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A Mathematical Modelling Approach towards Efficient Water Distribution Systems: A Case Study of Zomba - Malawi's Water Distribution Network

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Declaration of Authorship

I, **Charles Fodya**, declare that this thesis titled, **A Mathematical Modelling Approach towards Efficient Water Distribution Systems: A Case Study of Zomba - Malawi** and the work presented in it are my own. I confirm that:

- This work was done wholly or mainly while in candidature for a research degree at this University.
- Where any part of this thesis has previously been submitted for a degree or any other qualification at this University or any other institution, this has been clearly stated.
- Where I have consulted the published work of others, this is always clearly attributed.
- Where I have quoted from the work of others, the source is always given. With the exception of such quotations, this thesis is entirely my own work.
- I have acknowledged all main sources of help.
- Where the thesis is based on work done by myself jointly with others, I have made clear exactly what was done by others and what I have contributed myself.

SIGNED

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DATE

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I want to thank the Almighty God and Jesus Christ for taking me this far. Since I started school in 1987, I have gone through sicknesses, stresses enough to have costed my life, denials, etc, but You have been so faithful to me, God! I am more than indebted to proclaiming your greatness in me.

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To my family,
I'd never ask anything better from God than you all.
May I be an inspiration to everyone of you,
and a stepping stone for all,
...... with Love and Joy

Abstract

This thesis presents work on four problems identified in the Zomba Water Distribution Network. The research was carried out on the existing network infrastructure with the aim of improving efficiency by reducing Unaccounted-For Water (UFW). The first challenge was to develop a demand model for the city based on its influencing factors: daily and seasonal fluctuations as well as population dynamics. The model was to capture demand patterns for short-term, medium-term to long-term time periods, thereby becoming an important input factor into decision making. The developed model may be employed to generate demand which can be input into the city's 10 year infrastructure expansion plan. The second problem was to explain why houses built too close to tanks are at risk of inconsistent water supply. It was found that the requirement of observing the tank elevations, h_{elv}, had been violated. As a result, the discontinued supplies occurred. Once the required tank elevation h_{elv} had been established, an extra increase in the elevation so as to accommodate a larger population was determined to be on the order of one centimeter. Third problem was to explain the continued mains pipe failures observed in the network. It was established that the main cause was the hammering effects started through the process of manually closing flow control valves (FCVs) fitted next to tanks. A possible remedy was to rather fit the FCVs at joint nodes and not at the tanks. This was estimated to greatly reduce the hammering effects, eventually turning them into minor head losses due to elbow bends. Finally, a structural approach to designing tanks that optimize the use of gravity is presented. This is an input into the infrastructure expansion planning of the city. Regardless of any design shapes they may take, tanks with height-radius, h/r, ratio of less than 1 exhibit diminished, as opposed to increased, height changes with demand changes. Such a property would ensure consistency in the pressure at the tank, allowing for delivery of the demanded load.

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Nomenclature

Water Scarcity	When less than 1000m^3 of water per capita per year available to people
Physical Water Scarcity	Water scarcity caused by not having enough water to meet all demands
Economical Water Scarcity	Water scarcity caused by lack of investment in water or insufficient human capacity
Water Distribution Network	A network of tanks, valves, pumps and pipes that transport finished water to consumers
Potable Water	Water that has been treated and is ready for human consumption
Unaccounted-for Water	Water lost in the water distribution network between treatment plants and consumers due to leakages and other reasons
Water Disconnection	A situation whereby supply of water to a household is stopped mainly due to non payment of bills
Mains Pipe	A pipe in a network that has a large diameter and is used to transport large volumes of water at a given time
Feeder Tank	A tank that supplies water to another tank in a network
End Tank	A tank that supplies water to consumers

Cascade Effect	A situation in the water distribution network whereby a network design fault results into challenges like pipe failures and some parts of the network not having access to water
Longitudinal Stress	A stress coplanar with but perpendicular to the symmetry axis
Circumferential Stress	A normal stress in the tangential (azimuth) direction of a cylindrical pipe. Also called Hoop Stress or Tangential Stress

Chapter 1

Introduction

1.1 Water Scarcity

Lack of access to safe drinking water, technically termed Water Scarcity, remains a challenge in many parts of the world. According to the 2010 Water Economic Report [1], up to 45% of the world's population is affected by water scarcity. The effects of such are felt in diverse fields. For example, many lives are lost every year due to water borne diseases like Cholera because of drinking contaminated water as a result of lack of access to safe and treated drinking water [6].

Another area that gets severely affected by water scarcity is the business industry. Beverage companies, for example, require clean water as one of its major raw resources for production. In its absence, production is negatively affected [6].

The agricultural industry (plant and animal farming, processing, and other related activities) accounts for 70% of consumed water. This clearly says that in areas where water is scarce, agricultural production cannot be fully utilized to its maximum potential. Economically speaking, a country in water scarcity tends to import agricultural products, than exporting, to cover up for its needs [34]. Thus, according to the UN report [35], tackling water scarcity has a direct bearing on achieving the Millennium Development Goals.

There are two types of water scarcity: Physical Water Scarcity and Economic Water Scarcity. Physical Scarcity is when the land does not host enough water resources to satisfy the needs of its inhabitants. Places like Arid and Semi Arid regions are affected by this type of scarcity. Research conducted in Fodya et al. [37] explained that in such regions people attempted to resort to harnessing water from dew or air through air condensers. However, the harnessed quantities were far from the required amounts. Transporting water through pipes from long distances to the Arid and Semi Arid regions remains the sole viable option, albeit expensive. Economic Scarcity, on the other hand, is scarcity caused by lack of investment in water or insufficient human capacity to satisfy water demand [38]. While water sources may be available in abundance, resources such as infrastructure, political/ethnic differences and poor governance could mean that universal access to water is still limited [38]. Sub Saharan Africa's water scarcity falls under the latter. Besides being surrounded by the Indian Ocean and the Atlantic Ocean from east all through down to west (whose water can be made available for use with desalination processing), Sub-Saharan Africa has sources of fresh water like lakes (like Lake Malawi, Lake Tanganyika, Lake Victoria, just to mention a few) and perennial rivers (like Shire, Zambezi, Congo, Niger) [11].

It is interesting to note that economic water scarcity affects 25% of the world's population while physical scarcity affects 20% of the world's population [1].

1.2 Malawi: Zomba

As a country within the Sub Saharan Africa, Malawi is affected by Economic Water Scarcity. Despite having lakes which account for a third of the country's size, and despite having perennial rivers that run all year through, Malawi's water supply is only limited to cities, towns and designated townships. Even within reach of water supply infrastructure, water is rationed often and is not accessible at all times. The population lacking access to water supply infrastructure often survives with unsafe water sources like wells and boreholes.

Zomba city is the old capital of Malawi (Figure 1.1) located at the coordinates 15.3833°S, 35.3333°E. The City's topography is mountainous. As of the 2008 Population Census, the city's population was $88,314$. With a growth rate of 3%, the projected population for 2014 was set to be at $106,000$. Only 88% of the population has access to piped water [46].

The water source for the Zomba city is the Mulunguzi Dam (Figure 1.2) situated on top of Mount Zomba and as such the water supply is 95% gravity fed [10]. As is the case with all other cities and towns in developed areas, water is supplied to the population through water distribution networks that are managed by local municipalities or companies. Zomba's network is managed by the Southern Region Water Board.

The common performance challenge of each municipality, or a supply company, is efficiency. This efficiency can be defined in terms of many yardsticks. Some of the yardsticks are reducing unaccounted-for water (UFW), minimizing operating costs (energy costs, labour costs, maintenance costs, and other), increasing access to free water[1], as well as a multitude of others as defined from community to community. Whatever parameters may be used, the dominant focus of any efficient water distribution network is to attain notable deliverables such as serving a larger (or the entire) population, supplying uninterrupted water supply and maintaining required pressures at the end points.

1.3 Water Distribution Networks

All water distribution networks have a set of generalized problems, or objectives, as per the literature. Predominant problems are pump scheduling, pressure management, leakage minimization, as well as operations cost minimization [7, 9, 12, 26–29, 32, 63, 66–70]. These problems have been extensively researched on, and standard algorithms and software exist that give solutions for each given network's set of parameters.

[1] countries like Saudi Arabia and UAE provide water to their people for free

Figure 1.1: Zomba City: View of city from the Plateau and view of the Plateau

Figure 1.2: Mulunguzi Dam, Zomba, Malawi

Instead of considering the problem in a global fashion however, one could consider each network as exhibiting its own localized problems unique from another. Such problems are tackled on an individual basis as they emerge, and are better handled by understanding their localized causes thoroughly. The understanding is that a thorough study of these localized problems will produce localized solutions that may not only be effective to such networks but also to networks with similar structural conditions.

This thesis follows the latter approach by identifying localized problems for a specific network and presenting scientific explanations for each of them as well as proposing feasible solutions. Zomba city's water distribution network has been identified and is used as a research site. Four problems were identified and their causes and possible countermeasures are presented.

The first problem was to model the water demand of the city of Zomba while taking into consideration influencing factors as perceived by the supplier. The second problem was to explain why some sectors of the city's houses, especially those next to water tanks, do not receive constant water supply while other sectors further away do. A third problem discussed in this thesis is similar to the second but it involves the tanks and mains pipes. It, therefore, has far larger consequences than the second. The problem revolves around a subset of tanks which were not receiving water while others were receiving a constant supply. When some manual intervention was brought into play, another problem was created - persistent mains pipe failures. Finally, as a mountainous city, a scientific description of the recommended dimensions for tanks desirable for networks like (and including) Zomba is presented. This is an essential input into the city's infrastructure expansion plan.

1.4 Problem Definition

The main objective of this thesis is to overcome the problems discussed above while making the best possible use of the existing infrastructure of the Zomba network. We aim for answers to questions like *'how do we make the existing water distribution network more efficient?'* A mathematical approach is taken allowing us to gain insight into what can practically be done with regards to the existing engineered structure. In this manner we may hope to be able to improve upon water efficiency without adding or removing sections of the network, an engineering task that comes with some hefty costs.

The question is further broken down into specific sub-questions:

1. how can we model water demand correctly to its appropriate supply,
2. why does the network allow for certain sections of the city and certain tanks to fall short of constant water supply,
3. what are the causes for the persistent failures of the mains pipes in the network, and what are the corresponding corrective measures, and
4. what tank design properties must be employed to ensure optimal use of gravity and cut out costs associated with external power.

1.5 Rationale

At the moment, 88% of the population of Zomba city has access to piped water [46]. This corresponds to an operating supply efficiency of 60% for the distribution network since the system's unaccounted-for water (UFW) is currently sitting at 40% of total production (Table 1.1, [2]). If supply efficiency were to improve to 68%, the existing infrastructure can afford to supply the whole population of the city with water. If production was so efficient that the targeted 85% of produced water reaches the city's demand, then the existing infrastructure can affort to support up to 125% of the current population. There would be more than enough water for every resident of Zomba. Ideally one would hope that the network would operate at 100%, which means that Zomba's infrastructure would be capable of supporting 147% of the existing population (Table 1.2).

Table 1.1: Produced vs. Billed Water for Zomba City (Source: SRWB Annual Reports, Zomba)

Year	**Total Produced (m^3)**	**Total Billed (m^3)**
2009 - 10	5,329,339	3,388,982
2010 - 11	5,408,637	3,717,346
2011 - 12	5,785,611	3,945,060

Given this reasoning, and also considering the economic standing of Malawi as a country ranked amongst the poorest in the world [5], it is obvious that thoughts of expanding the current infrastructure as a means of supplying the city's inhabitants with more water may be a long term option for now - rather the focus should be on improving the efficiency of the current infrastructure. All that is required is ingenuity in terms of the use of the current infrastructure.

Table 1.2: Efficiency - Target Population Relationship

Efficiency (%ge)	**Target Population (%ge)**
60	88
68	100
85	125
100	147

By implementing the results and recommendation from this research, Zomba city will certainly be on its way to achieving the targeted 85% efficiency rate. Globally speaking, a contribution will be made towards achieving one of the Millennium Development Goals: Access to safe drinking water for all. The approach taken in this thesis was to identify localized problems in the water distribution network of Zomba city, present a scientific explanation to the causatives as well as some feasible solutions to them. It is expected, by all parties concerned, that the results will in fact be implemented by the Southern Region Water Board.

1.6 Research Process

The story of this research began in 2009 when Malawi's Southern Region Water Board decided to take a bold action to reduce levels of unaccounted-for water in its networks. At the time the unaccounted-for water levels were at around 40%. Since Zomba city alone provides for 70% of the Board's revenues, the city's water distribution network was given the highest priority. An informal meeting between the Southern Region Water Board engineers and this researcher was held in September 2009. The main problems observed in the meeting, the causes of which were not known to the engineers at the time, were persistent pipe failures in the network as well as the exclusion of other parts of the network from water supply. The latter was of special interest because the engineers observed that the most affected houses were those near the water tanks. Eventually it was also highlighted that the network did not have a structured mathematical model for predicting water demand. A desirable model would be able to capture demand patterns in the short, medium and long term taking into consideration the influencing factors as perceived by the Board.

Given the scope of the research, and the importance of its expected outcomes, a proposal to register for a Ph.D. programme was conceived so that a thorough research exercise can be carried out with adequate supervision from experts in the field. The results from the research would then be implemented within this practical context. The focus of this research was to come up with two main things: an explanation of the observed anomalies in the network, and a set of possible measures to control the observed problems. A third requirement was to identify any good measures that can be integrated into planning.

The first research visit to Zomba network was done in August 2012. The purpose of the first visit was to collect all water consumption readings available from the production plant and to physically go around the city following the pipes and tanks so as to have a clear understanding of the physical structure of the network.While diagrams for the network were available, it still was necessary to physically inspect the pipes so as to ap-

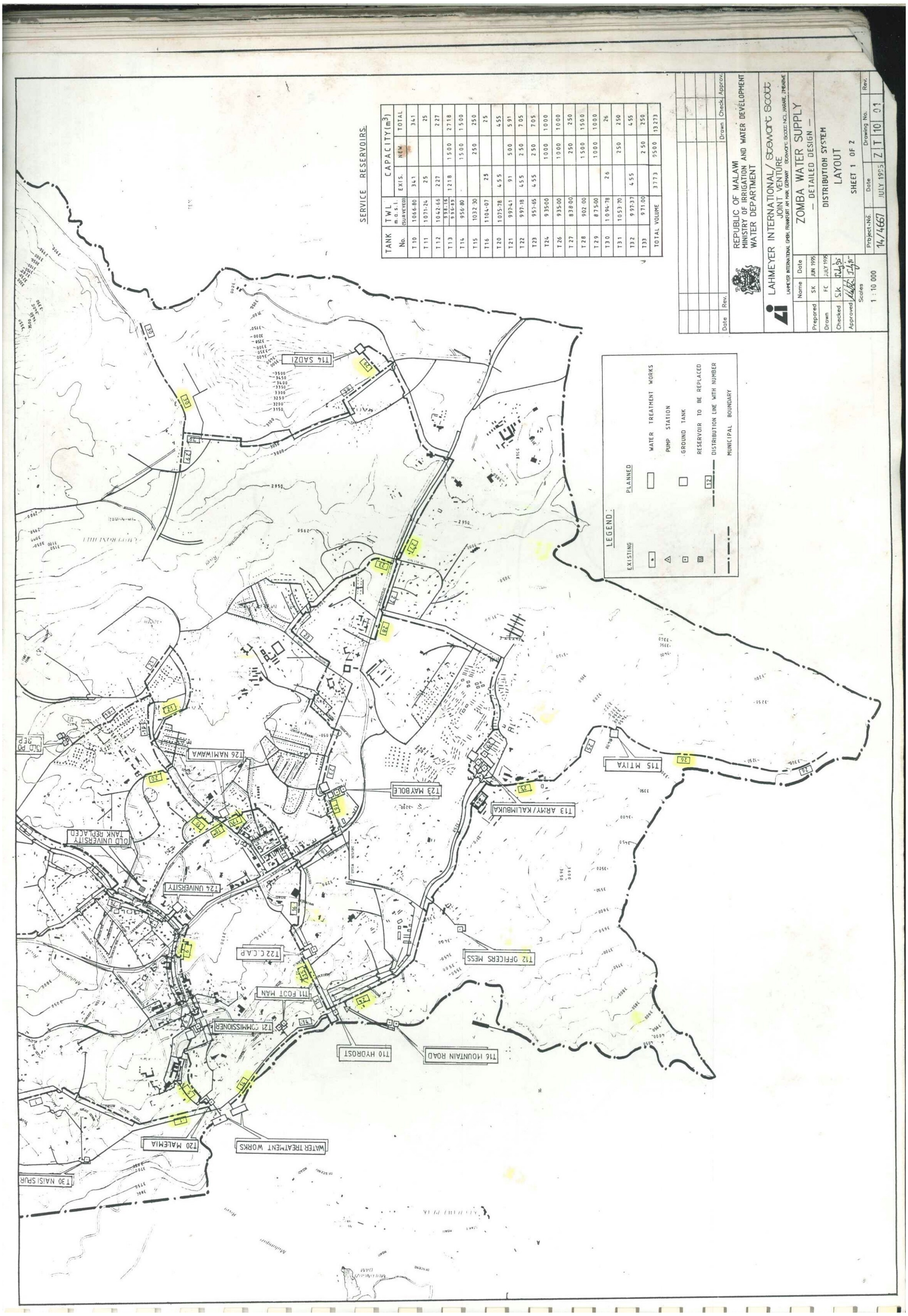

Figure 1.3: First Part of Zomba Water Distribution Network

Figure 1.4: Second Part of Zomba Water Distribution Network

preciate the cause of persistent failures considering that the pipes were of different ages some dating back to colonization periods. It also was observed that Zomba only records hourly consumption readings (which are the standard readings used in calculations) for half a day. The next recording is then taken the next day, 12 hours after the last recording of the previous day. To normalize the situation, complete 24-hourly readings were taken for seven days and the cumulative daily readings were distributed across the hours using the weighted average method. Only recordings from 2009 to 2012 were available for use in the research. To increase the size of data, the readings were then recorded for the next two years until 2014. In the end, consumption readings for a total of 5 years, from 2009 to 2014 were used in modelling the water consumption. Network diagrams with detailed geographical positions, capacities and properties of tanks and pipes were also obtained from the Water Board.

