan	6	306	11.71	104	54.56
Cookstown	15	749	71.64	640	335.74
Saggart	10.2	1768	114.99	1007	528.27
		Cork City PRVs			
Chetwind East	15.7	10.43	1.04	9.15	4.80
Chetwind SW	11.55	6.47	0.48	4.17	2.19
Southern Area NW	16.60	5.05	0.53	4.68	2.46
Southern Area W	15.60	5.05	0.50	4.40	2.31
High Central	29.20	6.28	1.17	10.25	5.38
High West Central	17.50	7.97	0.89	7.79	4.09
Mahon SE	21.90	15.93	2.22	19.48	10.22
South Area - SE and W	15.20	16.12	1.56	13.69	7.18
South Side	4.50	17.59	0.5	4.42	2.32
		Bray Town Council			
Palermo PRV	35.68	9.14	2.08	18.22	9.56
Town Hall PRV	25.48	14.12	2.29	20.1	10.54
Giltspur reservoir	50.97	69.63	22.63	198.24	103.99
		Tydavnet GWS, Co.Monaghan			
Reservoir outflow	2.7	11.84	0.2	1.79	0.94
		Waterford COCO			
Modeligo BPTs	15	0.97	0.09	0.81	0.43
PRV Portlaw WS	15	1.39	0.13	1.16	0.61
PRV Lismore	15	2.78	0.27	2.33	1.22

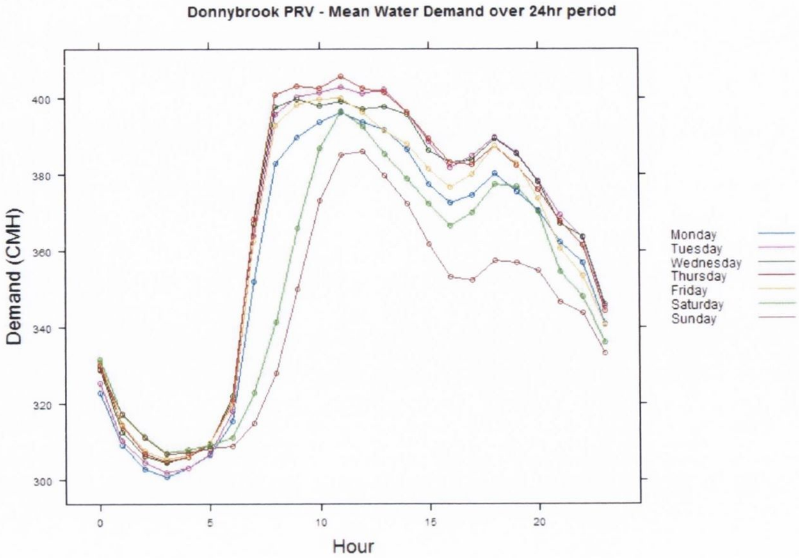
Summary of Irish energy recovery site conditions (continued)

Name	**Head (m)**	**Flow (l/s)**	**Power (kW)**	**Annual Power (MWh)**	**Annual CO_2 emissions savings (tonnes)**
		South Dublin COCO			
Alpine Heights (New)	32.67	3.53	0.74	6.45	3.38
Ashwood	44.67	4.19	1.19	10.46	5.49
Cherrywood	27	9.06	1.56	13.67	7.17
Gallanstawn WSA	26.75	61.96	10.57	92.59	48.57
Grange Castle	28.13	26.94	4.83	42.33	22.21
Laurel Park	21.67	2.67	0.37	3.23	1.7
Milltown/Baldonnell	20.75	4.96	0.66	5.75	3.01
Stocking Lane	21.33	11.54	1.57	13.76	7.22
Western Ind Estate	45.75	8.77	2.56	22.42	11.76
Woodford & Watery Lane	14.33	20.71	1.89	16.58	8.7
		Fingal COCO			
Donabate	30.39	30.76	5.96	52.22	27.40
Longlands	24.21	16.52	2.55	22.34	11.72
Glenellen	26.89	15.17	2.60	22.78	11.95
Feltrim	17.50	15.70	1.75	15.35	8.05
		Clonmel Town Council			
Apple Garage	18.01	27.78	3.19	27.94	14.66
Gurtnafleur	16.99	4.17	0.45	3.95	2.07
Old Bridge	21.41	2.69	0.37	3.21	1.68
Sergeants lane	48.93	2.22	0.69	6.07	3.19
Western rd	28.88	4.72	0.87	7.62	4.00
Loretto Sch	20.73	19.44	2.57	22.51	11.81
Loretto Barracks	9.85	12.04	0.76	6.63	3.48

