JIMMA UNIVERSITY

SCHOOL OF POST GRADUATE STUDIES

JIMMA INSTITUTE OF TECHNOLOGY

FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING

ENVIRONMENTAL ENGINEERING CHAIR

PERFORMANCE EVALUATION OF WATER SUPPLY DISTRIBUTION SYSTEM; A CASE OF JALDU TOWN, OROMIA, ETHIOPIA

BY: BEZAWIT BEKELE BALCHA

A THESIS SUBMITTED TO ENVIRONMENTAL ENGINEERING CHAIR, JIMMA INSTITUTE OF TECHNOLOGY, SCHOOL OF GRADUATE STUDIES, JIMMA UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTERS OF SCIENCE IN ENVIRONMENTAL ENGINEERING

JULY, 2021

JIMMA, ETHIOPIA

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MAIN ADVISOR: - ZERIHUN ASMELASH (PhD)
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JULY; 2021
JIMMA; ETHIOPIA

DECLARATION

I hereby declare that this submission is my original thesis work and it is not containing material previously published by another person nor which has been accepted for the award of any other academic degree of the University, except I have been used and where due acknowledgement has been made and cited in the reference part of this work.

Bezawit Bekele Balcha Signature_____________ Date____________

This Thesis has been submitted for examination with my approval as a university advisor.

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(Main Advisor) Signature Date

Mr. Wendosen Mekonnin (MSc) ___________________ _______________
(Co- Advisor) Signature Date

APPROVAL PAGE

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ABSTRACT

The aim of proper design of a water supply distribution network is adequately delivering of water to the consumption nodes. However, the hydraulic performance of the water distribution network for this study area was inadequate to transfer available water to a consumption node. Hence, the main finding of the study was to evaluate the water supply distribution system and its hydraulic performance. In order to obtain the main findings of the study, researcher used to, water GEMS v8i for model simulation; Auto CAD v2007i for exporting water distribution networks drawings to water GEMS; ArcGIS 10.1 for extracting junction elevation using geographic positioning system. The existing water supply production is 1693.62 m^3/day; whereas 811.6m^3/d is currently the maximum demand day. However, the deficit of water supply was observed b/n 2020-2025 years, which requires and hence additional 411.20 m^3/d of waters requires as other surplus source of water. This indicates that the current services of water supply yield are fixed and water demand is timely increasing and attains 3910.7m^3/d at the end of 2040 years (design projection time), hence, 2,217.70 m^3/d water supplies is requiring end of 2040 as additional sources of supply. This deficit of water supply is proposed for domestic water demand and non-domestic water demand and water loss consideration of all water demand variation. In the same manner the hydraulic performance of this distribution network were simulated at steady state and extended state condition using water GEMS Simulation and there is high junction pressure, negative junction pressure was recorded since simulation at maximum day water demand and minimum day water demand consumption hours. During steady state simulation, 24.60 % of the higher pressures junction of the area, which is ≥70 mH_2O, was observed at the different junctions due to low elevation and large pipe diameters. During this condition, most of the distribution pipelines has the optimum range (70-15 mH_2O) which is 59.89 % of distribution network is the normal ranges and 15.51 % of the lowest pressure junction ≤15 mH_2O due to low elevation and large pipe size. Whereas 51.93 % is the lowest water velocity < 0.56m/s and 13.94 % was the highest water velocity > 2.5m/s recorded with respectively. Since this extended state simulation from the total unfunction junction pressure, due to high pressure and low water pressure is 40.11%. Accordingly, the analyzed of hydraulic performance pressure 0.81 and velocity performance 0.64 and hydraulic performance indexed for both velocity and pressure simulation is 0.47 which ranges in acceptable performance indication according to performance standards.

Key words: Hydraulic Performance index, Water GEMS V8i, Water loss, Water supply deficit, Jaldu town

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ACRONYMS

ADD	Average Daily Demand
CSA	Central Statistics Agency
EPS	Extended Period Simulation
FD	Fire Demand
GIS	Geographical Information system
GPS	Global Positioning System
HC	House Connection
HGL	Hydraulic Gradient Lines
MASL	Meter Above Sea Level
MDD	Maximum Daily Demand
MDDF	Maximum Daily Demand Factor
MOWR	Ministry of Water Resource
NDD	Non-Domestic Demand
NRW	Non-Revenue Water
PF	Public Fountain
PIP	Performance Index Pressure
PIV	Performance Index Velocity
HPI	Hydraulic Performance Index
DSD	Developed System Dynamic
PHDF	Peak Hour Demand Factor
PHF	Peak Hour Factor
PVC	Polyvinyl Chloride
UFW	Unaccounted for water
WASH	Water, Sanitation and Hygiene
WDN	Water Distribution Networks
WDS	Water Distribution System
WHO	World Health Organization
YC	Yard Connection
YSC	Yard Shard Connections

CHAPTER ONE

1. INTRODUCTION

1.1. BACKGROUND OF THE STUDY

Water is a primary need to sustain life and every citizen has the right to have access to potable water (Datwyler, 2012; Ravi *et al.*, 2019; Hunde,2020; Peng and Mayorga, 2016). However; the performance of water supply distribution service is becoming a major issue as a worldwide (Agnew, 2006; Imneisi *et al*., 2016; Framework, 2019; Sahilu and Chaka, 2017). This issue may cause; the expansion of pre-urbanization and population increments, increasing pressure on local water supply distribution system, mostly in developing countries. According to WASH (2016) report, the global drinking water target was met 91% (ninety one percent) percent in 2010 while the Caucasus and Central Asia, Northern Africa, Oceania and Sub-Saharan Africa did not achieve this milestone.

The provision of safe and adequate water supply services are necessary components for sustainable development (WASH, 2016, Anisha *et al.*, 2016). This provision of adequate supply of potable water for use in urban areas in developing countries is crucial for the well-being of the people. As Such ways of water demand for such supplies in the developing countries has been increasing over time because of rising standards of living that occur with economic progress and population increase resulting from natural growth, and rural urban migration and rising per capital income. Accordingly, the estimated water supply service level of Ethiopia in terms of coverage, quantity, quality and reliability is very low (Wannapop and Jearsiripongkul 2018; Pandya, 2019). A well performing urban water supply system should provide water supply for human beings and livestock consumption, for industrial and other uses taking the existing and future realities of the city in to consideration.

The distribution network is responsible for delivering water from the source or treatment facilities to its consumers at serviceable pressures and mainly consists of pipes, pumps, junctions, valves, fittings, and storage tanks(Wang, 2019; Riis, 2016; Salunke *et al.*, 2018). Water distribution networks play an important role in modern societies being its proper operation directly related to the population's well-being. However, this water demand is increasing due to different factors (population growth rate and climate condition, which defect the performance of the water distribution network led to the negative influence in most of the socioeconomic sectors. Leakage is one of the causes of water loss in a network distribution system that currently needs attention (Peng *et'al*., 2016; Leta, 2018; and Tadesse, 2020). However, hydraulic performance is not investigated by the utility of Jaldu town and hence it's an attempt made to analyze and evaluate the hydraulic performance of the water supply distribution system while considering water loss. Water distribution systems are designed to adequately satisfy the water requirements for a combination of domestic, commercial, industrial, and firefighting purposes. The system should be capable of meeting the demands placed on it at all times and at the satisfactory hydraulic performance(Wannapop, Jearsiripongkul and Jiamjiroch, 2018). However, hydraulic performance is not investigated by the utility of Jaldu town. Therefore, this is an attempt made to analyze and evaluate the hydraulic performance of the water supply distribution system while considering water loss. Consequently, water supply distribution systems in urban areas are often unable to meet existing community. Additionally, as reviews indicate some consumers take unequal amounts of water and the poor is the first victim to the problem (Salunke *et al.*, 2018; Fekrudin and Ababa, 2019) it is expected to be for Jaldu town.

1.2. Statement of Problems

The performance of water supply distribution networking system should satisfy current and future demands. Majority of the study area householder's water supply services system were obtaining directly through private either connections or public taps, which cause to intermittent water distribution system. According to Jaldu town water supply service office, currently there is intermittent water distribution has impact on performance of water distribution system. This shortage is common for a long time before and continuing in serious manner currently. The other indications are queue on limited water points and going long distance to fetch water from sources out of the town is a common phenomenon especially for women and children in current time. This water supply shortage in the town drives to study the causes of this problem and give information that helps the concerned body to give solution for the problem. However, the Jaldu town water supply distribution system is providing inadequate services to its customers. These include low service coverage and irregular mode of water distribution system characterized by frequent cut-offs and technical incompetence for several years. This temptation could be occurred due to the limited water supply sources, pre-urbanization, insufficient hydraulic performance, and aging water distribution infrastructure, which coupled to increases water demand. Mostly for this study area water supply, sources do not satisfy the demand of present and future population, so the distribution system does not cover the whole part of the area. This need to found the Jaldu town water supply distribution system should perform as standardly accepted key performance indicators in water supply distribution networks.

1.3. Objective of the study

1.3.1. General Objective

The objective of the study was to investigate the performance of water supply distribution system by using water GEMS v8i

1.3.2. Specific Objectives

1) To evaluate water supply deficit and predict water demand for the future;
2) To analysis the hydraulic performance in water supply distribution network; and
3) To evaluate total water losses (Unaccounted for water) in distribution system.

1.4. Research question

1) What is the existing water demand and future water supply deficit?
2) What is the level of hydraulic performance in water supply distribution networks?
3) How much is the water loss in the existing water distribution system?

1.5. Significance of the study

This study evaluates the situation of hydraulic performance of urban water supply distribution system. Based on the factors that are negatively contribute the satisfaction level of the service, was listed out and alternative system operation, maintenance and management is recommended. In addition, hopefully, the insights that has drawn from this study was initiate further research on similar sites and was contribute to solving the existing problems of rural water distribution system

1.6. Scope of the study

The primary goal of this study was, to evaluate the performance of the existing distribution system of the Jaldu town using water GEMS. Thus, the scope of this study is estimation current and projected water demand, future water supply deficit of Jaldu town. Therefore, the scope of this study is limited to the stated objectives above over the study area. In this study the quality of water did not be considered due to , the lack of budget or funds and other zone or town's performance evaluation of water distribution system did not be studied in this study. This may provide an estimate of the overall position of the water billed authority, which may assist to make an overall conclusion on the performance of water loss and hydraulics parameter performance in water distribution network.

CHAPTER TWO

2. LITERATURE REVIEW

2.1. Urban water supply

Safe drinking water is the birthright of all humankind as much a birthright as clean air((Fekrudin and Ababa, 2019; Anore, 2020; Kanownik *et al*., 2019) while access to clean water can be considered as one of the basic needs and rights of a human being. (Ravi *et al.*, 2019; Dacombe *et al.*, 2016) stated that, water is important character which could be includes both domestic water demand and productive water demand uses. Thus, safe drinking water harmonized with improved sanitation contributes to the overall well-being of people; it has significant bearing on infant mortality rate, longevity and productivity. However, the majority of the world‗s population in both rural and urban settlements does not have access to safe drinking water. According to (Ketema and Bacha, 2015; Bank, 2017) only 16% of people in Sub-Saharan Africa had access to drinking water through a household connection (an indoor tap or a tap in the yard). Not only their poor access to readily accessible drinking water, even when water is available in these small towns there are risks of contamination due to several factors like inappropriate waste disposal and lack of water supply infrastructure such as pipe line for water (Hunde and Ing, 2020; Islam *et al.*, 2016). According to Water Utility Partnership (Fadaei and Sadeghi, 2014), the primary goal of all water supply utilities is to provide customers with a private ‗connection to the piped water supply network.

2.2. Water supply Distribution System

Water distribution systems convey water drawn from the water source or treatment facility, to the point where it is delivered to the users (Design and Enterprise, 2019; Bhatt and Paneria, 2017; Pandya, 2019). Water distributions systems in urban areas are continuously evolving to balance the increase in demand arising from urban development change in consumption patterns, industrial development and other domestic uses. Water distribution systems are important to the community to deliver clean water from storage facilities to consumers through a complex and extensive pipe network and this distribution system consist of a water supply source and a pipe network. The distribution network is responsible for delivering water from the source to its consumers at serviceable pressures and mainly consists of pumps, pipes, junctions (nodes), valves, fittings, and storage tanks. Water distribution networks is an important component of any water supply system accounting for up to 80% of the total cost of the system(Capt. *et al.,* 2021) and as a result operation and maintenance cost may soar higher if they are poorly designed, hence the need to have a well-planned, designed and constructed water distribution network cannot be over emphasized especially because of its importance to industrial growth and water's crucial role in society for health, firefighting and quality of life (Anisha *et al.*, 2016).

2.3. Components of water distribution network

In a water distribution system, the steady state analysis is an important component of assessing the adequacy of a network (Sonaje, 2015). The hydraulic problem in connection with pipe networks consist of solving for the distribution of flow and head loss in the individual elements for a given total discharge or for a given total head loss (Sadeghi, 2018). This problem is considered solved when the flow pattern in each pipe is determined under some specified pattern of supply and consumption. The supply may be from reservoirs, storage tanks and or pumps or specified as in flow or outflows at some points in the network and from the known flow rates the pressures or head losses through the system is computed (Panday, 2019). Thus, according to Pandya (2019), alternatively, the solution may be initially for the heads at each junction or node of the network and these can be used to compute the flow rates in each pipe of the network.

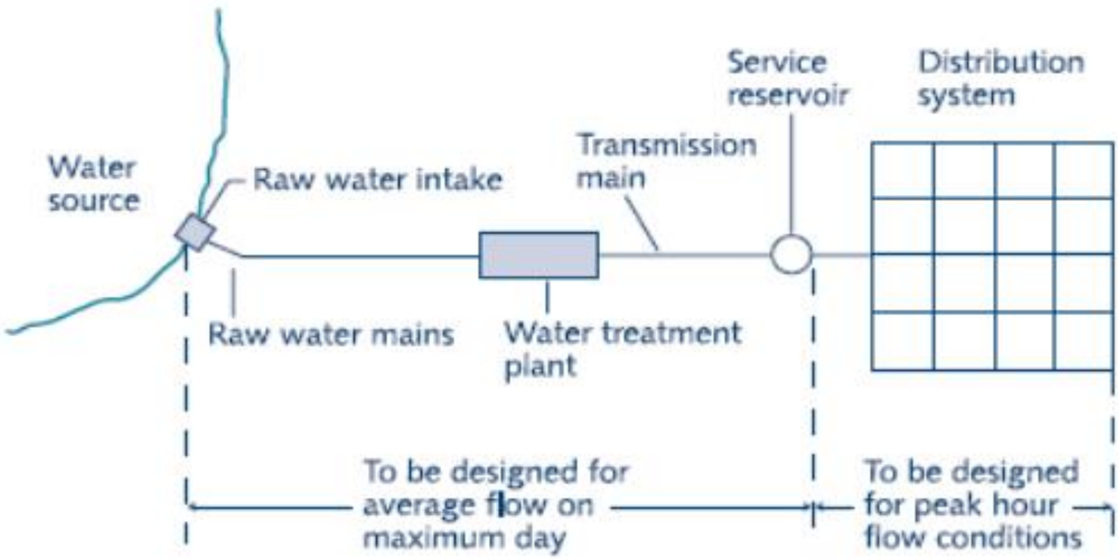


Figure 2.1: Components of water supply distribution system

Source: Water Utility Partnership (Fadaei and Sadeghi, 2014)

Transmission and distribution mains**: -** In this water distribution system, piping system is often categorized as transmission/trunk mains and distribution mains (Genetie and Befekadu, 2019, Kanownik and Policht-latawiec, 2019).

Transmission mains: Transmission mains were consisting of components that are convey large amounts of water over great distances, typically between major facilities within the distribution system. In most water supply system, transmission main is mainly used to transport water from treatment plant to service reservoirs/ storage tanks. Whereby, individual customers are usually not served from these mains (Sonaje, 2015).

Distribution mains: Distribution mains are an intermediate pipeline used to delivering water from transmission main to customers (Jagadesh, 2016). Hence, the mains distribution are smaller in diameter than transmission mains, and typically follow the general topology and alignment of the town streets. Different fittings such as elbows, tees, reducers, crosses and numerous other accessories are used in the main to connect pipes (Wang, 2020). While other maintenance and operational appurtenances, such as fire hydrants and valves are connected directly to the distribution mains. Further, this service also service line was laid and transmits water from the distribution mains to end customers.

2.4. Types of water distribution layout system

According to Mehta and Joshi, (2019) described that the water distribution networks have classified as explained

Below;

Dead end or tree system: In this system, a main starting from the reservoir is laid along the main road and sub mains are taken off from it along roads joining the main road (Sonaje, 2015). These distributors are taken off from the sub main along streets and lanes joining the road service connections are made from these branches. This system is suitable for towns develops in irregular manner and has the advantages of cheap initial cost, simple design calculation and easy extension of the system when desired (Sadeghi, 2018). The main disadvantages of this system are: the supply will be cut off if repair work is carried on the main or sub mains, there are dead ends which may contaminate the supply and it is difficult to meet the fire demand during repair (Bhoyar and Mane, 2017).

Grid-iron system: This system is most convenient for towns having rectangular layout of roads and improvement over dead-end system(Capt. *et al.*, 2021). Accordingly, the dead ends are interconnected with each other and water circulated freely throughout the system. In this system mainline is laid along the main road but for sub-mains are taken in both directions along other minor roads and streets (Jagadesh, 2016). From these sub-mains, branches are taken out and are interconnected to each other and water circulates freely throughout the system. This system removes all the disadvantages of dead-end system. From the above systems, Gridiron system is most suitable for towns that have a rectangular lag out of roads & for newly developed cities. The main advantages of this system are all dead ends are eliminated; very small area was affected during repair work, the friction losses and the sizes of pipes are reduced, and in case of fire demand, more quantity of water can be diverted to the affected area by closing the valves of nearby localities (Wang, 2020).

Circular or ring system: This system is adopted only in well-planned locality of cities. In this system each locality is divided into square and the water main are laid around all the four sides of the square. All the sub-mains and branches are taken off from the boundary mains and are interconnected. This system is the best of the other system but it requires many valves and more pipe length (Panday, 2019) and this system is most suitable for towns and cities having well planned road access.

Radial system: Actually, this is the reverse of ring system and water flows towards outer periphery from one point and the entire district is divided in to various zones and one reservoir is placed for each zone, which is placed at the center of the zone (Salunke, 2018).

2.5. Method of Population forecasting

The economic design period of the components of a water supply depends on their life, initial cost, rate of interest on loan, the ease with which they can be expanded of the likelihood that they was rendered absolute by technological advances (Wannapop, 2019). The future development of the town mostly depends on trade expansion, development of industries and surrounding country, discoveries of mines, construction of rail way station (Sadeghi, 2018). These elements may produce sharp rises, slow growth, and stationery conditions. The populations are increased by births, decreased by deaths, increased or decreased by migration and increased by annexation and factors affect the change in population (Mehta, 2019). The correct present and past population can be obtained from census office and hence, knowing the present population from the recent census is possible to design or forecast future population of the town (Mehta, 2019). By considering growth rate of the town, we use the following different methods of population forecasting to asses and estimate the future population of the town:

A. Arithmetic increase method

$$\mathrm{Pn} = \mathrm{Po} + \mathrm{K_n} \tag{2.1}$$

Where; P_n = population at n decade

n =decade or year

k =arithmetic increase

B. Geometric increase method

The method is based on the assumption that the percentage increase in population remains constant. It also known as uniform increase method. The increase is compounded over the existing population. This method is mostly applicable for growing towns and cities having vast scope of expansion.

$$\mathrm{Pn} = \mathrm{po}(1 + \mathrm{k})^{\mathrm{n}} \tag{2.2}$$

Where P_o=initial population

P_n = Population at n decades or year

n = decade or year

K = percentage or geometric increase.

C. Incremental increase method

In this method, the population in each successive future decade is first worked out by the arithmetical increase method and to these values; the incremental average per decade is added. Since the method combines both arithmetic as well as geometric increase method, it improves the few results that are obtained by arithmetic increase method.

$$Pn = Po + n * (K + r) \tag{2.3}$$

Where Po=initial population

Pn=population at n^{th} decade or year

n=number of decades

K=Arithmetic increase

r=incremental increase

D. Method used by Ethiopian statistics authority

The Ethiopian statistic authority uses the formula for most water supply project in the country to project population at the end of required decade/year.

$$P_n = p_o e^{Kn} \tag{2.4}$$

Where P_n=population at n decades (year)

P_o=initial population (from census)

K=growth rate

n=decade or year

Due to given population data Arithmetic increase, Geometric increase, Incremental increase and Ethiopian statistical authority methods are used for population projection.

2.6. Water demand projections

Estimating water demands for a particular town depends on the size of the population to be served, their standard of living and activities ,the cost of water supplied, the availability of waste water service and the purpose of demand(Anisha *et al.*, 2016; Anore, 2020) . This demand can be varies according to the requirement of the domestic demand, institutional, industrial and social establishments. In addition to these, demand allowances need to be included for leakage, wastage, and operational requirements such as flushing of mains.

2.7. Domestic water demand projections

The water demand for actual house hold activity is known as domestic water demand(Bank, 2017, Journal and Reuse, 2019; Hunde, 2020) which includes water for drinking, cooking, bathing, washing flushing and toilet. This demand can be depending on many factors, the most important of which are economic, social and climatic(Islam *et al.*, 2016;Anawr, 2018; Riis, 2016; Peng and Mayorga, 2016). Based on the available at obtained from the Jaldu water supply service has four major modes of service were identified for domestic water consumers.

i. Population Distribution by Mode of Service

The percentage of population to be served by each mode of service will vary with time. The variation is caused by changes in living standards, improvement of the service level, changes in building standards and capacity of the water supply service to expand. The water service offices serve the community by the three major modes of services namely public tap, yard connection and house connection. However, for this study, the following modes of services are adopted from the design criteria prepared by the Ministry of Water Resources 2006; (Capt *et al.*, 2021; Anisha *et al.*, 2016). House connection, Yard connection own, Yard connection shared and Public tap supplies. Therefore, the present population and projected percentage of population served by each demand category is estimated by taking that mode of services conditions.

This projection envisaged provision of the traditional source users with public taps, and yard connections (own & shared). Further decreases in public tap users are expected on the assumption that more and more people will have private yard connections as indicated in the following Table 2.1.

Table 2.1: Population Percentage Distributions by Mode of Service

Mode of Service	Year					
	2014	2016	2018	2020	2025	2030
HTC	1.5%	2.5%	5.0%	7.5%	9.0%	10.0%
YTO	11.3%	15.0%	20.0%	25.0%	26.0%	27.5%
YTS	9.2%	12.5%	17.5%	22.5%	25.0%	25.0%
PT	78.0%	70%	57.5%	45%	40%	37.5%
Total	100%	100%	100%	100%	100%	100%

Source: Ministry of Water Resources (2006)

Due to this, an increase in percentage of yard connections and house connections is anticipated by the end of the design period.

ii. Per Capita Water Demand

The per-capita domestic water demand for various demand categories varies depending on the size of the town and the level of development, the type of water supply scheme, the socioeconomic conditions of the towns and the climatic condition of the area(Anisha *et al.*, 2016; Salunke *et al.*, 2018). The per capita water demand for adequate supply level has to be determined based on the basic human water requirements for various activities of demand category(Al-Mashagbah, 2015; Olbasa, 2017; G, A and N, 2016).

According to the design criteria prepared in January 2006 by Ministry of Water Resources, Table 2.2 shows the per capita domestic water demand adopted for Urban Water Supply System for Stage I (2025) design Horizon.

Table 2.2: Per capita domestic water demand adopted for Urban Water Supply System

Purpose	Mode of Service			
	HTC	YTO	YTS	PT
Total (l/c/day)	50	25	30	20

Source: Ministry of Water Resources, January 2006

With an increase in awareness within the community on the advantage of using a clean water supply and provision of better services by the water supply unit, the consumption level of the community in each mode of service is expected to increase by the end of the project design period (2035). In view of this, and as per the recommendation in the MoWR design criteria, the per capita consumption for each mode of services at the end of the design period was estimated. The expected domestic water demand for each mode of service by the end of the project period is given in Table 2.5.

Table 2.3: Per-Capita Demand by Mode of Service (2035)

Purpose	Mode of Service			
	HTC	YTO	YTS	PT
Total (l/c/day)	70	30	40	25

It is difficult to estimate how exactly the per capita water demand will grow in between the design horizons. The values given in Table 2.5 have therefore been used for the first year of design horizon i.e., 2014 and assumed to remain constant until 2025 though the reality of the situation may be that per capita demand may gradually grow from existing usage up to the 2025 design horizon per capita demands. Between the 2025 and 2035, design horizons, the assumption has been made that will be a linear growth in per capita demands. It should be noted at this stage that the reality of the situation would only be able to be determined with constant monitoring of consumptions and tracking of data over the years between implementation and the final year of the design horizon. The estimation of these intermediate per capita demand figures will not have any effect on the design of Stage I and Stage II infrastructure, but may affect the calculation of water tariffs.

Table 2.4: Projected Per Capita Demand by Mode of Service

Mode of Service	Year				
	2015	2020	2025	2030	2035
HTC	50.0	50.0	50.0	60.0	70.0
YTS	25.0	25.0	25.0	27.5	30.0
YTO	30.0	30.0	30.0	35.0	40.0
PT	20.0	30.0	20.0	22.5	25.0

Source: ministry of water and energy water supply module for urban, 2003

Table 2.5: Population percentage distributions by mode of service

Year		2015	2020	2025	2030	2035
Connection type	House	5.7%	6.58%	7.48%	9.28%	10.18%
	Yard	24.6%	28.64%	32.44%	40.84%	45.04%
	Yard Shared	28.7%	33.26%	37.68%	47.48%	52.38%
	Public Tap	39.0%	30.0%	21.40%	1.40%	0.00%
	Non-Domestic	30.0%	30.0%	30.00%	30.00%	30.00%
	Unaccounted For	30.7%	28.9%	27.38%	26.18%	25.58%

Source: ministry of water and energy water supply module for urban, 2003

iii. Adjustment factors of Domestic Water Demand

The climatic condition size of the town, culture of people industries cost of wale, fault of water pressure in the distribution system and system of supply is one of the adjustment factors of domestic water demand.

a) Adjustment for climate

The climatic condition of project area has an impact for the quantities of water consumptions. Those who are living in hot area consume extra water and people who live in normal temperature area consume less water. In order to account for changes of average per capital domestic demand, the water demand is multiplied by climatic factor.

Table 2.6: Adjustment of climate factor with Altitudes for water demand

Group	Mean Annual Temp. (^{0}C)	Description	Altitude	Adjustment Factor
A	≤ 10	Cool	>3300	0.8
B	10-15	Cool temperate	2300-3300	0.9
C	15-20	Temperate	1500-2300	1.0
D	20-25	Warm temperate	500-1500	1.3
E	≥ 25	Hot	<500	1.5

(Source): Design Guideline for WSP, 2008, Urban Water Supply Design Criteria MoWE, 2006

b) Socio-economic adjustment factors

The socioeconomic adjustment factor is determined based on the degree of the development of the particular town under study as the socioeconomic conditions play great role on the amount of water consumption. The determination of the degree of the existing devolvement and potential of the towns depend on personal judgment due to difficult condition in quantifying many aspects of the development.

Table 2.7: Adjustment factor for socio-economic conditions

Group	Description	Factor
A	Towns enjoying high using standards added with high potential development	1.1
B	Towns having a very high potential or development but lower living standard at present	1.05
C	Town under normal condition	1.0
D	Advanced rural towns	0.9

Source: Design Guideline for WSP, 2008, Urban Water Supply Design Criteria MoWE, 2006

iv. Variation of Water Demand

The variations in the rate of water demand means the average consumption rate or demand of water per head per day. The rate of demand of water, however, does not remain constant but it varies with the seasons or month of the year, with the days of the week, and with the hours of the day. Variation in demand refers to changes in demand due to changes in price alone other factors remaining constant. Seasonal or monthly variation are prominent in tropical countries like India rate of consumption reaches maximum in summer season due to greater use of water for street and lawn sprinkling(Dayessa and Merga, 2019; Mehta, 2019; Beyene, 2014; Desta and Befkadu, 2020). Other things that change demand include tastes and preferences, the composition or size of the population, the prices of related goods, moreover, even expectations. A change in any one of the underlying factors that determine what quantity people are willing to buy at a given price will cause a shift in demand.