The second research visit was done a year later, in August 2013. The focus in this instance was to collect the readings that had been recorded since the previous visit.

The last research visit was in September 2014. A report on the research findings on the pre-identified problems in the network was presented. The findings discussed are provided in the Chapters to follow; the work has been structured and developed into scientific articles for publication. One article has already been published (Chapter 2), one has been accepted (Chapter 4) while the other two (Chapters 3 and 5) have been submitted for publishing. In total, four academic papers have come out of the research; it is likely that two more articles outside the scope of this thesis will also be developed based upon work conducted here.

1.7 Outline of the Thesis

This thesis is outlined as follows:

Chapters 2 to 5 are the four problems singled out individually and discussed in depth. Chapter 2 presents the water demand model of the Zomba city. Chapter 3 provides an insight into the causes and possible solutions to the problem of lack of supply to portions of the city's population from the supply tanks. Chapter 4 discusses the connection between water hammering effects and the persistent pipe failures in the network. Chapter 5 presents a desirable structural property for ideal tanks for mountainous places such as Zomba that optimise the use of gravity for supplying water. Chapter 6 provides a general discussion of the four preceding Chapters and concluding remarks are made in Chapter 7.

Chapter 2

Supplier's View of Water Demand for a City: Modelling the Influencing Factors

2.1 Introduction

The law of demand and supply dictates that any business system should be operated at equilibrium. In fact, a measure of the degree to which a system has deviated from its equilibrium is a good indicator of its efficiency. In the water industry, for example, a supplier's intent is to produce just enough water to meet demand. If less water than demanded is produced, consumers experience persistent water shortages as rationing has to be employed. If too much is produced, supplier's running costs (from wasted energy, chemicals, man power and other input resources) become high.

The need for suppliers to thoroughly understand their demand requirements cannot be overemphasized. It is as vital as the consumer's understanding of their requirements; the latter is a topic much explored in the literature [40–45, 58]. Such an understanding helps the supplier to make appropriate decisions for its operations: such as to demarcate limits between peak hours and off-peak hours, to map out communities demand needs, as well as to determine appropriate pricing methods.

Literature approaches demand from two angles; consumer's perspective and supplier's perspective. According to authors [40–45, 58] the consumer's perspective's approach lists the major influencing factors for the demand of water as household income and price elasticity of the water.

This work follows the alternative approach of looking at demand from a supplier's perspective, as is the work of authors like [3, 4, 57]. At the end of the day, what is produced must match what is demanded by the consumers. This is vital for planning how much to produce in the next time period to match the consumer demand. For example, [40,43,45] used multivariate analysis, artificial neural networks and time series analysis, respectively, to predict short term demand for up to a couple of weeks. There is however a need to be able to predict beyond a few weeks. For planning purposes, it is essential that we have an idea of what the demand may be over the next five years and as such, when the existing infrastructure (e.g. tanks, treatment plant) would become inadequate for the population at that time. Similar work was done in the research [36] which predicts that additional water sources may be required in Zomba by 2035. It is in the interest of this research that the results fall within agreement levels with those of authors like [36].

Unlike consumers, producers' influencing factors for determining the quantity of water demanded are different from income and price elasticity, and they vary from community to community. For example, Zomba city's Water Supply Company, the Southern Region Water Board, mentions daily fluctuations, seasonal fluctuations and population dynamics as its main influencing factors for its production (Figure 2.1).

Water consumption is high during the day in comparison to consumption in the night. This means more water must be treated and supplied to households during the day. Patterns are also observed when considering the various seasons. Water demand is higher in the dry season than in the rainy season. The Supply Company mentions availability of other sources of water like seasonal rivers, and rain harvested water as an explanation for the low demand during the rainy season. When considering population dynamics, it is evident that more water will be demanded with an increasing population size, household

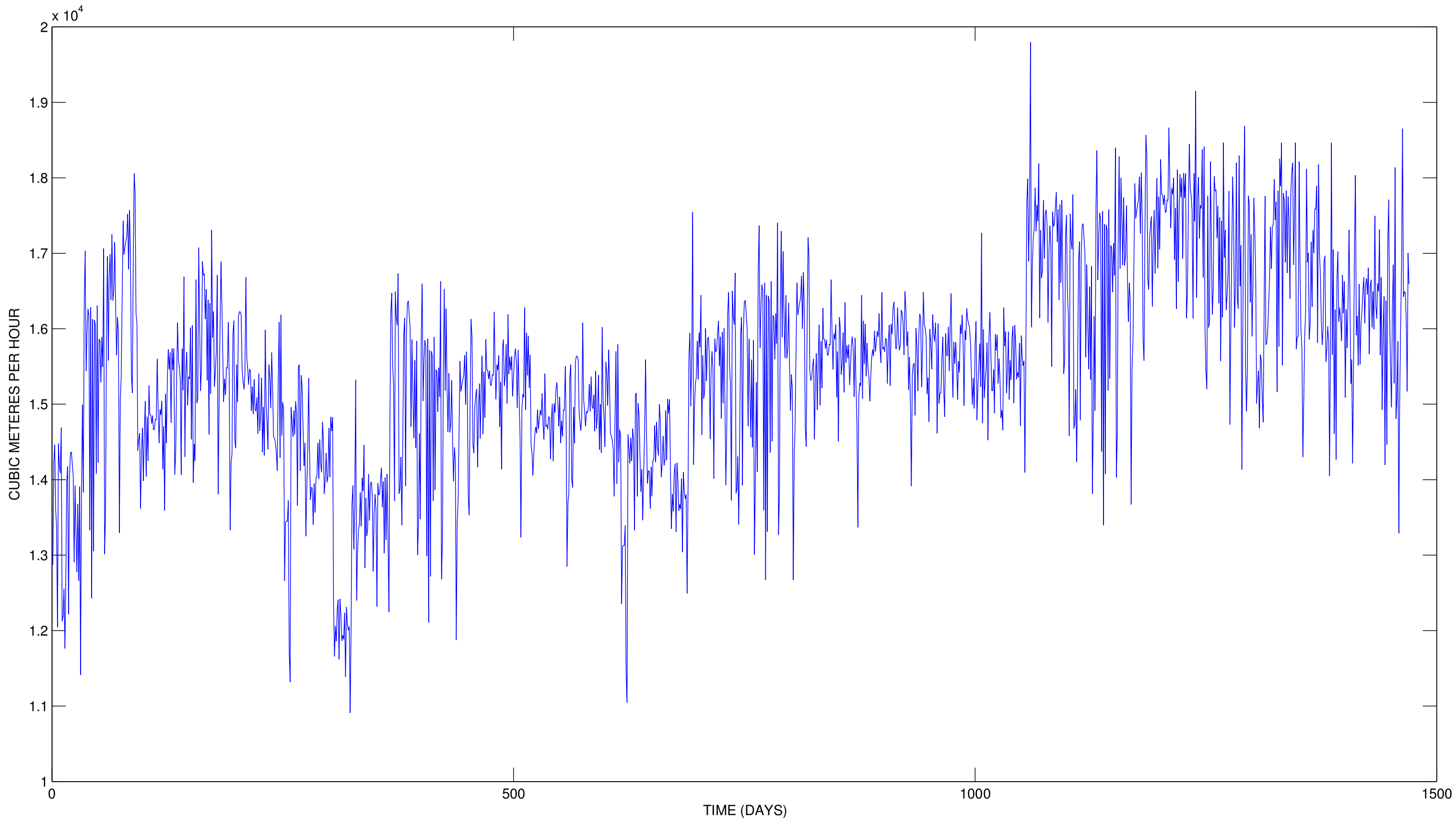

Figure 2.1: Consumption Readings for a Period of Four Years

numbers, and improved living standards, to name but a few factors.

The main aim of this Chapter is to present a mathematical model that depicts demand for water in Zomba city as a function of the three influencing factors as proposed by the Supply Company. Specifically, the objectives of coming up with this model are to incorporate the relationship between water demand in a single day, water demand in season cycles as well as population dynamics. It is expected that this model will be able to capture the observed patterns namely more water demanded during the day than at night, more water demanded during the dry season than during the rainy season, as well as an increase in water demand with an increased population size. This model must also be able to predict the demand levels which becomes inputs in decision making for the improvement of the supply network.

Zomba is the old capital of Malawi and is the major source of revenue for the Southern Region Water Board. Of the twelve districts the Board is mandated to supply water, the largest portion of the revenue is obtained from Zomba city - in fact 70% of its revenue comes from Zomba city alone. The city has a reservoir, Mulunguzi Dam, sitting on top of mount Zomba and the dam has a carrying capacity of $3,35$ million cubic meters. Being a mountainous city, 95% of its network is gravity fed. Yet only 88% of its population have access to piped water [46]. Before any improvement can be done on the city's network, it is important to first understand the demand needs of the city. This is essentially what this Chapter covers. Discussions regarding other network improvements will be discussed in the subsequent chapters.

The rest of the Chapter is outlined as follows: The next Section presents the proposed model with all necessary simplifications. Next is the methodology Section which describes how, when and why the data was collected. Fitting of the data into the proposed model is explained in the Results Section where the outcomes are mentioned. Section 2.6 is the Discussion Section which fits the current work into the current knowledge base and proposes future research alternatives to the work accomplished in this Chapter. Finally, conclusions are provided in Section 2.7.

2.2 The Water Consumption Model

In the literature there are two economic approaches with regard to modelling the demand of water supply: a consumer perspective [40–45, 58] and a supplier perspective [40, 43, 45]. The former pertains to the influences behind a consumer's water usage. Two main factors come up: consumer income levels as well as the price elasticity of water. Households cannot allocate all their income to water alone. There are many other things to be attended to with the same income. As a result, the amount they allocate towards buying water may vary over a period of time. As water is a necessity, if the price goes up, households may not reduce the amount of water they consume, even if it means the cost is now beyond their affordability levels. This however, leads to households not keeping up with required payments and, hence, a major cause of water disconnections.

A supplier, in turn, is more interested in modelling demand forecasting. This will help to demarcate peak times from non-peak times, and also to define an appropriate pricing plan. As such, a supplier's influencing factors for demand differ a consumer's. Zomba city's supplier, for example, sees a cyclic pattern in its demand on daily and seasonal basis as well as a general increasing tend with time. Some literature in pump scheduling uses parabolic functions to model demand patterns [7, 63, 64]. While it may serve its purpose to some degree, these parabolic functions are too much of a simplification as they lack the very basic property of a cyclic pattern which is required in order to be able to model the behaviour of demand.

Some literature, aimed at forecasting demand, uses methods such as multivariate analysis [40], artificial neural networks [43] and time series analysis [45]. There are however two shortfalls with these approaches. Firstly, no mention of the influencing factors appears in such models. Time series, for example, may capture patterns in the behaviour of the demand, but not the causes of the patterns. Secondly, forecasting is only done up to a maximum of a couple of weeks since a time series approach is considered. It it known that time series behaviour can completely deviate from the currently observed one in the long run. So it is safe to restrict the analysis to short time periods. This becomes a problem when the interest in modelling must include long term planning as an option where, for instance, a 10-year infrastructure expansion plan is being implemented as is the case with the Southern Region Water Board for Zomba city. As will be shown later in the chapter, the external factor population becomes the dominating factor, with fluctuations patterns not having as much impact as population.The approach followed in this research was in line of [3, 4, 62] where repetitive daily and seasonal patterns are captured by sinusoidal functions.

Simple models capable of capturing the relationship between water production and daily fluctuations are the sine or cosine models. Choosing which one of the two to use depends on the problem at hand. In our case, for example, standard production readings start at midnight of each day and end at 11:00pm of the day. Coincidentally, for a city like Zomba, where heavy production industry that requires factories to operate even at night, is not available, production readings are at their lowest level after midnight and before daybreak. They tend to start rising by daytime until they reach their maximum, and then drop towards evening to reach their minimum levels again after midnight and before daybreak. A sine function best suits the pattern. Differentiating the sine function, which is equivalent to determining the rate of change function for production readings, results into a cosine function.

Similarly, the financial year for the Water Board begins in every July of the year and ends in every June of the next year. As Malawi's rainy season, where one expects the demand for water to drop, start in October and ends in March, it is worthy noting that production readings are high at the beginning of the financial year. They eventually begin to drop until they reach their minimum levels in rainy season before rising up again to their maximum levels by end of the financial year. So the pattern starts from high dropping down to low and then rising up again to high. This is a pattern of a cosine function and, hence, the condition is better modelled by the cosine function. In the situation a differential of the production readings function is concerned, the derivative of the cosine function, the sine function, is used.

In the case of population dynamics, the number of households and the living standards for the population are the major determinants for access to piped water. However, according to census data from the National Statistical Office of Malawi [46–48] the proportion of the Zomba population with access to piped water has been approximately 80%, (78% in 1987, 80% in 1998 and 81% in 2008) of the prevailing population size. Thus, the population size can be considered an indicator of water access in the modelling process. As such, we propose the following water demand model:

$$\frac{dU}{dt} = \beta + \eta^* \cos(\omega_1 t) + \zeta^* \sin(\omega_2 t) + \kappa \frac{dP}{dt} \tag{2.1}$$

where

- $U = U(t)$ is the consumption, or demand, function of time t,
- $P = P(t)$ is the population function of time with P_0 the initial population,
- ω_1 is the short term (in our case, daily) fluctuation frequency, with period $T = 24$ hours
- ω_2 is the long term (in our case, seasonal) fluctuation frequency, with period $T = 365$ days, and
- β, η^*, ζ^* and κ are constants.

From literature on logistical differential equations [105–107], we have that

$$\frac{dP}{dt} = \gamma P(1 - \frac{P}{K}) \tag{2.2}$$

where γ is the population growth rate and K is the carrying capacity of the medium (in our case, the maximum population Zomba city can contain). In the context of our modelling, we can consider K as the total of individual maximum houses each tank can supply. As long as the households at a tank are below its carrying capacity as is described in this context, tanks may continue connecting new houses without need for expanding infrastructure.

Inserting (2.2) into (2.1) and integrating both sides gives

$$U(t) = \alpha + \beta t + \eta \sin(\omega_1 t) + \zeta \cos(\omega_2 t) + \kappa \frac{K}{1 + Ae^{-\gamma t}} \tag{2.3}$$

where α, $\eta = \dfrac{-\eta^*}{\omega_1}$, $\zeta = \dfrac{\zeta^*}{\omega_2}$, and $A = \dfrac{K - P_0}{P_0}$ are constants.

The model must be able to handle short term to long term patterns for the demand. It is important to also vary time to magnitudes which will simulate 'long run' scenarios. A possible approach to take this necessity into account is to take the limit of the function U as $t \to \infty$. This corresponds to a situation where the population is at its carrying capacity and that the dynamics involved with population changes are not effective anymore. It is important to note that

$$\lim_{t\to\infty} \frac{K}{1 + Ae^{-\gamma t}} = \frac{K}{1+0} = K. \tag{2.4}$$

Inserting the results from (2.4) into (2.3), we have the following:

$$U(t) = \Lambda + \beta t + \eta \sin(\omega_1 t) + \zeta \cos(\omega_2 t) \tag{2.5}$$

where $\Lambda = \alpha + \kappa K$.

An interesting result from the simplification is that while the third and the fourth terms of the right hand side of the model cater for the short term and long term fluctuations respectively, population dynamics is taken care of by the linear term. It must be emphasized that despite the assumption that the population is at it's carrying capacity, other factors such as well being of the people will still affect the demand. For example, when an individual buys a car, he or she will require extra water to clean it. The same principle applies to someone moving into an independent home. He or she may require extra water than when staying by parents home. However, on the overall, the proportion of the population that is connected remains at around 80% at any given time. All efforts to reduce the UFW are in fact aimed at increasing the said proportion and thereby adhering to MDG's requirements in the long run.

2.3 Methodology

Data on daily production readings from the Treatment Works Plant for Zomba city, spanning July 2009 to June 2013, were obtained from the Southern Region Water Board's Zomba office. To conform to the standard data used in literature, twenty-four hourly readings for a period of seven days were taken in the second week of September 2012. Based on the average weighting method, daily readings were distributed across the twenty-four hour period.

As mentioned above, given that there is an average of 4.4 persons per household in the Southern Region of Malawi for the past three census results [46–48], and that the percentage of the population that has access to piped water has remained at 80% across the 1987, 1998 and 2008 census reports, it seems reasonable to use the 2008 population size as an indicator of the number of households which exist and have access to piped water in the city [46–48].

Both daily and hourly production data were separately fitted into the developed mathematical model (2.5) and the corresponding parameters for the daily fluctuations, seasonal fluctuations as well as population dynamics were obtained for each of the two data sets.

The resulting functions with their associated parameters were compared to the raw daily readings obtained from Zomba in the next section. Readings generated by the hourly model were summed in twenty-four hourly intervals and compared to the raw daily readings.