Appendix C: Chapter 5 Further Details






Donnybrook PRV - Mean Water Demand over 24hr period
Demand (CMH)
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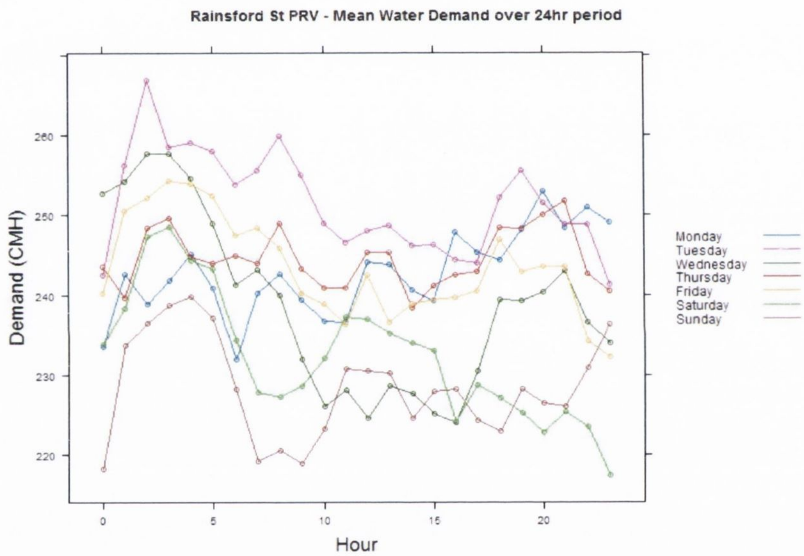
Rainsford St PRV - Mean Water Demand over 24hr period
Demand (CMH)
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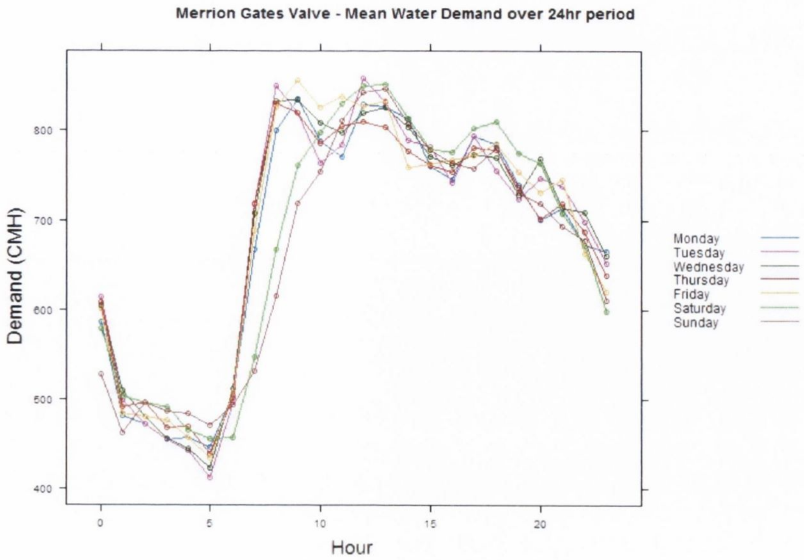
Merrion Gates Valve - Mean Water Demand over 24hr period
Demand (CMH)
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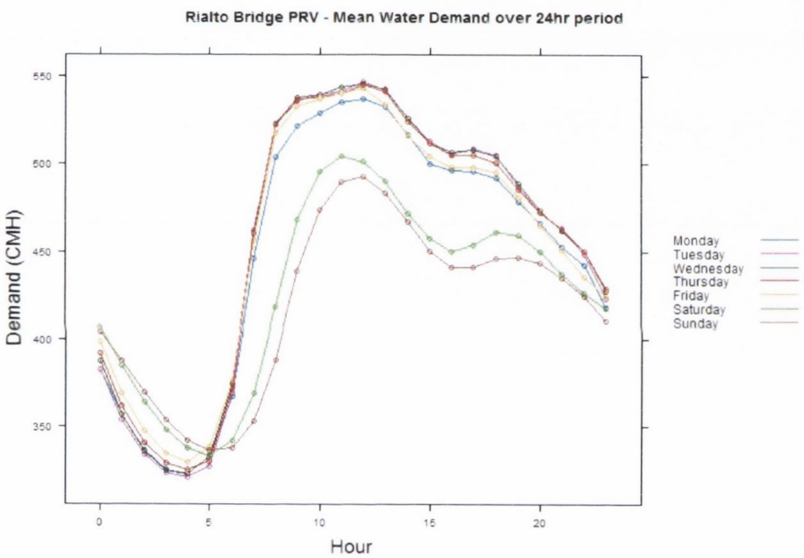
Rialto Bridge PRV - Mean Water Demand over 24hr period
Demand (CMH)
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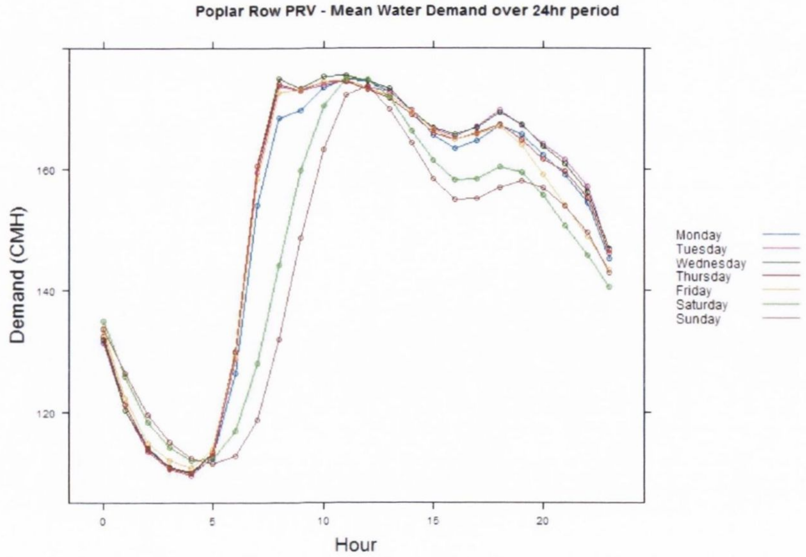
Poplar Row PRV - Mean Water Demand over 24hr period
Demand (CMH)
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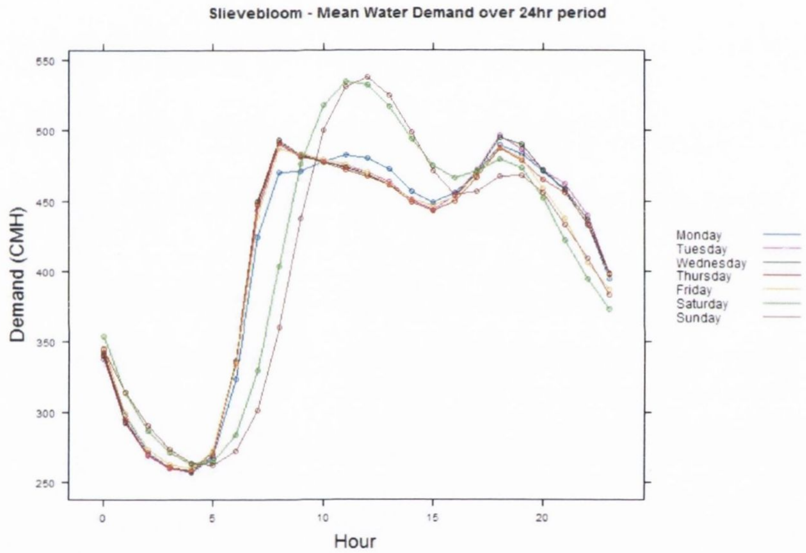
Slievebloom - Mean Water Demand over 24hr period
Demand (CMH)
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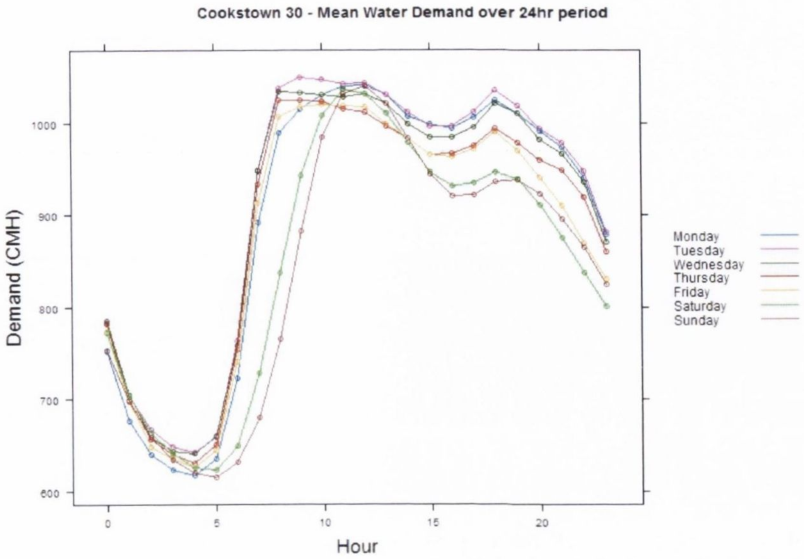
Cookstown 30 - Mean Water Demand over 24hr period
Demand (CMH)
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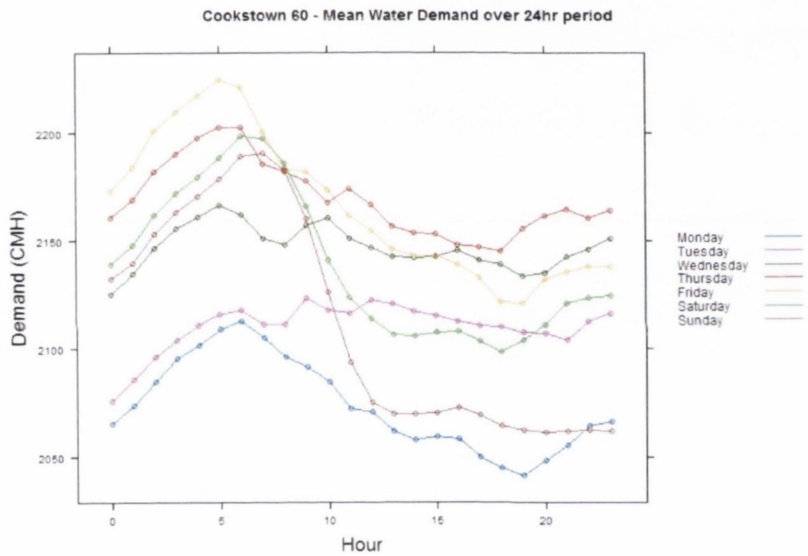
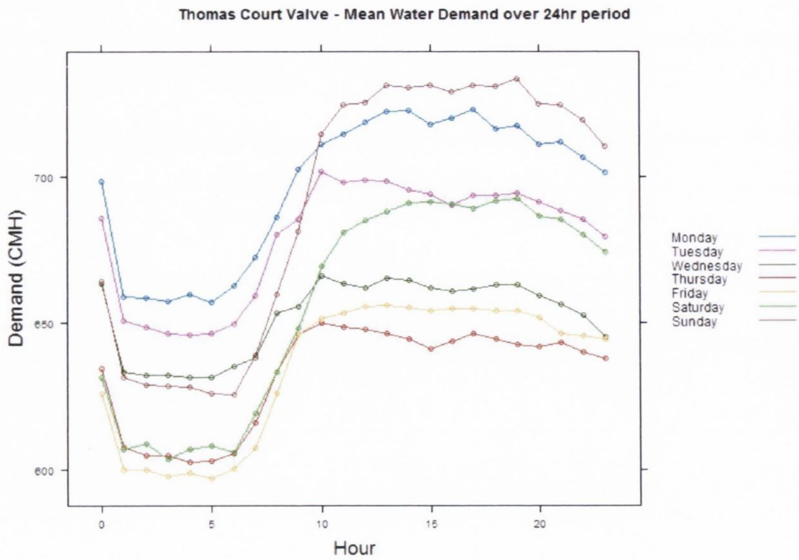
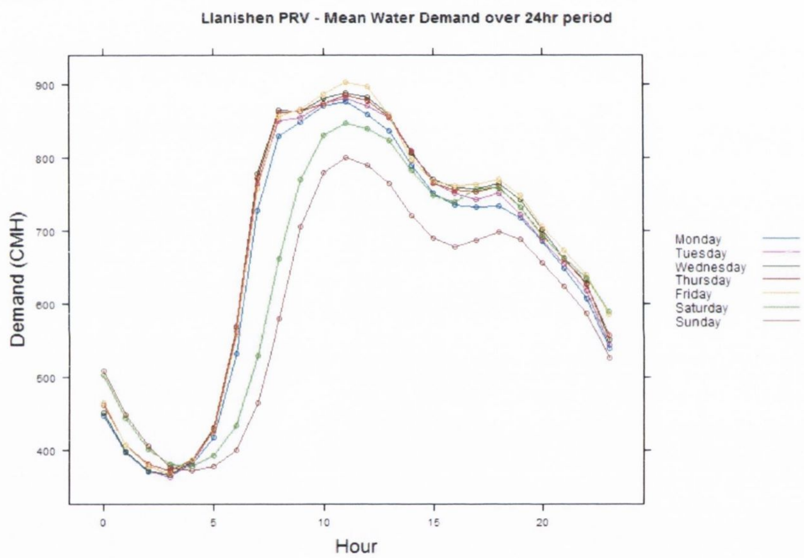
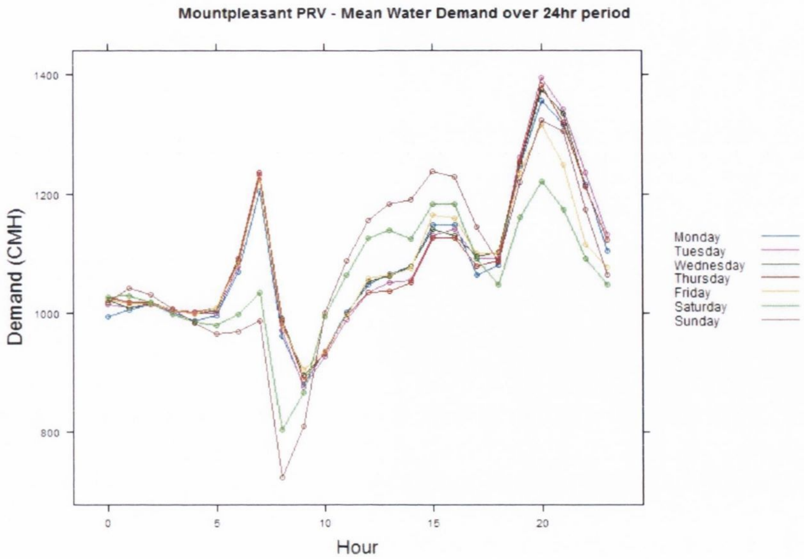
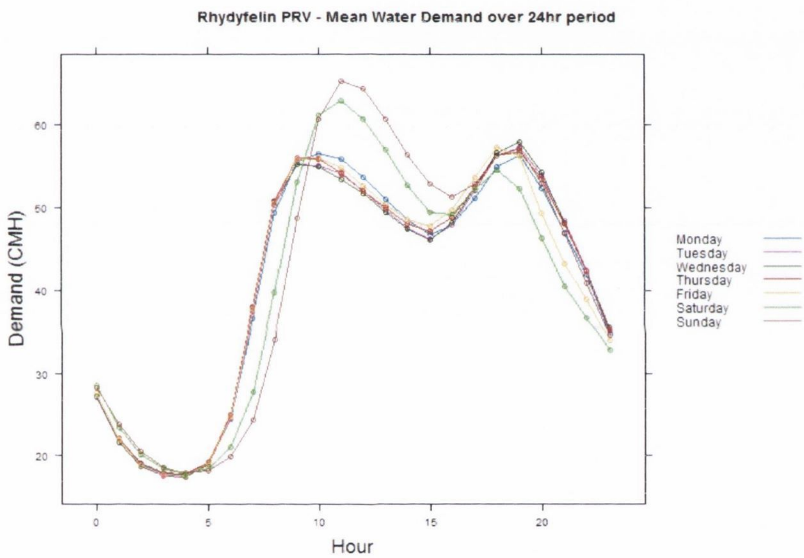
Saturday
Sunday