The daily water demand in a community area will vary during the year due to seasonal climate patterns, the work situation (harvest time) and other factors, such as cultural or religious occasions(Bhatt and Paneria, 2017). Accordingly, the maximum daily demand is usually estimated by adding 10-30% to the average daily demand. Thus, the peak factor for the daily water demand (k_1) is 1.1-1.3, whereas the hourly variation in domestic water demand during the day is much greater which 1.5 -2.5 ranges (Bhoyar and Mane, 2017; Mala-Jetmarova, Sultanova and Savic, 2018). The peak hour demand can be expressed as the average hourly demand multiplied by the hourly peak factor (k_2). For a particular distribution, area this factor depends on the size and character of the community served. The hourly peak factor tends to be high for small villages and usually lower for larger communities and small towns as shown in Figure 2.2.

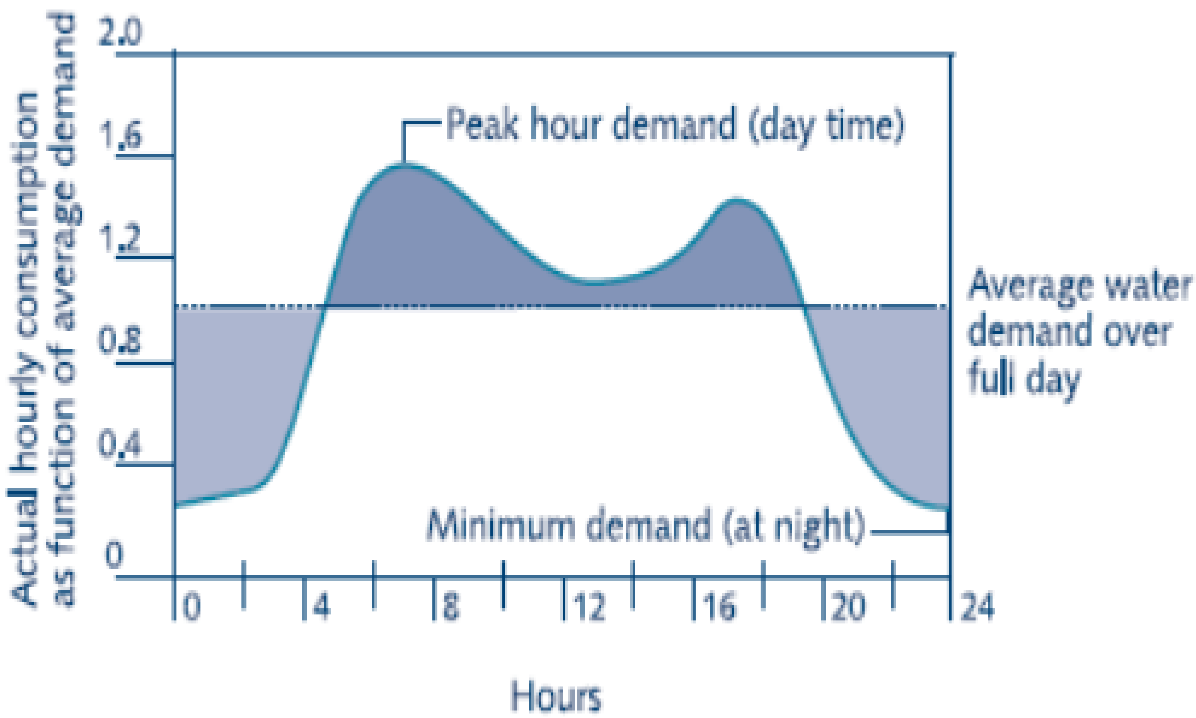


Figure 2.2: The variation of peak hourly demand and average water demand

Source: Ministry of Water and Energy water supply module for urban, 2003

v. Average Water Demand

The average daily water demand is the sum of the domestic, non-domestic and unaccounted for water, which is used to estimate the maximum day & the peak hour demand. The average day demand is used in economic calculations over the project's lifetime. Accordingly, 'Average daily water demand is the sum of the domestic, non-domestic and NRW which is used to estimate the maximum day & the peak hour demand'(Framework, 2019) which expressed as economic calculations over the project's lifetime as described in equation number 2.5 and 2.6.

$$\text{Q aveg.} = \text{ Per capital water consumption} * \text{ Total population of the town} \tag{2.5}$$

Where, Q_{avg} = Average day demand (m^3/s)

$$\frac{\text{Qaverage day}}{\text{f}} = (\text{K}_1 1\text{K}_2 + \frac{1}{100 - 1} \tag{2.6}$$

Whereas: k_1= Daily peak factor and

k_2=Hourly peak factor

Factor, f, in the equation is a unit conversion factor while I represent the leakage percentage of the total quantity supplied to the system. It is common to assume that 'demand = consumption + leakage'(Mavi and Vaidya, 2018).

vi. Maximum Day Water Demand

The maximum day water demand is considered to meet water consumption changes with seasons and days of the week(Pandya, 2019). Therefore, the maximum daily consumption to the mean annual daily consumption is the maximum day factor.

Table 2.8: Maximum daily factor

Population	Maximum daily factor
0-2000	1.3
2000-5000	1.25
Above 5000	1.2

Source: ministry of water and energy water supply module for urban 2003

vii. Seasonal Variation

The variation of water demand varies from season to season(Peng and Mayorga, 2016). In dry season, the water demand is maximum, because the people will use more water for bathing, cooling, lawn watering and street sprinkling. Therefore, 'maximum day water demand is considered to meet water consumption changes with seasons and it used to size source, treatment plant and rising mains. Hence, maximum day demands can be obtaining by multiplying the average day demands to the peaking factor applied to the node'(Kenasa, *et'al.*, 2018).

$$Q = PF * Q \tag{2.7}$$

Where, Q = Maximum day demand (m^3/s)

PF = Peaking factor between maximum day and average day demand

Q = Average day demand (m^3/s)

viii. Peak Hour Water Demand

The peak hour demand is greatly influenced by the size of the town, mode of service and social activity in the town(Bhoyar and Mane, 2017; Bhatt and Paneria, 2017). It is the highest demand of any one-hour over the maximum day, which represents the diurnal variation in water demand resulting from the behavioral patterns of the total population. The peak factor utilized to the peak hour demand show similar dependences that the maximum day factor for the maximum demand(Mala-Jetmarova, *et'al.*, 2018).

Table 2.9: Recommended peak hour Factors

Population size	Peak hour factor
<2,000	2.6
2,000-10,000	2.4-2.2
10,000-50,000	2.2-1.8
50,000-80,000	1.8-1.7
>80,000	<1.7

Source: Ministry of water and energy water supply module for urban, 2006

In most developing countries the maximum hour water demand is happen during morning and evening time over 24 hours, because in these times most people use water for bathing, washing and cooking purpose. Therefore, 'peak hour demand is the highest demand of any one hour over the maximum day. And it represents the hourly variations in water demand resulting from the behavioral patterns of the local population'(Dessie Tibebe, 2017).

$$Qhour = PF * Q\ average \tag{2.8}$$

Where, Q_{hour} = Peak hour demand (m3/s)

PF = Peaking factor between maximum hour and average day demand

Q_{avg} = Average day demand (m3/s)

ix. Demand diurnal pattern and multipliers factors

System demands vary throughout the course of a day and on a day-to-day basis. The Diurnal Demand Curve below illustrates the typical fluctuations in demand throughout a given day.

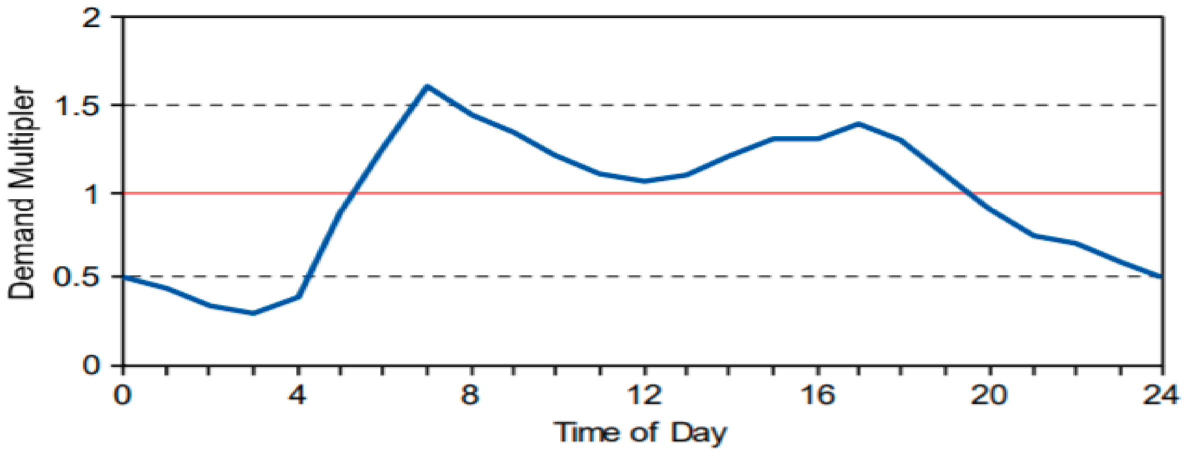


Figure 2.3: Demand diurnal pattern and multipliers factors

Source*:* Design and Analysis of Rural Water Supply System, Mehta, V. N. (2019)

The variations in water usage for water supply systems typically follow a 24-hour cycle. However, in reality, water demand varies over time and for extended period simulation to reflect the dynamics of the real system, these demand fluctuations must be incorporated into the model and it requires both baseline demand data and information on how demands vary over time.

2.8. Non Domestic Water Demand

This water demand is determined systematically and which can be broadly classified in to the following major categories: Institutional water demand, Industrial water demand and Commercial water demand(Vieira *et al.*, 2008, Datwyler, 2012, Anore, 2020).

a) Commercial water demand

This type of demand is the water furnished to commercial establishments (hotels, bars, butchery, miscellaneous shops, metal works, video house, vegetable sells shops, grinding mills, beer and soft drinking distributers, cloth toilers, teashops and restaurants). This quantity can vary considerably with the nature of the city, number and type of commercial establishments which is 10% of the total domestic water demand(Bhoyar and Mane, 2017,Anisha *et al.*, 2016, Bhatt and Paneria, 2017).

b) Industrial water demand

The water demand for industrial water demand was generally assessed separately. In case of Jaldu town, some categories of industries were included in domestic demand. The industrial water demand for Jaldu town is 5% of the total domestic demand of the future year(Mala-Jetmarova, Sultanova and Savic, 2018, Chaudhari *et al.*, 2017). However, water demand for large industries is expected to have their own water supply system. Hence future industrial water demand is not considered at this stage.

c) Institutional Water Demand

The water required for schools, hospital, health center offices, government offices and services, religious institutions and other public facilities is classified as institutional water demand which were 15% of total domestic water demand(Mavi and Vaidya, 2018; Framework, 2019; Kanownik and Policht-latawiec, 2019).

d) Fire Fighting Water Demand

Firefighting is a quantity of water required for fighting a fire outbreak. The quantity of water required for firefighting purpose is a function of population, but within minimum limit(Pandya, 2019). The quantity of water needed to extinguish fire depends up on population, contents of Buildings, density of buildings and their resistance to life (Wannapop, 2019). In our case the firefighting water requirement is taken care of by increasing10 % of the volume of storage reservoir can be meet from the storage but not from the sources(Pandya, 2019; Wannapop, Jearsiripongkul and Jiamjiroch, 2018). Therefore, the water required for firefighting shall be meeting by stopping supply to consumer for required time and directly it for firefighting purposes.

2.9. Water Demand Modeling

The most common method of allocating baseline demands is a simple unit loading method(Mala-Jetmarova, Sultanova and Savic, 2018; Pandya, 2019). This method involves counting the number of customers (hectares of a given land use, number of fixture units, or number of equivalent dwelling units) that contribute to the demand at a certain node, and then multiplying that number by the unit demand (for instance, number of gallons/ liters per capita per day) for the applicable load classification (Lencha, 2012;Ramesh, Santhosh and Jagadeesh, 2012). Therefore, average day demand was used to estimate the baseline demand and other demand in the water distribution system including unaccounted-for water. Hence, most modelers determine the water demand analysis of a given town by applying baseline demand to a variety of peaking factors and demand multipliers(Olbasa, 2017; Sahilu and Chaka, 2017). These demands can be determined by applying a multiplication factors or a peaking factor. Multiplication/ Peaking factors from average day to maximum day tend to range from 1.2 to 3.0, and factors from average day to peak hour are typically between 3.0 and 6.0 , but these values must be determined based on the demand characteristics of the system at hand'(Sahilu and Chaka, 2017). Therefore, when more than one demand type is served by a particular junction, the total demand for a junction at any given time is equal to the sum of each baseline demand times with its respective pattern multiplier, and it is used in most software packages to assign a different pattern to the different components of the composite demand as per below (Ravi et al., 2019).

$$Qi\ t = \sum Bij\ Pij\ t \tag{2.9}$$

Where, $Q_{i,\,t}$, = Total demand at junction i at time t (m^3/s)

$B_{i,\,j}$ = Baseline demand for demand type j at junction i (m^3/s)

$P_{i,\,j,\,t}$ = Pattern multiplier for demand type j at junction i at time t.

2.10. Hydraulic Performance analysis

As pre described under 2.11, hydraulic performance indices, is obtained from the penalty curves, are related to the elements of water distribution networking and the performance index of each node and pipe are generalized to the entire network. Thus, according to (Garoma, Kenasa and Jida, 2018) estimation of hydraulic performance indices is a function of the number of nodes, and pipes, nodal water demands, volume of the pipes.

Pump Capacity

A pump is device in which mechanical energy is applied and transferred to the water as total head, and these head is a function of flow rate through the pump(Bhoyar and Mane, 2017). While, the failures, location, size and capacity of pumps in water distribution are the major impacts for low flow or negative pressures arise in the system, and this can lead to intermittent water supply in the distribution system (Bhoyar and Mane, 2017). There are many reasons and factors why a pump is not performing well in a certain situation of water distribution system. But, as per (W. Bank, 2017), the important and possible reasons to less performing of pumps were identified an excessive of water pumped during periods of low consumption is stored in elevated tanks or directly without storage. For a power, failure would mean complete interruption in water supply. When water is pumping using electrically, the peak power consumption of water plant is likely to occur during high current consumption and this increases power cost. While distributing water adequate pressure, flow rate from source to all customers become issue of demand.

Accordingly, the performance of these centrifugal pumps is a function of flow rate, and is described by the following four parameters listed as below(Ramesh, Santhosh and Jagadeesh, 2012); Typically, only the head characteristic curve is needed for modeling; however, some models determine energy usage at pump stations as well as flow and head. To determine energy usage, the model must convert the waterpower produced by the pump into electric power used by the pump. This conversion is done using the efficiency relationships summarized below'(Ramesh, Santhosh and Jagadeesh, 2012).

$$\text{Pump efficiency } (\%) = (\text{Water Power}) / (\text{Pump Power}) \tag{2.10}$$

Pump power refers to the brake horsepower on the pump shaft, or the amount of power delivered to the pump from the motor. While, water power is the amount of power delivered to the water from the pump and it computed using the following relationship(Pandya, 2019);

$$\mathrm{WP} = \mathrm{Cf} * \mathrm{QHp} * \mathrm{C} \tag{2.11}$$

Where, WP = Water power, Watts,

Q = flow rate, l/s

H_P = head added at pump, m,

C = Specific weight of water, 9810 N/m3 and

C_f = Units conversion factor, 0.001 for SI

2.11. Hydraulic Performance Index

The hydraulic performance indices, is obtained from the penalty curves, are related to the elements of water distribution networking and the performance index of each node and pipe are generalized to the entire network. Thus, according to (Garoma, Kenasa and Jida, 2018) estimation of hydraulic performance indices is a function of the number of nodes, and pipes, nodal water demands, volume of the pipes were described in equation 3.6 for performance index for pressure junction and in equation 3.6 performance pipeline velocity. Hence, in both equation is depending on total number of networking nodes, node demands, and total number of pipeline and volume of the pipes. According to the standard codes(Lukubye and Andama, 2017), the values of Hdes and Hmax are considered 30 and 50 m, respectively. In addition, the values of V_{min}, V_{max}, $V_{opt,}$ and Voptu are defined 0.3, 2.5, 0.8, and 1.2 m/s, respectively. The desired values of pressure and velocity in these approaches are based on the defined standard values in Iran (Hunde *et'al.*, 2020) and hence these approaches is chosen because the authors evaluate the applicability of the proposed method for a part of System Dynamic framework methods in water distribution networks.

According Hunde et'al., (2020) the HPI is estimated from both System Dynamic framework methods as described in Figure 2.4 and from penalty curve in Figure 2.5.

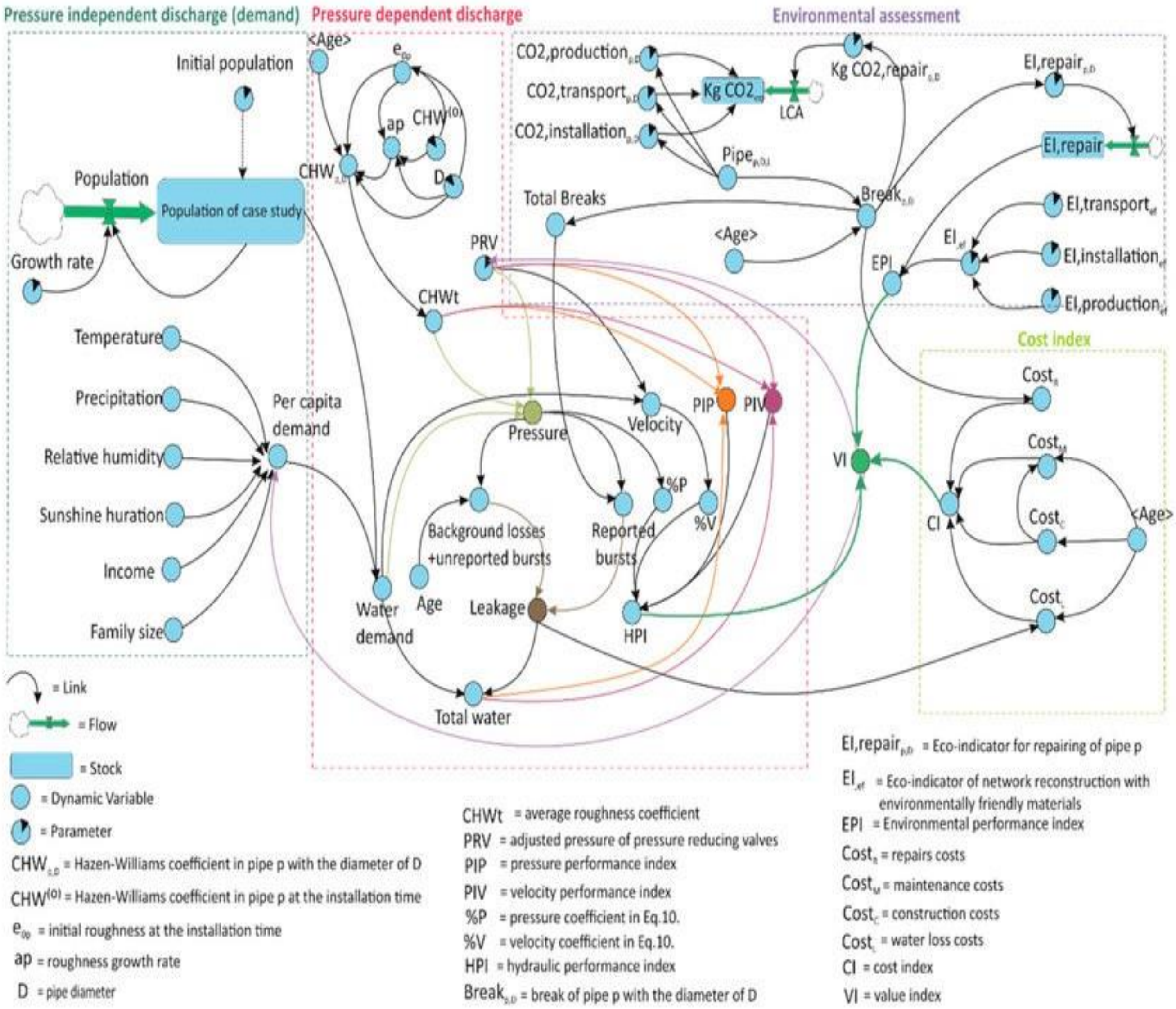


Figure 2.4: Hydraulic performance index by System Development Dynamic framework

Source (Lukubye and Andama, 2017) Hunde *et'al*., (2020)

However, it is worthwhile to mention that the Developed System Dynamic (DSD) framework makes it possible to consider other performance indexes and standard values for pressure and velocity and enables managers to evaluate their systems based on the defined boundaries in different countries without changing the overall method. In the SD model, due to the changes of pressure and velocity during the simulation time, PIP and PIV are changing in each time step. Since these indices are depended on the required demand and volume of the pipes, they would not depict the hydraulic performance of the whole system completely. Thus, HPI is introduced, which is dependent on the average pressure and average velocity of WDNs in the SD model, to combine the hydraulic variables of the whole system and WDNs' components (pressure, velocity).

Average values of the pressure and velocity are simulated in the SD model based on the variables affecting the system and also the proposed penalty curves by (Bhatt and Paneria, 2017) were employed to develop a hydraulic index in the SD framework. As shown in Figure 1, these curves indicate the different performance levels against the flow velocity in pipes and the pressure of nodes. The value of one shows the excellent level of performance and ≥0.75, ≥ 0.5, ≤ 0.25 describe the suitable, acceptable, and unacceptable performance of the system, respectively. In Figure 1a, Hdes is the minimum suitable pressure for which the demand is satisfied. The values of H_1, H_2, and H_3 are the pressures in which the outflows are equal to 0.25, 0.5, and 0.75 of the required nodal demand and are considered as H_2 = 1 /4Hdes, H_1 = 1/ 16Hdes, H_3 = 9 /16Hdes as shown by Vmax and Vmin, respectively. Furthermore, the domain of the optimum velocity is indicated by V_{opt} and (Ganjidoost, 2016).

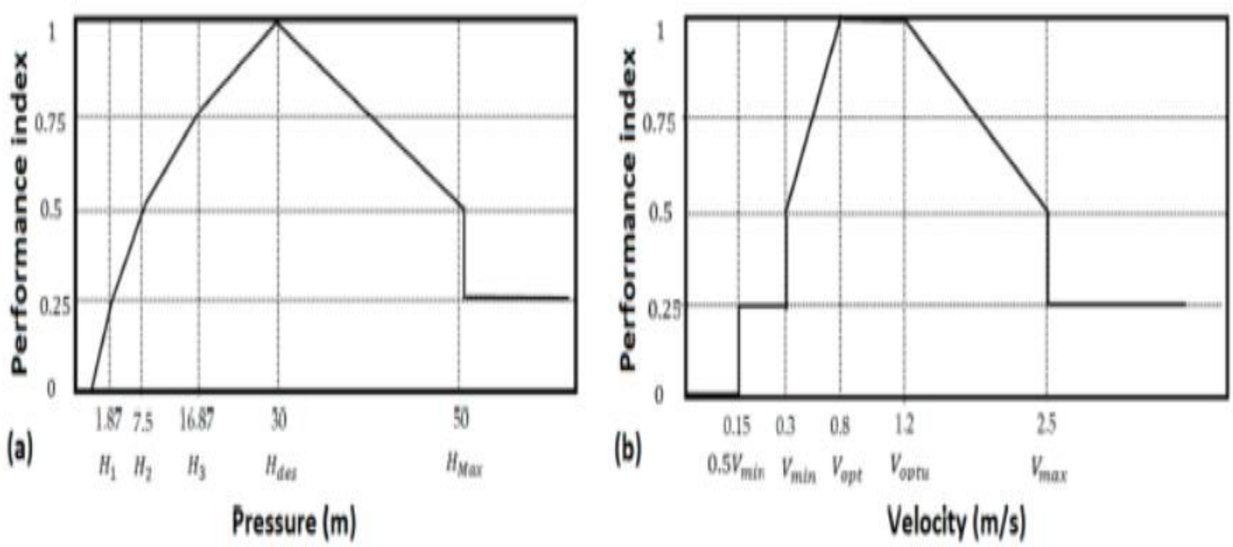


Figure 2.5: Performance penalty curves for: (a) pressure index and (b), velocity index based on the standard codes.

Source: (modified penalty curve hydraulic perfromance index , Capt *et al.*, 2021, Anore, 2020).

2.12. Hydraulic Performance indicators for water supply services

Performance is the degree to which infrastructure provides the services to meet the community expectations and it is a measure of effectiveness, reliability, and cost (Sahilu and Chaka, 2017). The performance of a water supply system depends on efficient and reliable working of all functional components including water resources, physical assets, operational activities, personnel, and environmental and financial activities.

The performance of a water supply system is evaluated to indirectly estimate the conditions and rehabilitation needs to ensure continuous and reliable working of these components of a water supply system during their entire service life before the occurrence of a failure(Anisha *et al.*, 2016). water supply system may face several problems associated with its continuous aging process, pressure fluctuation, water loss, water quality deterioration and so on, however to operate and maintain a water supply system at its maximum possible efficiency at a reasonable cost is one of the prime objectives of water utility. The performance of a water supply system can be assessed by selecting suitable indicators as indicated in Table 2.10.

Table 2.10: Performance indicators for water supply services (Husnain, et al., July 2013)

Items No	Indicators Categories	Description of sub group of performance Indicators
1	Water Recourse	✓ Water resources availability ✓ Reuse for Multipurpose
2	Personnel	✓ Personal data ✓ Personal per function ✓ Technical services personal per activity ✓ Personal health and safety ✓ Personal qualification and training
3	Physical	✓ Treatment ✓ Storage, transmissions and distribution ✓ Metering coverage ✓ Automation and control
4	Operation	✓ Inspection and maintenance of physical asset ✓ Electrical and signal transmission equipment's ✓ Mains valves and services connection rehabilitations ✓ Water losses ✓ Failure ✓ Water metering efficiency
5	Quality of services	✓ Coverage ✓ Public taps and standpipes ✓ Pressure and continuity of supply ✓ Quality of supply water ✓ Customer complains
6	Financial and economics	✓ Revenues, cost, investments, water losses ✓ Composition of running costs per main function of the water under taking ✓ Average water charges ✓ Efficiency indicators ✓ Composition of capital cost ✓ Profitability

The goals of term performance of a system are almost universally recognized, and it was commonly taken to consensus that the system should satisfy the demand of almost all customers (Mehta, 2019). Indirectly, it is mean that the system provides sufficient flows with adequate pressures and acceptable quality. The utilities recommended also are normally selected based on satisfying the requested need holding that it is sustainable and within reasonable economic limit (Wannapop, 2019). Therefore, any inconveniences in the quality and quantity of supply through the system are considered as poor performance. However, once one goes beyond general statements and attempts to flesh these issues out in more quantitative detail, a great deal of variation is observed in specific way systems are assessed and evaluated. It is helpful to classify performance based on physical and chemical characteristics of the supplied water into two primary aspects of quantity and quality. Meanwhile quantity of supplied water can be measured based on two major physical characteristics of supplied water, as quantity of pressure and quantity of outflows in the service life (Sadeghi, 2018).

2.13. Water distribution network simulation

The term simulation generally refers to the process of imitating the behavior of one system through the functions of another and used to predict system responses to events under a wide range of conditions without disrupting the actual system (Salunke, 2018). Using simulations, problems can be anticipated in proposed or existing systems, and can be evaluated before time, money, and materials are invested in a real-world project'(Vieira *et al.*, 2008), As per (Bhaskar *et al.*, 2017); in water distribution networks the most basic type of model simulations is either steady-state or extended-period simulation.

i. Steady state simulation

Represent a particular view of point in time and are used to determine the operating behavior of a system under static conditions. It computes the hydraulic parameters such as flows, pressures, pump operating characteristics, and others by assuming that demands and boundary conditions were not change with respect to time(Salunke *et al.*, 2018). In general, this type of analysis was used to determining the short-term effect of demand conditions on the system (Ganjidoost, 2016).

ii. Extended period simulation

The extended period simulations are determining the dynamic behavior of a system over a period of time, and it analyze the system on assumption that the hydraulic demands and boundary conditions were change with respect to time(Bhoyar and Mane, 2017). Hence, extended period analysis used to evaluate system performance over time and allows the user to model pressures and flow rates changing, tanks filling and draining, and regulating valves opening and closing throughout the system in response to varying demand conditions and automatic control strategies formulated by the modeler.