2.4 Results

2.4.1 Population Growth and Consumption

As previously mentioned, the linear term in the derived model models the population growth effect on the consumption. It is interesting to note that a simple population-consumption plot reveals a linear as well as a periodic relationship of the two - consumption against extrapolated population based on the growth rate - is in agreement with the model (Figure 2.2). It is worthy noting that the plot does provide an estimate of per capita consumption, of $0.22m^3$/day/person or (220 litres). This is equivalent to $80.3m^3$/year/person (which is below the threshold of $1000m^3$ as defined in Nomenclature section, despite being enough for basic consumption only, excluding water needed for extras like food production) which confirms that indeed water scarcity is a challenge in the city as is the case with the region.

2.4.2 Model Fitting

The model was used to fit two sets of data separately; the daily recorded data set and the hourly recorded data set. This was done by changing the values of the frequencies. When fitting the daily recordings data set, the periods used were 1 day for the daily fluctuation and 365 days for seasonal fluctuations. Thus ω_1 and ω_2 were assigned the values of 1 and $\frac{1}{365}$ respectively[1]. While fitting the hourly recordings set, ω_1 and ω_2 take the values $\frac{1}{24}$ and $\frac{1}{24 \times 365}$ respectively. MATLAB's curve fitting toolbox was used to design the solutions according to equation (2.5) for the two data sets (daily and hourly respectively) by calculating the appropriate coefficient values for either of the models.

[1]Recall that for period T and frequency ω, $\omega T = 1$

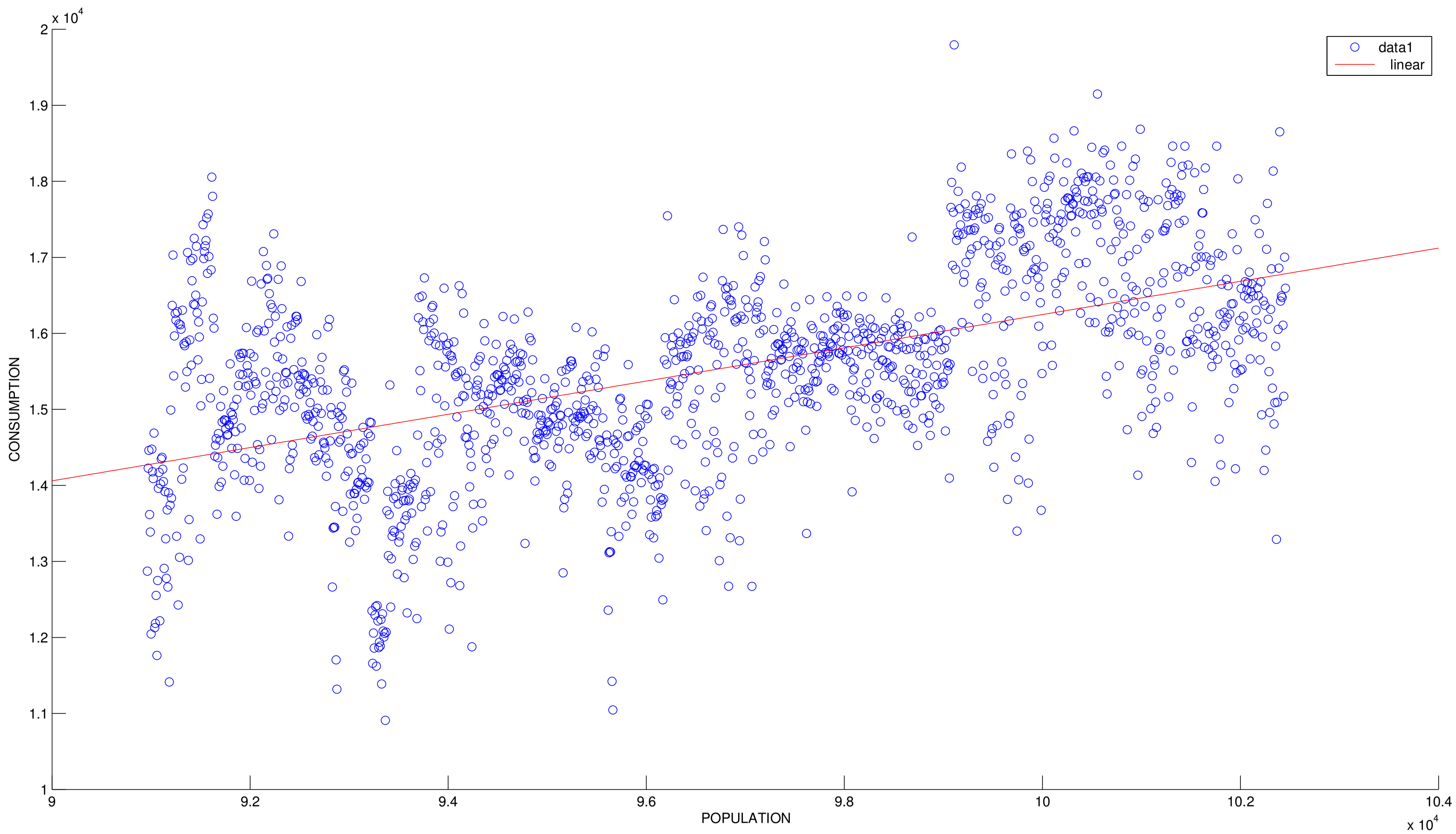

Figure 2.2: Linear Relationship between Estimated Population Growth and Consumption

The Daily and Hourly Models, as derived by the data fitting procedure are as follows:

$$U_D(t_D) = 14239 + 2t_D - 76\sin(t_D) + 13\cos(\frac{t_D}{365}) \quad (2.6)$$

$$U_H(t_H) = 594 + 0.003t_H + 0.997\sin(\frac{t_H}{24}) + 0.3418\cos(\frac{t_H}{24 \times 365}) \quad (2.7)$$

where t_D is time in days and t_H is time in hours.

2.4.3 The Hourly Model

The Hourly Model (2.7) was used to generate hourly data over the whole four year period over which raw readings were recorded. The generated readings were added in successive 24 hours to come up with daily readings for comparison with the actual daily readings. A linear regression for the raw daily readings against those generated by summing up output from the hourly model was structured. A linear relationship was observed. The observed deviations from the linear pattern are a result of inconveniences in the network such as closures to maintain pipes and pipe bursts, water disconnections and others (Figure 2.3).

2.4.4 The Daily Model

When raw daily recordings are fitted to a linear model, the function returned is $U_{RawFitted} = 14252 + 1.7003t_D$. It is interesting to note how the coefficients of the just mentioned function and those of the linear term from the U_D function are almost the same.

With the 2012 production capacity of $15000m^3/day$ (at that time another treatment plant was being erected that would increase the production to $27,000m^3/day$) from Zomba Treatment Works Plant and the projected population of $600,000e^{0.03\times3}$ (where $600,000$ is the 2009 population of Zomba City, 0.03 represents the growth rate of 3% and 3 the number of years to 2012 [46, 50]), it will take about 12.8 years for the plant to become inadequate for the demand when the new plant becomes operational. If efficiency

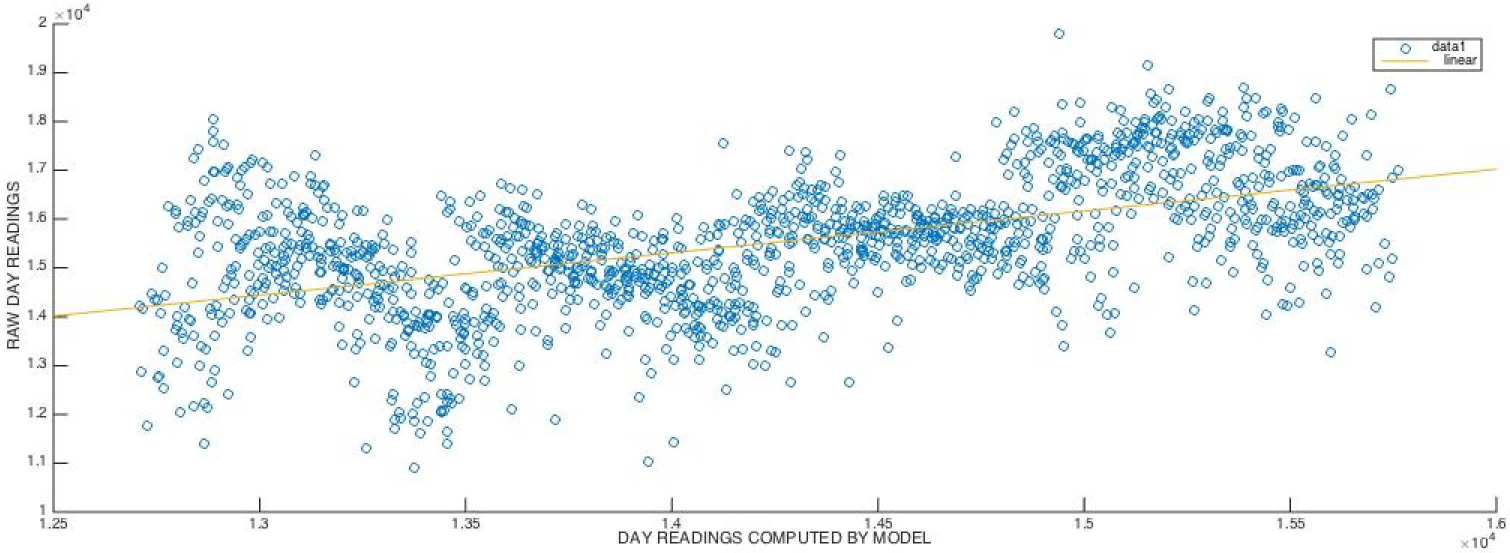

Figure 2.3: Linear Relationship between Raw Daily Readings and Daily Readings from the Hourly Model: All reading on both axes are in cubic metres (m^3)

were improved to the targeted 85%, then the infrastructure would be sufficient to carter for a population $820,000$ which is only attainable for the next 7.35 years. The capacity of the treatment plant has nothing to do with the capacity of the dam. The current water reservoir, the Mulunguzi Dam, has a carrying capacity of 3.35 million cubic meters of water and is enough to supply the city until 2035 according to research by [36]. This is a result of the replacement rate of the water in the dam. It is clear that the capacity of the treatment plant is very small and needs expansion to contain the projected dam's supply to 2035.

In practice, as a way of avoiding sediments interfering with pipes collecting water in a reservoir, a certain proportion of the water from the bottom of the reservoir is never used for water collection. Such pipes are raised above the bottom of the reservoir. This unused portion may account for a third of the reservoir capacity, and thus does affect the disposable carrying capacity of the reservoir. In the case of the Mulunguzi Dam in Zomba, two-thirds of the dam's carrying capacity of 3.35 million cubic meters is available.

2.4.5 Statistical Tests

By defining a Null Hypothesis as There is no statistical relationship between the observed data and the expected data from the developed model, we will try to find out whether the statistical methods agree or disagree with the Null Hypothesis.

The following table summarizes the statistics: Since there is no overlap in the confi-

Table 2.1: Performance statistics for Observed versus Expected hourly consumption readings

	Mean	**Std Dev**	**Std Error**	95% **Conf. Int**	
				Min	**Max**
Observed	647	131	0.6983	645.6034	648.3966
Expected	651	30	0.1601	651.3202	650.6798

dence intervals, it can be interpreted as a rejection of the Null Hypothesis. Therefore, there is a significant statistical relationship between the observed data and the expected data computed from the model. This indicates that the developed model may be a good starting point towards a perfect demand model with realistic simplifications as is the case with this thesis.

Next we shall try to compute the χ^2 value for the observed and expected annual annual consumption readings as depicted in Table 2.2. Again, our Null Hypothesis remains there is no statistical relationship between the observed data and the expected data generated from the model. Using the formula

$$\chi^2 = \sum \left(\frac{(Observed - Expected)^2}{Expected} \right), \tag{2.8}$$

the χ^2 value of the data is 7593.4. The critical value from χ^2 distribution table where p-value is 0.05 (equivalent to 5%, leaving the remaining 95% for our confidence interval) and degree of freedom of $4(= 5-1)$ is 9.49. Again, the χ^2 value is higher than the critical value, leading to the rejection of the Null hypothesis.

Table 2.2: Actual vs. Modelled Annual Water Production Readings

Year	**Actual Consumption** (m^3)	**Consumption by Model** (m^3)
2009 - 10	5,329,339	5,334,783
2010 - 11	5,408,637	5,597,547
2011 - 12	5,785,611	5,860,034
2012 - 13	6,173,984	6,213,154
2013 - 14	6,589,912	6,601,473

A run of hourly meter recordings against those generated from the model gave a R^2 value of 0.601 and adjusted R^2 value of 0.589. With the exclusion of outliers, which were determined by performing the operation

- Outlier = if ABSOLUTE VALUE (mean (raw hourly) - hourly data value]) > tolerance) [In our case, the tolerance was 100 since the meter readings were of the order of 1000],
- Assign NAN to all outlier values

- Drop all NAN values and their corresponding values on the expected data set
- perform statistics tests on the end data sets,

the number of data values used dropped. This too had an effect on the final computed values.

A similar run for coefficient of determination of population vs consumption readings using the same method as outlined above gave R^2 the value of 0.587 and adjusted R^2 value of 0.574.

In all cases, the expected data generated by the discussed model has a significant statistical relationship with the expected data from the developed model. This concludes that the developed model can be a good source of modelling water demand for the city of Zomba. Further improvements can be built on from this model.

2.5 Discussion

Now that we have a simple and basic model that is able to predict consumption levels with accuracy levels around 60% for Zomba city (Table 2.1), it is essential to note that we are now able to improve the efficiency of the current supply of water by building on the model. This efficiency does not only influence planning of consumption, but also becomes important for planning the expansion of the current infrastructure. For example, the Zomba city ought to plan for expansion of its Water Treatment Plant in the next 10 years (which was already under construction as of 2002 according to [10] and was completed in 2013), so that by the time the existing plant becomes inadequate, there is already a support structure for the extra demand. Issues of water rationing need never be experienced if proper planning is done.

As can also be deduced from the model, there should be enough piped water for every resident of Zomba city as well as the whole Zomba District for up to 2035 ([36]).

Even with the current reservoirs and treatment plants (both existing and the newly built one) as of 2012, Zomba has enough water for the city for at least 10 years. This is in agreement with the fact that, as a city in Sub Saharan Africa, Zomba is affected by economic water scarcity, rather than physical water scarcity [1, 34, 38]. Thus with efficient management of the existing infrastructure water should be supplied to many more people than is currently being achieved. This means that further research into efficient management of the supply systems is essential for cities like Zomba; the same can be said of other cities while allowing that certain differences may arise. In the end, when we have more efficient water distribution systems, the millennium development goal (MDG) of access to safe drinking water will be achieved in the region which currently experiences water shortages [6, 35].

Considering that hourly data readings had to be derived from daily readings, the model derived could be improved upon further if we had the exact hourly readings recorded. Also, a suggestion that the hourly readings be taken at the Treatment Works Plant, at both end of the Tanks (inflow and outflow) and at the consumers' end (usually can be obtained from utility bills) can help determine which area of the network is more inefficient than others. Targeting such areas will have a great impact on the efficiency of the systems.

Finally, the model is built on the understanding of the prevailing factors that affect the city's water demand. As cities have different factors affecting their demand needs, the model may differ from city to city although the three mentioned factors in Zomba are bound to have an effect on almost every city. Some cities, unlike Zomba, have big industrial activities going on for 24 hours a day which require consistent water supply day and night. Some cities would have more alternative permanent water sources like rivers that run all year or lakes. As such, the influencing factors may differ from city to city. This may affect the level of complexity of the demand model for each city.

2.6 Conclusions

This Chapter has presented a new mathematical model for determining demand or consumption of water in a typical city of Zomba in Malawi. Unlike the previous models which capitalized on the household's influencing factors to water demand namely income and price elasticity, this model takes into consideration the influencing factors as perceived by the supplier. For example, in Zomba city, Malawi, the supplier mentions daily and seasonal consumption fluctuations as well as population dynamics as the major influencing factors. With the constant ratio of 4.4 persons per household and a constant percentage of 80% of the population having access to piped water, the population size is a good indicator to work with. Daily consumption data over four years up to 2009 was collected from the Southern Region Water Board's Zomba Office. To conform with the standard data in literature, hourly data for seven days was recorded and the daily consumption data was converted into hourly data using the weighting method. Both daily and hourly data were fitted into the model separately and the corresponding parameters (constants in the model) were calculated using MATLAB. The corresponding daily as well as hourly models gave convincing results (Table 2.1). Minor differences were observed. The differences are in fact a vindication of an improvement in the network. It implies a decrease in water loss which leads to a decrease in total produced water. The predicted water production obtained via the model we have structured used data collected when no effort had been put into improving on the network. This explains why predicted readings obtained from the model are higher than the actual readings. At the current population growth rate, the existing treatment plant can serve the population for the next 12.8 years, and the current Mulunguzi Water Dam has over 20 years to serve the entire population of Zomba district as of 2012, which is in agreement with the research of [36] that sheds some important light on predicting acute shortage of supply by the year 2035.

