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In Holeta Town Wolmera Woreda West (Shewa Zone of Oromia region, Ethiopia)

Bachelor Thesis

Bibliographic information published by the German National Library:

The German National Library lists this publication in the National Bibliography; detailed bibliographic data are available on the Internet at http://dnb.dnb.de .

Imprint:

ISBN: 9783668651753

This book at GRIN:

https://www.grin.com/document/414284

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GRIN Verlag

WATER SUPPLY DISTRIBUTION SYSTEM DESIGN (IN HOLETA TOWN WOLMERA WOREDA WEST SHEWA ZONE OF OROMIA REGION, ETHIOPIA)

A Thesis Submitted to Arba Minch University Institute of Technology in Partial Fulfillment of the Requirements for the Degree of Bachelor of Science in Water Supply And Environmental Engineering

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Yonas Assefa

Arba Minch, Ethiopia
June 2016

ACKNOWLEDGEMENT

We want to provide our great full thanks to our advisors, Mr. Zelalem Abera (MSc), Mr. Sabkeebar Ararsa (BSc) next to God; for all his sincere, faithful and immense devotion to help us in providing all the necessary materials which are paramount for our final year project.

ABSTRACT

The provision of clean Water Supply is one of the major factors that greatly contribute to the socioeconomic transformation of a country by improving the health thereby increasing life standard and economic productivity of the society. However, most of the developing country like Ethiopia has still low potable water supply and sanitation coverage that result the citizens to be suffered from water Shortage, water born and water related diseases. A good water supply distribution infrastructure plays a key role for any kind development for a town. This project examined the theoretical framework for the design of an improved water distribution network for Holeta town. The aim of this water supply project is to provide potable water for present and future demand for targeted Holeta town which improve the existing water supply system of the town. The present and future population of the study area was determined and the water demand per day established. The hydrologic, hydro geologic and topographic data formed the basis of the design while laying emphasis on models and theories of pipe networking and performance. The pipe network layout was analyzed with the use of Epanet2.0 software which is based on Hazen William's equation.

Key Words: *EPA-NET software, population projection (forecast), pressure head, velocity head, water demand assessment and water distribution network system.*