Therefore, regardless of project size, mode based simulation can provide valuable information to assist an engineer in making well-informed decisions’ (Capt *et al.*, 2021).Since each simulation process pressure junction and pipe line velocity has great roles in this systems of modeling.

a) Low pressure

There is pressure loss by the action of friction at the pipe wall, but its magnitude also dependent on the water demand, properties of the fluid that is passing through the pipe, the speed at which it is moving, and the internal roughness of the pipe, pipe length, gradient and diameter of the pipe(Chaudhari *et al.*, 2017). Such situations may occur where there are properties on high ground, remote properties at the end of long lengths of pipe, demands that are greater than the design demand, pipes of inadequate capacity (too small diameter), rough pipes equipment failures such as pumps and valves. In general, poor pressures tend to be caused by inadequate capacity in a pipe or pump, high elevations, or some combination of the two (Framework, 2019; Chambers, *et al.*, 2004). Therefore, one of the hydraulic integrity is maintaining adequate water pressure inside the pipe. Hence, the water utilities should achieve a high degree of hydraulic integrity through a combination of proper system design, operation, and maintenance along with good monitoring.

b) High pressure during low demand conditions

High pressure during low demand conditions can cause pipe bursting, leakage and large amount of water losses through the distribution networks. Therefore, when dealing with high pressures, pressure reducing valve, should be used to reduce and regulate pressure in the system (Framework, 2019). Accordingly, pipes and pumps must be sized to overcome this problem and to provide acceptable pressure in the system. Although, sizing of control valves based on the desired flow conditions and pressure differential is vital (Mavi and Vaidya, 2018).

Table 2.11: Operating pressures in the distribution network (MoWR, 2006)

Pressure	Normal condition	Exceptional condition
Minimum	15m	10m
Maximum	60m	70m

c) Pipe line Velocity and Head loss

According to MoWR Urban Water Supply Design Criterion Water, (2006) velocities shall be maintained at less than 2m/sec, except in short sections & for pumps. Velocities in small diameter pipes (<DN100) may need even lower limiting velocities (Wang, 2020). A minimum velocity of 0.3 m/sec can be taken, but for looped systems there are also pipelines with sections having velocity <0.1m/sec whereas, the head loss is related to velocity and pipe roughness hence, the maximum head loss with therefore be governed by the maximum velocity criterion (Jagadesh, 2016). Experience shows that a pipe designed to flow at a velocity between 0.6 and 2 m/sec, depending on diameter, is usually at optimum condition (head loss versus cost). The shortest sections, particularly at special cases, at inlet and outlet of pumps, may be designed for higher velocities. Minimum static head is 20 m, which can supply a 4 Storey building from the distribution system and the maximum static head within a pressure zone was limited to 80m (Kanowink, 2017). Minimum dynamic head was established at 10 m at maximum velocities of major transmission mains < 2.5 m/s and maximum velocities of distribution mains < 2 m/s at the minimum velocities range 0.1- 0.3 m/s within the system (Kanowink, 2017).

2.14. Water loss in distribution network

Water losses occur in all water distribution networks but it can be variation of time, location and volume of losses occurred(Mehta and Joshi, 2019). Thereby, the volume of this losses reflects the capacity of water authorities to manage their distribution networks (Hajibabaei, Nazif and Sitzenfrei, 2019) and hence the water losses may be either real and apparent losses. Moreover, to most water utilities, the level of Non-Revenue water is a key performance indicator of efficiency and utility managers should use the water balance to calculate each component and determine where water losses are occurring. By quantifying NRW from the water balance concept, volumes of lost water into system can be calculate and they was then prioritize and implement the required policy changes and operational practices which lead to the proper understood and take the required actions'(Anisha *et al.*, 2016). Therefore, the water balance can guide water loss estimation in the distribution system while also indicating the level of accuracy of the Non-Revenue Water calculation.

a. Physical water loss

Physical losses, sometimes called 'real losses', are the annual volumes lost through all types of leaks, bursts, and overflows on mains, service reservoirs and service connections up to the point of customer metering. So, utility managers must be verifying the physical loss assessment of towns water distribution system'(Mala-Jetmarova, Sultanova and Savic, 2018).

b. Leakage from transmission and distribution mains

Leakages occurring from transmission and distribution mains are usually large in volume(Mavi and Vaidya, 2018). Thus, considerable volume of water is lost through bursts, leaking pipes, joints, valves and fittings of distribution system components. These causes are usually as result of age of the installations, bad quality of materials used, and poor workmanship. Although these factors were lead to reduction of pressure in the distribution network and intermittent in water supply (Pandya, 2019).

c. Leakage from transmission and distribution mains

Leakages occurring from transmission and distribution mains are usually large in volume(Mavi and Vaidya, 2018). Thus, considerable volume of water is lost through bursts, leaking pipes, joints, valves and fittings of distribution system components. These causes are usually as result of age of the installations, bad quality of materials used, and poor workmanship. Although these factors were lead to reduction of pressure in the distribution network and intermittent in water supply (Pandya, 2019).

d. Leakages from reservoirs and storage tanks

Leakage and overflows from reservoirs and storage tanks are easily quantified (Bhoyar and Mane, 2017). By observing overflows, utility experts can estimate the duration and flow rate of the events. While, most overflows occur at night when demands are low, therefore it is essential to undertake regularly night observations. 'Observations can be undertaken either physically or by installing a data logger which record reservoir levels automatically at preset intervals. Also, leakage from tanks is calculated using a drop test were the utility closes all inflow and outflow valves, measures the rate of water level drop, and then calculates the volume of water lost'(Wang, 2019).

e. Leakage on service connections up to the customer's meter

This leakage is more difficult to identify and it covers the greatest volume of physical losses. So that, utility experts can calculate the approximate volume of leakage in service connections by deducting the mains leakage and storage tank leakage from the total volume of physical losses(Hunde and Ing, 2020).

f. Commercial loss

Commercial loss is also referring to as apparent losses, and it consist of unauthorized consumption, all types of metering inaccuracies and data handling errors. It also includes water that is consumed but not paid by the users(Islam *et al.*, 2016). In the developing countries, metering inaccuracies (mainly under recorded problem) and illegal users of water within the distribution system is the common problem of water losses. Whereby, they contribute large coverage to apparent losses, so the levels of these losses were one of the significant concerns in developing country water distribution systems(Riis, 2016). Therefore, 'Apparent losses can amount to a large volume of water than physical losses and often have a greater value, since reducing apparent losses increases revenue, whereas physical losses reduce production costs. For any profitable utility, the water tariff will be higher than the variable production cost and sometimes up to four times higher. Thus, even a small volume of apparent loss will have a large financial impact'(Peng and Mayorga, 2016).

g. Non-Revenue water

According to the above water balance classification, Non-revenue water (NRW) is the total amount of water losses in the system from the water treatment plant outlet meter to the customer's meter and it consists of real loss and apparent losses (Dacomber, 2018). Thus, it is described as the difference of total amount of water production and authorized consumption figure.

$$\text{NRW} = \text{System Input Volume} - \text{Billed Authorized Consumption} \tag{2.12}$$

Unaccounted-for-water also expressed as a percentage and, has generally evaluated as the amount of water produced minus the metered customer use divided by the amount of water produced and multiplied by 100(Aswale *et al.*, 2015).

$$\text{Unaccounted for water} = \frac{\text{Water produced} - \text{Metered water used}}{\text{water produced}} * 100 \tag{2.13}$$

2.15. Performance indicator for physical loss

As per (Aswale *et al.*, 2015); The Infrastructure Leakage Index(ILI) is an excellent indicator of physical losses. Thus, the International Water Association developed the index, and the American Water Works Association and Water Loss Control Committee were recommending this indicator. Therefore, ILI described as the ratio of Current Annual Volume of Physical Losses (CAPL) to Unavoidable Annual Real Loss (UARL).

$$\text{ILI} = \frac{\text{CAPL}}{\text{UARL}} \tag{2.14}$$

Where, the ILI has no units and thus facilitates comparisons between utilities and countries that use different measurement units. According to IWA, Unavoidable Annual Real Loss (UARL) is also called the Minimum Achievable Annual Physical Losses (MAAPL); and its formula have been converted to a format using pre-defined pressure for a practical use as follow (G, A and N, 2016).

$$\text{UARL}\left({}^{\text{litres}}/_{\text{day}}\right) = 18 * \text{Lm} + 0.8 * \text{Nc} + 25 * \text{Lp} * \text{P} \tag{2.15}$$

Where Lm = mains length (km);

Nc = number of service connections;

LP = total length of private pipe, property boundary to customer meter (km); and

P = average pressure (m)

The ratio of the CAPL to UARL, or the ILI, is a measure of how well the utility implements the three infrastructure management functions. Although a well-managed system can have an ILI of 1.0 (CAPL = UARL), the utility may not necessarily aim for this target, since the ILI is a purely technical performance indicator and does not take economic considerations into account' (Vidigal, 2008)In general, the concept of infrastructure leakage index is identifying how well a distribution network is managed to control physical losses. Therefore, according to (Ramesh, Santhosh and Jagadeesh, 2012); the ILI target matrix shows the expected level of physical losses of countries at differing levels of network pressure. Hence, the water utility experts can use the matrix to guide further network development and improvement.

2.16. Causes of water loss in distribution network

In most of the developing regions, the design of water distribution systems is based on the assumption of direct supply, although most of these systems are intermittent systems which result in severe supply, insufficient pressure in the distribution system (pressure losses in several areas in the network), inequitable distribution of the available water and very short duration of supply (Hussni & Zyoud, 2003). However, the purpose of hydraulic integrity in the water distribution system is to supply water at adequate/acceptable pressure and flow. According to(Vieira *et al.*, 2008; Datwyler, 2012) the most common factors for intermittent water supply and loss of hydraulic integrity in the distribution system are, are low and high pressure junction.

CHAPTER THREE

3. MATERIALS AND METHODS

3.1. Description of the Study Area

Jaldu town is found in Oromia Regional State along Addis Ababa-Gindbrat asphalt road at about 72 km away from Addis Ababa. The town is located between geographical coordinates of 8° 31‟18.90” N to 8°32‟25.46” N and 38 $^{\circ}$37‟2.93” E to 38^{o} 38‟19.03” E (Dessisa, 2018).

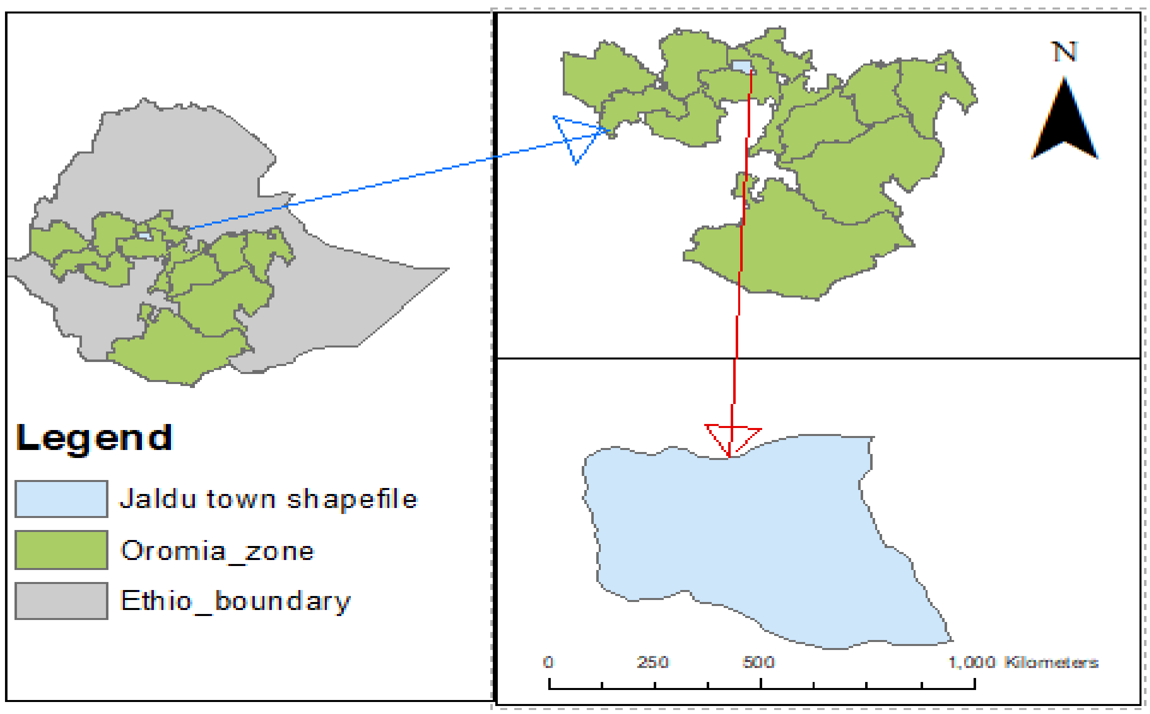


Figure 3.1: Map of the study area

As per described the topographic and climatic condition, the mean annual temperature of Jaldu town is grouped under the following groups i.e., the town has altitude in the range of 1500 – 2300 m and average temperature in the range of 16.5-19.10^{o} C as shown in Table 2.6.

3.2. Materials and tools used for the study

The study has used different material and tools in order to achieve the specific objective of the study as described below:

1) Pressure gauge meters’ / pressure meters

Measurement of pressure is a key point of water distribution network since it is a main importance for water distribution operation and maintenance. The pressure gauge should be installed in easily accessible places, so that it is convenient to read and to maintain a proper working condition. The most common pressure gauge used in water supply and distribution service is the Bourdon gauge, in which the primary element is an elastic metal tube that was used for calibration and validation.

2) Geographical Positioning System (Garmin72 tool)

This tool is used to collect the required elevation data during pressure reading, and used to collect the required elevation data, northing and easting of junction, tank, pump and air release valve. The use of handheld geo-referencing devices (global positioning system) and through pacing i.e., walking at a normal gait and counting the number of steps to cover measurement of position and velocity of ground, sea, air and space objects. Global positioning system hardware determines points in geographic coordinates, elevation and the name of specified location of junction pressures.

3) Water GEMS v8i

Analyzing the existing water distribution system using Bentleys water GEMS, hence the existing water distribution system is simulated through construct of a model using Bentley water GEMS. In order to assess the hydraulic performance of the distribution network some parameters were required like flow velocity and pressure. The analysis is beginning by feeding the diameter of distribution pipes in to software and the pressure, velocity and head loss are in the distribution system. The pressures were measured throughout the water distribution system to monitor the level of service and to collect data for use in calibration. In this case, for this study its used for the following activities as shown: Geospatial model building in the water distribution network and performing steady state as well as extended period simulations, to determine velocity and nodal pressure, hydraulic grade lines, and it helped in analyzing the entire network system, visualized the effects of constituent components and parameters as well as the pressure and velocity, at each node is detected

4) ArcGIS 10.1Vi

This tool was used for delineation of the study area, to display the overlapped shape file of the distribution network on the topographic map of the town. This software can help the users to give simply a location and descriptive data to create maps, tables and charts to apply, in other words ArcGIS software that provides location information to build a complete system. This application is one of the famous and powerful American company ESRI products in the field of geographic information systems applications. The ArcGIS software use GIS that enable users to simply spatial data and descriptive data to create maps, tables and charts to apply, in other words ArcGIS software that allows the construction of a full system provides location information. While, Microsoft Excel sheet were used to organize elevation data, to calculate a repeated work of nodal base water demand requirement of distribution network simulation and for manual pressure validation work.

3.3. Study design flow chart

For this research thesis, an integration of Arc GIS, Auto CAD and Water GEMS tool for modeling hydraulic parameters together with an artificial network acting as the decision support system are used to find the effective failure model for a particular in water distribution system. Therefore, In order to obtain the specific objective of the study, the researcher were investigated the appropriate procedures to gain the results of study as described in below Figure 3.2.

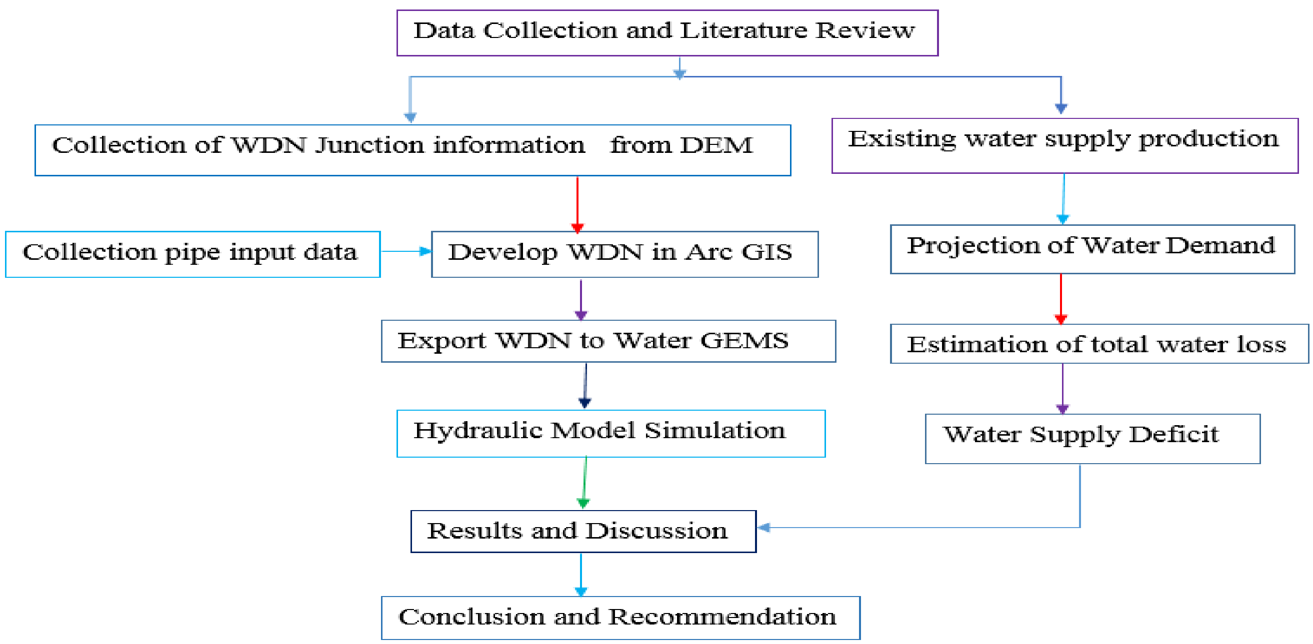


Figure 3. 2: Study flow diagram

3.4. Sources of data and Data Collection

a) Source of data

The study is used, the primary data as data source which were gained from pressure reading, elevation surveying and by made of discussion with water utility staff members to obtain additional relevant information on the subject matter. While, secondary data source was collected from different literature reviews, design report, the town water supply service office existing documents and annual reported papers. For hydraulic analysis in the software all, the required input data was collected from water treatment plant. Pipe data such as pipe diameter, C-value and length are assigned to the network. Input for nodes is elevation, water demand and time pattern. Pump head and flow are required data for the construction of pump curve. Layout of the water supply system in Auto CAD file is also an essential input. Both primary and secondary data were used to process the study in the realistic situation and techniques to get the required information.

b) Primary data Collection

This data can be collected from different perspectives of filed surveying (measuring the lowest pressure junction zone and higher-pressure junction, co-ordinate systems, elevation of nodes), interview with local administrative and observations of pipe material and leakages. Among this data collected, the researcher was used to display three lower pressure junction and three high-pressure junctions recorded for the validation of result measured near the corresponding location using pressure meter. Additionally; this data was collected from customers through household survey, face-to-face interview with local administrative about the real water supply production, amount of total water loss orally and system of distribution as described in Appendixes. And also from field surveyed data of collected on the pressure junction in the water distribution network of pipes at coordinates (x, y), nodal water demands estimated from per capita unit loads and water pressures at each junction from starting node to stop node of pipe connectivity survey. During this collection of data pressure gage, meter and GPS Garmin 72 tools used which to locate the latitude and longitude of the selected main node of the system.

On the other hands each co-ordinate of the selected main node of the system, elevation of pressure junction and location of the specified points is measured by using meter GPS Garmin 72 tools ,the lower pressure junction recorded is recorded in (J-58, J-61, J-7, J-8, J-83, J-133, J-30 J-33, J-26, J-76, J-158, J-41, J-39 J-156) are some of the nodes.

In the same manner, the highest junction pressure taken during as the primary data collection is identified. From the whole water distribution layout systems, in which the highest-pressure junction recorded in (J-103, J-104, J-191, J-111, J-121, J-186, J-150, J-190, J-161, J-156) are some of the over water pressures recorded since steady state run simulation at minimum day water demand as primary data used since calibration and validation of the study.

c) Secondary data collection

This data can be collected from water supply and pumping data, daily and monthly water production and consumption data, water supply network data. The data can be included, pipe length, size of pipe, elevation of each node, unit demand of each node and number of users at each mode of service (house, tap, yard and shared connection users). Each of this data collection contains numerous elements, for instance water supply distribution data (elevation of the distribution system, map of water distribution network, water distribution network layout, pipe data like material type, size and length, tanks and valves in the network). Since this data collection, most it was used as the input data for modeling the distribution network as shown in Table 3.1.

Table.3.1: Input parameters for WDN for model simulation as secondary data

Components WDN	Input data used for model simulation
Source /Reservoir	Elevation Head /height Co-ordinate
Tank	Base Elevation, Initional Elevation Max. Elevation, Min. Elevation, and Tank Diameter
Pressure Junction	Elevation of junction, Co-ordinate of junction and junction demand
Pipe	Pipe length, Diameter, Material and Roughness coefficient
Pump	Elevation, Pump definition (max. operation, shut off, design discharge and head of pump, pump capacity and pump efficiency)
Valves	Elevation, Diameter, Valve type,

3.5. Existing water supply distribution networks

The existing Jaldu town water supply system was constructed in 1989 E.C but unfortunately, due to population growth and expansion of the town, currently there is scarcity of water. This schemes of water supply also used for the currently population by providing serval temptation of water supply deficits without any expansion and improvement of pipe material and distribution layout system.

This source of the water supply system is from Keta spring and borehole, which have a total yield of 19.6 Ls, and the existing system consists of a spring intake structure, collection chamber, transmission main, storage reservoirs and distribution network.

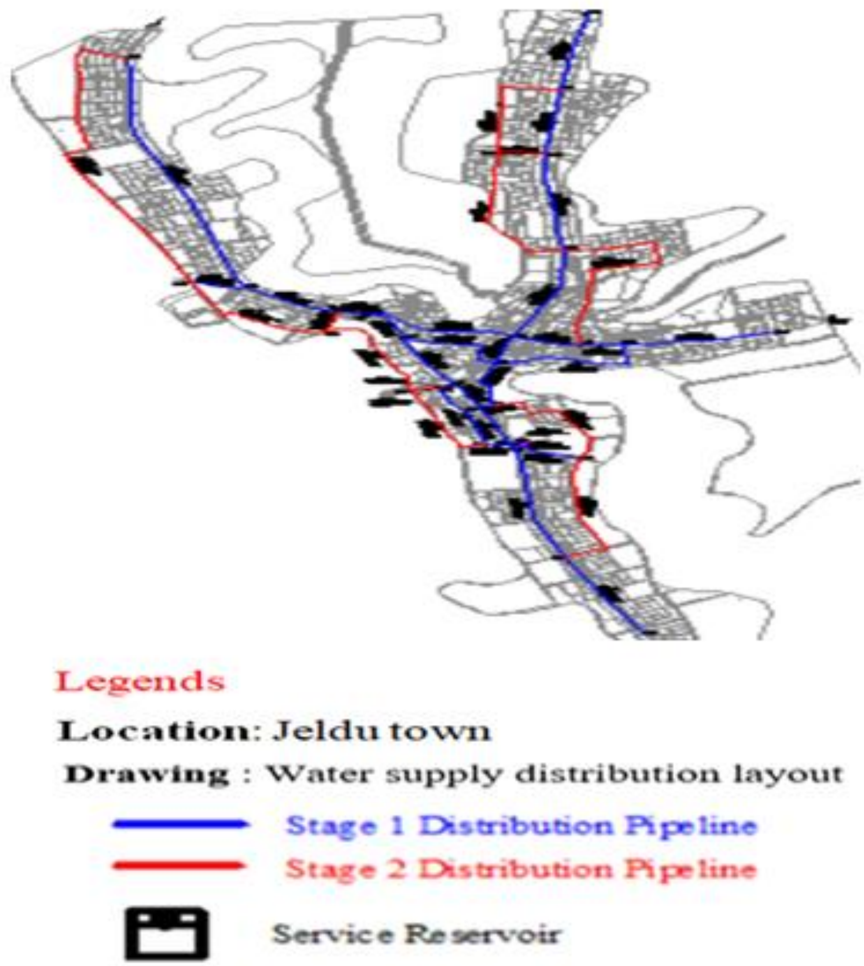


Figure 3.3: Existing water supply and distribution networks of Jaldu town

Source: Jaldu town Water supply and Sewerage Authority design document

3.6. Population projection

According to the CSA 2007 reports, the base population number of this town is 16970, and this population were including four rural kebeles, which was included by newly prepared master plan. Population projection has a paramount importance since it is the most important variable in all types of development planning at both macro and micro levels. The population of Jaldu town adopted from the structural plan study (2012) which is estimated to be 16,970 has been used as a base to project the population size of the planning period. As pre described in equation number (2.1, 2.2, 2.3 and 2.4) different population forecasting methods available as described in the literature that was used in this topic for population projection. However, their result varies from one method to another, so it is appropriate for particular town needs to consider overall current situations of the targeted town having the minimum error calculation among each method. Hence, by considering the standard level of town, in addition to the existing census results the population projection methods for the status were done as in Table 3.2 with considering minimum error among preferable.

Table 3.2: Methods population projection

Projection Methods	Formulas	Calculating error in each Method	Years/period
ECSA Methods	$P_n = P_O e^{kn}$	$\frac{\text{Actual popn} - \text{proj. popn}}{\text{Actual population}} \text{x}100\%$	2013-2035

Based up on the Table 3.2 the population projection can also be determined by population growth rate, this growth rate was different from time to time, and hence the Central Statistics Agency of Ethiopia calculated this population growth rate from 2008 to 2030 as justified below.

Table 3.3: Population growth rate

Years	2013	2015	2020	2025	2030	2035
Growth rate	4.60%	4.40%	4.20%	4.00%	3.80%	3.60%

Source: Central Statistics Agency of Ethiopia, 2014

3.7. Domestic Water Demand Projection

Estimating water demands for a particular town depends on the size of the population to be served, their standard of living and activities, the cost of water supplied, the availability of wastewater service and the purpose of demand. This demand can vary according to the requirement of the domestic water demand and non-domestic water demand. Accordingly, the water demand of town is calculated with due consideration of actual conditions of the town and pertinent to available data and domestic water demand is the amount of water needed for drinking, food preparation, washing, cleaning, bathing and other miscellaneous domestic purposes. The amount of water used for domestic purposes greatly depends on the lifestyle, living standard, and climate, mode of service and affordability of the users. The domestic water demand of the study area is including, determining population percentage distribution by mode of service and its future projection and adjustment of (climate and topographical location; socio-economic conditions)

a) Population percentage Distribution by Mode of Service

The percentage of population to be served by each mode of service will vary with time and this variation was caused by changes in living standards, improvement of the service level, changes in building standards and capacity of the water supply service to expand. Therefore, the present and projected percentage of population served by each demand category was estimated by taking the above stated conditions and by assuming that the percentage for the house tap connection (HTC), public taps connection (PTC), private yard connections (PYC) and yard tap connection (YTC) users will increase gradually during the project service period. Whereas, the percentage of tap users will dramatically reduce as more and more people will have private connections as the living standard of people and the socio-economic development stage increase as indicated in Table 3.5.