Chapter 3

Annihilating Flow Challenges: Why Some Parts of the Network are Unable to Receive Water

3.1 Introduction

A consistent water supply that's up-and-running most of the time, and has no persistent pipe burst problems, is desirable [51]. It is always expected that this supply must increase with time until a stability situation is achieved. This corresponds to reaching out to the maximum carrying capacity of the population. This, however, requires multi-disciplinary teamwork from scientists and engineers to say the least. As pointed out by [52], the scientific community has a crucial role to play to ensure individuals have access to potable water. It is the responsibility of governments to ensure individuals have access to safe water. In the case of Malawi, the Malawi Government oversees this responsibility through support from the World Bank through loans and grants. The implementation of the same is the jurisdiction of the five Water Boards and City and District Councils. Southern Region Water Board, together with District councils in the south oversee water connectivity for the whole Southern Region of Malawi except Blantyre which is managed by Blantyre Water Board.

One area where the scientific community has played a vital role in developing it is pump scheduling. This has become a field where efficient algorithms are being developed to ensure that while demand is being satisfied, energy costs are held low and an adequate amount of time for the servicing of pumps is allowed for [7, 62–65]. In terms of day-to-day maintenance the control of leakage is of prime importance. Algorithms are developed and implemented in networks to control unaccounted-for water (UFW) through pipe leakages [66–70].

Furthermore, an explanation for the problems which arise for specific networks and as such are not generic issues also require scientific investigation. In this Chapter, we discuss the notable lack of flowing water at certain sections within a network that relies almost entirely on providing water via gravity. Interestingly, houses that are very near the tanks are the ones suffering most from this problem. This problem is in fact a recent one, coming about as the city was undergoing expansion which led to houses being built close to the water tanks. As of 2012, this was a problem mainly on tanks located in the mountain locations: $T_{11}, T_{12}, T_{11}T_{20}, T_{21}, T_{30}, T_{32}$ and T_{33}. For example, some houses had to be connected to a mains supply pipe feeding T_{21} because the situation was so bad for the houses. Even at night, when most of taps in the other houses are closed and pressure demands are minimal, they could not have water supply.

The main aim of this Chapter is to provide a scientific explanation as to why houses right next to the tanks are not supplied with water while those some distance away are being supplied. Specifically, we look at the relationship amongst the pressure that drives the water to desired destinations, the discharges required at the demands nodes, altitudes of the places where demands are required, as well as tank elevations.

The rest of the Chapter is outlined as follows: Section 3.2 presents an analysis of the situation with appropriate models defined and some insight into the problem is provided. Section 3.3 simulates a part of the network with the developed models, providing more insight into why the problems really exist within the network. Section 3.4 discusses possible remedies to the challenge and we conclude with Section 3.5, a summary of the Chapter.

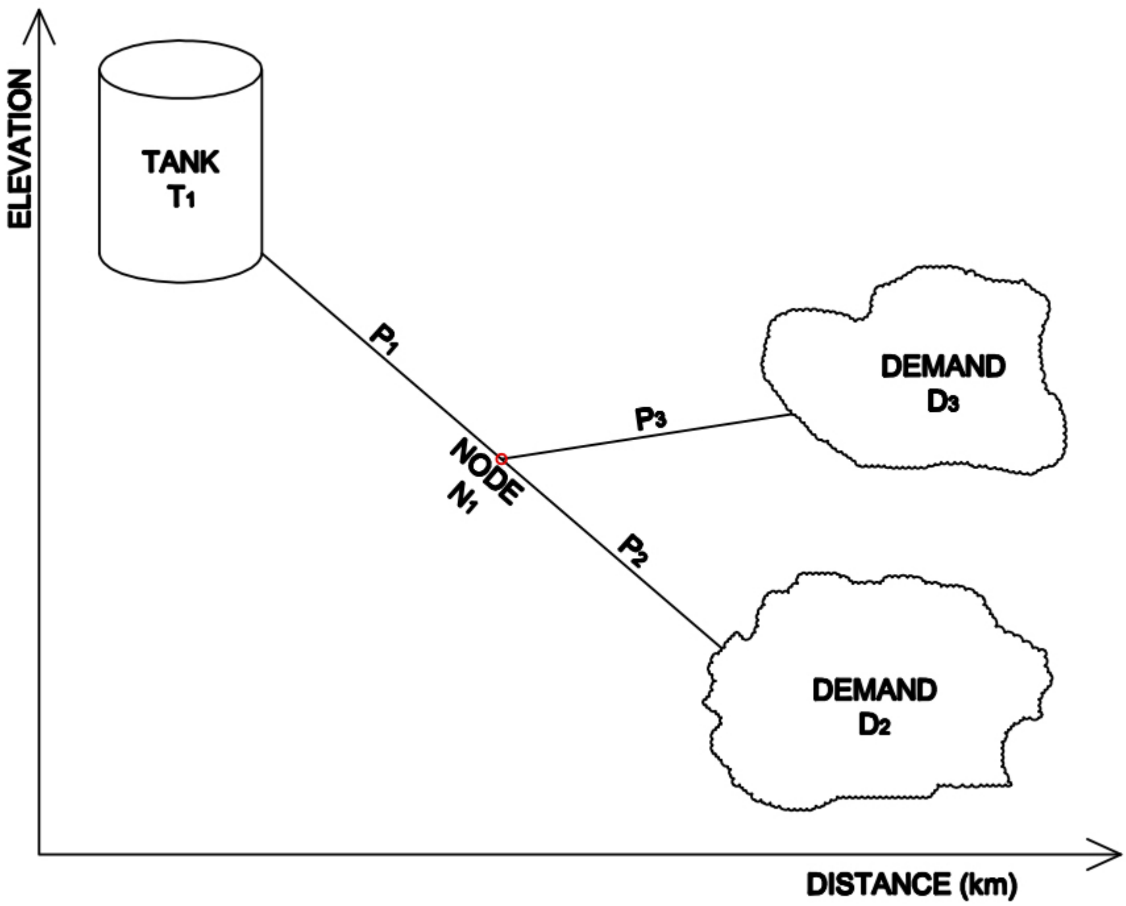

Figure 3.1: Sketch of the Model for Water Delivery to Demands at Different Altitudes

3.2 Situation Analysis and Modelling

To clearly understand the situation, a simple simulation experiment is described (Figure 3.1). In the experiment, tank T_1 supplies water via gravity to two areas D_2 and D_3.Thus

$$Q_1 = Q_2 + Q_3 \tag{3.1}$$

where Q_1 is the discharge from T_1 flowing through pipe P_1, and Q_2 and Q_3 are demands from D_2 and D_3 respectively.

Using Bernoulli's Equation, we find that the required pressure at the tank to supply water at both D_2 and D_3 can be calculated as follows:

$$\begin{aligned} \frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 &= \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + H_L \qquad (3.2) \\ &= \frac{p_3}{\gamma} + \frac{v_3^2}{2g} + z_3 + H_L \qquad (3.3) \end{aligned}$$

where

- v_1, v_2, v_3 are water velocities in P_1, P_2, P_3 respectively,
- p_1, p_2 and p_3 are pressures at T1, D_2 and D_3 respectively,
- $\gamma = \rho g$ is the specific weight of the water,
- ρ, g are the density and gravitational constant respectively,
- z_1, z_2, z_3 are reference heights of the tank and the demands, and
- H_L is head loss along the paths to D_2 and D_3 due to friction in pipes.

Since our main concern is delivering water to areas of high elevation, we will concentrate on equation (3.2) which can be rewritten explicitly in terms of the required pressure at T_1 as

$$p_1 = p_2 + \frac{1}{2}\rho(v_2^2 - v_1^2) + \gamma(z_2 - z_1) + \gamma H_L. \qquad (3.4)$$

A closer observation of equation (3.4) reveals that if the demand D_2 is at a higher elevation than the tank, i.e. $z_2 > z_1$, more pressure is required to satisfy the supply. A pump would be required in this case. On the other hand, if D_2 is at a lower elevation than T_1, i.e. $z_2 < z_1$, then less pressure is needed from the tank as much of it will be hauled through gravitational pull.

When water begins to flow from a high to low elevation it has initial potential energy, $\rho g(z_1 - z_2)$. It accelerates under the influence of gravity until the head loss generated by the kinetic energy of the fluid consume the potential energy. At this point the system attains equilibrium and the maximum flow for the given system parameters is attained.

In the absence of an external power source, e.g. a pump, this pressure, p_1 required from the tank must be derived from the potential energy generated from the elevation of the top water level (TWL) of the tank (a combination of the actual elevation of tank from the ground, the water head in the tank and the atmospheric pressure ATM). In the absence of tank lifts, which is typical of Zomba, the source of the pressure is derived from atmospheric pressure ATM and pressure from the head in the tank.

Thus we have the following governing equation for the pressure

$$\begin{aligned} p_{atm} + \gamma h_w &= p_1 \\ &= p_2 + \frac{1}{2}\rho(v_2^2 - v_1^2) + \gamma(z_2 - z_1) + \gamma H_L \end{aligned} \tag{3.5}$$

where

- p_{atm} is the atmospheric pressure, and
- $h_w = h_{tw} + h_{elv}$ is the water height defined as a sum of the actual height of water in the tank, h_{tw} and the tank elevation height, h_{elv}, is above z_1.

By setting $h_{tw} = h_{min}$, the minimum water height required in a tank so that it is able to supply according to demand with the assistance of gravity, (3.5) becomes

$$h_{elv} = \frac{p_2}{\gamma} + \frac{1}{2g}(v_2^2 - v_1^2) + (z_2 - z_1) + H_L - \left(\frac{p_{atm}}{\gamma} + h_{min}\right). \tag{3.6}$$

Tank elevation h_{elv} can also be expressed in terms of discharges, which is the most commonly recorded reading in water distribution systems, as follows

$$h_{elv} = \frac{p_2}{\gamma} + \frac{1}{2g}\left((\frac{Q_2}{A_2})^2 - (\frac{Q_1}{A_1})^2\right) + (z_2 - z_1) + H_L - \left(\frac{p_{atm}}{\gamma} + h_{min}\right). \tag{3.7}$$

where A_1, A_2 are the cross sectional areas for P_1, P_2.

The parameter h_{elv} plays a very important role in deciding where to position the tank.

If the tank has to supply any demand on its immediate outlet, then it should be lifted above the ground or a water tower must be erected to allow for this elevation. In hilly areas, the same can be achieved by placing the tank at an elevation which starts serving households located at an equivalent height of h_{elv} below its location. Households between the tank's location and its equivalent h_{elv} below its location will experience the water shortage problem due to inadequate pressure (Figure 3.2).

Holding $Q_1, Q_2, z_1, z_2, p_2, p_{atm}$ and H_L constant transforms the model into its most simplified form

$$h_{elv} = \kappa + (z_2 - z_1) \tag{3.8}$$

where $\kappa = \frac{p_2 - p_{atm}}{\gamma} + \frac{1}{2g}\left((\frac{Q_2}{A_2})^2 - (\frac{Q_1}{A_1})^2 \right) + (H_L - h_{min})$ is a constant.

In the absence of external power sources like pumps, we'd like to avoid supplying water to areas at higher elevations than the tanks, and hence, the worst case we'd like to have is to supply to areas at the same elevation as the tank, i.e. $z_1 = z_2$. Thus in order to find h_{elv} for a tank, we only need to obtain κ.

The derived model is tested on a part of the Zomba Network that exhibits the cascading effect and the results are reported in the next Section.

3.3 Numerical Simulations and Results

When hourly consumption readings are recorded at a treatment plant, the consecutive readings' differences represent the volume of water produced and distributed to the whole network in the specific hours between which the readings were taken. The water is channelled to various tanks according to their capacities that in turn represent the proportions of the population they are servicing.

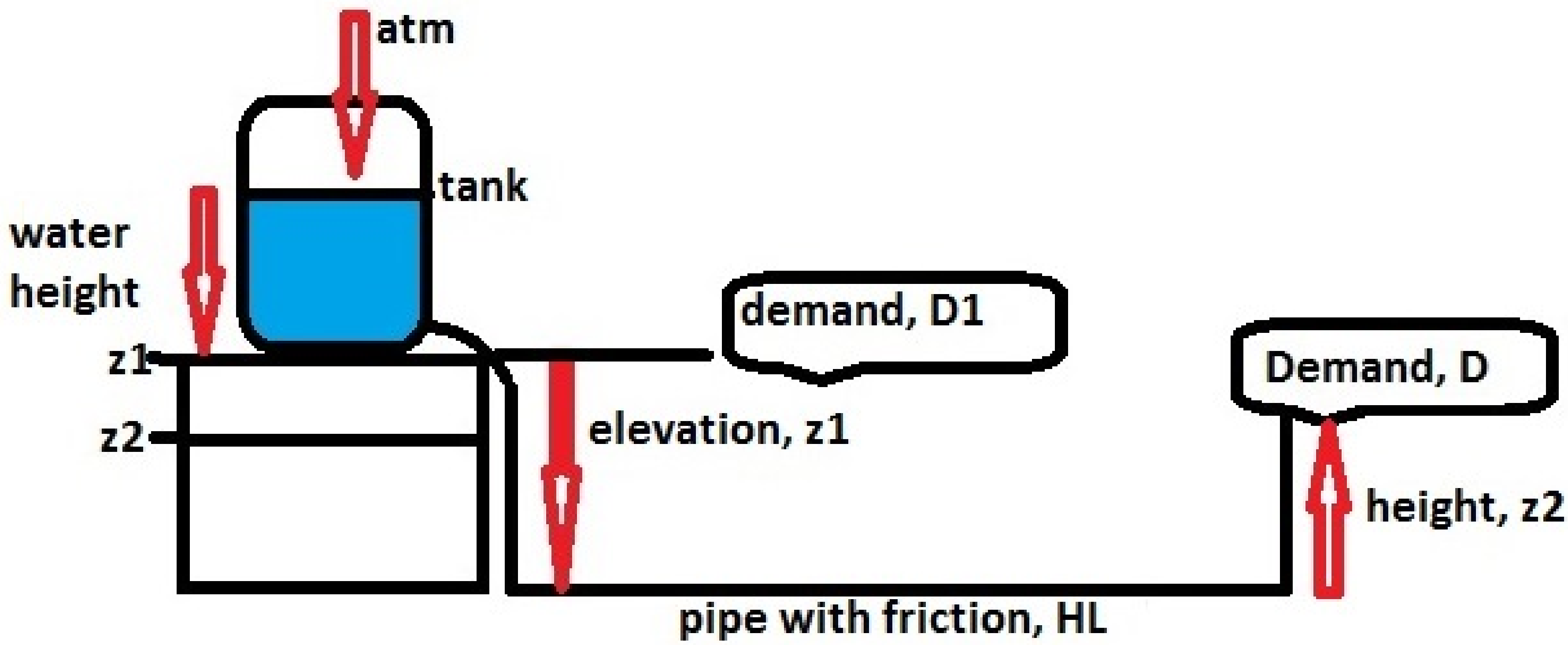

Figure 3.2: Sketch of effect of elevation on households

For the purposes of comparing the simulations produced with actual readings from the network we consider Tank T_{20}, also known as the Old Naisi Tank. This tank has a reported problem that houses close to it hardly have any access to water while houses at the bottom of the hill enjoy constantly running water. Usually, they have adequate supply at night when most of the houses downhill have closed their taps.

This problem was observed in tanks located in the mountain. These tanks are usually sitting on the ground. Specifically, we are talking of tanks $T_{11}, T_{12}, T_{20}, T_{30}, T_{32}$ and T_{33}.

We will try to employ the argument and model derived above to explain this situation and a possible solution. Specifically, we consider an 8am reading on the 9th of October, 2012, where $1152m^3$ of water was produced and discharged into the network system. This figure was the total hourly discharge for the whole network (i.e. $1152m^3/hr = 0.32m^3/sec$). The capacity of all the tanks in Zomba as of 2012 was $13,273m^3$ ([10]), and T_{20} had a capacity of $250m^3$ ($\sim$ 1.9% of total Zomba capacity ([10])). Thus $0.006m^3/sec$ (1.9% of total water produced at 8am of 9th October 2012) was estimated to have been at T_{20} and supplied to households connected to T_{20}. Three percent (3%) of the households downstream of T_{20} experienced a shortage of water [10]. Ideally, they're supposed to have accessed $0.03 \times 0.006 = 0.00018m^3/sec$ of the flow from the tank, meaning $0.00582m^3/sec$ was supplied to the rest of the houses connected through the tank.

The water supply service pipes through which households receive their water supplies are typically 100mm diameter pipes. To avoid pipe blockages between the tanks and pipe supply mains, the pipe supply mains are installed approximately 30cm above the base of the tank and the recommended pressure at delivery for Zomba is 1.5 bar [10]. With this information, we can find the value of κ as follows:[1]

$$\begin{aligned}\kappa &= \frac{150,000Pa - 101,325Pa}{1000kg/m^3 \times 9.8m/s^2} + \left(\frac{1}{2 \times 9.8m/s^2} \frac{(0.0018m^3/s)^2 - (0.06m^3/s)^2}{(\pi \times 0.05m)^2}\right) \\ &\quad +(H_L - 0.3m) \\ &= 4.64m + H_L \qquad (3.9)\end{aligned}$$

If the houses under concern are right next to the tank, in which case the piping is not considerably long, then the effects of Head Loss (H_L) are very minimal and can be ignored. In that situation, the required elevation height h_{elv} becomes

$$h_{elv} = \kappa = 4.64m. \qquad (3.10)$$

This is consistent with the observed situations in the network and specifically on T_{20}. While all of the affected houses may be more than $4.64m$ away from the tank, their vertical displacement from the tank is less than the said height.