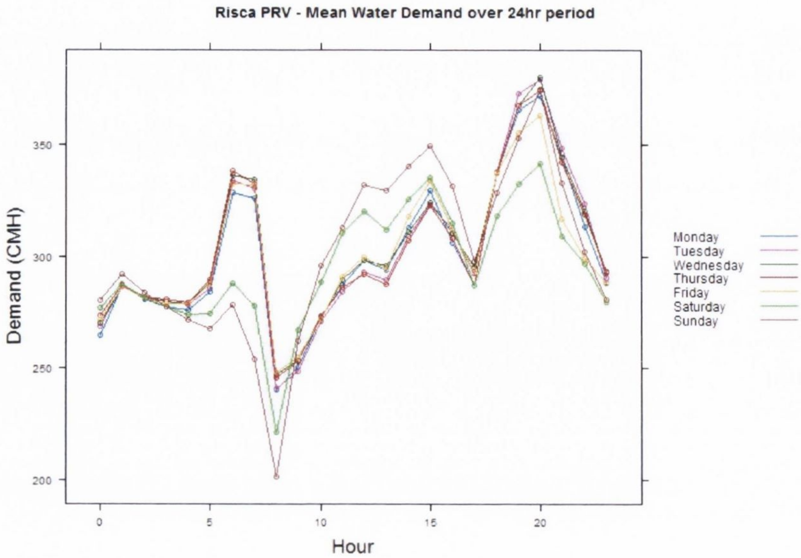

Cookstown 60 - Mean Water Demand over 24hr period
Demand (CMH)
Hour
Monday
Tuesday
Wednesday
Thursday
Friday
Saturday
Sunday

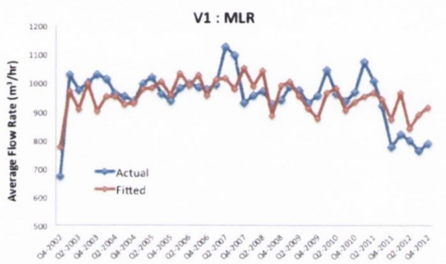

Thomas Court Valve - Mean Water Demand over 24hr period
Demand (CMH)
600
650
700
0
5
10
15
20
Hour
Monday
Tuesday
Wednesday
Thursday
Friday
Saturday
Sunday


Llanishen PRV - Mean Water Demand over 24hr period
Demand (CMH)
400
500
600
700
800
900
0
5
10
15
20
Hour
Monday
Tuesday
Wednesday
Thursday
Friday
Saturday
Sunday


Mountpleasant PRV - Mean Water Demand over 24hr period
Demand (CMH)
1400
1200
1000
800
Hour
0
5
10
15
20
Monday
Tuesday
Wednesday
Thursday
Friday
Saturday
Sunday

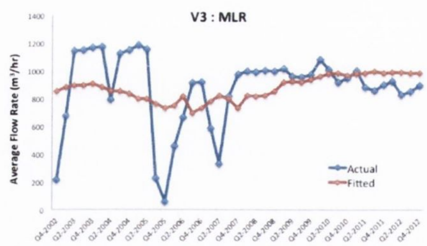

Rhydyfelin PRV - Mean Water Demand over 24hr period
Demand (CMH)
60
50
40
30
20
Hour
0
5
10
15
20
Monday
Tuesday
Wednesday
Thursday
Friday
Saturday
Sunday





(a) V1

(b) V2

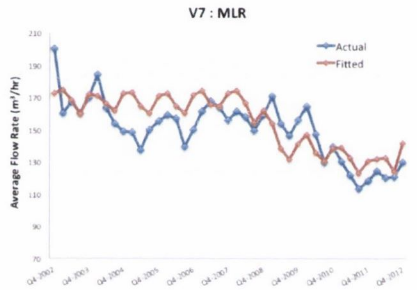


(a) V3

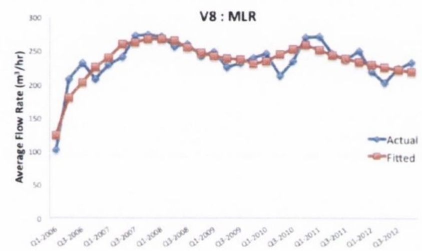
(b) V4

(a) V5

(b) V6



(a) V7



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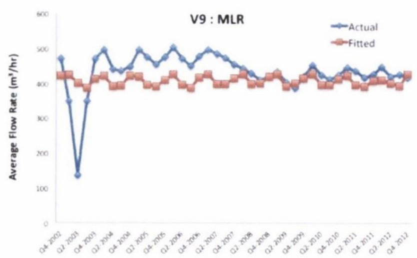
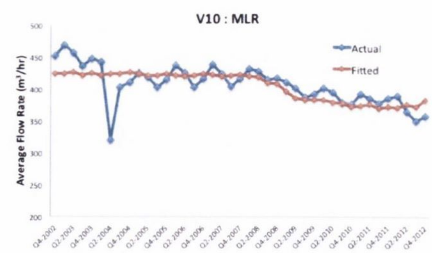
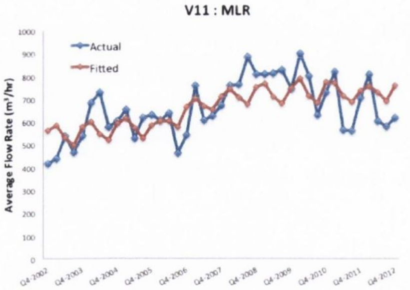
(b) V8