Table 3.4: Population Percentage Distributions by Mode of Service

Mode of Service	Percent of population served
HTC	1.50%
YTO	15.5%
YTS	12.6%
PT	70.4%
Total	100%

b) Per Capita Water Demand

The per capita water demand for adequate supply level has to be determined based on the basic human water requirements for various activities of demand category.

$$\text{Per Capital Demand} = \frac{Q}{P * 365} \tag{3.1}$$

Where Q: is the total quantity of water required by a town per year in liter

P: The population of the town

According to the design criteria prepared by Ministry of Water Resources (2006), Table 2.2 shows the per capita domestic water demand adopted for Urban Water Supply System for Stage I (2025) design Horizon. The study consideration the design standards of this Minster and assumed to almost constant consumption for the next 2025-2035 services of water supply project and used this guideline criterion's as shown in Table 2.4 in view of this, and as per the recommendation in the MoWR design criteria, the per capital consumption for each mode of services at the end of the design period was estimated.

c) Adjustment for Climate

As pre-described in Table 2.6 of literature the study area has a temperate climate according to agro-ecological classification of the country. The water consumption is less in area where the average temperature is low and high where temperature is very high. Accordingly, Jaldu falls under Mean Annual Temperature between 15 & 20, but the climatic factor of 1 is considered.

d) Adjustment for Socio-economic Conditions

The majority of the people in Jaldu town are driving their livelihood by undertaking agriculture and trade. As indicated in Table 2.7 the town is categorized as Group C -towns having normal living standards of the country and hence a socio-economic adjustment factor of 1.00 has been used for this study.

e) Water Demand Multiplicity Factors

i. Average Water Demand

The average daily water demand is the sum of the domestic, non-domestic and unaccounted for water that was used to estimate the maximum day & the peak hour demand. For this analysis, the projected average daily demand was determined using the most current average per capital consumption of water(Wang, 2019) described as the following formula.

$$\text{ADD} = \text{Popn} * \text{average per capita consumption of water} \tag{3.2}$$

$$\text{Where}: \text{ADD} = \text{average daily demand}, \text{popn} = \text{design population}$$

ii. Maximum Day Water Demand

The water consumption varies from day to day and the maximum day water demand was considered to meet water consumption changes with seasons and days of the week. The ratio of the maximum daily consumption to the mean annual daily consumption is the maximum day factor. The proposed maximum day factor usually varies between 1.0 & 1.3 as per the design criteria and hence, a maximum day factor of 1.2 was adopted. The maximum day demand was applied to determine source pumping requirement, size of rising mains and treatment system and this maximum day demand was used to determine treatment plant or wet well treatments.

iii. Peak Hour Water Demand

The peak hour demand is the highest demand of any one hour over the maximum day. It represents the daily variations in water demand resulting from the behavioral patterns of the local population. Experience clearly demonstrates that the peak hour factor is greater for a smaller population and Table 2.9 shows the recommended peak hour factors in relation to population size and according to this table, a peak hour factor of 1.9 is used.

3.8. Non-Domestic water demand projection

This demand is including several activities (industrial and commercial water demand, public, residential, firefighting etc.) of water consumption, but it depends up on size and level of the study area socioeconomic developments.

1) Institutional Water and Commercial Demand

In this town where there is sample supply of water, it is often the case that public service giving institutions like hotels, restaurants, offices, university, schools, etc. tend to consume large amount of water merely because they are able to afford better than other demand groups. Connection priority was also given to such institutions because of their economic and social significance in the town. It will not be reliable to estimate institutional & commercial water consumption from the existing water supply system of Jaldu town where there is uneven distribution of water in terms of flow and pressure. In the absence of adequate data, institutional and commercial demands are grouped together and termed ‘public’ water demand and are usually expressed as a percentage of the average domestic demand. For this study, the institutional & commercial water demand was assumed 20% of the average domestic demand.

2) Industrial water Demand

As per the urban water supply system design criteria, Ministry of Water Resource 2006, small-scale industrial enterprises was not categorized separately but have been included along with the domestic water demand. Even if there is no heavy industry, the researcher was considered micro and future Industrial parks plantation for the study area and hence allotted industrial water demand projection. Accordingly, the industrial demand has been considered 10 % of the average domestic water demand was considered for this study. This water demand can also be considered for the future expansion even if it was not present on the time being, since this may expanded on certain years and it have to consider for the future expansion of the industrial water demand consumption rates.

3) Institution and Commercial Demand

This water demand includes Institution, commercial buildings and commercial centers (office buildings, warehouses, stores, hotels, shopping centers, health centers, schools, temple, cinema houses, and railway and bus stations). In most the countries 10-30% of domestic water demand is considered as institutional and commercial water demands, but for this study 20% of average domestic water consumption is considered as commercial and institutional water demand.

4) Fire Fighting Demand

The firefighting water requirements is considered to be met by stopping supply to consumers and directing it for fire broke out purpose which is 5-10% of domestic water demand. This demand was estimated in different techniques, but for this research, its estimated 5% of the average domestic water demand.

3.9. Simulations and Performance analysis of the distribution system

a) Pump capacity and its efficiency

Pump is equipment is which deliver energy to the hydraulic system in order to overcome elevation difference and head losses due to pipe friction and fittings. For this study, raw water pump efficiency was conducted in order to determine the pumps capacity.

Therefore, using the finding the efficiency was assessed manually and computed as below;

$$\text{Pump Efficiency (\%)} = \frac{\text{QWater Power}_{\text{out maximum}}}{\text{Pump Power}_{\text{in}}} \tag{3.3}$$

The maximum capacity pump of delivering water to the distribution system was discussed as:

$$\text{Pump capacity} = \text{pump design capacity} * \text{effective pump operation time} \tag{3.4}$$

b) Steady-state simulations analysis

The model has been performed in steady state simulation analysis for the average daily demand, which is the demand at every node not changing throughout 24 hours of a day. It is required to run single period at the beginning of the simulation as to observe the model under snap shot situation. During normal and low flow periods in most water distribution systems, the HGL is relatively flat because the system's low velocities result in small head losses. This means that the effects of pipe roughness, demands, and closed valves are small, hence it is not recommended to start model calibration by adjusting pipe roughness. During normal flow conditions, the value of the HGL is primarily determined by water levels in tanks or by pump and/or pressure-reducing valve discharge pressures in pressure zones where there are no tanks with water levels that float on the system.

c) Extended Period Simulation

An extended-period simulation can run for any length of time, depending on the purpose of the analysis. The most common simulation duration is typically a multiple of 24 hours, because the most recognizable pattern for demands and operations is a daily one. Extended period simulation tracks a system over time, and it is a serious of linked steady state run and extended period simulation (links at average day demand). The need to run extended period simulation is because the system operations change over time demands vary over the course of the day as shown in Figure 3.6 for the variation of hours in 24 hrs.

Hydraulic Time Step: an important decision when running an extended period simulation is the selection of the hydraulic time step. The time step is the length of time for one steady-state portion of an extended-period simulation, and it should be selected such that changes in system hydraulics from one increment to the next are gradual. A time step, too large may cause abrupt hydraulic changes to occur, making it difficult for the model to give good results. Using an extended-period simulation model, we can simulate based on the peak, minimum and average day demands. In hydraulic simulation modeling, a distribution network was considered to be one in which all elements are connected to each other, every element is influenced by its neighbors, and each element is consistent with the condition of all other elements.

3.10. Hydraulic Performance Index

As pre described under 2.8, hydraulic performance indices, was obtained from the penalty curves, are related to the elements of water distribution networking and the performance index of each node and pipe are generalized to the entire network. According to (Chaudhari *et al.*, 2017) estimation of HPI is a function of the number of nodes, and pipes, nodal water demands, volume of the pipes as described (eqn 3.4, 3.5 and 3.6) for both case (pressure and velocity) hydraulic performance.

$$\mathrm{PIP} = \frac{\sum_{j\in Nj} Q_j^{req} * (\mathrm{PIPE_j})}{\sum_{j\in Nj} Q_j^{req}} \tag{3.5}$$

$$\mathrm{PIV} = \frac{\sum_{j\in NP} V_{ij} * (\mathrm{PIVE_j})}{\sum_{j\in NP} V_{ij}} \tag{3.6}$$

Where; PIP is the pressure performance index of the network, N_j is the number of the nodes, PIPEj is the pressure performance index of the node j, Q^{req} j is the required nodal demand of the node j, PIV is the velocity performance index of the network, NP is the number of the pipes, $PIPE_j$ is the velocity performance index of the pipe j, and V_j is the volume of the pipe j. Thus, according to (Capt *et al.*, 2021),HPI is introduced, which is dependent on the average pressure and average velocity of WDNs in the SD model, to combine the hydraulic variables of the whole system and WDNs' components (pipes, nodes) which is calculated as follows:

$$\mathrm{HPI} = \frac{\left(1 - \frac{\alpha + \beta}{Ni}\right) * \mathrm{PIP} + \left(\frac{1 - \gamma + \delta}{Nj}\right) * \mathrm{PIV}}{2} \tag{3.7}$$

Where ; HPI is the hydraulic performance index, PIP is the pressure performance index, PIV is the velocity performance index, Ni is the total number of the nodes, ἀ is the number of the nodes with the pressure of less than 30 m, β (beta) is the number of the nodes with the pressure of more than 50 m, Nj is total number of the pipes, ϒ (gama) is the number of the pipes with the flow velocity less than 0.8 m/s, and is the number of the pipes with the flow velocity exceeding 2.5 m/s. The ἀ coefficients' and β are calculated with respect to the average pressure and the coefficient's ϒ and δ are obtained based on the average flow velocity. Average values of the pressure and velocity are simulated in the water GEMS during simulation based on the variables affecting the system. In other words, these coefficients are representative of the average pressure and velocity of the system, while PIP and PIV is representative of the pressure of the nodes and the flow velocity of the pipes.

3.11. Performance Analysis of water loss

The water loss analysis in Jaldu was assessed at the town level based on the percentage of Non-Revenue Water that obtained from the total production and actual consumption. Using this data and equation below, the total Non-Revenue Water (NRW) in the system was calculated for each recorded year.

$$\text{NRW} = \text{System Input Volume} - \text{Billed Authorized (Consumption)} \tag{3.8}$$

In addition to non-revenue water loss there is also unavoidable water losses through the pipe line of distribution networks due to physical loss (leak flow, bursts of pipes etc.) and leak physical loss in the main was assessed base on the available data, and it was adopted by considering the minimum achievable annual physical losses (unavoidable annual real loss) in the system (Farley, et al., 2008).

$$\text{Unavoidable annual real loss } UARL\left(\frac{Litres}{Days}\right) = (18*L_m+0.8*N_c+25*L_p)*P \tag{3.9}$$

Where, Lm = mains length (km);

Nc = number of service connections;

Lp = total length of private pipe, property boundary to customer meter (km); and

P = average pressure (m)

3.12. Modeling scenarios

A Scenario is a set of Alternatives, while alternatives are groups of actual model data. Scenario and alternatives are based on a parent/child relationship where a child scenario or alternative inherits data from the parent scenario or alternative. The water distribution network in the continuous supply systems should be designed to with stands the range of pressures corresponding to the minimum and maximum supply conditions. Which means: at (average day demand (base demand), peak hour demand & low flow demand, (night flow demand).

3.13. Model calibration and validation

For model calibration and validation, effort data were collected from field selected sample locations and hence, the required collected data include pressure and nodal water demand but for each node, record was taken five times at different times in single days. Model calibration and validation were undertaken based on the different calibration standard criteria for hydraulic network and water quality modeling. Model calibration is the process of fine-tuning a model until it simulates field conditions for a specified time horizon to an established degree of accuracy. Therefore, model will not be hundred percent correct and to be calibrating it must be accurately simulating the observed data. Further, according to (Design and Enterprise, 2019); hydraulic model calibration is the necessary process of modeling and it is calibrated in order to have better confidence, understanding and identifying errors made during the model-building process.

Collecting pressures data throughout the water distribution system used to indicate the level of service. Pressure readings was done using pressure gauge commonly taken at pump stations, storage tanks, reservoirs, fire hydrants, home faucets, air release and other types of valves. However, different factors can contribute to deviation between model simulation and actual field data. Therefore, ‘calibration can be accomplished by adjusting only internal pipe roughness values or estimates of nodal demands until an agreement between observed and computed pressures and flows is obtained. In this calibration process, it was considered that the pipe length, diameter and material, nodal elevation, nodal demand, flow and head measurement at the source are obtained/measured reasonably accurately.

The model was calibrated by adjusting the NRW demand uniformly and the pipe roughness coefficients for the area of pipes by the same amount to minimize the summation of the sum of square of difference between the measured and simulated values of the pressure head at the observation points. Thus, if the observed pressure at the node is more than the modeled or computed pressure and the impact of increasing the roughness value for a particular area of pipes is to increase the pressure head at the observation node, then the pipe roughness value for that area of pipes needs to be increased. Hence, this calibration process assumed that the prepared data and predicted nodal demands were reasonably accurate and the model calibrated by adjusting only the pipe roughness coefficients for the group of pipes by the same amount to minimize the summation of the sum of square of difference between the measured and simulated values of the pressure heads at the observation points.

CHAPTER FOUR

4. RESULTS AND DISCUSSION

4.1. Evaluation of the existing water supply production

The source of water supply for the study is acquired from both surface water (keta spring) 6.4 L/s, from Borehole wells 13.2 L/s and hence, totally 19.6 L/s yields of water supply were produced. This amount of water supply were proposed to collect and distributing for the societies by using pump operation and electric power from source to water treatment plant. Thus, the amount of water supply reaped to the service reservoir by using the electric power 13.06 L/s of water was distributed or collected in 24 hours of the per day in working 18 hours were for about 9.8 L/s only. Hence, the total volume of water entered to the storage tank within 24 hours is 846.76 cubic meter per day by using the electric power distribution only. On the other hands when the electric power was off day`s the supply of water from source has stand by generator which is operated for only 18 hours per a day, supplying of water 447.12 metric cubic per day is limited. For this causes pump operation is requires which was generates 18 L/s by operating for 18 hours and hence, the total volume of water entered to the storage tank within 24 hours was 1293.88 cubic meters per day. Due to the interruption of the distribution, system only for about 399.64 cubic meters of water supply could not be stored into the storage tank, which is from the daily production of water decreases by 3.56 %. This condition is main consequence for the shortage of water supply distribution scarcity to satisfy the demand of the study area, and limited sources of water supply, which is unfit for the current level of rapidly growing of projected population on existing supply.

4.2. Population projection

As pre-described 2.5 and 3.6 parts of population projection, there are different methods available, but the study was considered the existing matter and minimum error of calculation used the geometric increase growth rate methods. In order to forecast the current population of the study area which is based on last population census report population and housing census report of 2007 which was prepared by Ethiopian Central Statistical Agency was accessed mainly to establish base population. For this study 4.7% to 3.2%, growth rate from 2014 up to 2040 was adopted as the coming year increase the population increase rate decrease. The population of the town in year 2015, according to projection from 2007 CSA report the base population of the study area is 16970.

Table 4.1: Past, present and projected population estimation in the study area

Years	2014	2020	2025	2030	2035	2040
Growth rate	4.70%	4.20%	4.00%	3.80%	3.60%	3.20%
Population	18603	24575	29387	35779	42564	46498

4.3. Water Demand Analysis

Analysis of water demand was typically evaluated based on the currently water consumption rate and future water supply production rate and estimating ongoing of sustainability of feeding rate without interruption of supply deficit. However, this water demand was used to for two different purpose domestic water demand and non-domestic water demand. Domestic water demand analysis is a consumption of water by population distribution of mode of service connections and the average daily per capita consumption was used to analyze the domestic water demand for the study area. However, the access of water supply may be evaluated using the amount of water consumed and the level of connection. For evaluating the amount of water consumption, the annual water consumption was converted to average daily per capita consumption using the population data of the study area.

4.4. Water demand analysis by mode of services and per capita water demand

As pre-described in Table 2.3 and 2.4, the per capita demand of water per mode of service of the study areas standard was 70, 40, 30 and 25 for Private house connections, Private yard connection, Private yard shared and Public taps urban respectively (MoWR, 2006). This can be expressed as the percentage of population distribution services, which changes in water demand annually. This service of water demand is varying from year to year and when same of their needs increase the other needs where decreasing. For instance, the private yard connection (50-70l/c/d) and Private house connections (25-30l/c/d) are in increasing order. For public tap users it is in decreasing (30 and 20 l/c/d) order for the year 2020 to 2025 and then it keeps constant for the rest of design period. The Private yard shared is decreasing throughout the design period. Based on these standards as a base line value, per capita water demand throughout the year was projected up to 2040 as shown in Table 4.2.

Table 4.2: Domestic water demand estimation by population distribution by mode services and per capita water demand

Years		2014	2016	2018	2020	2025	2030	2035	2040
projected Popn	No	18603	20315	22099	24575	29387	35779	42564	46498
Popn distribution by mode of Service %									
HC	%	0.03	0.03	0.05	0.08	0.09	0.10	0.125	0.15
YTS	%	0.14	0.15	0.2	0.25	0.26	0.27	0.3	0.32
YTP	%	0.115	0.13	0.18	0.23	0.25	0.25	0.27	0.29
PT	%	0.78	0.70	0.58	0.45	0.40	0.37	0.35	0.32
Popn- distribution service									
HC	No	446	508	1105	1843	2645	3578	5321	6975
YTS	No	2697	3047	4420	6144	7638	9839	12769	15112
YTP	No	2139	2539	3867	5529	7347	8945	11705	13717
PT	No	14510	14221	12707	11059	11755	13417	14897	14879
Per capita demand									
HC	l/c/d	40	45	50	60	70	70	70	70
YTS	l/c/d	20	22.5	25	27.5	30	30	30	30
YTP	l/c/d	25	29	30	35	40	40	40	40
PT	l/c/d	18	19	20	22.5	25	25	25	25
Domestic Water Demand	l/d	38647	435248	535900	721890	1002008	1238847	1596150	1862244
	m^3/d	386.50	435.20	535.91	721.91	1002.11	1238.81	1596.12	1862.12

4.5. Adjustments of Domestic Water Demand Projection

The average per capita domestic water demand for each year was computed by combining water demand by mode of service and population percentage distribution by mode of service for the year

2014 to 2040. After the per capita water demand for each mode of service has been determined, the adjustments for climate and socio-economic factors were assumed unit according to the town's design criteria. As indicated in Table 2.6 and 2.7 there is several factors (altitudes of the study area, climatic condition and socioeconomic standards) which determines the domestic water demands. The water consumption is less in area where the average temperature is low and high where temperature is very high. Jaldu has a temperate climate according to agro-ecological classification of the country with mean annual temperature is between 15 & 20 and hence the climatic factor of 1.0 is considered. The majority of the societies in this area are driving their livelihood by undertaking agriculture, small and medium trade. In this view, the town was categorized as Group C - towns having normal living standards of the country. Therefore, a socio-economic adjustment factor of 1.0 has been used for this study estimated in Table 4.3.

Table 4.3: Domestic water demand and its adjustment for each mode of service is presented

Items	Units	2014	2020	2025	2030	2035	2040
Domestic WD	l/d	386477	435248.9	535900.8	721890.6	1002008	1238847
	m^3/d	386.5	435.2	1002.11	1238.81	1596.12	1862.12
Multiplier factor for domestic water demand adjustment							
Climate condition		1	1	1	1	1	1
Domestic WD	l/d	386477	435248.9	1002008	1238847	1596150	1862244
	m^3/d	386.5	435.2	1002.11	1238.81	1596.12	1862.12
Socio-economic		1	1	1	1	1	1
Aveg. Adjust DWD	l/d	386477	435248.9	1002008	1238847	1596150	1862244
	m^3/d	386.5	435.2	1002.11	1238.81	1596.12	1862.12

4.6. Non-Domestic water demand projection

The evaluation of non-domestic water demands is interdepending on the domestic water demands, however in most of the areas this demand considered as the ordinary water demand. As pre described under 2.6 and 3.6 points, non-domestic water was estimated from the percentage of population projection by the mode of service (domestic water demand projected). Thus, as reviewed under those issues all non-domestic water demand was estimated from this domestic water by referenced percentage accordingly its requirement to the specified area. This demand has been estimated as for the commercial water demand (CWD) activities is taken to be 10% of domestic demand, for industrial water demand (IWD) 5% the domestic demand, for Fire Fighting Demand (FFWD) 10 % of the maximum day demand is included in sizing the service reservoirs as stipulated in the design criteria and Non-Revenue Water (unaccounted) (NRWD) 20% of domestic demand is used as indicated in Table 4.4.

Table 4.4: projection of non-domestic water demand for the study area

Years	Unit	Adjusted DWD	CWD10%	IWD5%	FFWD 5%	NRWD 20%	Ave. NWD
2014	l/d	386477.3	38647.7	19323.8	19323.8	77295.4	154590.9
	m^3/d	386.4773	38.6477	19.3238	19.3238	77.2954	154.5909
2016	l/d	435248.8	43524.8	21762.4	21762.4	87049.7	174099.6
	m^3/d	435.2488	43.5248	21.7624	21.7624	87.0497	174.0996
2018	l/d	535900.7	53590	26795	26795	107180.2	214360.3
	m^3/d	535.9007	53.59	26.795	26.795	107.1802	214.3603
2020	l/d	721890.6	72189	36094.5	36094.5	144378.1	288756.3
	m^3/d	721.8906	72.189	36.0945	36.0945	144.3781	288.7563
2025	l/d	1002009	100200.8	50100.4	50100.4	200401.7	400803.4
	m^3/d	1002.009	100.2008	50.1004	50.1004	200.4017	400.8034
2030	l/d	1238848	123884.7	61942.3	61942.3	247769.6	495539.2
	m^3/d	1238.848	123.8847	61.9423	61.9423	247.7696	495.5392
2035	l/d	1596150	159615	79807.5	79807.5	319230	638460
	m^3/d	1596.15	159.615	79.8075	79.8075	319.23	638.46
2040	l/d	1862245	186224.4	93112.2	93112.2	372449	744898
	m^3/d	1862.245	186.2244	93.1122	93.1122	372.449	744.898

Based up on the Table 4.4 of water demand projection indicates the domestic water demand is more required for the consumption purpose than the non-domestic water demand purpose. This means in the study area there was less or small industrial, commercial etc. of water consumption requirements. This average non-domestic water demand was estimated from domestic water demand and hence the total average water demand and annual domestic and non-domestic water demand is described in the below Figure 4.1 and basically based on the analyzed of water demand in Table 4.5 for this study area were projected. As justified on this figure the average water demand is greater than both domestic and non-domestic water demand, which implies that in the study area domestic water demand is more highly requires than non-domestic water demand i.e. there is less expansion of industry area and other non-domestic water consumptions. And hence in this cause it’s better to analysis more in detail for domestic water demand since its highly increasing due to pre-urbanization, high population growth rate than expansion of manufacturing industry.

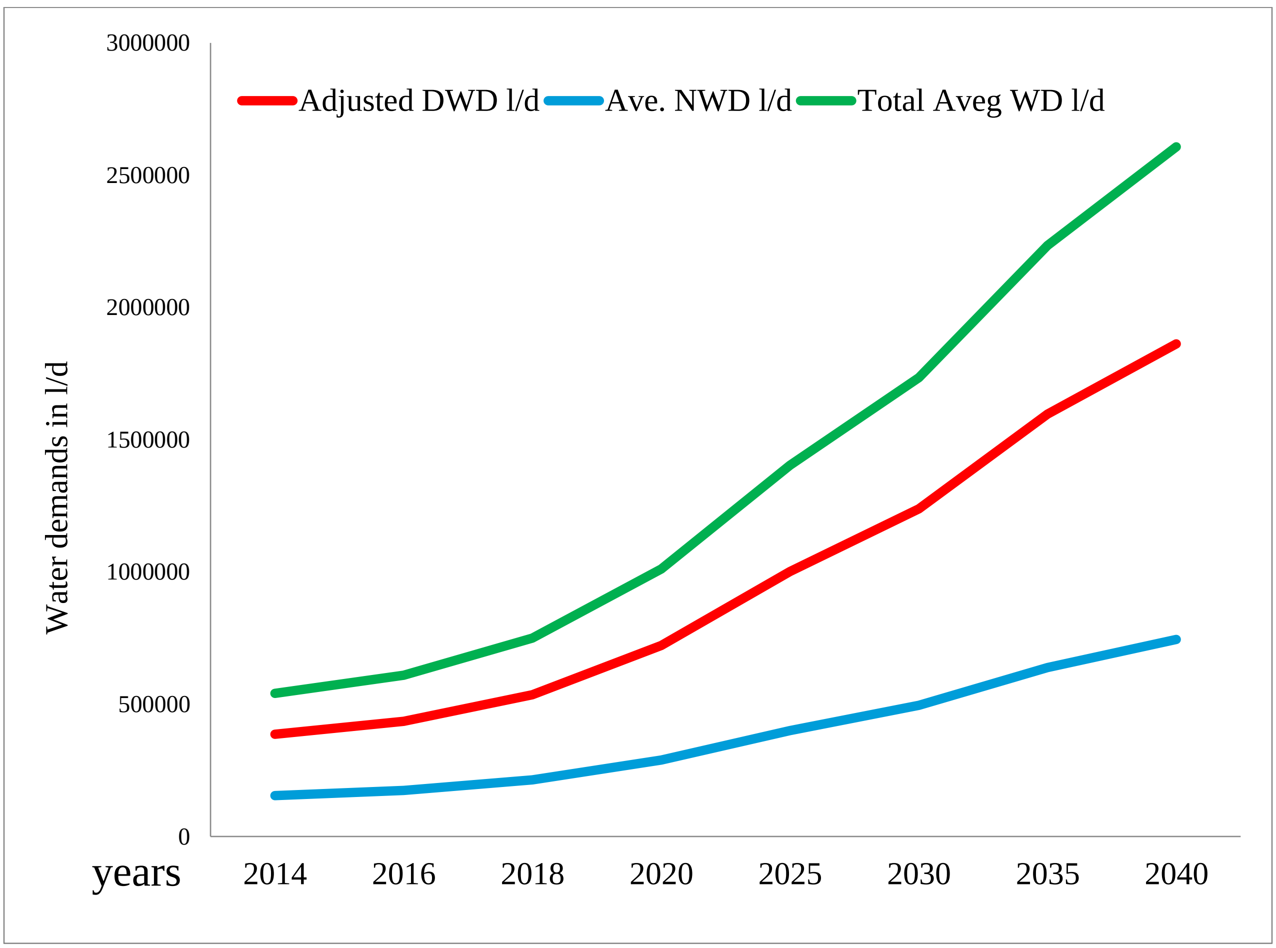


Figure 4.1: The domestic water demand vs non-domestic water demand

4.7. Analysis of water demand variation

The total average water demand was estimated from both domestic water demands and non-domestic water demands, but this demand can vary from year to year, day to days, hour to hours etc. Thus, in order to uniform this variation of demand it's should be multiplied by a certain multiplicity factors of maximum day demand and peak hourly water demands from each variation of water consumption rate of average water demands as shown in Table 4.5.

Table 4.5: Estimation of water demand variation and water demand adjusted

Years	Unit	2014	2020	2025	2030	2035	2040
Adjusted DWD	l/d	386477.3	721890.6	1002009	1238848	1596150	1862245
	m^3/d	386	722	1002	1239	1596	1862
Ave. NDWD	l/d	154590.9	288756.3	400803.4	495539.2	638460	744898
	m^3/d	155	289	401	496	638	745
Total Ave WD	l/d	541068.3	1010647	1402812	1734387	2234610	2607143
	m^3/d	541.1	1010.6	1402.8	1734.4	2234.6	2607.1
MDD factor		1.5	1.5	1.5	1.5	1.5	1.5
MDWD	l/d	811602.4	1515970	2104218	2601581	3351915	3910714
	m^3/d	811.6	1516	2104.2	2601.6	3351.9	3910.7
PHWD factor		2	2	2	2	2	2
PHWD	l/d	1082137	2021294	2805624	3468774	4469220	5214286
	m^3/d	1082.1	2021.3	2805.6	3468.8	4469.2	5214.3

The total water demand of the town was determined by summing up the adjusted domestic water demand and average of non-domestic water demands. In the study domestic water consumption rate is more than the consumption rates of non-domestic water, which implies that there were less institutional, commercial and fire broke out off in the in the town. Thus, the water consumption rate is highly used for domestic purpose than other giving services and hence further investigation of water source may reduce if the population growth rate almost constant. The current maximum daily demand is 811.6 m^3/ day whereas for end projected of this demand it is 3910.7 m^3/d. Whereas the current maximum peak hourly water demand is1082.1m^3/d and the forecasted peak hourly water demand is 5214.3m^3/d, this implies that for about 4132.2m3/d of water demand requires to on the existing currently maximum peak hourly water demands. Thus, this determination also necessary to design the capacity of services reservoirs. Therefore, the design maximum water production capacity of the source is 2539.76 m^3/d, which is very low due to less working hour, reduction of boreholes yields, pump failure and lack of maintenance and time increasing of water demand. As discussed on Table 4.5 the variation of water demand (maximum day demand and peak hour water demand always greater than domestic and non-domestic water demand throughout the designed projection times.