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ABBRIVATIONS

Asl……………………...........................Above sea level

ADD……………………………………Average Day Demand

BH…………………………..............Borehole

BOQ…………………………….……..Bill Of Quantities

c/c……………….……………..……Center to Center

CSA…………………………............Central Statistical Agency

DCI……………………………………...Ductile Cast Iron

DL …………………………………..…. Dead load

DN…………………………................Nominal Diameter

EIA…………………………………..….Environmental Impact Assessment

ETB……………………………………...Ethiopian Birr

GS……………………………………......Galvanised Steel

L/C/D…………………………............Liter per capita per day

HC………………………………………. House Connection

HCU…………………………………..... House Connection Users

Hr.………………………………………..Hour

HTP……………………………………....House tap connection

HTU……………………………….….House Tap Users

MDD……………………………………Maximum Daily Demand

MDF……………………………………..Maximum Daily Factor

MoWR……………………………………Ministry of Water Resources

NDD……………………………………..Non-domestic Demand

PHD……………………………………..Peak Hour Demand

PHF……………………………………... Peak Hour factor

PN………………………………………..Nominal pressure category, in bar

PTU……………………………………...Public Tap users

PVC……………………………………...Poly Vinyl Chloride

Qty………………………………………Quantity

RC………………………………………Reinforced Concrete

RM……………………………………Raising Main

TDD……………………………………..Total Domestic Demand

TDH……………………………………..Total dynamic head

TWSS……………………………………Town Water & Sanitation Services

TWSSE …………………………………Town Water Supply and Enterprise

UFW……………………………………Unaccounted For Water

UWD…………………………………… Unaccounted Water demand

UPVC……………………………………Unplasticised Poly-Vinyl Chloride

WWDSE………………………………...Water Works Design and Supervision Enterprise

WHO……………………………………World Health Organization

YCS……………………………………..Yard Tap Shared

YTC……………………………………..Yard Tap Connection

YTS……………………………………...Yard Tap Shared

YTU……………………………………..Yard Tap Users

NOMENCLATURE

Unit	Description
°C	Degrees centigrade
Cm	Centimetre
d_{req}	Required diameter
kg	Kilogram
km	Kilometre
km^2	Square kilometres
l,lit	Litres
l/c/d	Litres per capita per day
l/s	Litres per second
lpm	Litres per minute
m	Meter
m^2	Square meter
m^3	Cubic meter
mg/l	Milligram per litre
m/sec	Meter per second
m^3/day	Cubic meters per day
m^3/hr	Cubic meters per hour
mm	Millimetre
m^3/hr	Meter cube per hour
N/mm^2	Newton per square millimetre
Ø	Diameter
@	At
%	Percent

1. INTRODUCTION

1.1 General

Water is one of the necessities for human being and for all living things. Water means nothing but just life as it constitutes the major part of the core of the cell, the protoplasm which is about 70% in content of the cell , even though water is a critical necessity for life , it has on adverse effect to life unless and other wise properly handled.

In the world clean water that can be used for domestic purpose is not more than 2% of the natural water resources of the earth. (Source; WHO, 2009).This is very small in amount of wholesome water comparing with the saline water body. As the result of this the world is faced to the shortage of sufficient access of safe drinking water. The developing countries of the world are specially affected by the problem of safe access of drinking water supply. This is because of the lack of technologies and financial supports to utilize their water resources.

Ethiopia is very well known for its enormous potential all of which is generated in its own tertiary and it is still known the water towers in Africa. Access of water supply in Ethiopia is amongst the lowest in sub-Saharan Africa and the entire world, while access has increased substantially with funding from external aid much fill remind to be done to achieve the millennium development goal of halving the share of people without access to water and sanitation. To achieve this goal the Ethiopian government is working to address the problem of safe access of drinking water in different towns of the country

The accessibility to safe water in Ethiopia is about 23 %. That is a very low level when compared with the 54 % average for the Sub-Sahara area (UNDP). In the modern society, it is imperative to Plan and build sustainable water supply scheme which can provide potable water in accordance with their demands and requirements for Human and livestock.The use of water for mankind, plant and animal is universal. Without water, there is no existence of life. The access to a safe and affordable water supply for drinking universally recognized as a basic human need.

Water distribution systems carry drinking water from a centralized treatment plant or well supplies to consumers' taps. These systems consist of pipes, pumps, valves, storage tanks, reservoirs, meters, fittings, and other hydraulic appurtenances *(Drinking Water Distribution Systems assessing and Reducing Risks, page 2).* Distribution system infrastructure is generally the major asset of a water utility. The *American Water Works Association* (*AWWA, 1974*) defines

the water distribution system as “including all water utility components for the distribution of finished or potable water by means of gravity storage feed or pumps though distribution pumping networks to customers or other users, including distribution equalizing storage.”
Therefore; Water distribution networks design and analysis play an important role in modern societies being its proper operation directly related to the population’s well-being. In Holeta whereas a town which is at fast growing stage, is very imperative in modern society to insure the availability of potable water and to plan and design for a sustainable economic suitable pipe network system or water supply schemes.