In general, the water distribution systems supplied directly from reservoirs have large diameter principal trunk mains; because of this, there is very little head loss within the trunk main and the associated pressure loss at high demand flows is negligible. The pipework associated with each individual customer's supply, however, tends to be of a much smaller diameter and the minor losses associated with bends and fittings can be quite high at times of peak demand and can be generalised to be of the order of $2 - 3m$. The diurnal demand within water distribution networks can vary significantly over a 24 hour period with the highest head loss and consequently the lowest system pressures occurring at the peak flow. In general, the highest flow within a network is $2 - 3$ times the order of the mean [12].

Another important observation is that if the houses under concern are very close to

[1] The second term of the right part of equation has been multiplied by $1 = \frac{10}{10}$ so that the discharges can be equated to physical radii of pipes. For example, $0.05m^2 = 50mm$ radius.

the tank in question, then the head loss, H_L, effects can be ignored, while if the houses are some distance away, requiring long pipes for water supply, then the head loss effects cannot be ignored. In the case of the later, the required h_{elv} would be greater. It is worth mentioning that as the population size next to the tank increases the corresponding change in elevation height h_{elv} is not very significant (Figure 3.3). As can be inferred from the model, an adjustment in h_{elv} of $3cm$ is able to be sufficient for almost the entire population. This, in practice, is hardly an adjustment, meaning that the h_{elv} value for each tank is indifferent of how many households are near the tank, holding all other parameters constant. Another practical thing to also note is that it is very unlikely that the whole population being served by a tank in a mountainous area like Zomba can be at one level. Some would definitely be at lower elevation levels and eventually are not affected by the discussed problem. In both cases, h_{elv} plays a crucial role in deciding how far away from tanks should houses be built, or how high should the tank be elevated.

It can easily be seen by rewriting Equation 3.9 as

$$\kappa = \alpha + \beta \times (\%\text{ge of population})^2 \tag{3.11}$$

where

$$\begin{aligned}
\alpha &= \frac{150,000Pa - 101,325Pa}{1000kg/m^3 \times 9.8m/s^2} - \frac{(0.06m^3/s)^2}{2 \times 9.8m/s^2 \times \pi \times (0.05m)^2} + (H_L - 0.3m) \\
\beta &= \frac{(0.06m^3/s)^2}{2 \times 9.8m/s^2 \times \pi \times (0.05m)^2}.
\end{aligned}$$

This is a quadratic equation as is seen in Figure 3.3. Cases of population increase and per capita demand increase are taken care of in terms of pressure by the same h_{elv}. However, the nominal volumes of consumption depend on production capacity at treatment plant. It is very important that the capacity of treatment plant be increased with time to cater for the population increase and per capita demand increase.

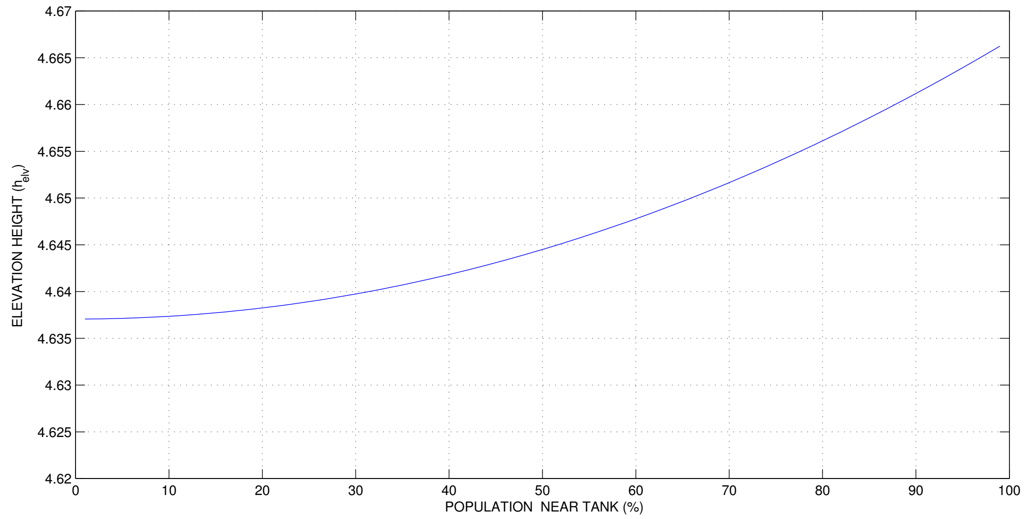

Figure 3.3: Adjustment of h_{elv} as population varies: A representation of $0.006m^3/s$ flow from the T_{20} tank

3.4 Discussion

Zomba's water distribution network is almost entirely supported by gravity. As a mountainous city, this is the most desired design. With a growing population, problems such with cascading effects start to take a hold. In ideal situations, tanks that are fully supported by gravity must be placed in elevated positions above the households they supply water to. Tanks can be raised to the desired elevation from the ground, or a water tower can be installed next to the tank to compensate for this elevation. This is typical in cases where the tanks are huge and would pose serious construction challenges if elevated from the ground. In that case external power is employed to pump water from the tank to the tower.

This elevation can also be observed in another way: in a typical mountainous area like Zomba, tanks can be placed on ground level as long as they start supplying water at a lower elevation than the one they are placed at. In that way, such structure of supplying from a lower elevation than tanks compensates for the tank's low elevation. This does not require any external power, but requires control of how close houses can be built next to the tanks. If the houses are within the range of this equivalent elevation height h_{elv}, then they are prone to the cascading effect, otherwise, they are safe.

It is important to note that this h_{elv} is a vertical elevation. If the mountain's slope is slanted at some elevation angle, say θ, from the ground, then the actual distance, d away from the tanks will be

$$d = \frac{h_{elv}}{\sin\theta}. \tag{3.12}$$

As observed on the T_{20}-Old Naisi tank, almost all the affected houses are at a distance of more than $4.64m$ away from the tank but their vertical displacement from the tank is less than the said height.

A more practical solution for Zomba City's network would be to lift the tanks above ground. Except a few recently built ones, most of the tanks, including five of the affected seven can be dismantled and reassembled. This will ensure no external energy is used to supply the extra increasing demand. The other alternatives are more costly both in the short run and long run because of use of external energy.

3.5 Conclusions

This Chapter has looked into a prevailing problem within the Zomba city network, and perhaps, one of the most common problems in any network that almost entirely relies on gravity, called the cascading effect. Specifically, the Chapter has sought to give a scientific explanation of the cause of the problem, and a possible remedy. As a mountainous city, Zomba's network is almost entirely powered by gravity. This implies that the pressure delivered to the end users, the households, is derived from atmospheric pressure and the water height in a tank. Because of this, there is a need to consider placing the tank at some elevation h_{elv} away from the houses. This elevation can be a physical lifting of the tank above the ground, or the installing of a water tower for the tank, or keeping it at a distance away from the first houses being supplied. The last case is common for mountainous places with tanks sitting on the ground.

If this elevation height h_{elv} is not observed, we end up in a situation where the natural atmospheric pressure as well as the pressure from the height of water in the tank won't be sufficient to deliver the water to some households in the network, i.e. we encounter the cascading effect. Usually houses that are within the perimeter of the elevation height h_{elv} are affected. This is consistent with the situation in Zomba, where houses recently built near tanks are the ones affected by this occurrence.

When designing city expansions, especially when opening new residential areas, municipalities must consider the implementation of h_{elv}. This can be achieved through observing the correct distance away from the tanks when building houses, erecting a water tower for each tank or physically lifting the tanks from the ground. The only challenge with water towers is that they require an external power to pump the water into them, which becomes an added operational cost for the supply companies. It therefore leaves the option of elevating tanks from ground to be the ideal options for fast growing and areas like Zomba. To some extent, use of tanks that can be dismantled and reassembled is also another option. This however has a limitation of containing pressure when the tanks are large.

Chapter 4

Saving Pipes from Flow Induced Failures: An Analysis of the Hammer, Circumferential and Longitudinal Stresses Effects

4.1 Introduction

Zomba city's water distribution network currently faces a number of operational challenges. Such challenges affect the efficiency of the network that, according to its design, is supposed to have been sufficient for the city's population. Two of the problems have already been discussed in detail in the two preceding Chapters. A mathematical model for the city's water demand was developed in Chapter 3, and the issue of tank elevation effects on households' access to water supply was discussed in Chapter 4. This Chapter discusses a third problem, similar to the one discussed in the previous Chapter [85], which is the occurrence of pipe failures in the network.

As a way of controlling pressure, the Zomba Distribution Network uses a network of tanks connected in series that act as pressure breaks. Some tanks (called receiver tanks) receive the water from other tanks (called feeder tanks) situated at higher elevations than the former. In turn, they supply the water to other tanks, communities or both. In the case of supplying water to other tanks, the receiving tanks are all situated at a lower elevation than the feeding tank. However, the receiving tanks are situated at different elevations relative to each other with some being at relatively lower elevations than others.

It is observed in the network that water will freely flow from a feeder tank to a receiver tank that is situated at a relatively lower elevation than other receiver tanks. This is governed by the rule of potential energy (Bernoulli's equation) where water will convert its potential energy into kinetic energy and move from a higher to a lower area. If two places are at different elevation, water will move to the lowest point first. Thus the receiving tank at a lower elevation continuously receives water until either it's full or the flow control valve (FCV) had been tightened to reduce flow to the tank at lower elevation (Figure 4.1). Mathematically speaking, tightening the FCV_2 reduces the flow to Q_2 to T_2 and thus increases flow Q_3 to T_3. In the absence of height measuring instruments in tanks, as is the case with over 50% of Zomba tanks, the process of closing FCV_2 starts when overflow at T_2 is noticed. An FCV fitted next to the tank is manually closed.

With time, this constant and repetitive manual closing and opening of the FCVs causes the connecting mains pipes on which the FCVs are fitted to fail. The volume of water lost when a failure involving mains pipes occurs is very significant.

An alternative way of using FCV is to close it up to a certain point where water is able to flow to both tanks T_2 and T_3. With exception of a number of tanks that are supplying a relatively fewer population because they were planned to be feeder tanks for future tanks under planning, the need to close the FCVs still remains. It has to be noted that it is not the overflowing that causes the observed pipe failures, but rather the closing and opening of FCVs.

The aim of this Chapter is to provide an explanation for the phenomenon described above, as well as suggesting possible solutions to the problem. Specifically, we aim to explain the connection amongst:

- pressure in the pipes,

- the observed unidirectional flow of water towards receiving tanks at lower elevations only, and
- the closing and opening of FCVs and the pipe failures observed.

As this problem is a real life problem happening in an existing network, a thorough understanding of the problem and implementation of its proposed solutions will open up avenues towards saving the water lost through tanks overflowing and mains pipe failures. The immediate result is that more water will be available for consumption in the Zomba network with the same existing infrastructure.

The rest of the Chapter is structured as follows: Section 4.2 presents a situational analysis of the problem considering the flows and the hammering effects induced in the pipes. The hammering effects form the basis for the failure. Section 4.3 presents results of a numerical simulation using the parameter values for network pipes and valves used in the Zomba network. Section 4.4 discusses the results in Section 4.3 and the theory in Section 4.2, and it shows that there is a connection between hammering effects and pipe failures in the network. Section 4.5 summarizes and draws conclusions from the work conducted.

4.2 Unaccounted for Water

However complex they may be, Urban Water Networks, obey the basic governing equation:

$$\textit{Water Produced} = \textit{Water Consumed} + \textit{Unaccounted-For Water}$$

and the main aim of each one of them is to reduce the Unaccounted-For Water (UFW) to acceptable levels as defined by individual sypply systems. To do this, supply companies must map out their consumption with accuracy and precision as demanded by their efficiency measures. The idea is simple: *produce however much water is required!* Unfortunately though unaccounted-for water can never be completely disregarded because of occurrences such as pipe failures, leakages, illegal water connections and tank overflows.

Such situations make it more complicated to manage urban water distribution networks as they begin to exhibit problems. Thus, beginning as localized in each network, network problems eventually adopted some commonalities and became problems on a global scale. Notable are the pump scheduling [7, 62–64], [65], leakage management [66–70], as well as demand forecasting issues [40, 43, 45]. Today, extensive literature is available with generalized models that can be applied to any network (sometimes with minor modifications), regardless of the complexity of the networks. On the other hand, individual networks continue to unearth localized problems that may with time become applicable to other networks with similar characteristics.

According to [115] through [2], UFW is categorized in two groups: The real (physical) UFW and the apparent UFW. Real UFW is comprises all water lost through pipe bursts, tank overflows, pipe leakages, tank leakages, as well as valve leakages. Apparent UFW, on the other hand, is all water losses due to unauthorized use (e.g. illegal connections) as well as inaccurate meter readings amongst master, industrial, commercial and domestic meters.

While it may be possible to control some of the UFW, it must be accepted that some UFW cannot be avoided. Thus, it is impossible to operate at 100% efficiency ([111]). For example, apparent UFW can be controlled by encouraging practices that would discourage illegal connections (like hefty penalties) while meter reading errors will always come with the meters. There is no meter that is 100% accurate.

Likewise, some of real UFW can be controlled, for example, using pressure management. By reducing pressure with 20%, [112, 113] showed that a reduction in 30% in UFW through leakage was observed.

This research follows the path of controlling real UFW in the city of Zomba. Four main sub problems were defined. The first one was to map the demand patterns of the city according to their influencing factors. Hammer and flow challenges are observed in the network, and they are discussed in the context of the network. Finally, as the network keeps expanding to match the growing population and urbanization, designing of network components that make optimal use of gravity becomes a priority. The research took a look at tank designing.

4.3 Problem Analysis

The flow problem described in the previous Section is simplified and explained by the following simulation (Figure 4.1). In this simulation, tank T_1 is a feeder tank which supplies water to tanks T_2 and T_3, besides its own demand. Our focus shall not be on T_1's demand, but rather its feeding of water into tanks T_2 and T_3. Tank T_1 is connected by pipe P_1 to node N_1 from which two pipes P_2 and P_3 connect tanks T_2 and T_3 respectively. The mains pipes under consideration are all of the same diameter. the diameters change for pipes supplying water from end tanks to houses. We will further assume that tank T_2 is at a lower elevation than tank T_3, and that the three tanks are at elevations z_1, z_2 and z_3 respectively.

What is observed in the Zomba Network is that water from tank T_1 will freely flow into tank T_2 and not into tank T_3. This is obviously all to the fact that water will freely move to the area with lowest potential energy. Because there are no automatic systems to determine the levels of water in tanks, when tank T_2 begins to overflow, an FCV at the inlet pipe into tank T_2 is manually closed by a Water Board staff. The closure allows water to begin to flow into tank T_3. Should the need for water arise again in tank T_2, or should an overflow be observed at T_3, the FCV is manually opened to let the water flow again into T_2. Sadly, the process of opening and closing the FCV is associated with pipe failures mostly experienced in the inlet pipe P_2 fitted with the FCV.

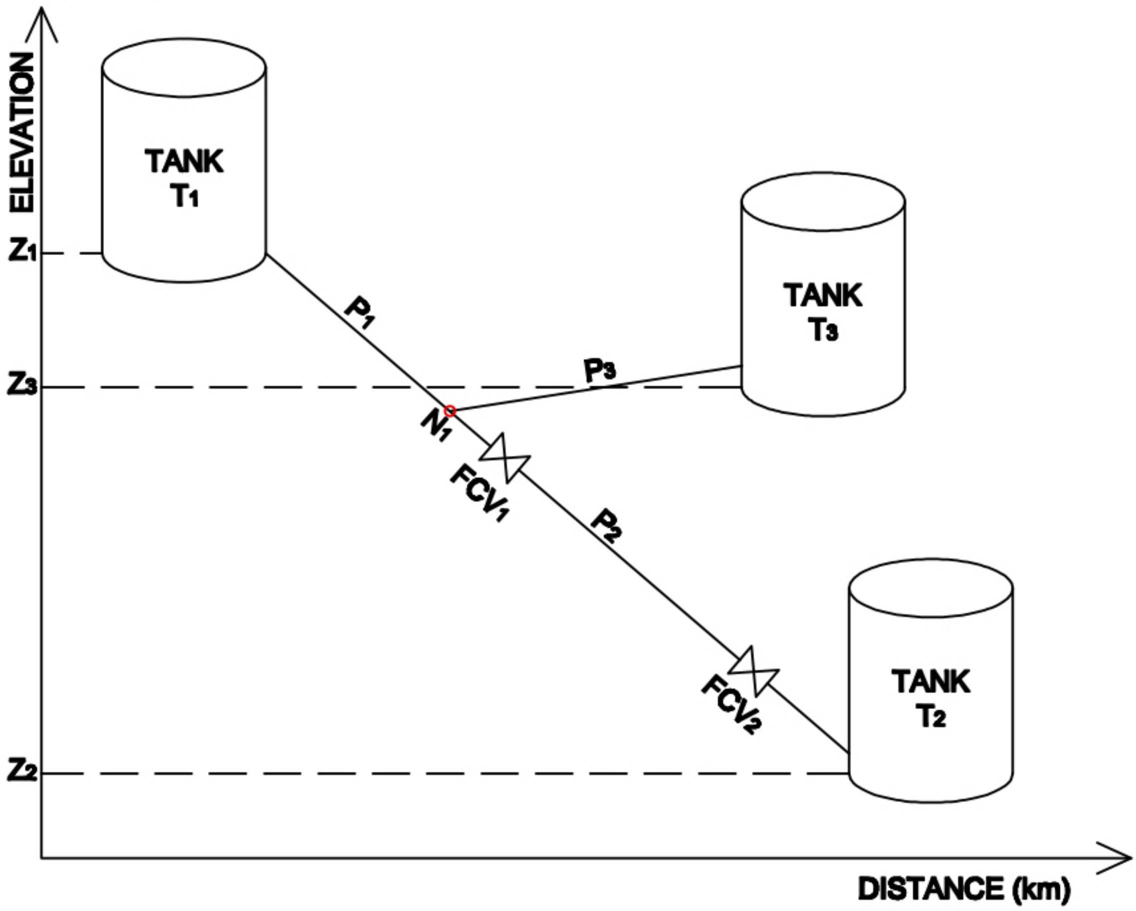

Figure 4.1: A Sketch for Water Delivery to Tanks at Different Altitudes

It needs to be mentioned here that, with more than 50% of the network inherited from old supply system ([2]), there are very few automated equipment in the network to determine flows and water height in tanks. As a result, manual observation of tanks is often employed.