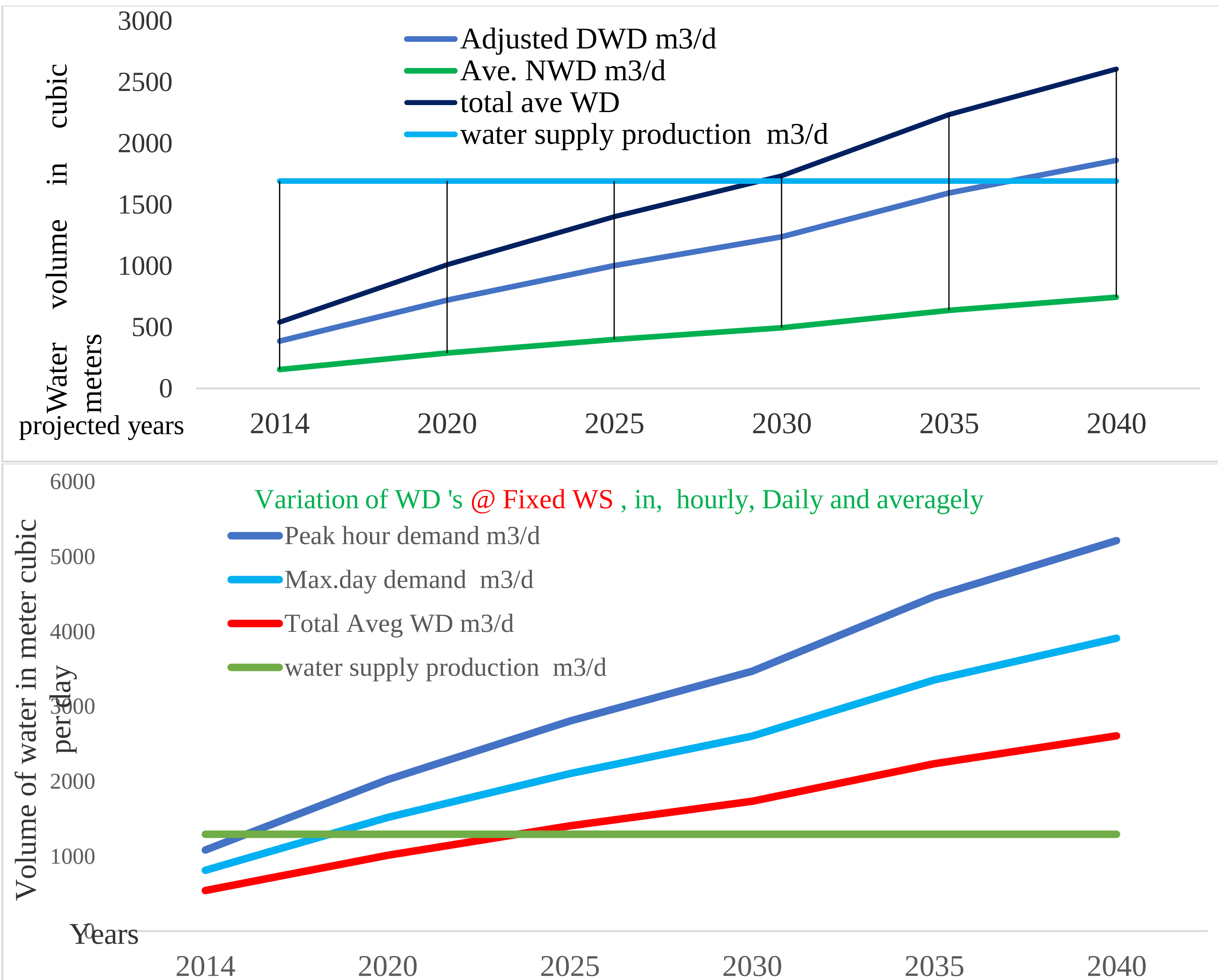


Figure 4.2: Estimated of water demand variation and water demand adjustment

4.8. Current and Future Water supply deficit

The maximum designed of water supply production from the existing source for this study area by supplying electricity power 861.87 m^3/d and when there is electricity is in operation and maintenance times by using pumps power only 414.72 m^3/d were considered to supply. Hence, currently the maximum daily water demand requires this area is 811.6 m^3/d of supplying water and after half quarters this demand were increased to 1516 m^3/d. This indicates that the current services of water supply are adequate for providing a desired requirement, but since the water production is fixed and human wish is unlimited, the water consumption rate is timely increasing and arrives 3910.7m^3/d at the ends of the design projection time. Consequently, the desired water supply requirement to the end of 2040 is 2,536.94 m^3/d of water supply is required as additional sources of supply. This deficit of water supply for the study area was proposed for total water demands (the average domestic and non-domestic water demand) by consideration of all water demand variation.

The deficit water supply is stared to occur after 2020-2025 years and in this time the additional 436.34 m^3/d quantity of waters needed for the area as other surplus source of water. Since this gap, existing between water supply and water demands was becoming doubled as described in Figure 4.2, especially the peak hourly water demand is higher than the other (maximum water demand, domestic water demand and non-domestic water demand) until the end of the projection time.

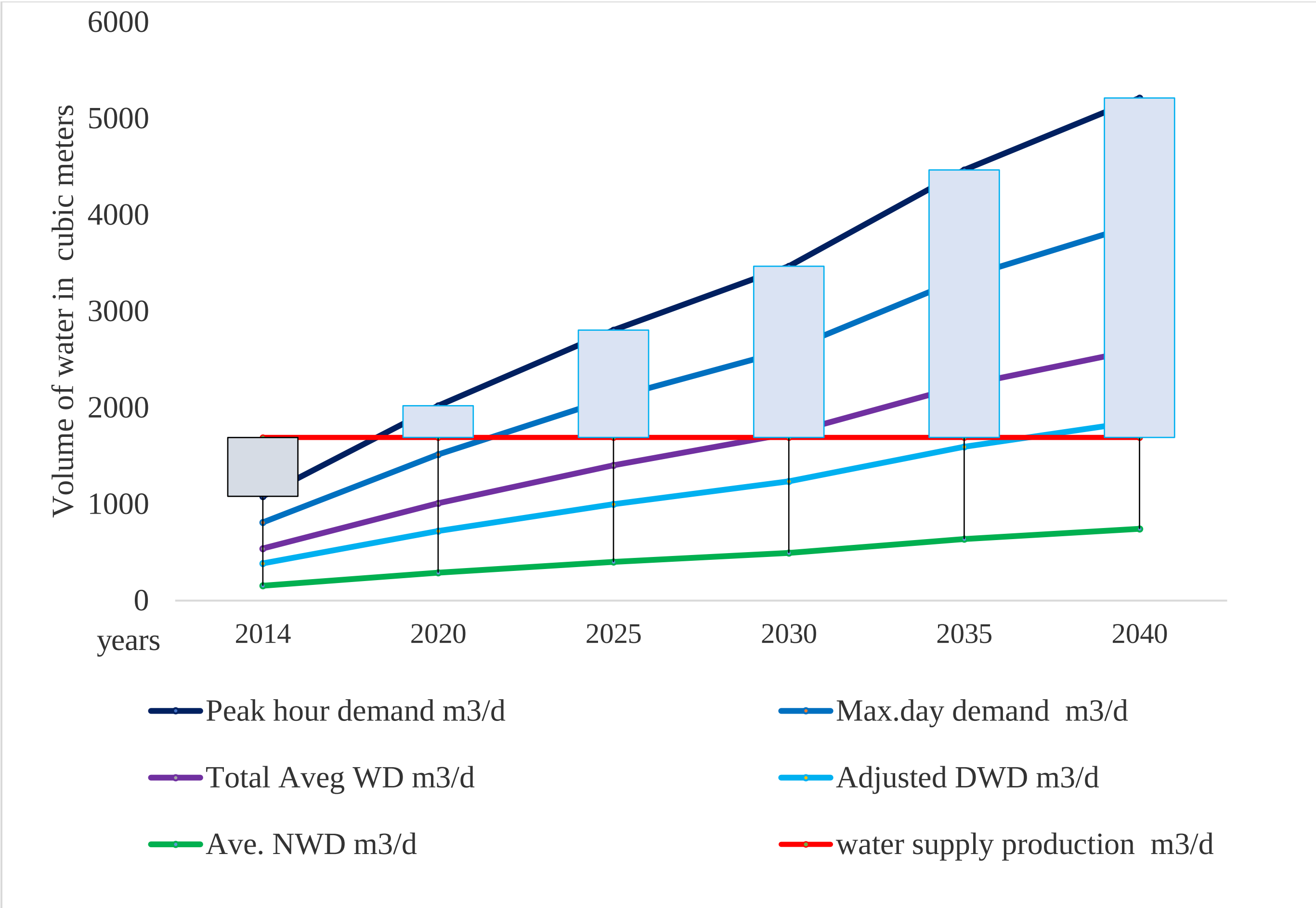


Figure 4.3: Current and Future Water supply deficit

4.9. Hydraulic performance analysis

The hydraulic performance analysis in distribution networks includes several hydraulic parameters like; water pressure, flow rate of water, water velocity, head loss, capacity and efficiency of pumps, size of reservoir has and pipes. Therefore, component of water distribution network has a great contribution on the hydraulic performance analysis and hence the value of each its summation resulted to evaluate its performance level.

i. Junction Pressures performance analysis

In the water distribution systems, pressure has two side effects (high water pressure and low water pressure or negative water pressure). Adequate positive pressure should be provided throughout a distribution network to ensure adequate water service and protect the system against backflow. Thus, the normal pressures in the water distribution network should be range from a minimum of approximately 15 mH_2O to a maximum of approximately 60 mH_2O. The minimum pressure allowed in a distribution system is 10 mH_2O and hence when pressures drop below 10mH_2O, a system could experience backflow conditions that could influence water quality. To same extents as conditional, the minimum pressure in water supply distribution is 10 mH_2O and the maximum pressure is up 70 mH_2O accordingly MoW, 2016. When a water supply distribution system service area has a wide range of ground elevations within its boundaries, water distribution networks are comprised of several pressure zones to provide acceptable pressures to the varying ground elevations. Flow rates in pipelines rapid changes in flow velocity within a distribution network can result in a pressure surge or water hammer.

ii. Pressure in distribution system during Steady State Simulation

A pressure, which is too high, raises the risk of bursts and leakages in the network and causes the energy expenses to rise, as more energy was needed to pump the water through the distribution network. A pressure, which is too low, can also increase the risk of water entrance and result in dissatisfied customers who experience an inadequate pressure in their households. This type of analysis was useful for determining pressures and flow rates under minimum, average, peak, or short-term effects on the system due to fire flows. For this type of analysis, the network equations are determined and solved with tanks being treated as fixed grade boundaries. The results that are obtained from this type of analysis are instantaneous values and may or may not be representative of the values of the system a few hours. For this study, since Extended state run simulation at average water consumption rate, for about 24.60 % is recorded as higher pressures junction for this study which is ≥70mH_2O and observed at different junctions, due to low elevation and smaller pipe sizes. In the same manner most of the distribution of the nodal pressure has the optimum range (15-70mH_2O) which is 59.89 % of distribution network is the normal ranges and whereas 15.51 % of junction demand can give service with the lowest pressure junction as indicated in Appendix 2 and 8.

a) Over Junction Pressure Simulation at average water demand

In the water supply distribution network should be designed to maintain a minimum pressure of 10 -15 mH_2O at ground level at all points in the distribution system under maximum day demand and fire flow conditions. Whereas the normal pressure in the water supply distribution system were 15-60 mH2O, rare case 70-80 mH_2O. However, in most of time there also the occurrence of over junction pressure due to; topographic complexity of the network; complexity of the network connections; different functional regulatory systems; temporal and spatial variations in water demand; the friction between water and the internal wall of the pipelines; the existence of non-Revenue water is contributing high percentages. Due to those reasons as shown in Figure 4.4 and Appendix 8 same of over pressure junction in the distribution networks during the extended state simulation.

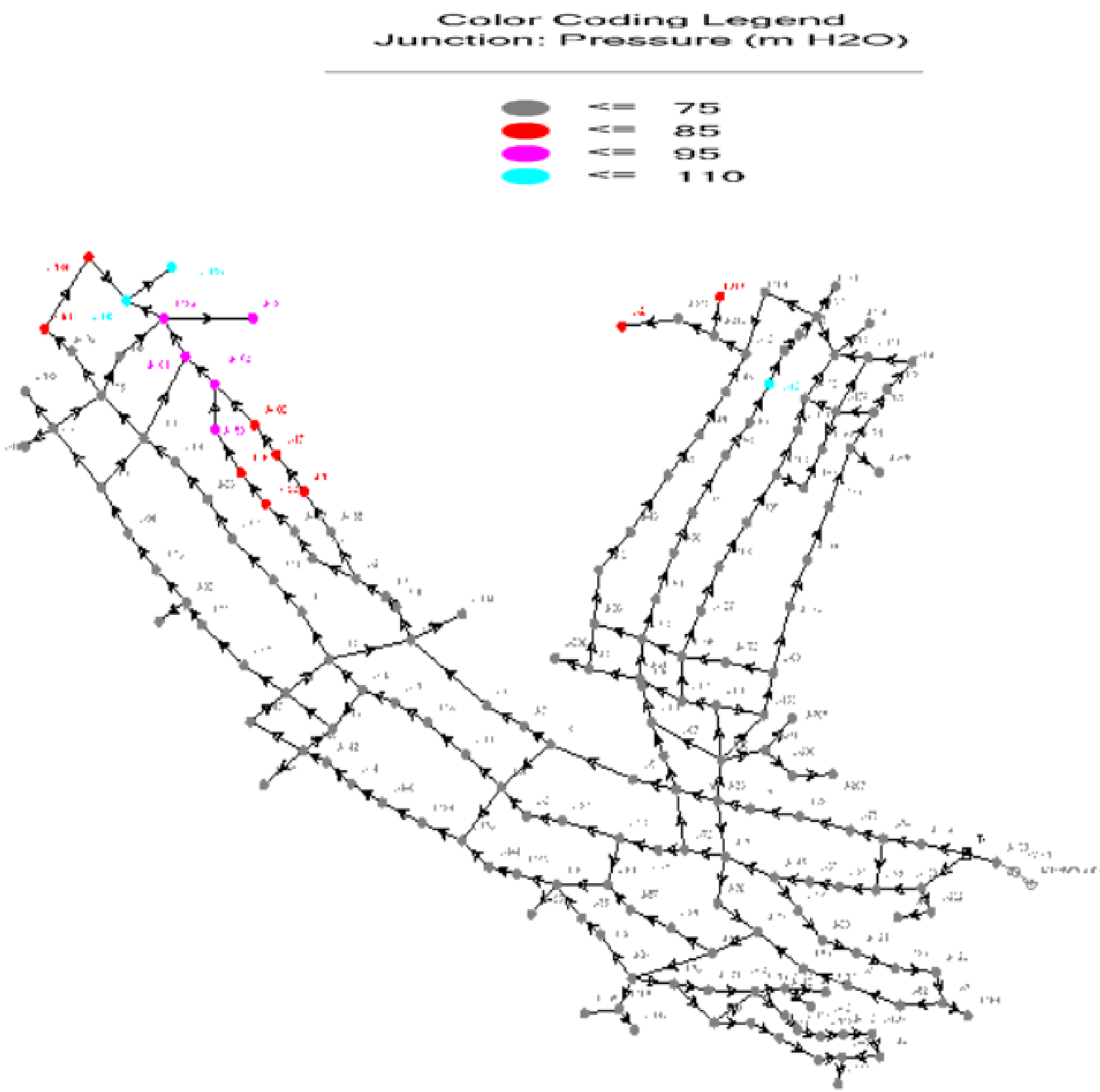


Figure 4.4: High junction pressure model simulated recorded at average water demand

Among the existing of total pressure junction (nodes) 18 nodes (J-182, J-104, J-150, J-105 , J 103, J-95, J-161, J-122, J-52, J-113) was records $\geq$ 70 mH2O water pressure during this simulation as shown in Figure 4.4. Thus, to reducing high junction pressure will reduce the leakage flow rate as well as the possibility of pipe burst which frequent variations in pressure, are associated with higher frequency of new leaks. Mostly over pressure was reduced by using resizing pipe material and control valves, which are used to control the flow or pressure in a distribution system. They are normally sized based on the desired maximum and minimum flow rates, the upstream and downstream pressure differentials, and the flow velocities. Therefore, minimization of excessive junction pressure was performed using a system of; pressure reducing valves, which used to limit the pressure in the pipe links, throttle control valves, used to control the pressure in a specific zone in the water networks and flow control valves and using variable speed pumps. This can change the speed of the electric motor that can change the hydraulic performance of the pump (such as power consumption, outlet flow, and pressure) whereas the pressure breaker valves which was used to force a specified pressure loss across the pipeline.

b) Low junction pressure simulation at average water demand

Pressure in an urban water system was typically maintained either by a pressurized water tank serving an urban area, by pumping the water up into a water tower and relying on gravity to maintain a constant pressure in the system or solely by pumps at the water treatment plant and repeater pumping stations. In these distribution systems of extended state, simulation same of the nodal demand where distributed with low pressure transient which may promote the collapse of water mains, leakage into the pipes at joints and seals under sub-atmospheric pressures, and back siphon age. Therefore, to analysis the pressure performance indexes; it's better to deal which nodal demand is low and over pressure in order to calculate the summation of pressure contribution of a junction due to the change in demand at another junction.

Generally, about 15.51% of the lowest water pressure is occurred in the junction (J-220, J - 202, J-4, J-85, J-228, J-87, J-11, J-62, J-214, J-32, J-192, J-224, J-55, J-197, J-73, and J-67) were recorded ≤15 mH_2O due to high elevation and small pipe material diameters. The pressure at nodes depends on the adopted minimum and maximum pressures within the network, topographic circumstances, and the size of the network. The minimum pressure should maintain to ensure that consumers' demand provided at all times. The maximum pressure also contains limitation of leakage and leads to water losses in distribution system. This pressure junction was described from the distribution-networking layout of water supply since the extended state simulation as shown in Figure 4.5.

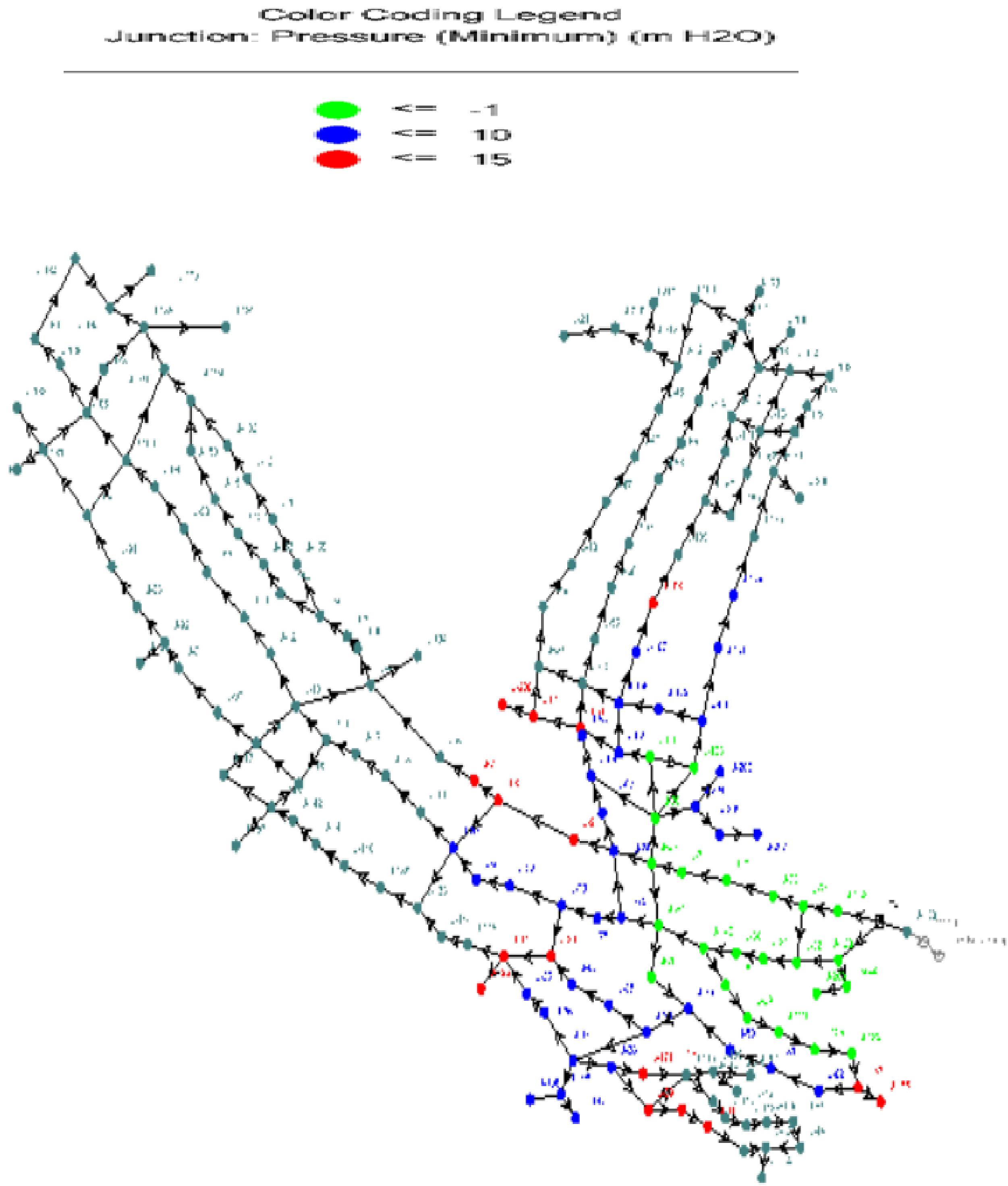


Figure 4.5: Lowest junction pressure performance since extended state simulation condition

c) Low and high Pressure performance of contour maps since Extended state simulation

Normal contouring routines only include model nodes, such as junctions, tanks and reservoirs whereas spot elevations was added to the drawing, however, this can build more detailed elevation contours and enhanced pressure contours. These enhanced contours include not only the model nodes but also the interpolated and calculated results for the spot elevations. Therefore, the enhanced pressure contours can help the modeler to understand the behavior of the system even in areas that have not been included directly in the model. Thus, as per described in the Figure 4.4 and 4.5 of the highest junction pressures in the distribution networks and the lowest junction pressure during the extended state of simulation. In the same manner the during this simulation the low and high-water pressure contour mapping also identified as the following Figure 4.6 simultaneously and the Contour Smoothing option displays the results of a contour map specification as smooth, curved contours.

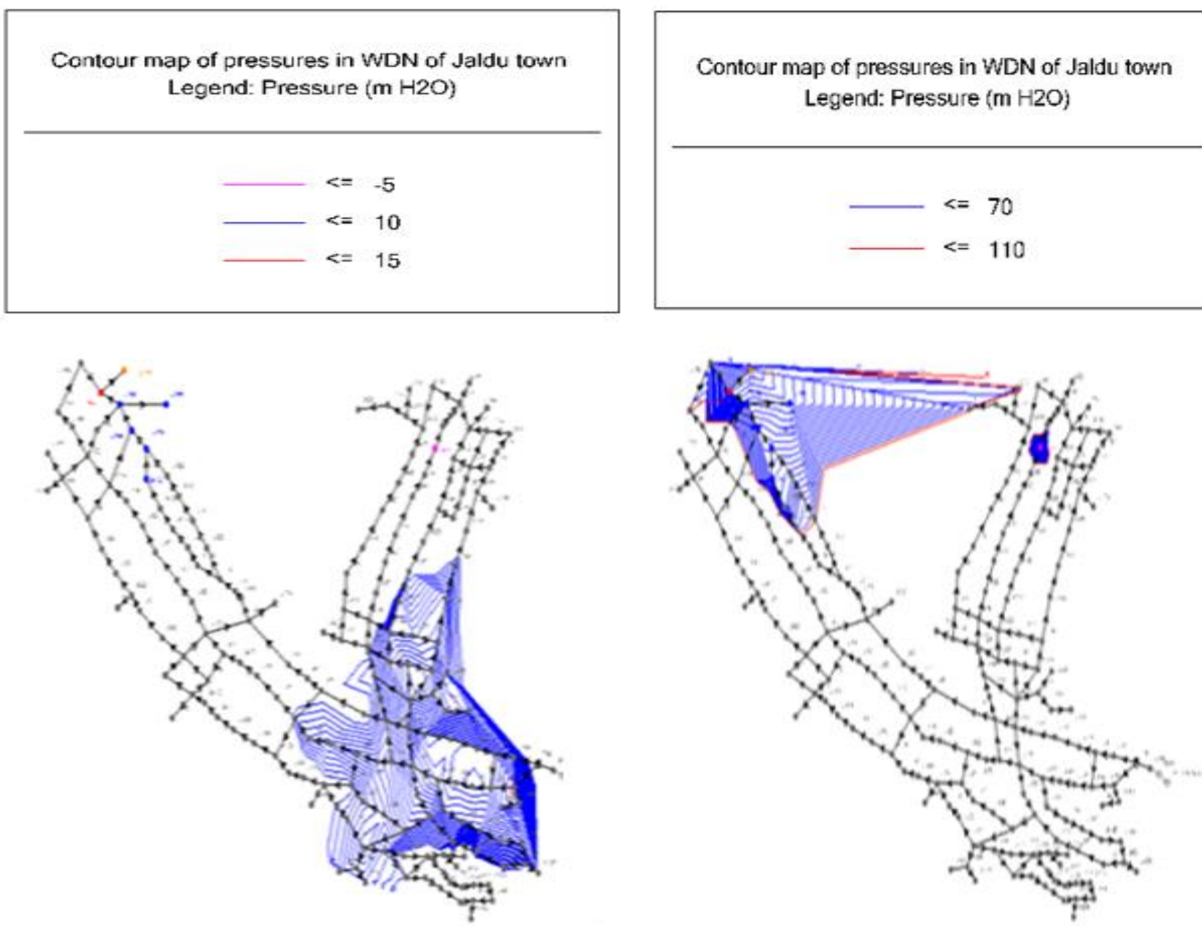


Figure 4. 6: Low water pressure recorded of contour during Extended state simulation

d) Pressure contouring maps of distribution networks during steady state simulation

The pressure of the distribution system was extrapolated by water GEMS to areas with pipe networks of 187 junctions. This extrapolation indicates the contour plot of pressure in the distribution system. The pressure distribution is not an illustration of pressures at taps only, but it includes the pressure at all different nodes in the water distribution network system. The pressure contour plot works true due to pump heads and flows. The pump curves and reservoir heads were displayed during the network analysis. There are extremes of high and low pressures throughout the system due to topographic variation of the town. A pressure zone is defined as the area bounded by both a lower and upper elevation, all of which receives water from a given hydraulic grade line or pressure from a set water surface. One usually provides the hydraulic grade line or more storage tanks located at the same elevations so they share high and low water surfaces.