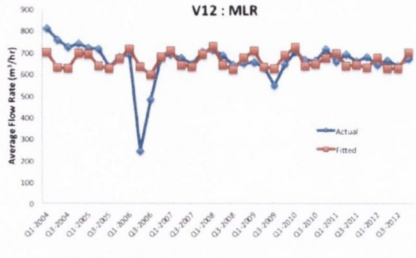
(a) V9



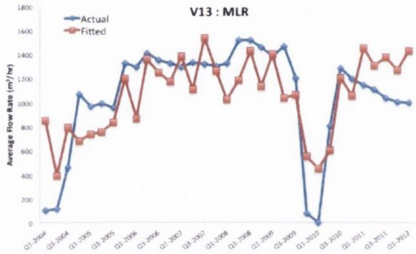
(b) V10



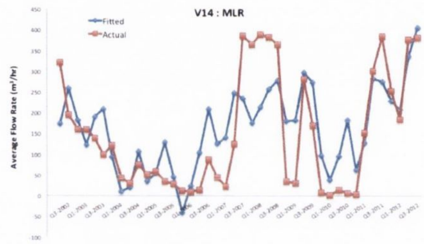
(a) V11



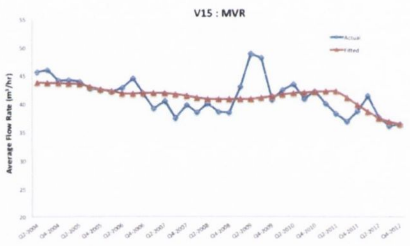
(b) V12



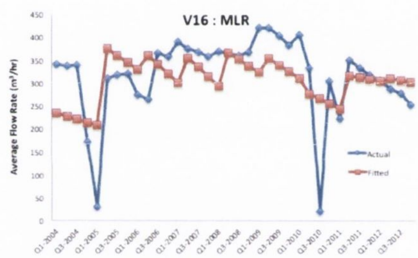
(a) V13



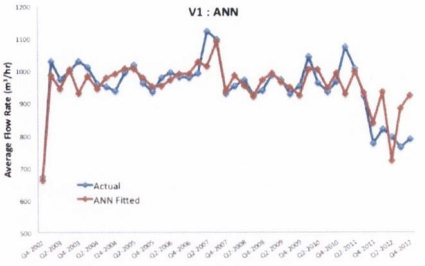
(b) V14



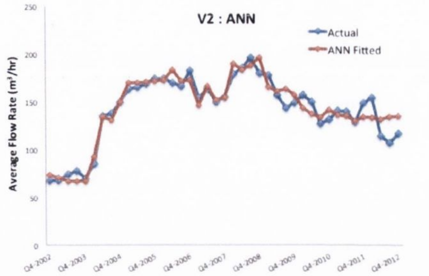
(c) V15



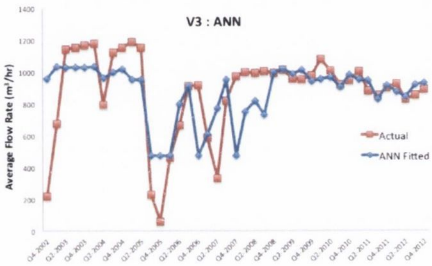
(d) V16



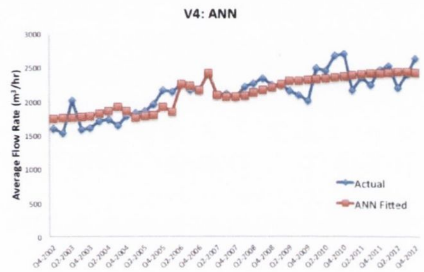
(a) V1



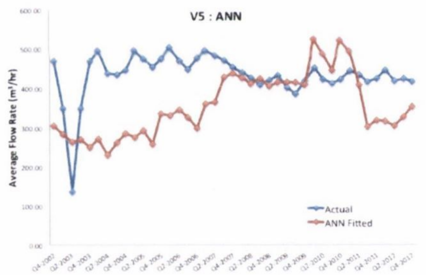
(b) V2



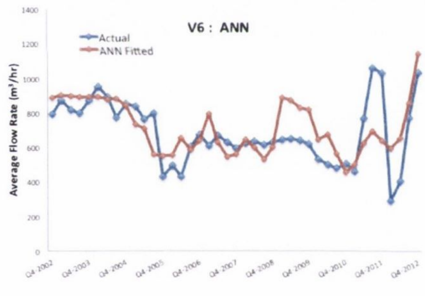
(c) V3



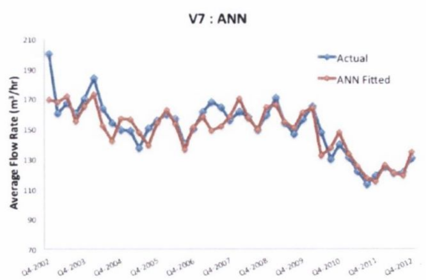
(d) V4



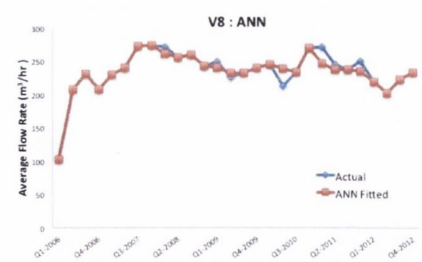
(a) V5



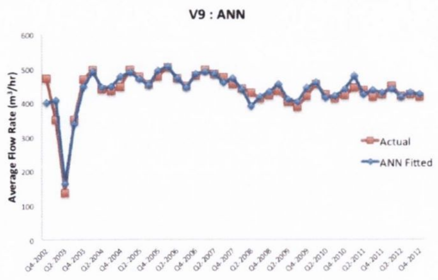
(b) V6



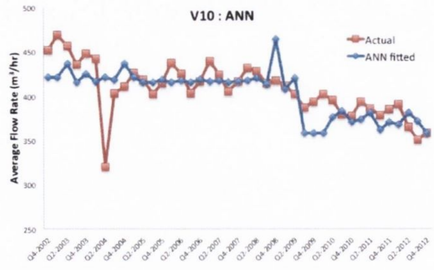
(c) V7



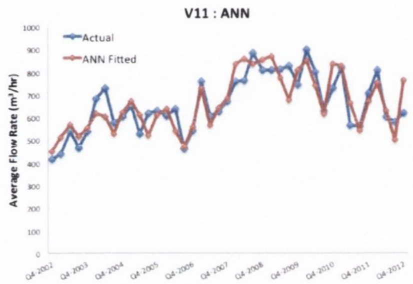
(d) V8



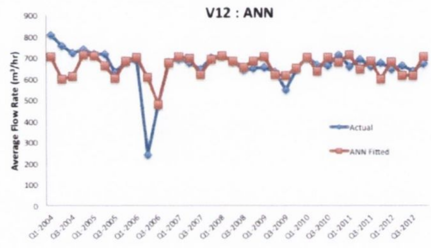
(a) V9



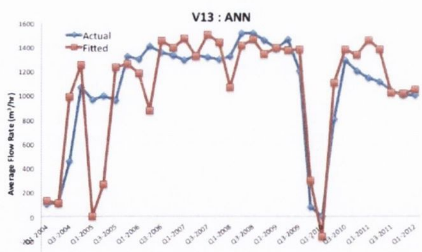
(b) V10



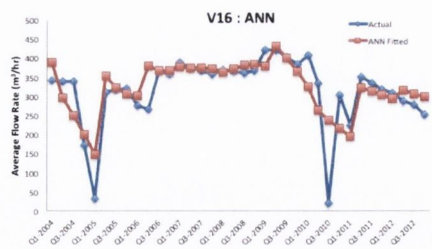
(c) V11



(d) V12



(a) V13



(b) V16

Year	Q	Black-horse	Donny-brook	Cookstown 30	Cookstown 60	Slieve-bloom	Rialto
2002	4	672.45	245.01	214.48	1595.91	452.08	470.46
2003	1	1029.04	311.89	672.43	1529.48	469.23	349.24
2003	2	973.08	243.13	1141.28	2018.51	456.96	136.52
2003	3	999.10	288.27	1150.65	1577.18	435.69	349.87
2003	4	1028.90	228.73	1165.24	1603.17	447.80	469.67
2004	1	1011.01	267.28	1174.72	1711.27	442.00	496.50
2004	2	959.92	245.31	790.05	1727.94	319.97	439.52
2004	3	950.09	251.51	1122.72	1647.22	403.10	434.28
2004	4	936.31	262.30	1149.27	1775.91	410.67	447.18
2005	1	995.20	286.40	1186.25	1828.86	425.98	495.08
2005	2	1017.77	284.15	1152.14	1850.82	418.46	475.84
2005	3	961.93	250.90	226.39	1953.66	402.54	454.68
2005	4	935.38	349.07	58.47	2169.91	414.64	475.85
2006	1	978.87	333.63	459.14	2138.62	437.63	503.74
2006	2	995.90	327.74	662.67	2254.42	425.18	470.73
2006	3	981.97	331.22	911.79	2167.00	403.26	449.46
2006	4	977.82	346.29	916.67	2169.80	416.49	478.58
2007	1	993.25	363.12	584.99	2412.22	439.04	495.92
2007	2	1123.43	360.74	327.29	2095.37	424.44	484.08
2007	3	1096.76	363.98	815.56	2109.94	405.04	472.17
2007	4	929.94	398.71	971.35	2067.13	416.39	454.70
2008	1	953.35	426.63	995.53	2217.30	432.16	441.37
2008	2	971.94	405.16	991.63	2257.85	427.82	428.04
2008	3	925.89	427.33	1001.75	2332.20	415.03	410.59
2008	4	938.82	417.12	994.04	2238.42	417.19	421.59
2009	1	988.00	418.25	1014.67	2234.60	411.58	433.34
2009	2	972.71	400.93	957.84	2147.47	401.80	401.58
2009	3	928.74	441.73	953.22	2090.69	386.94	386.52
2009	4	953.27	475.57	975.74	1997.45	393.13	419.18
2010	1	1044.52	520.15	1080.36	2492.68	402.01	452.63
2010	2	962.51	502.92	1008.51	2452.28	395.43	423.72
2010	3	934.73	516.62	918.53	2668.29	379.10	412.82
2010	4	966.30	522.66	945.60	2703.55	376.81	422.51
2011	1	1072.71	483.18	1002.35	2170.27	393.18	444.45
2011	2	1005.55	335.98	878.92	2355.61	385.47	435.99
2011	3	919.68	307.68	854.99	2238.67	378.54	416.83
2011	4	773.86	282.16	900.62	2461.52	385.31	425.06
2012	1	819.53	324.20	925.18	2522.64	390.51	447.60
2012	2	795.98	336.57	825.59	2199.23	365.07	418.48
2012	3	763.28	336.48	853.14	2401.14	351.00	424.91
2012	4	788.99	331.52	890.85	2637.21	247.67	417.71

Quarterly climate data: Casement Aerodrome Weather Station, Dublin:

Year	Quarter	Ave Temp	Max Temp	Min Temp	Total Rain	Ave Rain
2002	4	7.53	15.5	-1.9	283.1	0.19
2003	1	5.75	14.4	-4.3	110.8	0.05
2003	2	11.21	21.7	-2.4	221.2	0.10
2003	3	15.09	25.5	4.9	99.5	0.05
2003	4	7.48	17.9	-5.1	221.3	0.10
2004	1	5.63	15.7	-6.4	146	0.07
2004	2	11.39	24.3	-0.1	132.7	0.06
2004	3	14.52	23.8	5	218.8	0.10
2004	4	8.04	15.7	-1.2	205.1	0.09
2005	1	6.38	16.1	-2.1	126.1	0.06
2005	2	10.94	23.2	-0.2	170.3	0.08
2005	3	14.86	24.4	2.4	165.2	0.07
2005	4	8.14	18.9	-3.1	221.5	0.10
2006	1	5.38	13.7	-5.3	123.8	0.06
2006	2	11.07	23.1	-3.1	184.5	0.08
2006	3	15.67	25.9	4	160.7	0.07
2006	4	8.72	16.9	-2	271	0.12