4.3.1 Flow Analysis

In an ideal situation, where all desired water reaches its destination in tanks T_2 and T_3, we let the discharges in the respective pipes P_1, P_2 and P_3 be Q_1, Q_2 and Q_3. These discharges are related by the equation

$$Q_1 = Q_2 + Q_3 \tag{4.1}$$

or alternatively

$$A_1 v_1 = A_2 v_2 + A_3 v_3 \tag{4.2}$$

where A_i, v_i are the respective cross-sectional area and velocity for pipes P_i, $i = 1, 2, 3$.

According to Bernoulli's Equation, discussed in [85], the pressure required at tank T_1 to deliver water discharge Q_3 to tank T_3 at pressure, say, p_3 is given by

$$p_1 = p_3 + \frac{1}{2}\rho(v_3^2 - v_1^2) + \gamma(z_3 - z_1) + \gamma H_L \tag{4.3}$$

where

- $\gamma = \rho g$ is the specific weight of the water,
- ρ, g are the density of water and gravitational constant respectively, and
- H_L is the sum of both major and minor head losses along the pipe.

For a network that is completely dependent on the gravity force, the required pressure p_1 is entirely derived from the pressure from the atmosphere and pressure from the height of the water in the tank, i.e.

$$p_1 = p_{atm} + \gamma h_w \tag{4.4}$$

where

- p_{atm} is the atmospheric pressure, and
- h_w is the height of the water in the tank combined with the elevation in case the tank was raised above the ground.

If it happens that the atmospheric pressure combined with the pressure from the height of the water in the tank is insufficient to produce the required pressure p_1, i.e.

$$p_1 > p_{atm} + \gamma h_w, \tag{4.5}$$

then the water flow cannot reach tank T_3 unless either the FCV is manually closed or some external energy is added to increase the pressure at tank T_1.

This is typical of the Zomba network, and hence, explains why water will always flow to tank T_2 unless the FCV is closed manually. The consequences of closing the FCV will be considered in the subsequent Subsection.

Tanks such as T_{14}, T_{15}, T_{27} and T_{29} are far from the treatment plant and are large. They were built with the intention of using them as feeder tanks for future expansions that had not materialized as of the time the research was conducted. As a result, when they are full their capacities are enough to supply the benefiting population for a lengthy period of time without refilling. Thus, they have to be closed when full and opened when going towards empty since and this process sometimes takes a week to repeat. The mains pipes supplying these tanks are often vulnerable to failures.

4.3.2 Flow Control Valves and the Water Hammering Effect

In the absence of automated flow regulators fitted to tanks or pipes, an indicator which prompts the Water Board staff to close the FCV is when the tank T_2 starts to overflow. There is very little, if any, leeway when it comes to the time allowed for the water overflow to continue. Normally, the FCVs are shut immediately to prevent any excess loss of treated water from the tank. In the process of saving the overflowing water through closing the FCV, another problem is created - *the water hammering problem.* A strong shock wave of pressure propagates backward through the closed pipe until it reaches the node N_1. The discharge now moves in the direction of tank T_3.

Just to have an appreciation of the kind of orders of pressure magnitudes we'd be talking about, consider the following situation. Pressure p_2 at tank T_2 will be

$$p_2 = p_1 + \gamma(z_1 - z_2) - \gamma H_L. \tag{4.6}$$

since $A_1 v_1 = A_2 v_2$ when all the water is flowing to tank T_2 and all pipes have the same diameter. According to [12], the change in pressure due to the hammering effect is a function of the length of the pipe, the time taken to close the valve, as well as the water properties such as density and the bulk modulus. If L is the length of the pipe, t_c the closing time (with $t_c = \frac{2L}{C}$ the critical closing time; $t_c < \frac{2L}{C}$ means the closure is instantaneous[1]; $t_c > \frac{2L}{C}$ means the closure is gradual[2]) [12,105,106], E_b the bulk modulus of water, and $C = \sqrt{\frac{E_b}{\rho}}$ the speed of the pressure wave, then the pressure build up in the pipe P_2 would either be

$$\Delta p = \frac{\rho L v_2}{t_c} \tag{4.7}$$

when gradual closures are experienced, i.e. $t_c > \frac{2L}{C}$, or

$$\Delta p = \rho C v_2 \tag{4.8}$$

in the case of sudden closures where $t_c \leq \frac{2L}{C}$.

Thus, the total pressure in pipe P_2 will be the sum

$$p_{tot} = p_2 + \Delta p. \tag{4.9}$$

It will equally also be important to look at pressures in the pipes in terms of the amount of stress subjected to the pipes, circumferential stress f_c and longitudinal stress f_L . The circumferential stress f_c for the pipe, according to [12, 105, 106], will be given by the equation

$$f_c = \frac{(p_2 + \Delta p)D}{2\tau} \tag{4.10}$$

where

- D is the inner diameter of the pipe, and
- τ is the wall thickness of the pipe.

[1]Instantaneous valve closure is defined to occur if the valve is closed faster than the wave travel time. The wave travel time is $w_c \leq \frac{2L}{C}$

[2]Gradual valve closure is defined to occur if the valve is closed slower than the wave travel time.

The longitudinal stress f_L for the pipe is

$$f_L = \frac{1}{2} f_c. \tag{4.11}$$

The next Section simulates typical averaged values for the Zomba Distribution Network. The values of the hammering pressures and the two stresses are calculated and plotted.

4.4 Numerical Simulations and Results

We use typical values of the Zomba distribution network, whose tanks are located at altitudes of between $838m$ and $1104m$ above sea level. The feeder-to-receiver tank altitude difference has an average of $74m$ with connecting pipes ranging from $1000m$ to $5,000m$. Inner diameters of the pipes come in three varieties: $150mm$, $200mm$ and $250mm$. The pipe thickness ranges from $10mm$ to $20mm$ depending on the standard of the pipe in use.

Three length measurements of $1,000m$, $3,000m$ and $5,000m$ were used in the simulations. Inner diameters of $150mm$, $200mm$ and $250mm$ were used, and pipe thicknesses of $10mm$, $15mm$ and $20mm$ were used. Closing times of $1s$, $5s$, and $10s$ were used.

Thus, the following measurements were obtained and used for simulations: $L = 1000, 3000$, 5000, $D = 0.15, 0.2, 0.25$, $\tau = 0.01, 0.015$, 0.02. A velocity value of $9m^3/s$, which is typical for Zomba's mains pipes, was also used. Equations 4.9, 4.10 and 4.11 were used to compute the stresses with a combination of the stated measurements. A length-diameter-thickness and time combination of eighty-one simulations were performed for each of the stresses.

The calculations were done in accordance to [14] where the units of feet, feet-per-second were used for length, thickness, diameters and velocity. All such units were converted accordingly, and the converted values are the ones used in the Tables below (4.1, 4.2, 4.3).

The parameters of interest were total hammer pressure (p_{tot}), circumferential stress (f_c) as well as longitudinal stress (f_L, which is inferred from f_c) for pipes. A further assumption of hedging (or cancelling out) the head loss with the head from the height of the water in the tank was considered in modelling so that we can concentrate on the atmospheric pressures. Thus, equation 4.3 can be rewritten as

$$p_{atm} + \gamma h_w = p_3 + \frac{1}{2}\rho(v_3^2 - v_1^2) + \gamma(z_3 - z_1) + \gamma H_L \tag{4.12}$$

where a simple rearranging of terms results into

$$p_{atm} = p_3 + \frac{1}{2}\rho(v_3^2 - v_1^2) + \gamma(z_3 - z_1) + \gamma\left(H_L - h_w\right). \tag{4.13}$$

The hedging assumption implies $H_L - h_w = 0$ and the equation reduces to

$$p_{atm} = p_3 + \frac{1}{2}\rho(v_3^2 - v_1^2) + \gamma(z_3 - z_1). \tag{4.14}$$

It was observed (Tables 4.1, 4.2, 4.3) that water hammering pressures were in the magnitude of 2 to 5 times that of the normal flows in the pipes. Of interest, were the circumferential stresses as well as longitudinal stresses. With all the possible combinations of pipe parameters used in the Zomba Network as classified according to diameter, thickness, closing time, and length, the circumferential stress and longitudinal stress assumed values of orders of up to 12 times the hammer stresses, or 60 times pressures under normal operating pressures. In an ageing network like that of present Zomba city, these pressures may be sufficient to be the main cause of pipe failures (Table 4.1, 4.2, 4.3). The stresses values are highest with the longest pipe values used in the network, 5000m.

Also of interest is when the closing time is less than the critical time t_c. For example, if the FCV is a flap-valve and can be closed in $1s$ which is less than $t_c = 1.3s$ then the stress values surge to above 50 normal flow pressures.

Table 4.1: Hammer, Circumferential and Longitudinal Stresses for Pipe Length = 3280.84ft (1000m)

Length (ft)	Time (s)	Initial P (Psi)	Hammer P (Psi)	Diameter, D (ft)	Thickness, τ (ft)	Ratio $\frac{D}{2\tau}$	f_c, (Psi)	f_l, (Psi)
3280.84	1	87	168.13	0.49	0.033	7.4242	1248.2	624.12
					0.049	5	840.65	420.325
					0.066	3.721	624.12	312.058
				0.66	0.033	10	1681.3	840.65
					0.049	6.7347	1132.3	566.153
					0.066	5	840.65	420.325
				0.82	0.033	12.4242	2088.9	1044.4
					0.049	8.3673	1406.8	703.4
					0.066	6.2121	1044.4	544.2
	5	87	103.24	0.49	0.033	7.4242	766.5	383.24
					0.049	5	516.2	258.1
					0.066	3.721	384.2	192.08
				0.66	0.033	10	1032.4	516.2
					0.049	6.7347	695.3	347.65
					0.066	5	516.2	258.1
				0.82	0.033	12.4242	1282.7	641.34
					0.049	8.3673	863.8	431.92
					0.066	6.2121	641.3	320.669
	10	87	95.13	0.49	0.033	7.4242	706.3	353.13
					0.049	5	475.6	237.83
					0.066	3.721	354.0	176.99
				0.66	0.033	10	951.3	475.65
					0.049	6.7347	640.7	320.34
					0.066	5	475.6	237.83
				0.82	0.033	12.4242	1181.9	590.96
					0.049	8.3673	796.0	397.99
					0.066	6.2121	591.0	295.48

Table 4.2: Hammer, Circumferential and Longitudinal Stresses for Pipe Length = 9842.52ft (3000m)

Length (ft)	Time (s)	Initial P (Psi)	Hammer P (Psi)	Diameter, D (ft)	Thickness, τ (ft)	Ratio $\frac{D}{2\tau}$	f_c, (Psi)	f_l, (Psi)
9842.52	1	87	330.33	0.49	0.033	7.4242	2452.4	1226.2
					0.049	5	1651.6	825.8
					0.066	3.721	1229.2	614.6
				0.66	0.033	10	3303.3	1651.6
					0.049	6.7347	2224.7	1112.3
					0.066	5	1651.6	825.8
				0.82	0.033	12.4242	4104.1	2052.0
					0.049	8.3673	2764.0	1382.0
					0.066	6.2121	2052.0	1026.0
	5	87	135.68	0.49	0.033	7.4242	1007.3	503.7
					0.049	5	678.4	339.2
					0.066	3.721	504.9	252.4
				0.66	0.033	10	1356.8	678.4
					0.049	6.7347	913.8	456.9
					0.066	5	678.4	339.25
				0.82	0.033	12.4242	1685.7	842.9
					0.049	8.3673	1135.3	567.6
					0.066	6.2121	842.9	421.4
	10	87	111.35	0.49	0.033	7.4242	826.7	413.3
					0.049	5	556.7	278.4
					0.066	3.721	414.3	207.2
				0.66	0.033	10	1113.5	556.7
					0.049	6.7347	749.9	375.0
					0.066	5	556.7	278.4
				0.82	0.033	12.4242	1383.4	691.7
					0.049	8.3673	931.7	465.8
					0.066	6.2121	691.7	345.9

Table 4.3: Hammer, Circumferential and Longitudinal Stresses for Pipe Length = 16404.2ft (5000m)

Length (ft)	Time (s)	Initial P (Psi)	Hammer P (Psi)	Diameter, D (ft)	Thickness, τ (ft)	Ratio $\frac{D}{2\tau}$	f_c, (Psi)	f_l, (Psi)
16404.2	1	87	492.54	0.49	0.033	7.4242	3656.7	1828.4
					0.049	5	2462.7	1231.4
					0.066	3.721	1832.7	916.37
				0.66	0.033	10	4925.4	2462.7
					0.049	6.7347	3317.1	1658.6
					0.066	5	2462.7	1231.4
				0.82	0.033	12.4242	6119.4	3059.7
					0.049	8.3673	4121.2	2060.6
					0.066	6.2121	3059.7	1529.9
	5	87	168.13	0.49	0.033	7.4242	1248.2	624.12
					0.049	5	840.65	420.32
					0.066	3.721	625.61	312.81
				0.66	0.033	10	1681.3	840.65
					0.049	6.7347	1132.3	566.15
					0.066	5	840.65	420.32
				0.82	0.033	12.4242	2088.9	1044.4
					0.049	8.3673	1406.8	703.4
					0.066	6.2121	1044.4	522.22
	10	87	127.57	0.49	0.033	7.4242	947.11	473.55
					0.049	5	637.85	318.92
					0.066	3.721	474.69	237.34
				0.66	0.033	10	1275.7	637.85
					0.049	6.7347	1859.15	429.57
					0.066	5	637.85	318.92
				0.82	0.033	12.4242	1585.0	792.48
					0.049	8.3673	1067.4	533.71
					0.066	6.2121	792.48	396.24

Besides allowing for longer closure times of FCVs, another possible remedy to this problem would be to rather fit the FCVs next to pipe junctions (as is shown in Figure 4.1, Node N_1) than next to tanks. To explain this scenario, let us again consider equation (4.7), i.e.

$$\Delta p = \frac{\rho v L}{t_c}. \tag{4.15}$$

Fitting the FCV at the node is equivalent to taking the limit of the length to approach zero ($\lim_{L\to 0}$). The equation becomes

$$\begin{aligned} \lim_{L\to 0} \Delta p &= \frac{\rho v}{t_c} \lim_{L\to 0} L && (4.16)\\ &= 0. && (4.17) \end{aligned}$$

What this result means is that if the FCV is fitted next to the node, and the FCV closures are gradual, then the hammering effects are greatly reduced. When the FCV is closed, it is equivalent to creating an elbow at the node N_1 that allows water to flow from tank T_1 to tank T_3. Usually closure times are bigger than the critical times by design for most FCVs in use, including those of Zomba. Thus, in situations like the one currently being discussed, the eventual associated pressure effects are the head losses due to the elbow bend. Such effects are much smaller in magnitude than compared to the hammering effect. When the FCV closures are instant, then the ripple effects are propagated along the other pipes. This necessitates the emphasis of gradual closing in networks.

4.5 Discussion

As can be seen from the analysis and on the ground, atmospheric pressure, fluid pressure and gravity alone may not be the only needed driving force to distribute water to all tanks in Zomba city. It is evident that some feeder tanks are unable to deliver water to some end tanks at a relatively higher elevation than others by gravity alone. This inadequacy best explains the phenomenon observed in the distribution whereby some tanks do not receive water while others constantly do until they start to overflow. The idea of manually closing the FCVs may be feasible in a network like Zomba since they do not have automated equipment yet on the whole network[3], albeit having some serious consequences on the network itself. Much as it may not be desirable to let water overflow in tanks for some time when observed, there is a need to strike a balance on the closing time. As can be seen from the simulations, large circumferential as well as longitudinal pressures - as much as 60 times those of the normal flow pressures - can be experienced in the network. Depending on the pressure capacity of the pipes, age of the pipes and other contributing factors , pipe failures would be inevitable in the network as is constantly being observed.

It is an obvious fact that FCVs are placed next to tanks for security reasons. The same security system that looks after tanks would take care of the valves as well. However, looking at the surging costs for repairing networks as a result of such high pressures created in the process, it might be a good idea to look into the suggestion of fitting them at nodes with some consideration. If implemented, all vulnerable pipes (inlet pipes into tanks and on which FCVs are fitted) become safe from hammering pressures.

It is also worthy noting that when FCV is closed at tank T_2, the hammer wave propagating backwards gets distributed into pipes P_1 and P_3 after reaching the node N_1 [[12]]. As such, the hammer effects are of smaller magnitudes in the pipes P_1 and P_3 as compared to P_2. This is because P_2 carries the source hammer effects that get spread to the other two pipes. Essentially the sum of pressures due to hammer effects in pipes P_1 and P_3 may not exceed those of pipe P_2. However, the fact that closures are gradual does not help with the tank overflow problems. An ideal option would be to install intelligent flow regulators that are equipped with modern technology, and are programmable, so that they may adjust the flow automatically. Such intelligent regulators will monitor the amount of water required at any given time and thus completely eliminate the possibility

[3]an effort towards installing the electronic system on the network is in place. In April 2016, an advert appeared in local newspapers Daily Times and Nation requesting bids for the supply, installation and commissioning of a SCADA system on the network

of requiring the manual closure of FCVs.