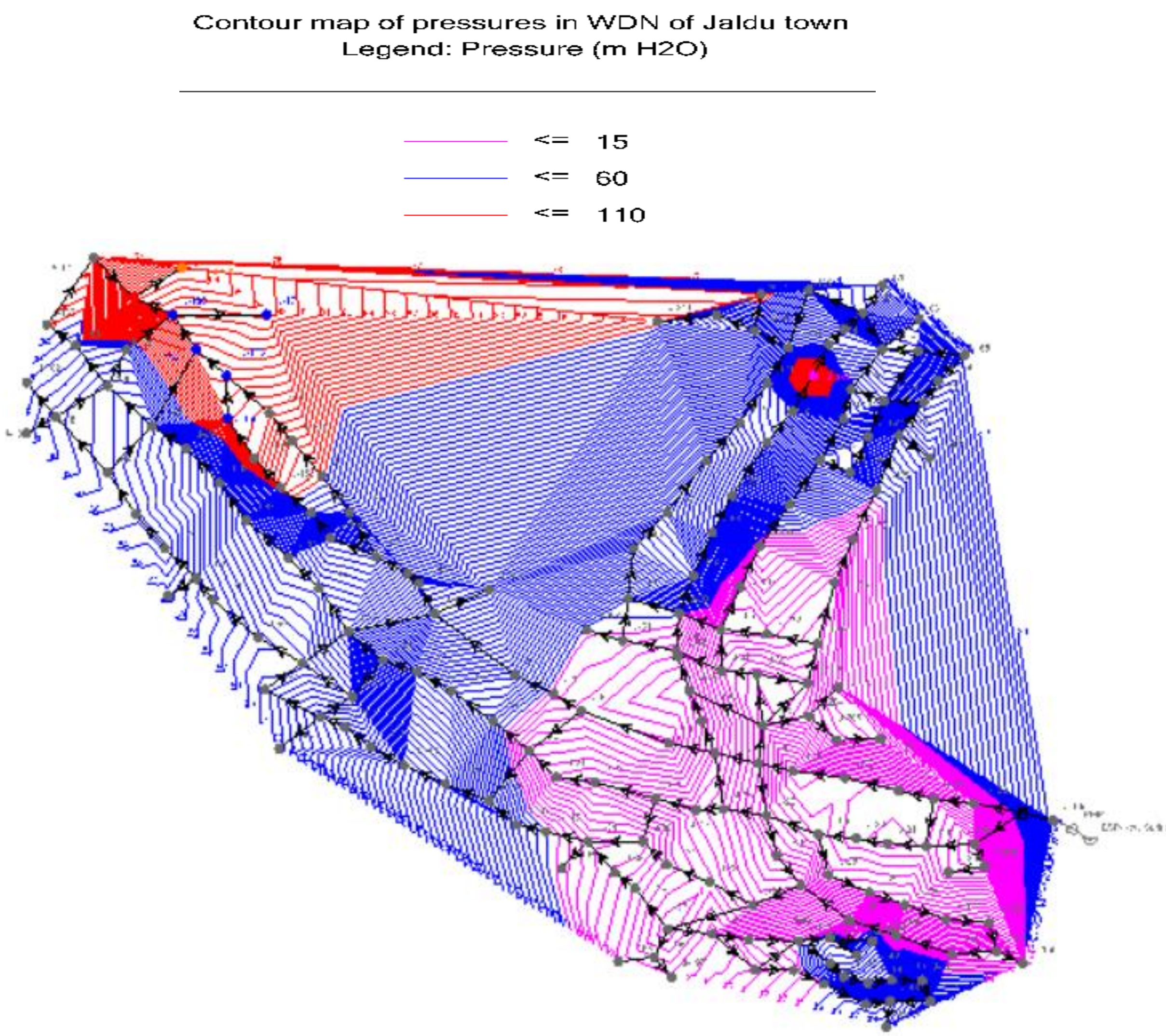


Figure 4.7: Pressure junction contour maps during Extended state simulation

With the use of pressure zones, gravity service was provided and low pressure pumping from a treatment plant clears well or lower pressure zone storage tank only occurs to fill an upper zone storage tank. Pump control can be automated based on water levels in the storage tank. When the tank's level drops from water demands in the system, pumps in the lower zone activate and start to fill the tank and/or satisfy the demands. Since the pump's design discharge pressure is set to overcome head loss enrooted to the tank's high-water level, service taps essentially, receive the same pressure as they would if the water was flowing by gravity to them from the storage tank. As described in the previous section, delivery pressures to service connections will be the difference between the tank's water level, less head losses in the pipe, and less the ground elevation at the service connection. In addition, as stated, line losses are usually minimal since the pipelines are sized for peak flow conditions.

e) Pressure Distribution at Peak Hour Consumption

The peak hour water consumption of is one the highest water demands in the distribution which is always greater than the maximum day demand of for the study area as described in Figure 4.3. During this consumption rates, the junction pressure of each nodes was obtained at the lowest water pressure as indicated in Appendix 2. Throughout this pressure distribution of networking systems, most of the nodes have below 15 mH_2O pressure junctions. Generally, by concerning to this, for about 117 nodal demands of model simulation results were recorded less than 25 mH_2O which is 62.57% and 60 nodal demand simulation was less than 15 mH_2O (14.97%) of nodes junction can distribute water at this low flow rate of water which can cause for back flow of water. The total area of low-pressure head is 14.97% during peak hour consumption and 1.60% over pressure since this model simulation time.

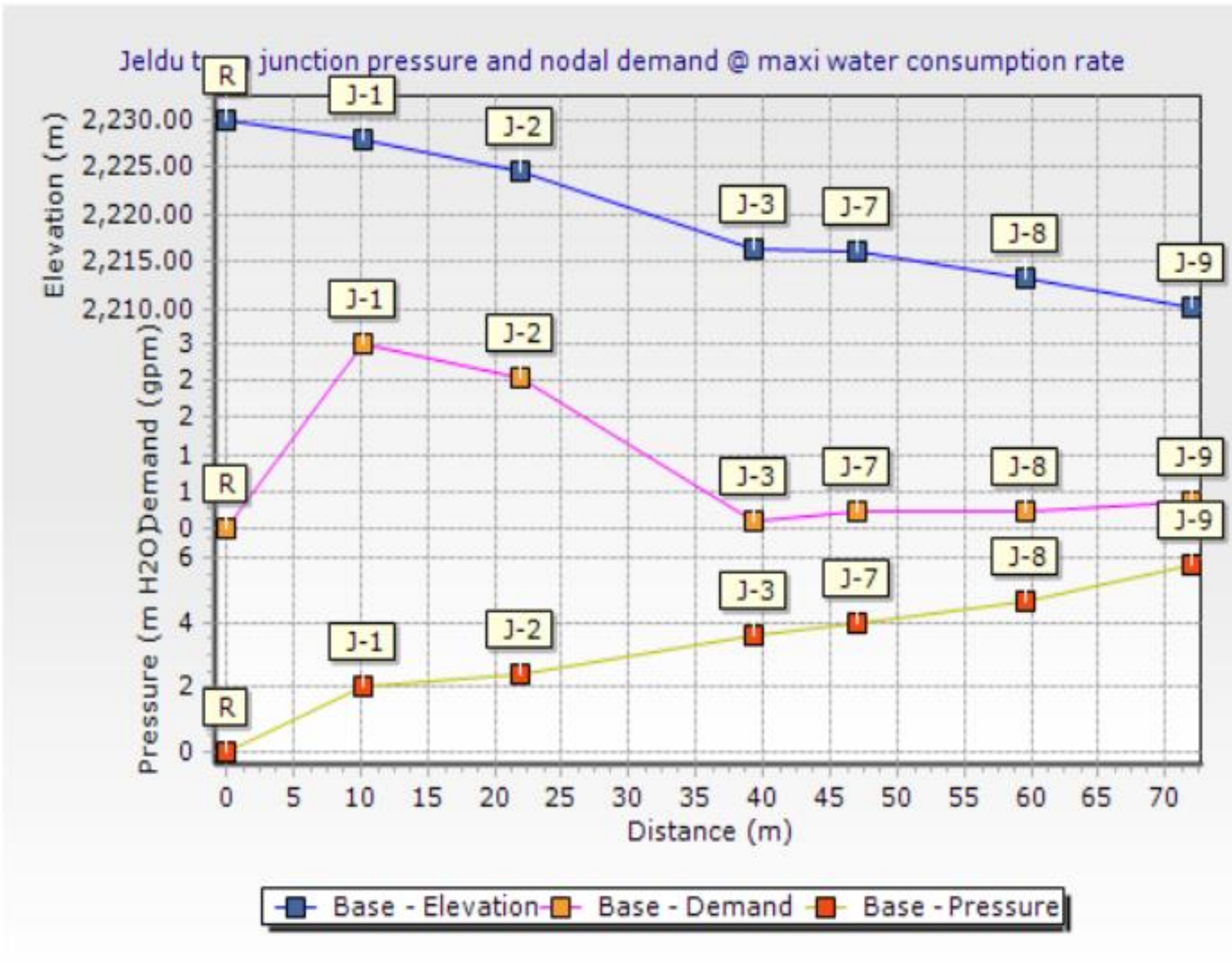


Figure 4.8: Pressure Distribution at Peak Hour Consumption

f) Pressure Distribution during Minimum Hour Consumption

During minimum hour consumption, 1.98% of residents get water at low pressure due to improper pipe size (the maximum diameters in this distribution zone is less than 40mm), and this area is almost the long distance from the services reservoir. Thus, the performance of junction pressure index at this minimum hour consumption rate was estimated as pre described in equation 3.5 and which is for about 0.54. Additionally, in this distribution area the topographical it has high elevation of the area, which creates a low level of reliability of water users on the supply system. As shown in Figure 4.10, most of the identified nodes have pressure below 15 mH_2O and some of the nodes have pressure above 70 mH_2O. Thus, only 0.17% of the areas have pressure unfit to the recommended limit (15-70 mH_2O) during minimum consumption of the steady state simulation at averagely minimum water demand consumption.

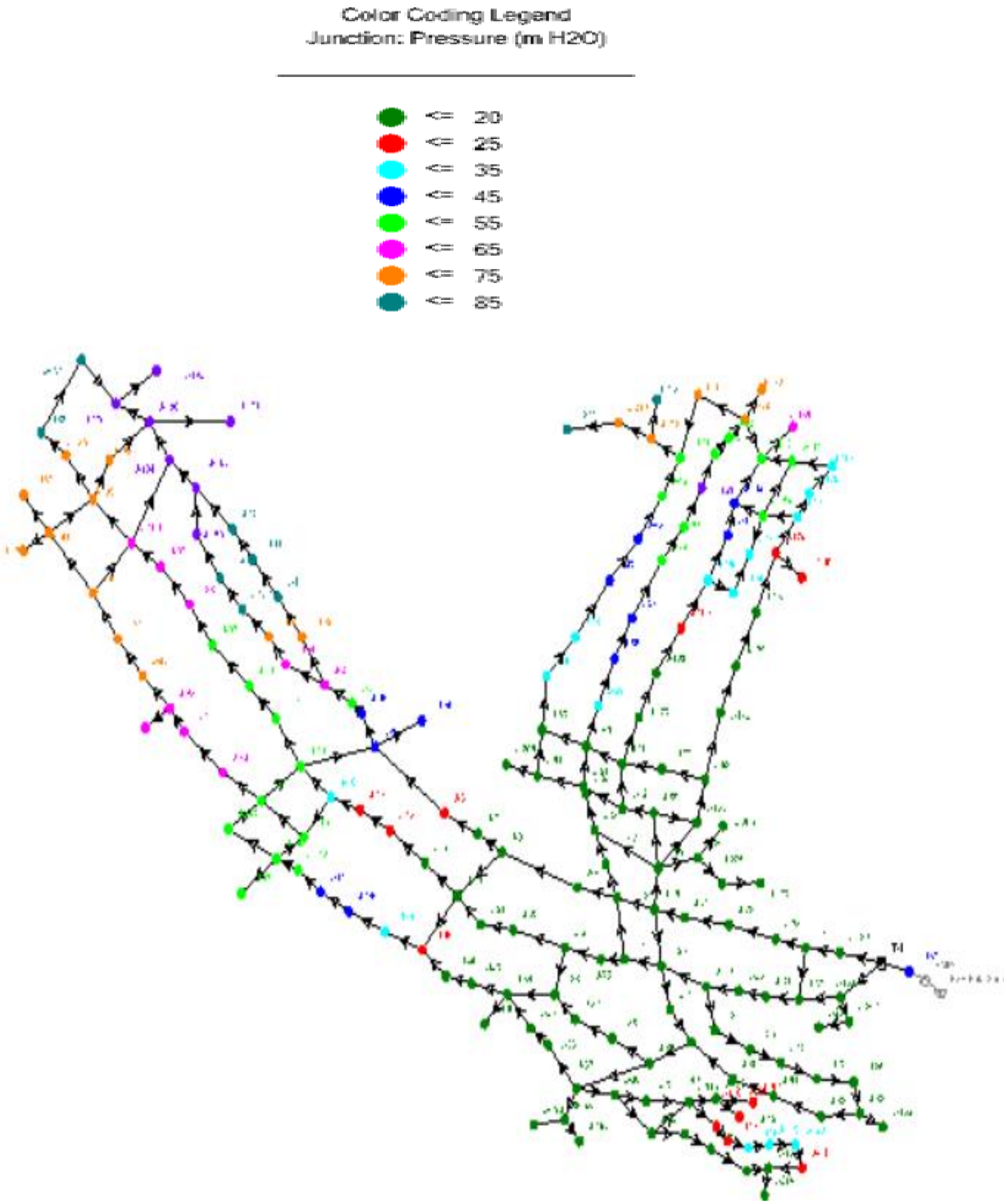


Figure 4.9: Pressure Distributions during Minimum Hour Consumption

4.10. Velocity Distribution during Peak Hour Consumption

During peak hour, water demand consumption for about 0.51 % of velocity in water distribution network is without the desired criteria of MoW 2006 hence only 34.13% distribution network was in standard value, whereas 51.93 % and 13.94 % of the velocity during peak hour consumption is less than 0.56m/s and 2.5m/s respect as shown in Figure 4.10. The hydraulic parameters performance (velocity performance) in this distribution system at minimum hourly or average water day water demand consumption rate almost most of the pipeline were distribute water accordingly to water supply distribution guidelines. However, some of them were beyond to the standard level and below due to mismatch of discharge flow versus to the pipe size, due to this in certain areas even if at this minimum demand consumption rates it could not be properly arriving at the required nodal demands. In this session of peak hour water demand consumption for about 108 of pipes lines conveyed 0.6-2.5 m/s of water flow velocity, which is in below standards level, whereas 72 pipelines distributed in normal and 29 of pipelines are distributed water flow without the limited criterial above the standards.

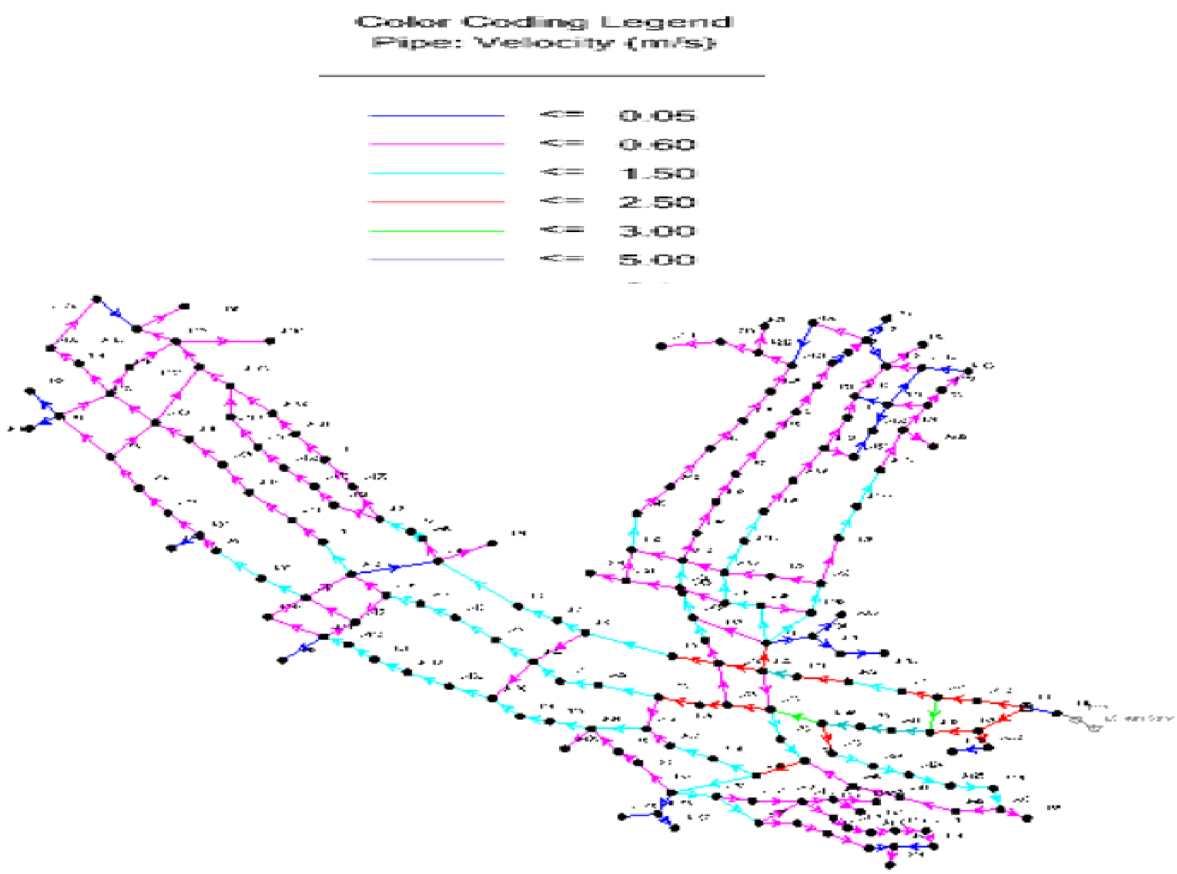


Figure 4.10: Velocity distributions during peak hour consumption

During this simulation, model same of the selected junction in the form Figure 4.11, water distribution network is low pressure in the distribution system due to the topographic variation of the town. The area of kebeles keta near to St. Church and mosques was marked by very low pressure < 15 m. This minimum pressure should be maintained in the system to avoid the water column separation and to ensure that the consumer's demand was provided at all times. As described in the follow Figure 4.11, the selected pipelines of water velocity within the pipe water distribution systems. Since this in same of the pipe line can distributed in with the standard level due to its shortest length and small size of the pipe materials, in the same way, almost half of the selected lines were also distributes above and below the level of standard values throughout the pipe lines as indicated in the Figure 4.12 in the pipe lines.

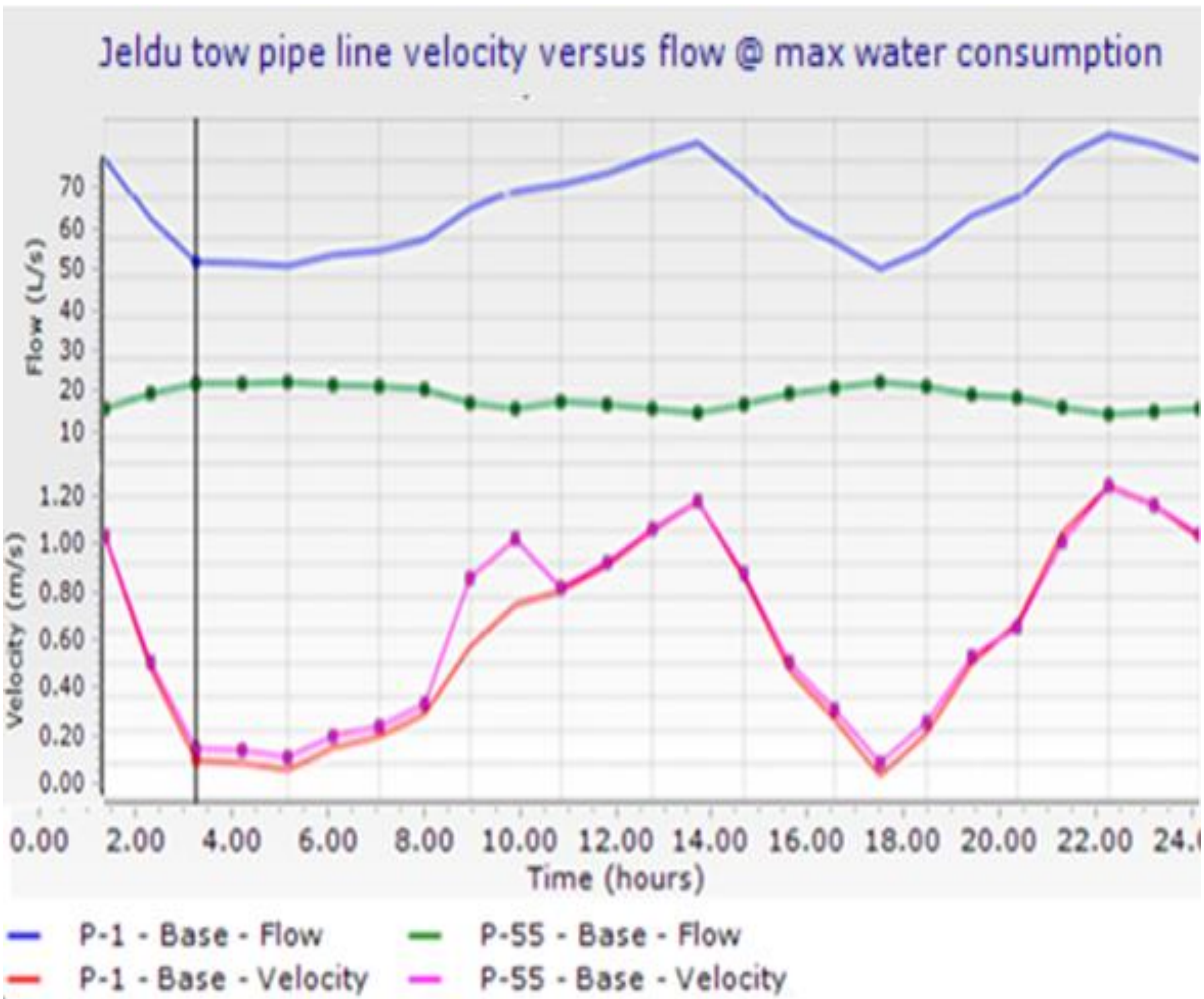


Figure 4.11: Flow of water in the selected pipes

The velocity of water flow in a pipe is also one of the important parameters in hydraulic modeling performance evaluation of the efficiency of water supply distribution and transmission line. The velocity ranges can also be adopted as the design criteria, low velocities for hygienic, while too high-velocity cause exceptional head loss reason are not preferred velocity distribution is also varying with demand pattern changes. At the peak time demand, the values are different as compare to minimum consumption hour and hence, water distribution network velocity during peak hour demand is summarized in Figure 4.12 and expressed in Appendix 7.

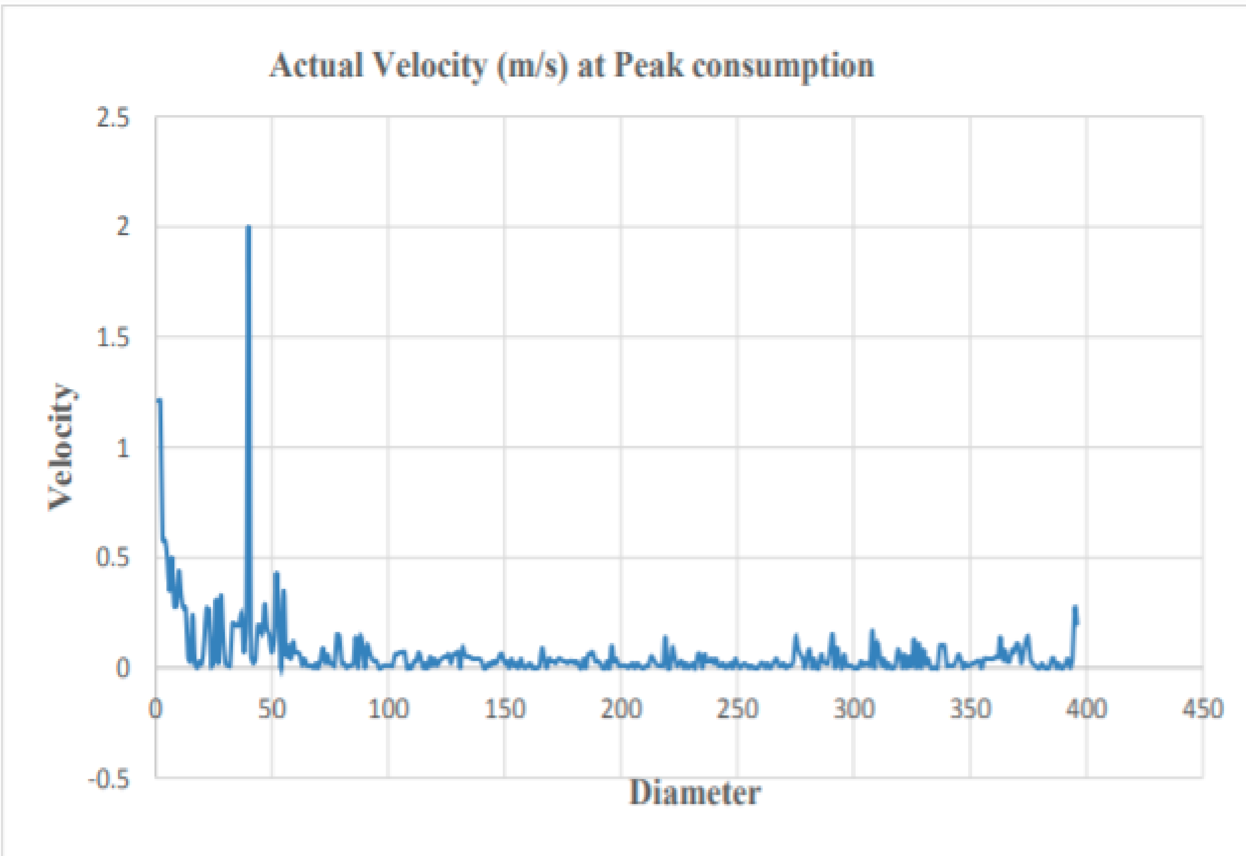


Figure 4.12: Velocity distribution at peak hour consumption

4.11. Hydraulic Performance Index

As pre- discussed under 3.9, the hydraulic performance indexes levels against the flow velocity in pipes and the pressure of nodes (junction pressure). According to (Ravi *et al.*, 2019) the value of excellent level of performance index, shows 0.75, 0.5, and 0.25 describe the suitable, acceptable, and unacceptable performance of the system, respectively and this performance indices, which are obtained from the penalty curves, are related to the elements of WDNs. For this study, the pressure performance index of the network is 0.81 were obtained from the penalty curves analysis and the velocity performance index of the network distribution is 0.64 independently. Thus, from the above basic performance analysis the estimated hydraulic performance indexes for the study is 0.47 that indicates almost the acceptable ranges according to the performance analysis indexes.

4.12. Water Loss in the Distribution

Water loss is the amount of distributed drinking water that does not reach customers, and that water utilities therefore do not receive payment for it (Non-Revenue Water). Non-Revenue Water can occur through physical losses from leaking and broken pipes, which was caused by poor operations and maintenance, the lack of active leakage control, and poor quality of underground assets. According to the billed and produced water supply data of this study area were 95,952 meter cubic of water is annual water lost, which is for about 18.34% is lost without any services before it arrives to the customer services. Thus, the total water loss in distribution networks is estimate by subtracting the amount of water billed or consumed from the amount of water produced for study area as shown in Table 4.8.

Table 4.6: Water Loss in the Distribution (Non-Revenue Water)

Month years	Water production (m^3)	Water consumption (m^3)	Water loss(m^3)	water loss %	Com. Production m^3	Com. consumes .m^3	Com.loss %
Jan	68,636	58,341	10,295	11	68,636	58,341	11
Feb	65,204	55,424	9,781	10	133,841	113,764	21
Mar	58,341	32,087	26,253	27	192,181	145,852	48
May	67,950	57,757	10,192	11	260,131	203,609	59
Jun	44,614	37,921	6,692	7	304,745	241,531	66
Jul	51,134	43,464	7,670	8	355,878	284,994	74
Oct	54,909	46,673	8,236	9	410,787	331,667	83
Dec	51,477	43,756	7,722	8	462,265	375,423	91
Apr	60,743	51,632	9,111	10	523,008	427,054	100

Water losses and the associated financial losses represent a serious problem for water supply companies all over the world. High levels of water losses result from poor management and poor condition of distribution systems. The increase in water losses forces water supply companies to implement systems for control and evaluation of water losses, make organizational changes and to develop and implement modernization programmes in order to improve the technical condition of water supply networks.

The analysis of data on distribution system failures indicates a direct relationship between the operation and the failure rate in water distribution systems, but the scale of the problem has not been sufficiently evaluated so far. The cumulative average water loss of the system is shown in the Table 4.7 above. Water loss is usually expressed in terms of percentage (UFW), loss per kilometer length of main pipes and loss per properties or number of connections. In this study, calculate the water loss in using above as percentage of (UFW).