2007	1	6.40	13.7	-4.1	189.8	0.09
2007	2	11.69	22	0.3	176.1	0.08
2007	3	13.90	22.3	1.3	261.8	0.12
2007	4	8.79	17.6	-3.1	133.3	0.06
2008	1	6.15	14.2	-6.6	213.9	0.10
2008	2	10.86	22	-1.2	136.7	0.06
2008	3	14.06	21.8	4.4	406.1	0.18
2008	4	6.86	16.4	-3	185.9	0.08
2009	1	5.43	14.3	-4	144.4	0.07
2009	2	11.14	24.4	0.3	210.9	0.10
2009	3	14.20	21.9	2.8	259	0.12
2009	4	7.47	17.3	-6.5	303.2	0.14
2010	1	3.27	13.9	-9.2	137	0.06
2010	2	10.90	23	-3.7	114.9	0.05
2010	3	14.28	22.8	2.6	230.9	0.10
2010	4	5.27	18.4	-11.5	188.9	0.09
2011	1	5.64	15.2	-6.1	124.5	0.06
2011	2	11.30	23.1	0.2	129.9	0.06
2011	3	13.90	24	4.5	147.3	0.07
2011	4	9.19	19.6	-2.1	270.1	0.12
2012	1	7.14	18.6	-5.6	109.1	0.05
2012	2	9.86	21.9	-2.7	298.3	0.14
2012	3	13.79	23.4	1.1	246	0.11
2012	4	6.81	14.1	-3.1	196.1	0.09

Quarterly climate data: Casement Aerodrome Station, Dublin:

Year	Quarter	Ave RH	Ave VP	Ave Sun	Total Sun
2002	4	86.70	9.14	0.06	87.3
2003	1	83.13	7.75	0.16	350.1
2003	2	78.71	10.53	0.24	531.8
2003	3	79.92	13.73	0.21	472.3
2003	4	85.74	9.07	0.11	238.3
2004	1	82.69	7.69	0.15	324.5
2004	2	78.33	10.60	0.26	566.8
2004	3	81.37	13.46	0.20	448.4
2004	4	85.95	9.36	0.10	221.7
2005	1	83.46	8.18	0.10	219.1
2005	2	79.52	10.59	0.23	498.1
2005	3	81.32	13.75	0.19	427.7
2005	4	87.14	9.65	0.10	230.1
2006	1	84.76	7.74	0.10	222.9
2006	2	78.66	10.45	0.28	618.9
2006	3	79.79	14.11	0.23	518.6
2006	4	85.67	9.84	0.13	287.1
2007	1	83.89	8.15	0.13	288.1
2007	2	79.95	11.04	0.27	599
2007	3	83.40	13.24	0.20	440.4
2007	4	85.25	9.85	0.10	230.9
2008	1	82.11	7.86	0.13	274.9
2008	2	77.97	10.19	0.28	603.6
2008	3	83.90	13.46	0.17	370.8
2008	4	88.70	9.01	0.12	262.1
2009	1	85.71	7.81	0.12	266.6
2009	2	82.07	10.89	0.28	619.8
2009	3	83.39	13.52	0.20	445.8
2009	4	89.06	9.49	0.10	227.4
2010	1	88.46	6.93	0.14	305.5
2010	2	79.46	10.50	0.29	632.6
2010	3	82.88	13.51	0.23	498.7
2010	4	89.15	8.33	0.13	280.9
2011	1	87.01	8.06	0.15	326.5
2011	2	79.09	10.56	0.27	588.3
2011	3	81.24	12.84	0.19	419.1
2011	4	86.53	10.37	0.10	217.8
2012	1	85.02	8.69	0.12	263.6
2012	2	82.85	10.25	0.22	482.9
2012	3	84.08	13.32	0.20	447.6
2012	4	87.16	8.76	0.12	254.1

Quarterly Socio-Economic Data for Dublin Region:

Year	Quarter	Unemp. (%)	Pop. (thousands)	Part (%)	Cons.	Houses	Leakage (%)
2002	4	4.2	1128	62.5	247.4	1172	34.31
2003	1	4.2	1130.6	62	250.3	937	34.11
2003	2	3.9	1133.2	62.3	248.2	848	33.91
2003	3	4.6	1136.1	62.7	265.9	842	33.71
2003	4	4.1	1139	62.3	272.1	743	33.51
2004	1	4.8	1141.9	62.1	317.8	915	33.55
2004	2	4.2	1144.8	61.8	337.5	1168	33.58
2004	3	4.3	1148.75	62.9	326.2	1140	33.62
2004	4	3.9	1152.7	62.8	313.8	1307	33.65
2005	1	4.2	1156.65	62.7	329.1	1553	33.74
2005	2	4.7	1160.6	64	357.6	1553	33.82
2005	3	4.6	1154.9	64.7	366.8	1854	33.91
2005	4	4.3	1149.2	64.3	367.4	2064	33.99
2006	1	4.7	1143.5	64.8	369.7	1935	34.23
2006	2	4.9	1183.4	64.8	358.5	1438	34.47
2006	3	4.6	1195.35	65.5	364.4	2315	34.71
2006	4	4.3	1207.3	65.6	369.4	2058	34.95
2007	1	4.5	1219.25	65.7	345.3	1685	35.34
2007	2	4.9	1231.2	65.6	336	1365	35.73
2007	3	4.7	1236.58	66.3	303.5	1551	36.12
2007	4	4.5	1241.95	65.8	279.6	2077	36.51
2008	1	4.9	1247.33	65.5	262.8	1373	36.34
2008	2	5.1	1252.7	65.6	240.8	1445	36.18
2008	3	6.6	1254.15	66.5	210	1390	36.01
2008	4	7	1255.6	64.8	188.3	1140	35.84
2009	1	9	1257.05	64.2	165.8	658	35.87
2009	2	10.6	1258.5	64	148.6	585	35.89
2009	3	11.2	1258.78	63.4	141.7	629	35.92
2009	4	11.1	1259.05	63.2	120.4	525	35.94
2010	1	11.2	1259.33	62.4	105.7	312	36.58
2010	2	11.7	1259.6	62.5	105.2	186	37.225
2010	3	12	1260.08	62.4	99.7	139	37.87
2010	4	13	1260.55	62.4	90.4	274	38.51
2011	1	12.7	1261.03	61.5	84.1	174	38.04
2011	2	12.3	1261.5	62.2	81.2	168	37.57
2011	3	13.4	1261.85	61.7	83.4	69	37.09
2011	4	13	1262.2	62.2	85.9	146	36.62
2012	1	13.2	1262.55	61.6	82.1	115	36.40
2012	2	12.3	1262.9	61.7	78.2	88	36.175
2012	3	12.9	1262.78	62.2	77.7	149	35.95
2012	4	11.1	1262.65	61.9	81.8	155	35.73