On the basic principle of automatic control of flow to T_2, the regulators will be able reduce Q_2 and thus reduce $Q_1 - Q_2$ which, in turn, increases the flow Q_3 to T_3. They could also prove vital to minimizing unaccounted-for water (UFW) by eliminating tank overflows. The only challenge with such regulators is the need for a substantial financial investment. They require an external source of power (usually electricity) and the training of manpower for usage and maintenance.

It is also worth mentioning that the amount of water contributing to UFW when mains pipes fail is relatively higher, compared to other pipes, per unit time since the mains pipes transport vast amounts of water at any given time when compared to all other parts of the network. For example, one mains pipe connects to a tank that may supply, on average, 100 households. This means that, at any given time, a failure in a mains pipe is equivalent to losing water of the order of equivalent to supply for 100 households. On the other hand, if the pipe to one house bursts, the loss is equivalent to one-hundredth of the capacity carried by one mains pipe. Policing this situation means more UFW is arrested and channelled back to consumers, and the 85% targeted efficiency can more easily be reached within a shorter period of time. This could mean that more water would be available to Zomba city's population using the existing infrastructure.

4.6 Conclusions

This Chapter has presented a mathematical explanation and possible solutions to a prevailing problem in the Zomba city's water distribution network. Water from a feeder tank flows to receiver tanks at relatively lower elevations to others. When the tanks receiving water start to overflow, flow control valves (FCVs) are manually closed to allow water to flow to the dry tanks.

A simple simulation model has been developed to understand the situation. The model reveals that there is not enough pressures at feeder tanks to deliver water to all receiving tanks because of elevation differences unless the FCVs are manually closed. However, when the FCVs are manually closed, water hammering effects are activated which, according to the actual pipe and elevation dimensions governing the Zomba network, can see pressures in pipes rise to more than ten times the normal levels in the flow. Circumferential stresses of 60 times normal flow pressures are observed. Such levels of pressures are enough to burst pipes in the network. Due to the nature of the pipes involved, i.e. mains pipes, a failure in one contributes to a large proportion of UFW to the network. Thus, stopping such failures may mean a significant drop in UFW levels and thus increase the water being supplied without expanding the current infrastructure.

As a way of avoiding such pressures in the network, putting FCVs next to joint nodes as opposed to tanks proves a viable option. In the event of the FCV closing (gradually or suddenly), the distance to the node is almost zero making the hammering effects negligible. Instead, only minor head losses due to a bend will be experienced which are far lower than the levels experienced due to the pressures observed during hammering effects.

The use of automated FCVs could prove more efficient. With these, only the required amount of water at any time would be allowed to pass through. This eliminates the requirement of manual closing of FCVs. This last option requires some financial investment and may be considered as a long term plan for the network.

Finally, it is worthy reporting that a post research presentation to Southern Region Water Board saw the Board accept the mistake of closing the FCVs at tanks than closing FCVs at nodes. To this effect, the practice had been changed. Correct FCVs at nodes are now the ones being closed unlike before. After some time, it was reported (informally though) that pipes that were giving them severe failing problems had now started to 'behave'.

Chapter 5

Optimizing the Use of Natural Gravitation Force in Tank Designs

5.1 Introduction

The supply of water is a complex process. Beginning from the water source (which can be a river, lake or dam), water is ferried to the treatment plants where it is treated first before it can be considered ready to be consumed. Once water has been treated, it is hauled in mains pipes to storage tanks which normally supply households through gravity. These tanks can also supply other tanks with water - a situation that defines them to be feeder tanks while the receiving tanks are called end tanks.

Storage tanks play a crucial and important role in the management of demand [61,87,88], continued supply of water during pipe failures until they have run empty, provision of supply for emergency reserves e.g. fire fighting, as well as assisting the continuous water supply during pump scheduling [89–92]. In a typical tank situation, water is allowed to flow into storage tanks until some level, h_{max} is attained. The flow is stopped and the water level starts to decrease because of consumption until a minimum level h_{min} is attained. At this point, the water flow into the tank is restarted until h_{max} is reached again [12, 63, 65, 93].

The time difference between stopping and restarting the flow into the storage tank while holding the number of stoppages constant is very critical in terms of operation costs (e.g.

energy in pumped systems, or electronically regulated systems to open or shut valves). A longer time lapse between stopping and restarting the flow into the storage tank is most desired as that would mean relatively less energy is being used. On the other hand, a short time span between stopping and restarting would mean more energy is required [94]. The former is associated with storage tanks that require more time for their water height to drop from h_{max} to h_{min}. The latter takes less time.

A consequence of quick drops and rises in water levels between h_{max} and h_{min} is the fluctuations in pressure at the tank. A typical tank that supplies its demand through gravity depends on three sources of energy: atmospheric pressure; potential energy from tank elevation as well as pressure from the column of water in the tank [12, 85, 95]. The first two are usually constant, implying that a change in the latter affects pressures in the pipes.

When designing water distribution systems, not much consideration was given to the role of tanks in ensuring an efficient system until recently [90, 96, 97]. Some authors, for example, [98–103] and [104], do take the approach of modelling hydraulic aspects and of late the structural approach of physical designing of materials is has attracted a significant consideration [90]. The structural role of a tank cannot be separated from the efficiency of the network system. In spite of coming in different shapes and sizes, there must be some desired structural properties that each one of the storage tanks must possess.

This Chapter follows the path of [90] and discusses one of the most crucial structural features required for a storage tank to contribute towards the efficiency of the network while making best use of gravity in the context of Zomba Distribution Network for optimal use of gravity. Since Zomba is mountainous, it is in the interest of this research direction to make use of gravity and minimize the use of external energy when transporting water. Specifically two dimensional properties of a tank are studied - the height and radius of the tank. We try to answer the question: how should the two be related so as to produce a tank that would complement an efficient gravity reliant water distribution network system for the city of Zomba?

The rest of this Chapter is structured as follows: Section 5.2 discusses the relationships between the pressure and water height in the tank as well as the water height in the tank and the tank radius. A relationship between the rate of change of the height of the water column in the tanks to the ratio of height to radius of the tank is proposed. Section 5.3 presents the simulations and results of the proposed relationship. Section 5.4 discusses the results in light of identifying the desired height - radius values necessary for optimizing the use of gravity, while Section 5.5 provides concluding remarks.

5.2 Tank Design for Optimizing Gravitational Forces

To find out the relationship between rate of change of pressure $\frac{dp_1}{dt}$ and the tank properties, let us suppose we have a tank, say T_1, that is supplying water to either demand D (or another tank, say, T_2). Assume further that no external power is used to deliver this load to the demand (or tank T_2) and that, for the purposes of simplicity, there is no inflow into T_1 in the meantime. A minimum pressure, p_1 is required at tank T_1 to successfully manage the delivery of water to the demand D (or tank T_2) [85, 95]. In the absence of pumping, the pressure p_1 is derived from atmospheric pressure and the height of the water in tank T_1. We therefore have the relationship

$$p_1 = p_{atm} + \gamma \left(h_{elv} + h_w \right) \tag{5.1}$$

where

- p_{atm} is the atmospheric pressure,
- $\gamma = \rho g$ is is the specific weight of water,
- ρ and g are the density of the water and the gravitational constant respectively,

- h_{elv} is the tank elevation height from the ground, and
- h_w is the water height in the tank.

In a typical tank setup, p_{atm} and h_{elv} are constant. Thus, the change in pressure p_1 is necessarily a result of the change in the water height h_w. A linear relationship between the pressure p_1 and water height h_w can be represented with the equation

$$p_1(t) = \frac{1}{\gamma}h_w(t) + \beta \tag{5.2}$$

where $\beta = p_{atm} + \gamma h_{elv}$ is a constant.

Such a representation makes it easier to see that

$$\frac{dp_1}{dt} = \frac{1}{\gamma}\frac{dh_w}{dt}. \tag{5.3}$$

On the other hand, a change in the volume V of the water in a typical cylindrical tank is given by

$$\begin{aligned} \frac{dV}{dt} &= \frac{d}{dt}\left(\pi r^2 h_w\right) \\ &= 2\pi r h_w \frac{dr}{dt} + \pi r^2 \frac{dh_w}{dt}. \end{aligned} \tag{5.4}$$

Since the radius of the tank, r, is constant, we do have that $\frac{dr}{dt} = 0$ and hence

$$\frac{dV}{dt} = \pi r^2 \frac{dh_w}{dt}. \tag{5.5}$$

This change in volume is essentially the water demand that has been consumed by users in the specified time. This shows that, for the demand at a specified time D_t, we have

$$\frac{dV}{dt} = D_t \tag{5.6}$$

and hence

$$\frac{dh_w}{dt} = \frac{1}{\pi r^2} D_t. \tag{5.7}$$

If we make the simplification of fixing demand D_t as a constant (e.g. hourly demand, daily demand, etc.) we can, therefore, conclude that

$$\frac{dh_w}{dt} \propto \frac{1}{r^2}. \tag{5.8}$$

Re-writing equation (5.8) with a constant of proportionality αh_t^2, we get

$$\frac{dh_w}{dt} = \alpha \left(\frac{h_t}{r}\right)^2 \tag{5.9}$$

where h_t is the height of the tank and $\alpha = \frac{\kappa D_t}{\pi}$ for κ constant of proportionality. This substitution allows us to realize the second order relationship between the rate of change $\frac{dh_w}{dt}$ in the water height and the ratio $\frac{h_t}{r}$.

The ratio $\frac{h_t}{r}$ is a very important parameter in determining the rate of change $\frac{dh_w}{dt}$. Ideally, one would want the ratio $\frac{h_t}{r}$ to remain constant. But in practice, due to operational issues like pump scheduling, flow control and others, $\frac{dh_w}{dt}$ keeps changing according to inflow and outflow rates of the tank. In that situation, gradual changes would be more desirable as that would mean gradual adjustments to total pressure $\frac{dp_1}{dt}$ at a tank (atmospheric pressure, height pressure and elevation pressure). The idea is that the smaller the fluctuation levels in pressures, the smaller the disturbance in terms of transporting water to end users.

The next section discusses simulations, using MATLAB, indicating the consequences due to varying both the tank height h_t and the radius r on the rate of change $\frac{dh_w}{dt}$ as derived in equation (5.9) which, in turn, influences the rate of change $\frac{dp_1}{dt}$.

5.3 Simulations and Results

A simulation of the quadratic relationship between the ratios $\frac{dh_w}{dt}$ and $(\frac{h_t}{r})^2$ was plotted, first in 1-D (Figure 5.1).

As is expected from properties of a quadratic model, the rate of change $\frac{dh_w}{dt}$ is gradual for $\frac{h_t}{r}$ values of less than 1 with the low levels of change in slope as $\frac{h_t}{r} \rightarrow 0$.

The same was also observed in a 2-D plot of the relationship. Figure 5.2 picks up the relationship in a wider context. Again, when the $\frac{h_t}{r}$ exceed 1, the values of $\frac{dh_w}{dt}$ increase in quadratic order. While it looks like the rest of the graph behaves the same, a zoom into the flat part of the graph shows that even in that region, the ratio $\frac{h_t}{r}$ plays a very crucial role in determining changes in the ratio $\frac{dh_w}{dt}$ (Figure 5.3). The bottom line remains, as long as the ratio $\frac{h_t}{r}$ has values less than 1, the changes in the ratio $\frac{dh_w}{dt}$ are gradual as can be seen in the slope of the graph.

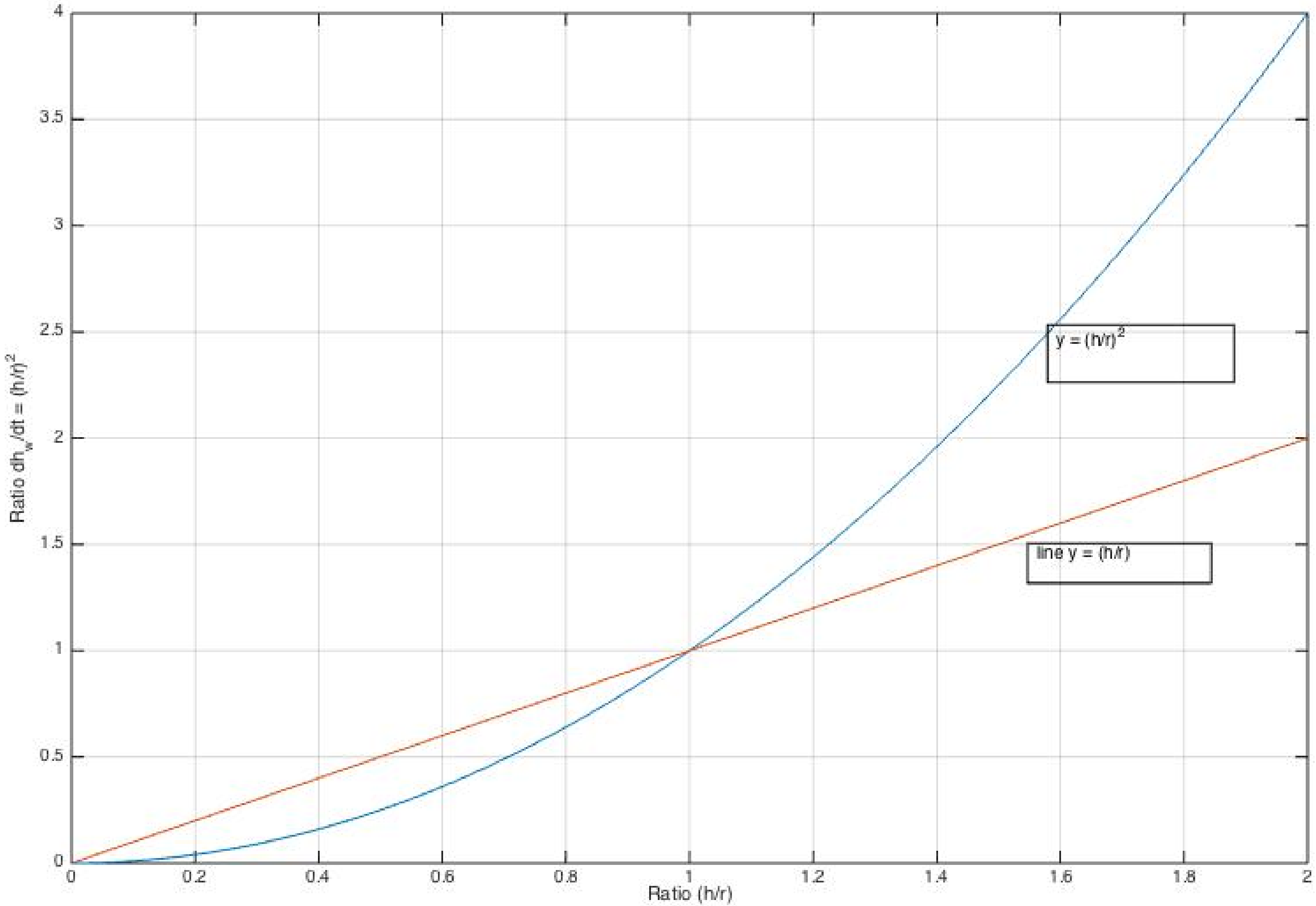

Figure 5.1: 1-D Sketch of Rate of Change $\frac{dh_w}{dt}$ with varying h_t/t

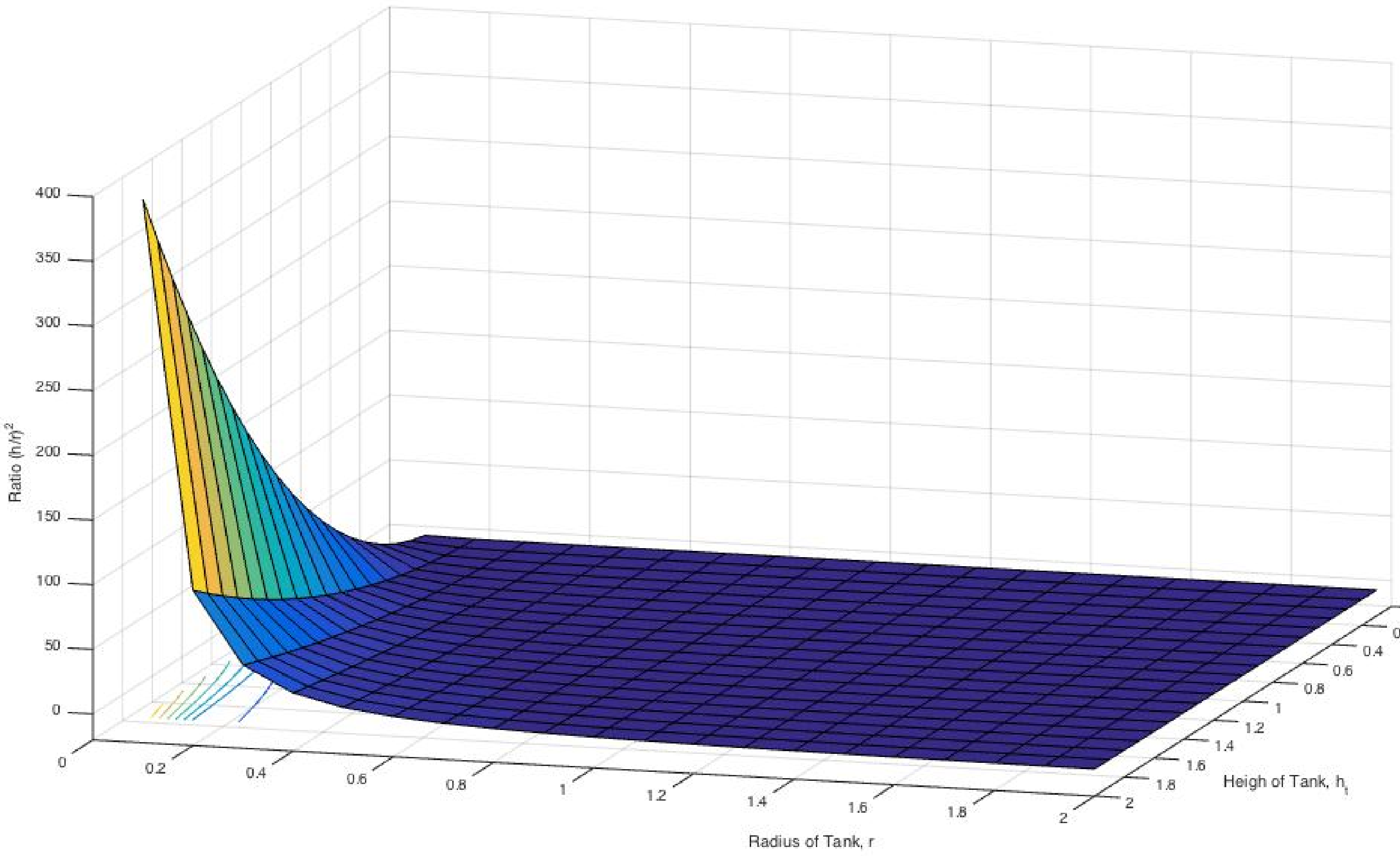

Figure 5.2: 2-D Sketch of Rate of Change $\frac{dh_w}{dt}$ with varying h_t and t