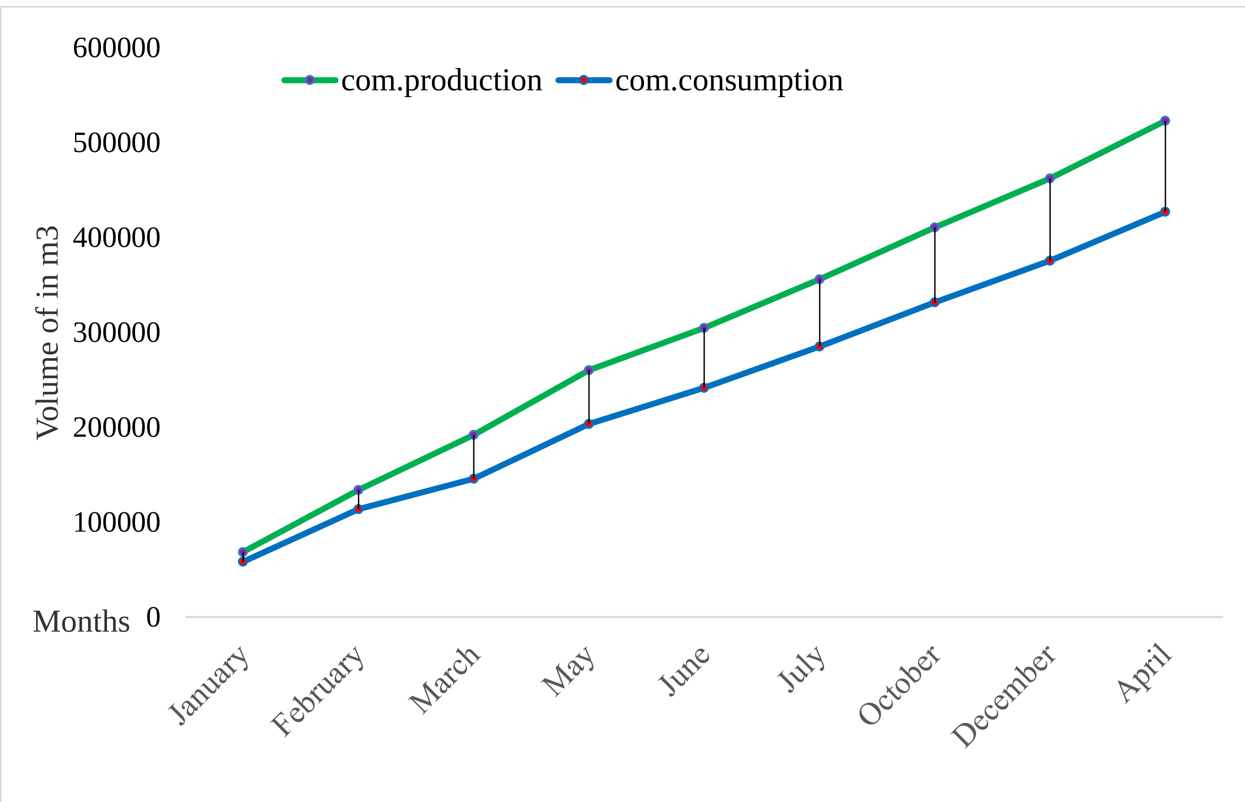
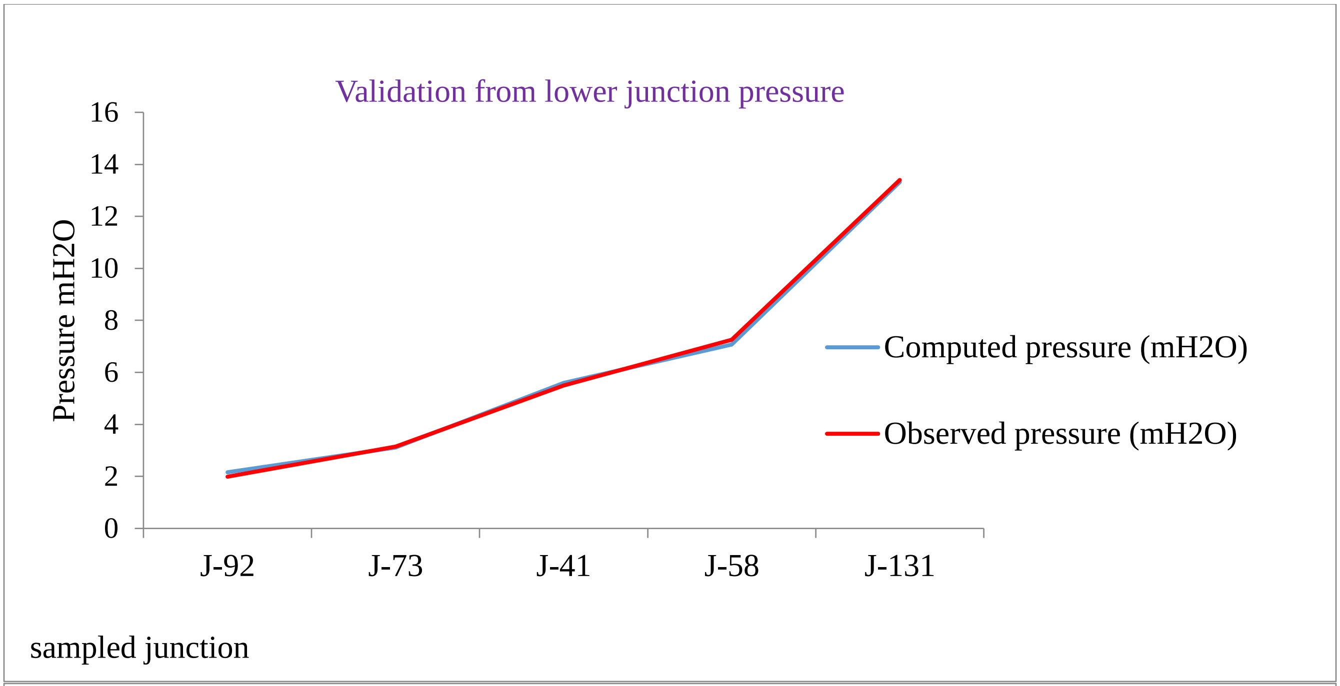


Figure 4.13: Cumulative water production and cumulative water consumption

The seven-month water produced and distributed to the distribution and the water billed that was aggregated from the individual customer meter readings were used to quantify the total water loss for the study area. As the authorized non-metered consumption is insignificant while compared with the water production, the unaccounted-for water has been as a synonymy of the water loss in this analysis.

4.13. Model Calibration and validation

For model calibration and validation, effort data were collected from field selected sample locations as described in Appendix 3. Model calibration was determined based on the results of model pressure and measured pressure in the selected nodes has been used for calibration. According to (Mavi and Vaidya, 2018) the percentage of measurement nodes satisfied the essential minimal amount of measurement points (2% of all nodes) for the designing and operation purposes. Thus, the study is used pressure gauge to measure the pressure junction demand at five nodes (J-102, J-27, J-34, J-17, and J-164) from 187 nodes in particular hours (6:00 AM, 9:00 AM, 6:00 PM and 9:00 PM) for five days to check the simulation results. For each node, record was taken five times at different times in single days. Model calibration and validation were undertaken based on the different calibration standard criteria for hydraulic network and water quality modeling. 'Model calibration is the process of fine-tuning a model until it simulates field conditions for a specified time horizon to an established degree of accuracy'. Therefore, model will not be hundred percent correct and to be calibrating it must be accurately simulating the observed data. Further, according to(Mala-Jetmarova, Sultanova and Savic, 2018); hydraulic model calibration is the necessary process of modeling and it is calibrated in order to have better confidence, understanding and identifying errors made during the model-building process. As in Figure 4.14; during the comparison of water pressure with measured values to simulated values, the gaps were recorded up to 14m head which was out of the pressure standard according(Anisha *et al.*, 2016) limitation suggested. Therefore, the computed pressure for both upper and lower zone, value was calibrated until the result was approach to the measured pressure value.



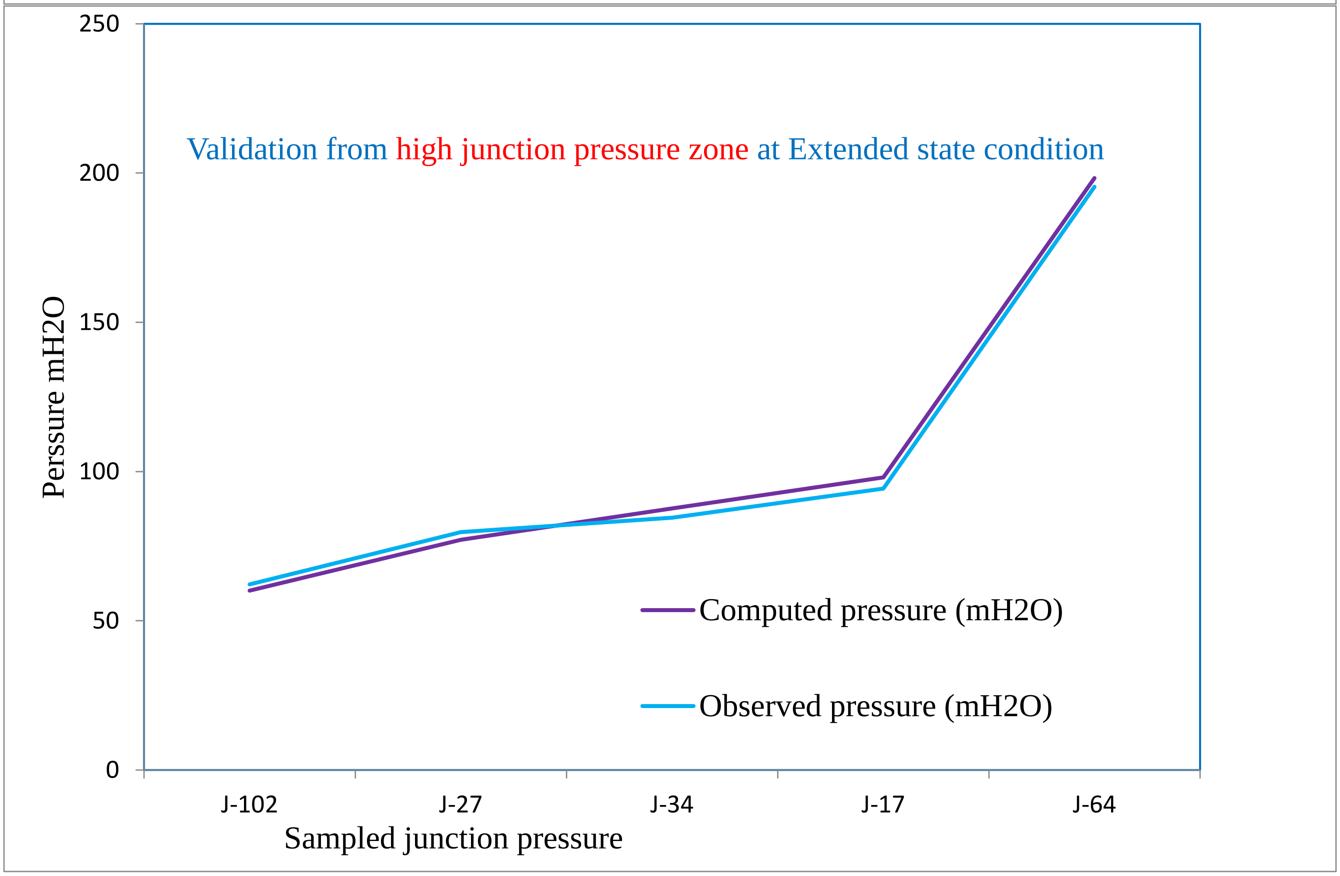


Figure 4.14: Simulated and measured values of upper zone and lower zone pressure

While, as per discussion with the water utility manager, in Jaldu town the maximum hour water demand is happen during morning and evening time, when most people use water for bathing, washing and cooking purpose so that in case of higher and lower zone the computed pressure and observed pressure are almost close to each other.

4.14. Model Calibration

As the study results(Capt. *et al.*, 2021 and Bhatt and Paneria, 2017) showed that the water GEMS model has a good capability to predict the pressure at the node as confirmed with a higher coefficient value ($R^2 \geq 0.95$). The liner regression relationship of pressure which showed a typical $R^2 \geq 0.5$ is considered acceptable of model performance as per AWWA (2012). The model calibration result showed that the Water GEMS model is a good predictor of pressure in the study area. The model validation work was taken manually using the correlation coefficient equation (R^2) method and it were described and represent graphically in Figures 4.15. From this Figure explanations, the results of correlation value (R^2) for both high- and Low-pressure zone was representing as 99.99% and 99.97%, respectively. Thus, based on hydraulic model calibration result based on difference error: In this study, the pressure data measured at the near to node, home faucet of the system was used to assess the model performance. The model performance measure such as the degree of accuracy (error of difference) and the coefficient of determination (R^2) are two techniques to be considered for the calibration model check as mentioned below the results and the value this is explained in the Appendix 4. All observed pressures were equal to the simulated pressures, giving a link coefficient of one that is the best correlation between observed and simulated.

Thereby, the calibrated Pressure value was validated within the recommended standard as described in Figure 4.15 for upper and lower zones.

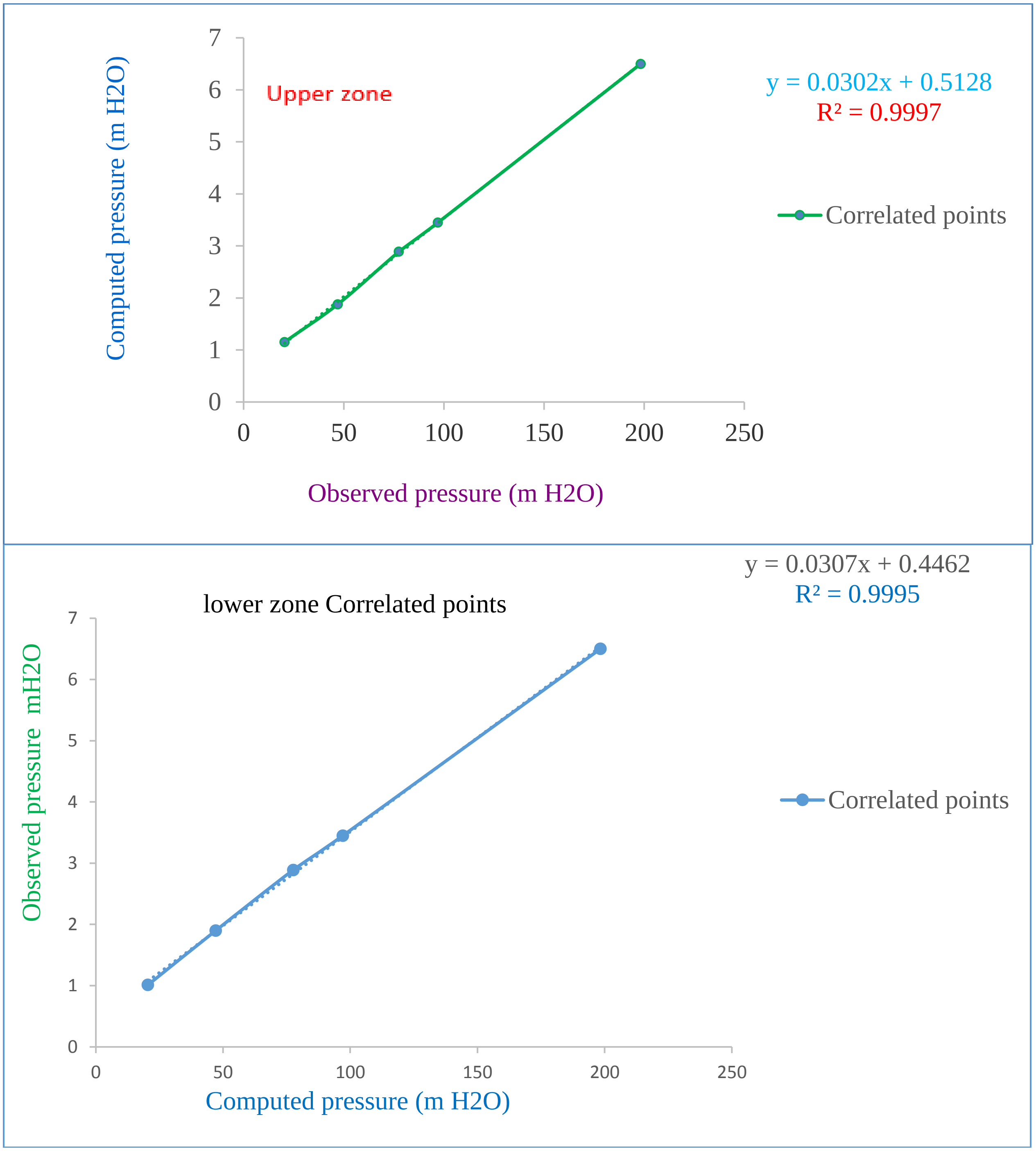


Figure 4.15: Correlated plot during pressure calibration (lower zone) for average demand

CHAPTER FIVE

5. CONCLUSION AND RECOMMENDATION

5.1. CONCLUSION

The existing water supply project of the Jaldu town was constructed in 1989 E.C for giving service to an estimated 16970 of population. However, the current and the projected population of this area was 24575 and 46498 from 2014 and 2040 yrs. with respectively, this indicates that doubling of water demand at a fixed water supply production which give services beyond to the design period with inadequate performance. The existing water supply production is for about 1693 cubic meters per day whereas the current water demands for the study area is 811.62 cubic meters per day, which is almost safe for a few years only. However, beyond to 2021- 2040 years since water production is limited and lastly 3910.7 m^3/d of water is requiring for this area. Hence, this materializes the scarcity of drinking water supply or water supply deficit is increasing. In addition to this in the distribution networks, there is a problem of proper pipeline layout system with appropriate dimension to estimated demand capacity. Due to this reason, there is a number of negative and over junction pressure and water velocity were recorded since extended state and extended state simulation analysis. This is 15.51 % of the lowest water pressure is occurred in the junction were recorded ≤ 15 mH_2O and 24.60 % records the highest water pressure > 70 mH_2O, whereas 51.93 % is the lowest water velocity < 0.56 m/s and 13.94% is the highest water velocity > 2.5m/s recorded with respectively. The pressure performance index and velocity performance index were for about 0.81 and 0.64 with respectively for which recognizes the model simulation results and penalty curve values as a good situation. To ample, the performance analysis of the water distribution network is evaluating by estimating junction pressure and pipeline water velocity, so for this study the hydraulic performance index is 0.47, which ranges almost in the acceptable standard or in good condition. The study also identifies the major water loss encountered in the distribution, which a serious problem for water supplies due to poor management and lack of operation and maintenance system. This system supplies affects the hydraulic performance of the network and exposes it to high values of pressure and velocities; which results adverse effect on the readings of the customer water meters due to the pushed and sucked air in the network. This was ratified calibrated and validated from high and low-pressure junction recorded nodes, which implies ended observed and simulated values.

5.2. RECOMMENDATION

The existing water demand of the study area is greater than daily water production of the system mainly, due to pre-urbanization and unaccounted water losses. Therefore, it is important to revise the design and rehabilitate the water distribution system by improving the size of reservoirs capacity and replacing the new raw water pumps with the required hydraulic performance and checking out performance of the other components of the system deliberately. Generally, in order to improve the performance of Jaldu town water supply and distribution system the following activity should be required.

- To increase the performance of water supply distribution system finding additional source of water and water losses control measures should be taken.
- Uses of pressure sustaining valves are recommended as to control the occurrence of minimum pressures. These valves start closing if the upstream pressure falls below the predetermined value as to guarantee allowable minimum pressure for isolated parts of the network.

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APPENDIXES

Appendix 1: Junction pressure at minimum and peak hour consumptions

≤ 70 mH2O Junction pressure	No of Junction pressure recorded @ minimum hour consumption	Percentage (%)
≥ 70	56	29.95
70-65	42	22.46
65-55	22	11.76
55-45	21	11.23
45-35	19	10.16
35-25	17	9.09
25-15	7	3.74
≤15	3	1.60
Total	187	100

≤ 70 mH2O Junction pressure	No of Junction pressure recorded **@** : peak hour consumption	Percentage (%)
≥ 70	3	1.60
70-65	6	3.21
65-55	11	5.88
55-45	13	6.95
45-35	17	9.09
35-25	20	10.70
25-15	28	14.97
≤15	89	47.59
Total	187	100

Appendix 2: Negative junction pressure recorded in distribution systems

Message Id	Scenario	Element Type	Element Id	Label	Time (hours)	Message
40004	Base	Junction	31	J-2	0.000	Negative pressure at Junction J-2.
40004	Base	Junction	42	J-8	0.000	Negative pressure at Junction J-8.
40004	Base	Junction	227	J-73	0.000	Negative pressure at Junction J-73.
40004	Base	Junction	229	J-74	0.000	Negative pressure at Junction J-74.
40004	Base	Junction	231	J-75	0.000	Negative pressure at Junction J-75.
40004	Base	Junction	239	J-78	0.000	Negative pressure at Junction J-78.
40004	Base	Junction	241	J-79	0.000	Negative pressure at Junction J-79.
40004	Base	Junction	243	J-80	0.000	Negative pressure at Junction J-80.
40004	Base	Junction	247	J-82	0.000	Negative pressure at Junction J-82.
40004	Base	Junction	251	J-84	0.000	Negative pressure at Junction J-84.
40004	Base	Junction	259	J-86	0.000	Negative pressure at Junction J-86.
40004	Base	Junction	261	J-87	0.000	Negative pressure at Junction J-87.
40004	Base	Junction	276	J-94	0.000	Negative pressure at Junction J-94.
40004	Base	Junction	280	J-95	0.000	Negative pressure at Junction J-95.
40004	Base	Junction	288	J-98	0.000	Negative pressure at Junction J-98.
40004	Base	Junction	323	J-107	0.000	Negative pressure at Junction J-107.
40004	Base	Junction	227	J-73	0.000	Negative pressure at Junction J-73.
40004	Base	Junction	229	J-74	0.000	Negative pressure at Junction J-74.
40004	Base	Junction	231	J-75	0.000	Negative pressure at Junction J-75.
40004	Base	Junction	235	J-77	0.000	Negative pressure at Junction J-77.
40004	Base	Junction	239	J-78	0.000	Negative pressure at Junction J-78.
40004	Base	Junction	241	J-79	0.000	Negative pressure at Junction J-79.
40004	Base	Junction	243	J-80	0.000	Negative pressure at Junction J-80.
40004	Base	Junction	245	J-81	0.000	Negative pressure at Junction J-81.
40004	Base	Junction	247	J-82	0.000	Negative pressure at Junction J-82.
40004	Base	Junction	249	J-83	0.000	Negative pressure at Junction J-83.
40004	Base	Junction	251	J-84	0.000	Negative pressure at Junction J-84.
40004	Base	Junction	257	J-85	0.000	Negative pressure at Junction J-85.
40004	Base	Junction	259	J-86	0.000	Negative pressure at Junction J-86.
40004	Base	Junction	261	J-87	0.000	Negative pressure at Junction J-87.
40004	Base	Junction	272	J-92	0.000	Negative pressure at Junction J-92.
40004	Base	Junction	276	J-94	0.000	Negative pressure at Junction J-94.
40004	Base	Junction	280	J-95	0.000	Negative pressure at Junction J-95.
40004	Base	Junction	284	J-96	0.000	Negative pressure at Junction J-96.
41605	Base	(N/A)	-1	(N/A)	0.000	More than 20 negative pressures reported this timestep (only the first 20 are lis

Appendix 3: Model calibration regression results

SUMMARY OUTPUT								
Regression Statistics								
Multiple R	0.96							
R Square	0.928							
Adjusted R Square	0.904							
Standard Error	1.22							
Observations	5							
ANOVA								
	df	*SS*	*MS*	*F*	*Significance F*			
Regression	1	58.28	58.28	38.95	0.0082			
Residual	3	4.48	1.49					
Total	4	62.77						
	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 99.0%*	*Upper 99.0%*
Intercept	0.302	1.07	1.54	0.22	-1.76	5.09	-4.63	7.9588
X Variable 1	0.462	0.20	6.24	0.0082	0.615	1.89	0.081	2.42
RESIDUAL OUTPUT								
Observation	Predicted	Residuals	Standard Residuals					
1	3.596108936	-1.29611	-1.2236					
2	5.163653659	0.336346	0.317529					
3	6.952712909	0.347287	0.327857					
4	8.00129867	1.398701	1.320448					
5	13.58622583	-0.78623	-0.74224					

Appendix 4: Representation of pressure value (upper zone); for peak demand time

Sampling point	Measured Time (LT)	Computed pressure (m)	Observed pressure (m)	Location		
				X(m)	Y(m)	Z(m)
J-69	2:30	20.47	21.00	604,014.61	895,736.95	2,910.00
J-92	2:45	47.11	46.75	604,173.55	896,035.09	2,916.80
J-1	3:25	77.6	76.27	603,125.26	896,134.07	2,905.00
J-34	4:00	97.07	97.25	603,758.49	895,361.32	2,973.70
J-105	4:30	198.32	198.00	604,067.83	896,121.22	2,983.70

Appendix 5: Representation of pressure value (lower zone); for peak demand time

Sampling Point	Measured Time (LT)	Computed pressure (m)	Observed pressure (m)	Location		
				X(m)	Y(m)	Z(m)
J-110	2:00	32.42	32.00	604,132.00	895,675.26	2,900.00
J-54	2:45	55.89	56.07	603,645.78	895,689.02	2,898.90
J-150	3:30	67.79	68	604,034.37	895,547.70	2,897.50
J-208	4:10	75.31	75	603,854.25	896,978.00	2,897.60
J-226	4:45	184.14	185.19	603,960.25	896,870.25	2,895.70

Appendix 6: pump result; calculated water result

Time (Hr)	Calculated water power (kw)
0:00:00	31.2
1:00:00	31.2
2:00:00	31.2
3:00:00	31.2
4:00:00	0.00
5:00:00	0.00
6:00:00	26.30
7:00:00	13.48
8:00:00	18.50
9:00:00	0.00
10:00:00	0.00
11:00:00	26.14
12:00:00	13.40
13:00:00	18.24
14:00:00	30.60
15:00:00	0.00
16:00:00	0.00
17:00:00	26.20
18:00:00	19.12
19:00:00	0.00
20:00:00	0.00
21:00:00	0.00
22:00:00	25.18
23:00:00	0.00
24:00:00	29.50

Appendix 7: Flex table of extended state simulation for Pipe report result run; during average day demand time

Label	Length (m)	Start Node	Stop Node	Diameter (mm)	Material	Hazen-Williams C	Flow (L/s)	Velocity (m/s)	Head loss Gradient (m/m)
P-2	520	J-2	J-3	85	PVC	130	10.12	0.8	0.010
P-6	650	J-6	J-7	85	PVC	130	12.98	1.18	0.020
P-7	455	J-7	J-8	85	PVC	130	5.32	1.26	0.023
P-8	1365	J-8	J-9	85	PVC	130	5.52	1.5	0.031
P-10	585	J-11	J-12	85	HDPE	130	-0.43	0.55	0.005
P-11	780	J-12	J-13	85	HDPE	130	-0.23	0.61	0.006
P-12	780	J-13	J-14	85	HDPE	130	-0.08	0.17	0.001
P-13	845	J-14	J-15	85	HDPE	130	12.32	0.5	0.004
P-14	715	J-15	J-16	85	PVC	130	13.12	0.35	0.002
P-15	520	J-16	J-17	85	PVC	130	14.15	0.92	0.013
P-16	585	J-17	J-18	85	PNC	130	14.63	0.98	0.014
P-17	715	J-18	J-19	85	PVC	130	14.52	1.01	0.015
P-18	715	J-19	J-20	85	PVC	130	14.56	1.04	0.016
P-19	520	J-20	J-21	85	PVC	130	14.25	0.92	0.013
P-20	520	J-21	J-22	85	PVC	130	14.26	0.94	0.013
P-21	910	J-22	J-23	85	PVC	130	14.19	0.97	0.014
P-23	650	J-24	J-25	85	PVC	130	14.85	2.13	0.060
P-24	780	J-25	J-26	85	PVC	130	12.23	0.44	0.003
P-27	650	J-28	J-29	85	PVC	130	12.15	1.48	0.030
P-29	455	J-30	J-31	85	PVC	130	8.12	4.57	0.246