Welsh Quarterly Data

Year	Q	Risca Flow	Llanishen Flow	Unemp. Cardiff	Unemp. Caer-philly	Pop. Caerphilly	Pop. Cardiff
2004	1	339.96	806.00	5.7	6.9	172111.5	316135.75
2004	2	337.63	754.88	5.7	6.9	172361	317099
2004	3	338.11	720.47	5.7	6.9	172465	318074.5
2004	4	170.82	737.42	5.7	6.9	172569	319050
2005	1	30.91	716.14	6.6	6.6	172673	320025.5
2005	2	310.58	715.52	6.6	6.6	172777	321001
2005	3	317.34	631.33	6.6	6.6	173018	321692.25
2005	4	319.14	677.15	6.6	6.6	173259	322383.5
2006	1	274.78	687.88	5.6	5.9	173500	323074.75
2006	2	264.44	240.93	5.6	5.9	173741	323766
2006	3	366.19	478.78	5.6	5.9	174052.5	324873.5
2006	4	358.40	678.16	5.6	5.9	174364	325981
2007	1	389.26	689.20	6.9	6.2	174675.5	327088.5
2007	2	374.53	672.99	6.9	6.2	174987	328196
2007	3	367.23	645.09	6.9	6.2	175305	329344.5
2007	4	358.02	700.25	6.9	6.2	175623	330493
2008	1	368.63	707.90	6.4	7.5	175941	331641.5
2008	2	365.36	683.71	6.4	7.5	176259	332790
2008	3	359.61	639.28	6.4	7.5	176484	334006.5
2008	4	366.88	648.29	6.4	7.5	176709	335223
2009	1	420.19	653.57	8.4	11.8	176934	336439.5
2009	2	420.29	631.20	8.4	11.8	177159	337656
2009	3	403.40	544.56	8.4	11.8	177394.5	338592.5
2009	4	383.46	645.33	8.4	11.8	177630	339529
2010	1	406.03	699.50	10.3	9.2	177865.5	340465.5
2010	2	333.17	664.64	10.3	9.2	178101	341402
2010	3	20.77	660.39	10.3	9.2	178271.25	342412
2010	4	304.37	712.81	10.3	9.2	178441.5	343422
2011	1	222.44	658.01	8.4	11.2	178611.75	344432
2011	2	350.97	689.75	8.4	11.2	178782	345442
2011	3	332.85	655.63	8.4	11.2	178842	346204.75
2011	4	317.00	675.31	8.4	11.2	178902	346967.5
2012	1	307.19	641.60	10.9	9	178962	347730.25
2012	2	287.40	659.32	10.9	9	179022	348493
2012	3	278.05	635.34	10.9	9	179078.25	349297.25
2012	4	252.16	668.55	10.9	9	179134.5	350101.5

Welsh quarterly data

Year	Q	WW Leakage	Wales Cons.	Houses built	Price (£)
2002	1	256	101.1	1800	126
2002	2	253	106	2060	124
2002	3	249	109.2	2050	124
2002	4	245	115.5	2260	124
2003	1	243	108.6	1940	124
2003	2	242	116.9	2290	122
2003	3	240	129.3	1800	122
2003	4	238	136.9	2230	122
2004	1	237	135.5	1980	122
2004	2	237	141.2	2260	123
2004	3	236	138.7	2120	123
2004	4	235	132	2540	123
2005	1	234	135	1580	123
2005	2	233	130.8	2080	142
2005	3	231	127.7	1960	142
2005	4	230	122.5	2170	142
2006	1	229	122.3	2050	142
2006	2	228	125.1	2160	147
2006	3	226	130.2	2260	147
2006	4	225	129.4	2250	147
2007	1	221	136.2	2670	147
2007	2	217	133.2	2580	155
2007	3	213	134.7	1980	155
2007	4	209	124.5	2290	155
2008	1	208	131.4	1820	155
2008	2	207	126.2	2160	165
2008	3	205	115.6	1790	165
2008	4	204	112.1	1690	165
2009	1	202	104.2	1470	165
2009	2	200	105.5	1750	170
2009	3	197	106.3	1290	170
2009	4	195	105.1	1830	170
2010	1	205	89.4	1300	170
2010	2	215	100.1	1480	168
2010	3	225	107.2	1300	168
2010	4	235	103.3	1540	168
2011	1	226	109.1	1190	168
2011	2	217	109.3	1620	177
2011	3	208	114	1130	177
2011	4	199	102.4	1720	177
2012	1	196	93.5	1110	177
2012	2	192	95.4	1630	178
2012	3	189	106.2	1230	178

Welsh quarterly data: Pontypool PRV

Year	Q	Pontypool Flow	Pop. Torfaen	Unemp. Torfaen
2002	1	198.28	90753	5.8
2002	2	321.05	90700	5.8
2002	3	195.52	90704.75	5.8
2002	4	159.76	90709.5	5.8
2003	1	158.94	90714.25	5.5
2003	2	139.23	90719	5.5
2003	3	99.26	90668.25	5.5
2003	4	120.37	90617.5	5.5
2004	1	42.92	90566.75	4.8
2004	2	30.48	90516	4.8
2004	3	74.78	90548	4.8
2004	4	50.52	90580	4.8
2005	1	58.96	90612	6.2
2005	2	34.38	90644	6.2
2005	3	28.81	90688	6.2
2005	4	12.31	90732	6.2
2006	1	9.61	90776	4.9
2006	2	14.31	90820	4.9
2006	3	87.78	90858.5	4.9
2006	4	44.63	90897	4.9
2007	1	21.22	90935.5	6.8
2007	2	124.93	90974	6.8
2007	3	384.56	91004	6.8
2007	4	365.14	91034	6.8
2008	1	388.32	91064	8.2
2008	2	381.50	91094	8.2
2008	3	364.91	91120.25	8.2
2008	4	35.38	91146.5	8.2
2009	1	29.16	91172.75	11.1
2009	2	281.91	91199	11.1
2009	3	169.55	91164.25	11.1
2009	4	6.88	91129.5	11.1
2010	1	0.42	91094.75	8.8
2010	2	14.65	91060	8.8
2010	3	5.17	91092.5	8.8
2010	4	3.5	91125	8.8
2011	1	151.37	91157.5	11.3
2011	2	299.76	91190	11.3
2011	3	383.93	91235.5	11.3
2011	4	252.51	91281	11.3
2012	1	183.51	91326.5	13
2012	2	374.93	91372	13
2012	3	380.17	91380.75	13

Quarterly St. Athan Weather Station Data

Year	Q	Ave. Cloud	Ave. Temp	Ave. Rainfall
2002	1	72.1	7.57	0.18
2002	2	82.7	11.47	0.11
2002	3	119.1	15.54	0.08
2002	4	94.9	9.12	0.24
2003	1	135.2	6.15	0.09
2003	2	136.2	12.54	0.11
2003	3	145.4	16.71	0.08
2003	4	116.4	8.38	0.14
2004	1	112.0	6.39	0.13
2004	2	136.1	12.28	0.07
2004	3	116.9	15.91	0.13
2004	4	102.9	9.08	0.18
2005	1	108.6	6.69	0.09
2005	2	123.0	11.97	0.10
2005	3	128.8	16.31	0.10
2005	4	114.1	8.93	0.15
2006	1	100.5	4.96	0.07
2006	2	130.5	12.27	0.10
2006	3	127.5	17.38	0.06
2006	4	93.8	10.28	0.20
2007	1	106.0	7.60	0.14
2007	2	136.1	12.97	0.11
2007	3	119.4	15.14	0.13
2007	4	110.5	8.94	0.12
2008	1	115.3	6.98	0.12
2008	2	134.3	12.04	0.09
2008	3	98.3	15.28	0.25
2008	4	117.9	7.96	0.11
2009	1	102.3	5.39	0.10
2009	2	122.4	12.28	0.09
2009	3	99.5	15.59	0.14
2009	4	99.6	9.02	0.14
2010	1	112.4	4.25	0.08
2010	2	150.1	12.02	0.04
2010	3	109.6	15.62	0.14
2010	4	120.0	6.27	0.11
2011	1	100.4	6.33	0.11
2011	2	127.9	12.84	0.10
2011	3	103.2	15.49	0.13
2011	4	89.2	10.67	0.17
2012	1	109.3	6.90	0.10
2012	2	101.8	11.29	0.20
2012	3	102.0	15.25	0.18