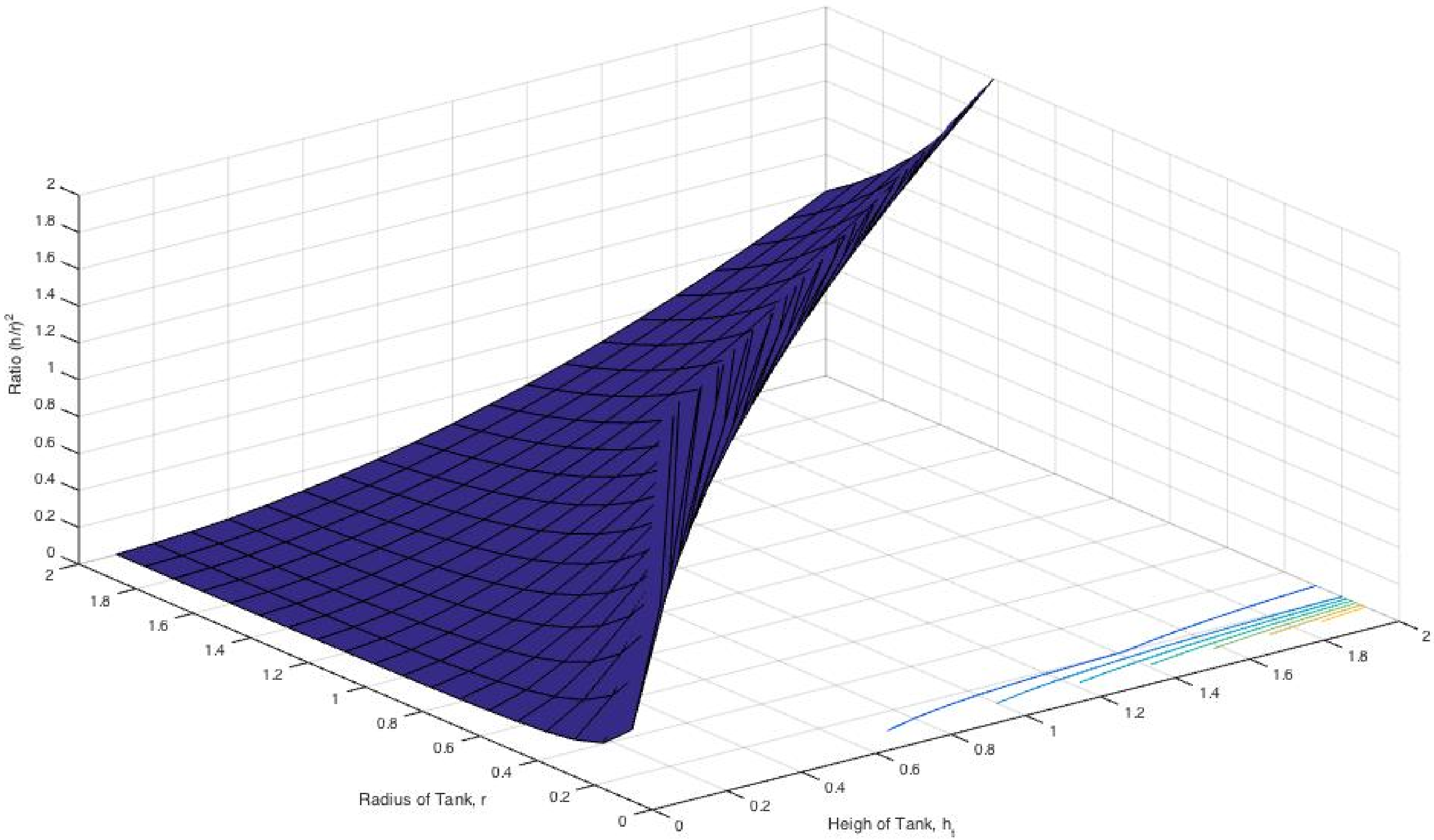

Figure 5.3: 2-D Zoomed Sketch of Rate of Change $\frac{dh_w}{dt}$ with varying h_t and t

5.4 Discussion

We first discuss a rather interesting question a reader may ask, why are we interested in values of $\frac{h_t}{r} < 1$ and not $\frac{h_t}{r} \geq 1$? The answer is simple. Let us consider Equation 5.9 again. We write it below for reference sake.

$$\frac{dh_w}{dt} = \alpha \left(\frac{h_t}{r}\right)^2.$$

Regardless of what values α assumes, when $\frac{h_t}{r} < 1$ then $(\frac{h_t}{r})^2 < 1$ and hence we have that

$$\alpha \left(\frac{h_t}{r}\right)^2 < \alpha. \tag{5.10}$$

On the other hand, when $\frac{h_t}{r} \geq 1$, then we have

$$\alpha \left(\frac{h_t}{r}\right)^2 \geq \alpha. \tag{5.11}$$

These results mean the overall changes in the ratio $\frac{dh_w}{dt}$ are diminished by quadratic order when $\frac{h_t}{r} < 1$ and are compounded, again by quadratic order, when $\frac{h_t}{r} \geq 1$.

As can be inferred from the simulations above, tanks designed with radii greater than their heights are better suited in maintaining the required pressures needed to transport water to the demand sites. In an ideal situation, highest levels of change are attained when $h \to \infty$ and $r \to 0$. Similarly, lowest changes would be recorded when $h \to 0$ and $r \to \infty$. These are not practical situations. In practice, tanks have specific space they must be built on, a time frame for completion and require finite material and man power resources to use for building. As is the case with Zomba, tank sizes are determined with the proportion of population they will serve, and also whether they will be used as feeder tanks or end tanks in the process. However these may be combined, it still remains advisable to adhere to the requirement of the ratio $\frac{h_t}{r} < 1$ when gravity is to play a vital role in transporting water. This condition may be very useful in a network like Zomba where the, despite being mountainous, external power sources like electricity are but a luxury in the country's economy.

Persistent pressure fluctuations may also account for pipe failures as was discussed in the hammering topic. It therefore is desirable to keep pressure fluctuation to as low levels as possible (usually defined from network to network according to individual network standards).

Also, typical tanks do not necessarily need to always be cylindrical, i.e. do not always have to have a circular base. Some tanks can have a square base, some a rectangular base, others a triangular base. Regular shapes can be directly computed using the same arguments. For example, a square base tank with length L would follow the equation

$$\frac{dh_w}{dt} = \frac{1}{L^2} D_t. \tag{5.12}$$

and all other arguments would hold.

Another way of going about different shapes from the usual and simple ones (triangle, square, rectangle, circle, etc) is to first determine the base area. Once the base area A is calculated, the equivalent length or radius can be derived from equating A to the area of the equivalent square or circle. For example, in the case of a circle, the equivalent radius can, therefore, be calculated as

$$r = \sqrt{\frac{A}{\pi}}. \tag{5.13}$$

As long as this radius r is larger than the corresponding height of the tank h_t, all of the above arguments hold.

5.5 Conclusion

A typical cylindrical tank set-up was discussed in this Chapter where the relationship between the height change $\frac{dh_w}{dt}$ was analyzed with respect to the ratio $\frac{h}{r}$. The reason for this analysis was to determine the ratio properties that exhibit minimal height changes. Such a property ensures that atmospheric, elevation and water height pressures play a vital role in transporting water in a network before the need for external energy come in. If places like Zomba would adopt such designs, they would minimize the use of external energy to transport its water.

It was found out that tanks that follow the simple ratio property of $\frac{h_t}{r} < 1$, irrespective of their size and capacity, are better placed to exhibit diminished, as opposed to increased, height changes with demand changes.

Chapter 6

General Discussion

6.1 Introduction

Water distribution network systems have become an integral part of our living. As such, their day to day management as well as short term to long term planning is vital for their performance. Just like any typical network, the Zomba water distribution network serves the same purpose which is the transportation of water from the source to the treatment plant, then to storage tanks through a network of tanks and pipes, finally reaching the end users. A simple equation that governs every water distribution network is

$$\textit{Water Produced} = \textit{Water Consumed} + \textit{UFW}$$

and the aim of every one of them is to reduce Unaccounted-for Water (UFW) to acceptable levels as defined from time to time. For example, Zomba has aimed at attaining a limit of UFW of at most 15% of the water produced. At the moment, Zomba's UFW is sitting at 40%.

To control UFW, there is need to understand the demand function of Zomba city. This involves understanding the influencing factors as perceived by the Southern Region Water Board. A final product is a demand model that takes on board all these factors.

Equally, a thorough understanding of distribution dynamics is essential towards controlling UFW. Leakage control through pipe bursts are one way of arresting the situation. Thus a thorough understanding of the network dynamics reveals that appropriate tank elevations must be observed as well as usage of flow control valves next to pipe junctions. Finally, tank design properties also play a vital role in controlling UFW.

6.2 Demand Modelling

This research follows the work of the authors like [3, 4, 57] where cyclic patterns in the demand are captured by sinusoidal functions. The developed model has been able to take on board the influencing factors perceived by the Southern Region Water Board. This model may be a starting point for modelling of water demand over longer periods of time, just to build on the time series modelling techniques. Such long term modelling may help a lot in correct mapping of demand. It is arguably a problem in Malawi, and much of Sub Saharan Africa that water shortages are a result of lack of proper planning and other factors.

A demand model like the one discussed in this thesis may not be generalized to all urban places unless they share the same influencing factors. Even in such cases, the capacity of demand levels will dictate the parameter values, allowing them to take on different values. So, in general, demand models may differ from place to place.

6.3 Flow Effects versus Demand

When the network was commissioned in 1889 [24], no literature records any incidence of houses built next to tanks experiencing water supply problems. We may argue that there were not may houses yet, and the elevation h_{elv} requirements were being followed by default, even if people didn't know the importance of it. It is only until the dawn of democracy in Malawi, in 1994, when the cities saw rapid expansion in terms of infrastructure [25]. In the absence of external power to drive the water, which is typical of Zomba Network as at the time the research was being conducted (2012 - 2014), all houses within the perimeter of h_{elv} from the tank will continue to experience this problem.

As outlined already in the chapter discussion, the more practical way of handling this problem is to dismantle the tanks and raise them above the ground. Most of Zomba tanks can be re-assembled. Doing so would cut costs on the electricity being used to supply some parts of the city. Otherwise, it must be a responsibility of all parties, the Water Board, City Authorities, Household owners to adhere to the elevation requirements for a continued use of natural pressure ways of transporting water. External energy comes at a cost, and obviously such costs are transferred to the end users. If elevation requirements are adhered to, it may ensure the tariff costs remain lower than when using electricity.

Another option for households is to use external house tanks. These are tanks put at an elevation at a house. Water is allowed to collect into the tank and the house uses water from the tank. Firstly, such tanks come at a cost. This means if our main network is consistent in delivery, the house tanks may be redundant. Secondly, such tanks are placed at an elevation. The challenge is if the house already has pressure problems at tap level, putting water into a higher elevation may become a challenge.In the end, water may never even find its way into the house tank. Thirdly, it will be very difficult to control overflow in house tanks. As a result, the Water Board may not guarantee a control in UFW.

It therefore remains that lifting the re-assemblable tanks in Zomba is the best solution for now.

6.4 Hammering Effects

The hammering, circumferential and longitudinal stress values, as can be inferred from the simulations, are quite of big magnitudes fin the network.

Sudden closures are by far prohibited. Most FCVs have an inbuilt design mechanism that controls closures to not be below critical closure time. However, as shown in the chapter, gradual closures still come with the hammering, longitudinal and circumferential stresses. If the FCV is closed at a Node, unlike at the tank, the stress effects are greatly reduced. The available stresses are shared in the two pipes connecting the other two tanks and thus cannot have the same effect as having one pipe handle all those. The tank at a higher elevation absorbs all those stresses when water finally flows into it. In a way, it acts as a pressure break.

As a way of controlling this situation in Zomba, whose network already has FCVs at every node, it should be encouraged to use FCVs at the nodes first before involving those at tanks. This will greatly reduce pipe failures. Also, a process of learning the required flows to tanks may be introduced. In that was, FCVs may be closed up to some level to just allow the required water to pass through.

A long term solution would be to invest in automated FCVs (e.g. prepaid meters, etc) that would allow the control to be done automatically from office.

6.5 Tank Design

When designing tanks that ought to be installed in mountainous networks that use more of gravity, like the Zomba city's network, it is important to realize that there is a positive relationship between pressure and the height of water in a tank. Desirable tanks must provide for a gradual decrease in the water height to ensure consistency in pressure values. It turns out that, whatever shapes they may take, tanks with a height - radius ratio of $\frac{h}{r} < 1$ exhibit diminished, as opposed to increased, height changes with demand changes. This property must be integrated with the other properties, like time, space, financial resources and others, to achieve a better tank. As such, it is also interesting to note that such a property must always be considered in the future infrastructure expansions while making optimal use of atmospheric, water height and elevation pressures in a network. It also must be mentioned that other pressures, like atmospheric pressure, do change with time, weather changes, altitude and other factors. However, humans have very little control over such changes. As such, controlling the water height changes is the easier option for humans.

Chapter 7

Conclusions

In this thesis, four aspects of a water distribution network have been studied. The Zomba network in Malawi has been the focus of this investigation. The first consideration was the modelling of the water demand patterns in Zomba city. Influencing factors as viewed by the Southern Region Water Board, a water supply company for Zomba city, were used to come up with a mathematical model that governs the demand pattern for the city. The factors considered were daily as well as seasonal fluctuations and population dynamics. Interesting results derived from the model are that, despite the tank's capacity being able to supply the city up to the year 2035, the current treatment plant infrastructure can support the population for the next 12 years. As such, there is a need for an expansion of current treatment infrastructure before the time. Actually, another plant was under construction during the research period and was completed in 2013. The research conducted further supports the fact that improving efficiency of the current network is all that is needed for the Zomba city. As of 2016, the existing infrastructure - given appropriate adjustments and expansions - is able to support the whole population of Zomba.

The second consideration of this work concerns explaining why houses built next to the tank have a shortage of water supply. It was discovered that these shortages occur due to the fact that the necessary elevation requirements for tanks, especially in a gravity fed network like Zomba, are not observed. In cases where tanks are elevated from the ground to the required elevation h_{elv}, such problems are not observed. In an ideal situation one would expect that, if tanks are installed on the ground, then houses must start at an equivalent h_{elv} distance away from the tanks. This may be far from practical as is the case with Zomba. A practical solution would be to dismantle tanks and raise them to a correct elevation.

The third problem encountered when investigating the Zomba network links closely to the second case, but has far greater consequences. When water flowing from a feeder tank only reaches end tanks that are at relatively lower elevations to others, a manual closing of the flow control valves fitted next to the tanks induces hammering pressures and stresses that can be as large as 65 times the magnitudes of normal flow pressures in the concerned pipes. This is a source of numerous unexplained pipe failures observed in the network. It is recommended that flow control valves must instead be fitted next to the joint nodes as opposed to the tanks. Technology fitted flow control valves may be ideal for the network as they will only allow the required flow to pass through them, but they would require an external power source to operate. This would be an additional cost to the company.

The last part of this thesis studies the relationship between the ratios $\frac{dh_w}{dt}$ and $\frac{h_t}{r}$ and how they relate to the optimal use of atmospheric, water height and elevation pressures to transport water in the network. Tanks with the ratio property $\frac{h_t}{r} < 1$, on top of physical, financial and time constraints, are better suited for optimal use of the above mentioned pressures. This may be input in future designing of the network, not only for Zomba, but for all networks that want to make optimal use of atmospheric, water height and elevation pressures. If adopted in Africa, the continent would see a reduced use of external energy, like electricity, in its water supplies, a move that would help reallocate the external energy to other uses.

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