P-30	520	J-31	J-32	85	PVC	130	-0.14	4.73	0.262
P-33	585	J-34	J-35	85	PVC	130	-0.1	0.08	0.029
P-34	390	J-35	J-36	85	HDPE	130	9.25	0.09	0.027
P-35	780	J-36	J-37	85	HDPE	130	-2.38	0.1	0.003
P-36	650	J-37	J-38	85	HDPE	130	-10.17	1.07	0.008
P-37	780	J-38	J-39	85	HDPE	130	-11.57	0.57	0.032
P-38	585	J-39	J-40	85	HDPE	130	2.36	0.31	0.000
P-39	455	J-40	J-41	85	DCI	130	-0.37	0.27	0.007
P-40	650	J-41	J-42	85	DCI	130	14.25	0.15	0.048
P-41	390	J-42	J-43	85	DCI	130	14.56	0.02	0.020
P-42	585	J-43	J-44	100	DCI	85	14.89	0.04	0.023
P-43	715	J-45	J-46	120	DCI	130	10.25	0.14	0.162
P-44	715	J-46	J-47	100	DCI	130	6.25	0.3	0.068
P-45	975	J-47	J-48	85	DCI	130	-2.94	0.43	0.028
P-46	715	J-48	J-49	85	DCI	130	-0.32	0.56	0.019
P-47	715	J-49	J-50	85	HDPE	130	4.86	0.7	0.032
P-48	650	J-50	J-51	85	HDPE	130	10.63	0.43	0.003
P-50	390	J-52	J-53	140	HDPE	130	9.25	0.16	0.004
P-51	325	J-53	J-54	260	HDPE	130	11.89	0.05	0.020
P-53	585	J-55	J-56	85	HDPE	130	12.36	0.43	0.020
P-54	910	J-56	J-57	85	HDPE	130	12.87	0.43	0.026
P-55	585	J-57	J-58	85	HDPE	130	9.25	0.43	0.009
P-56	650	J-58	J-59	85	PVC	130	14.25	0.43	0.003
P-57	585	J-59	J-60	85	PVC	130	8.45	0.43	0.003

P-58	520	J-60	J-61	85	PVC	130	10.75	0.68	0.007
P-60	650	J-9	J-24	85	PVC	130	10.26	1.53	0.032
P-61	780	J-51	J-61	85	PVC	130	10.36	0.51	0.004
P-62	715	J-61	J-62	85	PVC	130	10.87	0.17	0.001
P-65	130	J-61	J-65	85	PVC	130	10.56	1.11	0.018
P-66	520	J-65	J-66	85	PVC	130	6.91	1.17	0.020
P-68	455	J-66	J-67	85	PVC	130	12.36	0.79	0.010
P-69	520	J-67	J-24	85	PVC	130	12.36	0.43	0.003
P-70	520	J-24	J-67	85	PVC	130	13.2	0.43	0.003
P-73	715	J-69	J-70	85	PVC	130	5.58	0.97	0.014
P-74	585	J-70	J-25	85	PVC	130	10.12	2.37	0.073
P-75	520	J-25	J-71	85	PVC	130	23.4	4.93	0.283
P-76	715	J-71	J-72	85	PVC	130	10.25	5.01	0.292
P-77	780	J-72	J-73	165	PVC	130	10.25	1.36	0.012
P-78	520	J-73	J-74	85	PVC	130	8.26	5.23	0.316
P-79	585	J-23	J-75	85	PVC	130	-9.04	1.57	0.034
P-80	390	J-75	J-76	85	PVC	130	-18.1	1.74	0.041
P-81	650	J-76	J-26	85	PVC	130	11.49	2.06	0.056
P-84	585	J-26	J-78	85	PVC	130	12.36	1.17	0.020
P-85	520	J-78	J-79	85	PVC	130	12.75	1.14	0.019
P-87	845	J-79	J-80	120	PVC	130	12.56	0.44	0.002
P-88	715	J-80	J-81	120	PVC	130	12.56	0.47	0.002
P-89	845	J-81	J-82	120	PVC	130	10.25	0.48	0.003
P-90	650	J-82	J-83	120	PVC	130	10.25	0.51	0.003

P-93	910	J-8	J-20	85	PVC	130	2.99	0.23	0.001
P-94	650	J-23	J-86	85	PVC	130	14.61	0.48	0.004
P-95	780	J-86	J-34	85	PVC	130	10.12	0.96	0.014
P-96	455	J-86	J-87	85	PVC	130	15.12	0.56	0.005
P-97	650	J-87	J-88	85	PVC	130	12.1	0.57	0.005
P-98	780	J-79	J-89	85	PVC	130	10.36	2.03	0.054
P-99	1300	J-89	J-37	85	PVC	130	15.12	1.28	0.023
P-100	715	J-88	J-89	85	PVC	130	15.45	0.58	0.005
P-101	715	J-14	J-90	85	PVC	130	15.63	0.72	0.008
P-102	845	J-90	J-91	85	PVC	130	12.45	0.62	0.006
P-103	390	J-91	J-92	85	PVC	130	8.15	0.49	0.004
P-104	650	J-92	J-93	85	PVC	130	8.69	0.47	0.004
P-105	650	J-93	J-94	85	PVC	130	8.25	0.4	0.003
P-106	715	J-94	J-95	85	PVC	130	10.45	0.36	0.002
P-107	1300	J-13	J-4	85	PVC	130	10.75	0.01	0.000
P-108	195	J-3	J-96	85	HDPE	130	10.25	0.94	0.013
P-109	520	J-96	J-4	85	HDPE	130	10.63	0.53	0.004
P-110	520	J-4	J-96	85	HDPE	130	10.25	0.53	0.004
P-111	845	J-11	J-97	85	HDPE	130	12.14	0.49	0.004
P-112	650	J-97	J-98	85	HDPE	130	10.26	0.49	0.004
P-113	715	J-98	J-99	85	DCI	130	10.36	0.45	0.003
P-114	585	J-99	J-100	85	DCI	130	10.25	0.42	0.003
P-115	650	J-1	J-101	85	DCI	130	10.34	0.3	0.002
P-116	520	J-101	J-102	85	DCI	130	5.93	0.25	0.001

P-117	845	J-102	J-103	85	DCI	130	-10.15	0.2	0.001
P-118	585	J-103	J-104	85	DCI	130	-0.94	0.07	0.000
P-119	1300	J-104	J-100	85	DCI	130	21.5	0.23	0.001
P-120	910	J-100	J-95	85	DCI	130	20.15	0.07	0.000
P-121	845	J-100	J-105	85	DCI	130	13.25	0.25	0.001
P-122	585	J-62	J-106	85	DCI	130	10.25	0.66	0.007
P-123	650	J-106	J-107	85	PVC	130	10.63	0.7	0.008
P-124	650	J-107	J-108	85	PVC	130	10.85	0.6	0.006
P-125	715	J-108	J-109	85	PVC	130	10.45	0.48	0.004
P-128	455	J-111	J-112	85	PVC	130	10.89	0.06	0.006
P-129	715	J-112	J-113	85	PVC	130	10.78	0.06	0.009
P-130	520	J-113	J-114	85	PVC	130	9.75	0.08	0.000
P-131	780	J-39	J-115	85	PVC	130	14.25	0.15	0.009
P-132	585	J-115	J-116	85	PVC	130	14.25	0.23	0.013
P-133	260	J-116	J-117	85	PVC	130	14.63	0.2	0.007
P-134	325	J-117	J-118	85	PVC	130	14.52	0.18	0.010
P-135	325	J-118	J-119	85	PVC	130	14.89	0.15	0.650
P-136	455	J-119	J-120	85	PVC	130	14.56	0.13	0.330
P-138	585	J-45	J-121	150	PVC	130	14.57	0.07	0.240
P-140	520	J-29	J-124	85	PVC	130	9.56	1.44	0.029
P-141	650	J-124	J-125	85	HDPE	130	9.25	1.36	0.026
P-142	585	J-125	J-126	85	HDPE	130	9.47	1.26	0.023
P-146	325	J-120	J-44	85	HDPE	130	12.25	0.11	0.000
P-147	845	J-76	J-24	85	HDPE	130	10.16	0.25	0.001

P-149	455	J-130	T-1	85	HDPE	130	10.96	0	0.000
P-150	715	T-1	J-132	85	HDPE	130	15.21	8.1	0.708
P-151	585	J-132	J-74	85	HDPE	130	15.54	8.1	0.708
P-152	715	J-74	J-32	85	HDPE	130	15.75	2.76	0.097
P-153	715	J-32	J-133	85	HDPE	130	2.94	1.96	0.051
P-154	845	J-133	T-1	85	HDPE	130	-0.76	10.47	1.140
P-155	325	ESP-Keta Spring	PMP-1	85	PVC	130	8.14	(N/A)	(N/A)
P-156	325	PMP-1	J-130	85	PVC	130	4.59	(N/A)	(N/A)
P-164	585	J-52	J-113	180	PVC	130	-0.21	0.01	0.000
P-165	910	J-20	J-138	85	PVC	130	-0.15	0.1	0.000
P-166	650	J-138	J-139	85	PVC	130	10.98	0.85	0.011
P-167	650	J-139	J-140	85	PVC	130	10.45	0.82	0.010
P-168	520	J-140	J-141	85	PVC	130	5.18	0.79	0.009
P-169	455	J-141	J-142	85	PVC	130	2.7	0.76	0.009
P-170	390	J-142	J-143	85	PVC	130	1.56	0.73	0.008
P-171	520	J-15	J-143	85	PVC	130	10.63	0.21	0.001
P-172	520	J-138	J-144	85	PVC	130	15.25	0.75	0.009
P-173	455	J-144	J-145	85	PVC	130	10.14	0.78	0.009
P-174	650	J-145	J-34	85	PVC	130	10.86	0.8	0.010
P-176	1430	J-4	J-6	85	PVC	130	5.25	1.14	0.019
P-177	780	J-26	J-146	85	PVC	130	-17	2.93	0.108
P-178	520	J-146	J-30	85	PVC	130	-25	4.43	0.232
P-179	585	J-146	J-28	85	PVC	130	9	1.51	0.031

P-180	910	J-143	J-147	85	PVC	130	3	0.46	0.003
P-185	650	J-147	J-14	85	PVC	130	3	0.44	0.003
P-186	650	J-13	J-16	85	PVC	130	-3	0.51	0.004
P-187	585	J-103	J-150	85	PVC	130	-1	0.13	0.000
P-188	715	J-150	J-151	85	PVC	130	-1	0.18	0.001
P-189	585	J-151	J-152	85	PVC	130	-1	0.23	0.001
P-190	585	J-152	J-153	85	HDPE	130	-2	0.27	0.001
P-191	455	J-153	J-154	85	HDPE	130	-2	0.3	0.002
P-193	715	J-154	J-2	85	HDPE	130	-2	0.38	0.002
P-194	650	J-1	J-155	85	HDPE	130	-2	0.37	0.002
P-195	715	J-155	J-2	85	DCI	130	-1	0.2	0.001
P-196	715	J-2	J-155	85	DCI	130	1	0.2	0.001
P-197	455	J-126	J-83	85	DCI	130	7	1.22	0.021
P-198	390	J-83	J-156	85	DCI	130	0	0.08	0.550
P-202	715	J-111	J-160	85	DCI	130	-1	0.12	0.380
P-203	780	J-160	J-109	85	DCI	130	-2	0.34	0.630
P-204	455	J-159	J-160	85	PVC	130	-1	0.15	0.007
P-206	520	J-54	J-161	85	PVC	130	-2	0.43	0.630
P-207	585	J-161	J-55	85	PVC	130	-1	0.22	0.850
P-208	585	J-55	J-161	85	PVC	130	1	0.22	0.001
P-209	585	J-159	J-162	85	PVC	130	0	0.02	0.055
P-210	520	J-62	J-69	85	PVC	130	-5	0.85	0.095
P-211	585	J-68	J-163	85	PVC	130	-5	0.96	0.089
P-212	715	J-163	J-69	85	PVC	130	0	0.05	0.009

P-213	715	J-69	J-163	85	PVC	130	0	0.05	0.026
P-214	585	J-162	J-164	85	PVC	130	0	0.02	0.105
P-215	845	J-164	J-114	85	PVC	130	0	0.04	0.000
P-217	650	J-113	J-166	85	PVC	130	1	0.1	0.023
P-221	650	J-168	J-114	85	PVC	130	0	0.04	0.162
P-222	650	J-114	J-168	85	PVC	130	0	0.04	0.068
P-223	910	J-121	J-169	200	PVC	130	-1	0.04	0.028
P-224	845	J-169	J-52	85	PVC	130	-1	0.26	0.019
P-225	520	J-52	J-170	85	PVC	130	0	0.02	0.032
P-226	910	J-163	J-70	85	PVC	130	-5	0.88	0.000
P-227	1170	J-70	J-66	85	PVC	130	3	0.48	0.032
P-229	715	J-171	J-115	85	PVC	130	2	0.36	0.043
P-231	715	J-68	J-172	85	PVC	130	2	0.29	0.023
P-232	650	J-172	J-106	85	PVC	130	1	0.21	0.033
P-233	650	J-106	J-60	85	PVC	130	1	0.11	0.017
P-234	780	J-60	J-50	85	PVC	130	2	0.31	0.012
P-235	520	J-112	J-164	85	PVC	130	0	0.04	0.066
P-236	585	J-164	J-173	85	PVC	130	-1	0.13	0.066
P-237	585	J-173	J-174	85	PVC	130	-2	0.3	0.088
P-238	845	J-174	J-175	85	PVC	130	-3	0.52	0.029
P-241	910	J-176	J-68	85	PVC	130	-2	0.32	0.000
P-242	910	J-68	J-176	85	HDPE	130	2	0.32	0.021
P-245	520	J-168	J-178	85	HDPE	130	-1	0.12	0.033
P-246	390	J-178	J-173	85	HDPE	130	0	0.07	0.084

P-247	390	J-173	J-178	85	HDPE	130	0	0.07	0.030
P-250	780	J-175	J-179	85	HDPE	130	-3	0.58	0.029
P-251	650	J-179	J-176	85	HDPE	130	-4	0.63	0.121
P-253	520	J-38	J-171	85	HDPE	130	2	0.42	0.020
P-255	845	J-105	J-181	85	HDPE	130	-1	0.1	0.050
P-256	455	J-181	J-180	85	HDPE	130	0	0.03	0.049
P-257	455	J-180	J-181	85	HDPE	130	0	0.03	0.166
P-258	650	J-181	J-182	85	HDPE	130	0	0.03	0.038
P-260	1170	J-183	J-184	85	HDPE	130	0	0.06	0.084
P-261	845	J-184	J-185	85	PVC	130	0	0.03	0.172
P-262	585	J-185	J-186	85	PVC	130	0	0.05	0.126
P-264	845	J-186	J-188	85	PVC	130	0	0.08	0.450
P-267	585	J-188	J-105	85	PVC	130	-1	0.16	3.230
P-268	715	J-105	J-189	85	PVC	130	1	0.1	1.360
P-269	520	J-189	J-183	85	PVC	130	1	0.14	0.550
P-270	715	J-105	J-189	85	PVC	130	1	0.1	0.380
P-271	585	J-186	J-104	85	PVC	130	-1	0.22	0.630
P-272	780	J-185	J-190	85	PVC	130	0	0.06	0.007
P-273	1365	J-186	J-191	85	PVC	130	0	0.09	0.630
P-275	1105	J-95	J-181	85	PVC	130	1	0.2	0.850
P-276	455	J-92	J-192	85	PVC	130	0	0.02	0.450
P-277	780	J-143	J-193	85	PVC	130	0	0.04	0.650
P-278	845	J-4	J-194	85	PVC	130	0	0.06	0.330
P-279	520	J-34	J-195	85	PVC	130	1	0.1	0.240

P-280	455	J-37	J-196	85	PVC	130	0	0.04	0.006
P-281	390	J-196	J-197	85	PVC	130	0	0.02	0.010
P-282	520	J-196	J-198	85	PVC	130	0	0	0.025
P-283	455	J-115	J-199	85	PVC	130	1	0.22	0.009
P-284	455	J-199	J-200	85	PVC	130	0	0.08	0.009
P-285	650	J-199	J-201	85	PVC	130	1	0.09	0.036
P-286	325	J-133	J-202	200	PVC	130	48	1.54	0.006
P-287	520	J-202	J-203	155	PVC	130	0	0.01	0.015
P-288	650	J-70	J-204	85	PVC	130	0	0.05	0.015
P-289	585	J-204	J-205	85	PVC	130	0	0.01	0.050
P-290	520	J-204	J-206	85	PVC	130	0	0.03	0.011
P-291	650	J-206	J-207	85	PVC	130	0	0.02	0.025
P-292	520	J-51	J-208	85	PVC	130	0	0.07	0.052
P-293	520	J-174	J-209	85	PVC	130	0	0.06	0.038
P-295	845	J-210	J-211	85	PVC	130	0	0.08	0.004
P-296	520	J-121	J-212	85	PVC	130	2	0.38	0.020
P-297	585	J-212	J-210	85	PVC	130	1	0.24	0.020
P-298	520	J-212	J-213	85	PVC	130	1	0.1	0.026
P-299	390	J-43	J-214	85	PVC	130	0	0.07	0.009

Appendix 9: Flex table for Junction pressure results for average day demand

Label	Elevation (m)	Demand (L/s)	Hydraulic Grade (m)	Pressure (m H2O)
J-133	2,958.10	0.5	2,942.44	77.6
J-202	2,956.80	0.2	2,942.37	26.21
J-203	2,955.50	0.14	2,942.37	46
J-31	2,951.10	0.04	2,939.67	0.86
J-32	2,952.80	0.5	2,941.89	97.23
J-30	2,948.80	0.9	2,937.94	27.01
J-71	2,947.60	0.5	2,936.93	84.57
J-73	2,951.00	0.75	2,940.42	84.58
J-74	2,953.20	0.08	2,942.93	95.77
J-72	2,949.80	0.8	2,940.28	12.62
J-146	2,944.40	0.14	2,936.00	2.11
J-28	2,943.30	0.2	2,935.66	81.9
J-132	2,955.10	0.16	2,949.20	81.25
J-29	2,941.20	0.18	2,935.34	81.22
J-26	2,939.90	0.14	2,934.68	47.26
J-124	2,940.00		2,935.07	
J-125	2,938.70	0.14	2,934.80	12.5
J-70	2,937.60	0.18	2,934.10	96.19
J-78	2,937.90	0.18	2,934.49	81.32
J-69	2,936.80	0.14	2,933.94	-0.84
J-25	2,937.40	0.14	2,934.72	12.96
J-126	2,936.10	0.14	2,934.54	59.46

J-163	2,935.40	0.14	2,933.94	305
J-79	2,934.50	0.14	2,934.28	22.93
J-62	2,933.50	0.18	2,933.85	59.66
J-204	2,933.50	0.14	2,934.10	69.88
J-76	2,933.30	0.13	2,934.14	91.09
J-68	2,932.50	0.14	2,933.81	78.11
J-89	2,932.30	0.14	2,933.69	69.5
J-176	2,931.40	0.14	2,933.79	66.09
J-206	2,931.60	0.09	2,934.10	85.73
J-75	2,931.10	0.18	2,933.73	97.73
J-37	2,930.30	0.14	2,933.35	0.6
J-23	2,930.30	0.25	2,933.55	9.19
J-172	2,930.40	0.14	2,933.80	97.07
J-106	2,930.20	0.09	2,933.79	18.72
J-80	2,930.40	0.09	2,934.30	81.81
J-24	2,929.90	0.18	2,934.12	87.66
J-88	2,929.30	0.16	2,933.64	13.44
J-207	2,929.70	0.18	2,934.10	27.01
J-107	2,929.20	0.25	2,933.71	84.91
J-22	2,928.80	0.14	2,933.36	15.11
J-205	2,928.80	0.14	2,934.10	56.18
J-21	2,927.50	0.14	2,933.24	84.56
J-196	2,927.30	0.18	2,933.35	69.68
J-81	2,928.10	0.18	2,934.33	84.71

J-87	2,926.90	0.5	2,933.58	89.97
J-67	2,927.00	0.14	2,934.10	17.56
J-197	2,926.20	0.2	2,933.35	76.01
J-65	2,926.30	0.14	2,933.87	20.35
J-38	2,925.50	0.18	2,933.20	67.38
J-198	2,925.60	0.09	2,933.35	8.4
J-179	2,925.80	0.09	2,933.72	29.07
J-66	2,926.10	0.09	2,934.03	12.97
J-36	2,925.40	0.25	2,933.35	55.89
J-82	2,926.20	0.09	2,934.37	7.53
J-20	2,924.60	0.09	2,933.13	28.27
J-35	2,923.60	0.14	2,933.35	52.43
J-39	2,923.00	0.14	2,933.14	50.06
J-83	2,924.20	0.14	2,934.40	91.46
J-86	2,922.70	0.09	2,933.51	8.99
J-108	2,922.40	0.14	2,933.65	76.71
J-34	2,921.90	0.14	2,933.35	2.86
J-61	2,922.40	0.18	2,933.85	110.25
J-8	2,921.50	0.14	2,933.15	146.7
J-156	2,922.40	0.09	2,934.40	63.06
J-171	2,921.00	0.09	2,933.16	30.15
J-9	2,921.50	0.09	2,933.79	34.21
J-40	2,920.70	0.09	2,933.11	10
J-7	2,919.50	0.25	2,932.98	20.47

J-51	2,920.30	0.14	2,933.80	48.89
J-195	2,919.80	0.18	2,933.34	51.42
J-41	2,919.00	0.2	2,933.09	52.19
J-208	2,919.50	0.16	2,933.79	22.51
J-115	2,918.00	0.09	2,933.13	60.2
J-145	2,918.00	0.09	2,933.25	63.94
J-60	2,918.50	0.23	2,933.79	55.6
J-42	2,917.60	0.14	2,933.09	97.21
J-175	2,917.80	1	2,933.66	76.89
J-50	2,917.50	0.14	2,933.77	26.38
J-43	2,916.20	0.18	2,933.09	26.21
J-19	2,915.60	0.14	2,932.98	61.66
J-144	2,915.00	0.14	2,933.19	76.72
J-214	2,914.00	0.14	2,933.09	164.14
J-199	2,914.00	0.09	2,933.12	48.37
J-200	2,912.50	0.14	2,933.12	0.89
J-109	2,912.80	0.18	2,933.61	79.45
J-201	2,912.00	1	2,933.12	1.45
J-18	2,911.50	0.09	2,932.84	52.02
J-116	2,911.00	0.09	2,933.12	108.03
J-174	2,911.30	0.14	2,933.60	52.91
J-6	2,910.40	0.09	2,932.79	47.83
J-17	2,910.00	0.09	2,932.69	47.11
J-138	2,910.20	0.09	2,933.13	38.33

J-44	2,910.00	0.18	2,933.09	77.67
J-209	2,909.70	0.14	2,933.60	27.63
J-117	2,909.00	0.09	2,933.10	47.96
J-49	2,906.80	0.09	2,933.68	8.44
J-118	2,905.00	0.18	2,933.10	43.66
J-160	2,903.70	0.14	2,933.59	58.88
J-59	2,903.70	0.09	2,933.76	57.05
J-173	2,902.50	0.09	2,933.59	46.16
J-119	2,902.00	0.09	2,933.09	46.71
J-159	2,901.70	0.18	2,933.58	34.89
J-16	2,900.00	0.23	2,932.68	78.76
J-139	2,900.00	0.09	2,933.01	98.32
J-48	2,900.60	0.09	2,933.63	33.5
J-120	2,900.00	0.2	2,933.09	26.2
J-162	2,900.20	0.09	2,933.58	52.74
J-178	2,900.00	0.18	2,933.58	59.91
J-168	2,898.90	0.25	2,933.58	32.42
J-140	2,897.50	0.18	2,932.90	38.05
J-47	2,897.60	0.14	2,933.58	-1.61
J-4	2,895.70	0.09	2,932.59	48.67
J-130	2,920.00	0.09	2,957.00	48.43
J-194	2,894.80	0.14	2,932.59	84.13
J-111	2,892.80	0.13	2,933.58	83.92
J-58	2,891.60	0.23	2,933.73	-3.05

J-96	2,890.00	0.2	2,932.53	0.37
J-141	2,889.70	0.2	2,932.81	55.65
J-112	2,889.80	0.2	2,933.58	52.72
J-46	2,889.70	0.16	2,933.57	170.53
J-57	2,888.90	0.09	2,933.70	17.49
J-142	2,887.60	8.07	2,932.74	-2.56
J-13	2,887.00	0.18	2,932.59	80.49
J-56	2,887.50	0.18	2,933.65	22.65
J-45	2,886.90	0.16	2,933.56	-1.78
J-164	2,886.70	0.14	2,933.58	48.98
J-143	2,885.40	0.25	2,932.68	27.47
J-3	2,884.80	0.09	2,932.40	59.45
J-11	2,884.70	0.18	2,932.47	107.44
J-12	2,883.00	0.14	2,932.52	54.96
J-55	2,883.80	0.14	2,933.62	-0.81
J-97	2,881.90	0.14	2,932.42	59.86
J-15	2,881.00	0.14	2,932.66	91.22
J-193	2,880.00	0.18	2,932.67	91.21
J-54	2,880.80	0.09	2,933.58	48.9
J-147	2,879.00	0.14	2,932.64	48.98
J-113	2,879.50	0.23	2,933.58	-0.21
J-14	2,878.00	0.09	2,932.60	32.73
J-121	2,878.90	0.09	2,933.56	-1.84
J-53	2,878.90	0.8	2,933.58	-1.74

J-114	2,878.80	0.14	2,933.58	104.97
J-2	2,876.90	0.09	2,932.32	50.33
J-98	2,876.90	0.18	2,932.36	159.66
J-90	2,877.00	0.23	2,932.51	281.17
J-154	2,875.80	0.14	2,932.30	50.06
J-166	2,876.60	0.25	2,933.58	22.14
J-91	2,874.70	0.09	2,932.43	-3.4
J-92	2,872.80	0.23	2,932.41	97.29
J-99	2,872.60	0.14	2,932.32	67.79
J-192	2,870.90	0.05	2,932.41	105.15
J-100	2,868.80	0.09	2,932.26	69.44
J-93	2,866.90	0.09	2,932.35	52.5
J-94	2,866.80	0.14	2,932.32	152.31
J-212	2,867.90	0.2	2,933.54	31.74
J-95	2,865.70	0.09	2,932.26	40.02
J-105	2,865.60	0	2,932.24	63.13
J-210	2,865.90	0	2,933.53	41.54
J-169	2,865.90	0	2,933.56	39.57
J-181	2,863.70	0.14	2,932.24	63.88
J-155	2,862.80	0.2	2,932.31	70.3
J-188	2,862.20	0.14	2,932.24	110.18
J-180	2,861.60	0.14	2,932.24	134.19
J-153	2,860.60	0.18	2,932.28	1.5
J-52	2,861.70	0.18	2,933.58	59.19

J-182	2,860.20	0.18	2,932.24	68.62
J-189	2,860.10	0.16	2,932.24	92.9
J-170	2,859.80	0.09	2,933.58	55.69
J-183	2,856.90	0.09	2,932.24	105.39
J-1	2,856.70	0.09	2,932.29	99.68
J-211	2,857.90	0.14	2,933.53	60.73
J-152	2,854.90	0.14	2,932.26	-0.25
J-101	2,854.50	0.09	2,932.27	61.64
J-213	2,855.70	0.09	2,933.54	-3.47
J-184	2,852.80	0.2	2,932.23	-109.06
J-151	2,848.70	0.8	2,932.25	24.45
J-102	2,848.70	0.14	2,932.26	111.97
J-150	2,846.80	0.14	2,932.25	68.32
J-103	2,844.80	0.18	2,932.24	-31.29
J-104	2,843.80	0.2	2,932.24	114.16
J-186	2,839.70	0.14	2,932.24	204.06
J-191	2,838.60	0.14	2,932.23	105.79
J-185	2,835.60	0.14	2,932.23	-2.66
J-190	2,834.80	0.14	2,932.23	101.7
J-161	2,823.60	0.32	2,933.62	53.12