Quarterly St. Athan Weather Station Data (continued)

Year	Q	Ave Global Radiation	Ave. Pressure
2002	1	222.0	1014.6
2002	2	733.1	1014.1
2002	3	679.6	1017.6
2002	4	158.8	1008.8
2003	1	282.4	1018.8
2003	2	759.1	1015.4
2003	3	643.6	1018.0
2003	4	178.2	1014.2
2004	1	249.6	1016.8
2004	2	787.5	1015.9
2004	3	626.5	1015.0
2004	4	159.5	1015.7
2005	1	229.1	1020.6
2005	2	749.6	1016.1
2005	3	685.3	1017.5
2005	4	171.0	1015.8
2006	1	228.0	1016.2
2006	2	777.9	1016.5
2006	3	692.2	1014.9
2006	4	166.3	1013.4
2007	1	276.9	1013.9
2007	2	764.9	1015.1
2007	3	669.5	1016.5
2007	4	175.9	1021.5
2008	1	263.4	1013.6
2008	2	781.5	1014.1
2008	3	569.2	1013.4
2008	4	186.0	1015.9
2009	1	270.6	1014.0
2009	2	774.7	1016.0
2009	3	609.4	1016.4
2009	4	173.7	1006.8
2010	1	254.6	1012.7
2010	2	854.3	1018.6
2010	3	629.7	1015.3
2010	4	193.0	1011.4
2011	1	259.4	1018.2
2011	2	822.9	1017.3
2011	3	657.7	1013.1
2011	4	168.3	1014.6
2012	1	288.8	1026.5
2012	2	722.8	1010.2
2012	3	657.7	1013.9

Quarterly St. Athan Weather Station Data (continued)

Year	Q	Ave. Rel. Humidity	Min. Temp	Max. Temp	Total Rainfall
2002	1	85.2	-4.8	14.5	299.8
2002	2	78.6	1.6	23.4	197.4
2002	3	79.0	6.5	23.7	157.0
2002	4	85.9	-0.3	17.1	444.8
2003	1	80.8	-5.6	16.9	171.6
2003	2	75.7	-0.5	25.0	204.0
2003	3	76.2	3.9	30.4	159.8
2003	4	83.6	-1.3	18.7	261.6
2004	1	80.7	-3.7	15.9	238.8
2004	2	80.3	1.6	25.9	136.8
2004	3	82.1	6.8	26.4	238.4
2004	4	85.8	-2.0	16.5	332.0
2005	1	81.8	-3.0	16.9	157.6
2005	2	80.2	0.2	24.6	200.8
2005	3	79.8	5.9	26.3	199.6
2005	4	85.2	-5.1	20.0	297.8
2006	1	82.9	-3.6	13.7	139.0
2006	2	79.1	-0.9	25.4	202.0
2006	3	77.9	8.9	30.7	129.0
2006	4	85.8	-1.4	18.9	377.8
2007	1	83.6	-3.6	15.1	271.4
2007	2	78.5	1.9	23.9	230.6
2007	3	80.6	5.1	25.1	272.0
2007	4	85.4	-2.4	17.2	253.0
2008	1	82.9	-1.8	12.2	217.0
2008	2	79.2	-0.8	24.3	171.6
2008	3	84.1	7.2	26.8	489.2
2008	4	85.2	-3.1	19.5	212.0
2009	1	87.4	-5.3	14.8	191.8
2009	2	80.4	1.9	25.1	190.2
2009	3	82.7	7.3	26.5	276.4
2009	4	86.5	-4.2	17.9	253.2
2010	1	86.6	-6.1	12.8	142.4
2010	2	75.0	1.7	24.9	80.2
2010	3	80.2	4.3	23.6	263.0
2010	4	83.8	-8.4	19.2	189.4
2011	1	86.5	-5.8	16.6	196.2
2011	2	79.1	4.1	24.8	197.2
2011	3	81.7	7.2	24.3	247.6
2011	4	85.1	-0.5	25.8	299.4
2012	1	84.1	-6.3	19.3	190.0
2012	2	81.4	0.3	26.5	359.6
2012	3	82.3	6.2	26.5	338.2

Appendix D: Chapter 6 Further Details

25-Node Benchmark Network: Junction data

Junctions		
ID	Elevation (m)	Demand (L/s)
1	18	5
2	18	10
3	14	0
4	12	5
5	14	30
6	15	10
7	14.5	0
8	14	20
9	14	0
10	15	5
11	12	10
12	15	0
13	23	0
14	20	5
15	8	20
16	10	0
17	7	0
18	8	5
19	10	5
20	7	0
21	10	0
22	15	20

25-Node Benchmark Network: Pipe data

Pipes

ID	Node1	Node2	Length (m)	Diameter (m)	HW coefficient
1	23	1	606	0.457	110
2	23	24	454	0.457	110
3	24	14	2,782	0.229	105
4	25	14	304	0.381	135
5	24	10	3,382	0.305	100
6	24	13	1,767	0.475	110
7	14	13	1,014	0.381	135
8	25	16	1,097	0.381	6
9	1	2	1,930	0.457	110
10	2	3	5,150	0.305	10
11	13	12	762	0.457	110
12	15	16	914	0.229	125
13	16	17	822	0.305	140
14	17	18	411	0.152	100
15	20	18	701	0.229	110
16	17	19	1,072	0.229	135
17	19	20	864	0.152	90
18	21	20	711	0.152	90
19	15	21	832	0.152	90
20	22	15	2,334	0.229	100
21	12	15	1,996	0.229	95
22	12	11	777	0.229	90
23	11	10	542	0.229	90
24	12	8	1,600	0.457	110
25	10	8	249	0.305	105
26	8	9	443	0.229	90
27	8	6	743	0.381	110
28	8	22	931	0.229	125
29	22	21	2,689	0.152	100
30	3	4	326	0.152	100
31	5	4	844	0.229	110
32	6	3	1,274	0.152	100
33	6	5	1,115	0.229	90
34	6	7	615	0.381	110
35	5	22	1,408	0.152	100
36	7	5	500	0.381	110
37	9	6	300	0.229	90

Appendix E: Chapter 7 Interview Protocol Documents

Vartry Hydropower Case Study: Interview Protocol

Interview Date: Thursday 19th July 2012, 10:30am
Location: Vartry Waterworks, Co.Wicklow, Ireland
Operator: Dublin City Council
Interviewee: Vartry Waterworks Engineer-in-Chief

Introduction

1. Great example. Project overview; Would like to understand why you did it, with whom, how well it has worked and what you would recommend to other county councils or water companies considering similar energy recovery schemes?
2. Book (Corcoran, 2005) describes history of Vartry reservoir, so mainly looking for information on the history of the application of hydropower at Vartry.
3. History of old Pelton turbine installed (1947) when, by who, how long did it run for, generating capacity, why was it de-commissioned? Power output?
4. Who was involved in the initial brainstorming? Was everyone interested / excited by the idea; were there many concerns? Was anyone dead set against it or was everyone enthusiastic? Was there any resistance to change? Did anyone have experience on similar projects?
5. Recommendations for anyone else to contact involved with the project (contractors, consultants, other DCC members, SEAI representatives, Bord Gais etc)
6. Main phases of the project - Planning / construction / grid connection - Any setbacks?
7. Likelihood for future collaboration? Any other hydro projects considered? Did members of the team work together again on any other project?

Model of Scheme

1. Technical specifications of turbine rated for X kW? Was this the expected power output? Does it vary much, daily, seasonally?

2. The door installed as per grid connection requirements what were the changes required connecting to the grid? Grid connection is a key factor with these projects; some people say it is better to use all the electricity on site if possible.

3. When did the initial idea for this project come about? Who would have been involved at this early stage?

4. Did anyone have experience on similar projects?

At Turbine

1. Where (from whom) and when did the idea first come about; how long did it take before plans began to take place. Who was involved in the initial brainstorming?

2. Was it the first of this kind for DCC in recent years?

3. Technical spec of turbine rated for X kW? Was this the expected power output? Does it vary much, daily, seasonally?

4. Turbine type? Most efficient over what flow rate range?

5. Have there been many issues mech/elec over the past few years of running the turbine? Any additional costs in terms of maintenance etc? Or does it just run without issue?

6. Do you do the maintenance yourselves or does the turbine supplier come back? How long did the turbine manufacturer remain involved in project for? Did they train you in how to maintain and run the turbine? How long did that take?

7. History of old pelton turbine installed (1947) when, by who, how long did it run for, generating capacity, why was it de-commissioned? Power output? Would you have learned anything from the previous turbine

Q&A After Site Visit

1. Who was involved in the initial brainstorming? Was everyone interested / excited by the idea; were there many concerns? Was anyone dead set against it or was everyone enthusiastic? Was there any resistance to change? Did anyone have experience on similar projects?

2. Would you have a Gantt chart or rough project timeline available showing the main stages involved etc that you could share with us?

3. What would the key stages on a Gantt chart look like? (feasibility, tender, planning, construction, commissioning, grid connection, grant application)
4. Who was involved at those stages?
5. Organisational structure? (Draw diagram) Organisational structure of DCC? How dependent were you on decisions from XXX at DCC, or XX at SEAI or XXX at ESB?
6. Recommendations for anyone else to contact involved with the project (contractors, consultants, other DCC members, SEAI rep, Bord Gais etc.
7. What were the key decisions during the project?
8. Were there any challenges at any of these stages?
9. Planning permission How long did this take? Were there any unexpected setbacks?
10. Feasibility study? How long did this stage take? How many options considered?
11. Tender stage How did the initial tendering process work? Was there a list of approved contractors and consultants? How was the decision made on who to engage with on the different tasks? How long did this stage take?
12. Construction phase Again, how long? Any setbacks?
13. Grid connection phase Again, how long? Any setbacks? Tender process- Bord gais, ESB etc?
14. SEAI support- this was eligible for a grant from the SEAI, when did the grant application begin? Was it straightforward? Was there any mention of eligibility for renewable energy feed-in-tariffs through the DCENR?
15. Were there any setbacks encountered due to communication or relationship issues?
16. How did the team get the job done?
17. Details on any delays/setbacks encountered during the project is there a lessons learned document? Any recommendations for other water companies embarking on a similar type of project?
18. Currently have you achieved investment payback yet or when is that expected? Dont need to know the number but did you have a budget?

19. How did the project come in?

20. Have any further hydro projects been considered?

21. Would there be any obstacles for you sharing this experience with other local authorities for example?

22. Recommendations for anyone else to contact involved with the project (contractors, consultants, SEAI, involved at different stages?)

Pen y Cefn Treatment Works: Interview Protocol

Interview Date: 13th November 2013 at 9:00am
Location: PenyCefn Waterworks, Dolgellau
Operator: Welsh Water
Interviewees: Zeropex Business Development Manager;
Zeropex Project Engineer;
PenyCefn Waterworks Plant Manager.

Main aim of interview:

Learn the story of this project from the initial idea, to project completion and beyond. Aim is to understand why WW embarked on this project, with whom, how well it has worked and what would be recommended to other water authorities considering similar energy recovery schemes?

Plant details (Zeropex, 2014): Application: Raw water flow control entering the treatment process
Key facts:

- Difgen Model: DG13-14
- Differential Pressure: 9 - 10.5bar
- Flow Range: 10 to 30l/s
- Power Output: 8-17kW
- Annual Revenue: 29,000 GBP
- Payback Period: 2.8 years

Background:

Pen y Cefn Water Treatment Works is gravity fed from the nearby Llyn Cynwch reservoir with raw water at a pressure of up to 10 Bar. The raw water is fed into an open dissolved oxygen flotation (DAF) tank so this pressure is not required for the process and would have previously just been removed through a pressure reduction valve. As the end of the pipe into the DAF plant is elevated to a height of around 5m, normally a turbine would have had to be mounted at this height which would have proved challenging and expensive. Welsh Water chose to use a Zeropex turbine which could be sited at ground level. Due to its unique operation, it could generate renewable electricity whilst maintaining a minimum pressure on its outlet, which allows water to enter the elevated tank. The turbine also controls the raw water flow into the treatment process by controlling its speed to match the desired flow. To allow a seamless operation, the Zeropex turbine was equipped with a load bank which allows the turbine to continue running even when the electricity supply from the grid fails which allows a controlled shutdown with no hydraulic issues resulting (Zeropex, 2014).

Introduction

1. How much time do you have to give us?
2. What is your role at Pen Y Cefn / at Zeropex / at Welsh Water? What does that involve?
3. When did you first become involved in the Difgen hydro project here?
4. And who would you have worked with initially / reported to? Sketch
5. Would you have worked on any similar projects to this? Examples
6. Would you have worked with any of the people involved with this project in previous projects? Who?
7. Why did the project come about? Who had the original idea? Who approached who?
8. Who was involved in the initial brainstorming? Was everyone interested / excited by the idea; were there any concerns? Was anyone dead set against it or was everyone enthusiastic? Was there any resistance to change? Did anyone have experience on similar projects?

9. Could you give me an idea of the main stages involved in the project and aidea of the project timeline? Would you have a Gantt chart or similar available showing these stages and timelines that you could share with us?

10. If not, what would the key stages on a Gantt chart look like? (feasibility, detailed design, tender (e.g. pre-qualified contractors), planning, construction, commissioning, grid connection, grant application, continuous operation)

11. Who was involved at each of these stages?

12. Was there anyone else involved other than Zeropex and Welsh Water? E.g. electrical consultants, civil contractors etc? (Black & Veatch were they contracted by WW or Zeropex?)

13. What issues emerged at each of these stages? Were there any challenges or setbacks at any of these stages? E.g. planning issues/ unexpected issues during construction/commissioning?

14. How did trouble shooting work, who spoke to who? How long did it take? Would on-site managers phone Zeropex/Welsh Water when issues arose? Or fix things yourselves?

15. Planning permission Was this necessary? How long did this take? Were there any unexpected setbacks?

16. Feasibility study? How long did this stage take? How many options considered? What criteria were applied?

17. Tender stage How did the initial tendering process work? Was there a list of approved contractors and consultants? How was the decision made on who to engage with on the different tasks? How long did this stage take?

18. Construction phase Again, how long? Any setbacks?

19. Grid connection phase Again, how long? Any setbacks? Tender process? Or is all electricity used on site?

20. Commissioning phase Again, how long? Any setbacks?

21. Renewable energy grants? Was this project eligible for a REFIT scheme?

22. If yes, when did the grant application begin? Was it straightforward?

23. (Were there any setbacks encountered due to communication or relationship issues?)

24. (How did the team get the job done?)

25. Currently have you achieved investment payback yet or when is that expected? Dont need to know the numbers but did you have a budget?

26. How did the project come in? What did you learn about such projects?

27. (Likelihood for future collaboration?) Have you worked on any other hydro projects since this?

28. If yes Which projects? And did you do anything differently next time around? Did you apply the learning to new projects? Did you use the same team again?

29. Did members of the team work together again on new projects?

30. Had anyone worked with each other on previous projects?

31. Any future plans for other projects with Welsh Water?

Technical at turbine

1. Technical spec of turbine power output range of 8-17kW. What would it generate on average, e.g. per year? Was this the expected power output? Does it vary much - daily, seasonally?

2. Was the turbine design an off-the-shelf design? Or were there some site specific adjustments to be made to it?

3. Turbine type rotary lobe. Patented Zeropex technology? When was this Difgen design developed and patented? And where?

4. Turbine by-pass system?

5. Previously the excess pressure was removed using a PRV? Is there a PRV installed still on the by-pass system?

6. Does the turbine run continuously 24/7?

7. Is it off-line much? i.e. for maintenance/repairs?

8. Is there much wear on the turbine from the raw water? More than the expected wear or predictable wear?

9. Where (from whom) and when did the idea first come about; how long did it take before plans began to take place. Who was involved in the initial brainstorming?

10. Was this the first project of this kind undertaken by welsh Water? If not, what other projects? How many years of experience would WW have in hydropower schemes like this?

11. Have there been many issues mechanical/electrical over the past few years of running the turbine? Any additional costs in terms of maintenance? Or does it run without issue?

12. Do you do the maintenance yourselves on site or does the turbine manufacturer come back to do it? How long did the turbine manufacturer stay involved for? Did they train you in how to maintain and run the turbine? How long did that take?

Q&A After

1. Who was involved in the initial brainstorming? Was everyone interested / excited by the idea; were there many concerns? Was anyone dead set against it or was everyone enthusiastic? Was there any resistance to change? Did anyone have experience on similar projects?

2. Organisational structure? (Draw diagram) Organisational structure of WW WW energy team? How dependent were you on decisions from different stakeholders at Pen Y Cefn or Zeropex?

3. Is there a lessons learned document, at Welsh Water, or at Zeropex? Any recommendations/tips for other water authorities embarking on a similar type of project?

4. Would there be any obstacles for you sharing this experience with other water companies, for example?

5. Would you have any photos during construction stage? Or construction drawings available?

6. Recommendations for anyone else to contact involved with the project (contractors, consultants, REFIT, involved at different stages?)