1.2 Statement of the problem

In Sub-Saharan Africa in general and particularly in Ethiopia beyond the impact of climate change on water availability, other major factors such as: population growth and poor water governance are exacerbating the water supply situation of the countries (Ndaruzaniye, 2011). Based on the 2010 statistical abstract published by CSA there are 970 towns in Ethiopia, 45% of the urban population live in 907 towns with less than 30,000 population. 55 % the urban population live in 63 towns with greater than 30,000 population. They all need piped water supply systems, and local utilities to oversee and operate them. The strategic action is to construct facilities that are well managed and can be expanded to meet the needs of a growing population.
The water supply in Ethiopia, where large portion of the population are challenged due to poor institutional, infrastructural and socio-demographic factors. Moreover, poor accountability and lacks of community participation in water projects were identified as constraints of sustainability (Yacob, et al, 2010). Similarly the study of Aschalew (2009) revealed the absence of community participation and technical constraints are responsible for frequent water interruption and sustainability challenges of urban water supply projects. As a result in spite of the vigorous efforts made to improve the coverage and the system of water supply in country, the functionality rate of water supply source in Ethiopia in 2007 was about 33 % percent (Tamene, et al., 2011). According to the World Health Organization, between 1990 and 2015 the access to improved drinking water sources increased from 13.2 per cent to 57.3 per cent.
In Ethiopia, after the intervention of the Multiple Use Service approach by the non-government projects, the productive role of water to the urban population has got great momentum by the

Ministry of Water (Butterworth, et al 2011). But except a mere recognition of the productive role of water, the nature and the factors of productive water use at household level has no yet studied. Even attempts are shown to give much emphasis to link water and urban farming while the domestic home based productive use of water by the urban households are overlooked (James, 2003). Moreover most of the studies conducted in Ethiopia concerning water supply and sanitation have been found to focus on either the supply or demand part of water research which overlooked the equilibrium between urban water supply and multiple needs of water (Kebede, 2003; Zelalem, 2005; and Gossaye, 2007). Similarly studies that are aimed at identifying the factors that affect the demand of water by the households found to pay little or no attention to include the productive demand of water, as a result aggregate demand are used to investigate the determinants of water consumption level. For instance the study of Bihon, (2006), Sileshi, (2008) and Dessalegn, (2012) assessed the main factors for water consumption level, demand for improved water and aggregate water demand respectively without consideration of the productive use of water by the households.

Though the water supply challenges remained intact in various parts of the country, adequate water supply to the urban as well as rural people remained one of the most crucial resources for survival, health and prosperity (WHO, 2006). Particularly to the rural poor water plays a significant contribution as a direct input into agricultural production and as the basis for health and welfare (Narcisse, 2010).

According to the data from the feasibility study, the majority of Holeta town population is partially supplied by town's water supply system. It is reported that the demand of water couldn't fulfilled the required demand of water supply. This occurs due to rapidly growth of Population, expansion of the town and development of economy of the town. It is reported that 90% of householders collect part of their total water needs from the town's water supply system fed directly through private connections or public taps. However, there is insufficient water to meet all demands and the deficit is made up from other sources including wells and water vendors. Though the existing infrastructure is old and in poor condition, interruptions of water supply were occurred from time to time.

1.3 Objective of the project

1.3.1 Main objective

The project aimed to improve the water supply and sanitation system both in quality and quantity at a reasonable cost without affecting the environmental circumstance of the project area.

1.3.2 Specific objective

The specific objective of this project is to:-

- ❖ To provide adequate and sufficient water to the town of Holeta.
- ❖ To solve the problem of water born disease by providing potable water.
- ❖ To develop new water sources, increase the number of pipes lines and construct additional reservoir, and
- ❖ To extend the distribution system to those areas of the town.

1.4 Expected output

At the end of design we expected that the water supply and sanitation system of the Holeta town will be improved.

- ❖ Potable and sufficient water should be provided
- ❖ Problem of water borne disease should be reduced or eliminated.

1.5 The research questions

The general and specific objectives of the study would be achieved by way of seeking answers to the following questions.

1. What are the types of existing water sources and supply in Holeta town?
2. What is the state of existing water supply?
3. Why demand for water exceeds the supply of water?
4. Do the urban communities have willingness to pay higher price for improved water service than the existing water supply service?
5. What are the comments of beneficiaries on the proposed strategy of water supply by the government?
6. What are the major challenges of water supply in the town?
7. What are the factors affecting water supply and consumption of Holeta town?

1.6 Significance of the study

Studying the extent and coverage and dynamics of urban water supply service in holeta helps to identify the pressing problems in service delivery. This study is expected to increase the knowledge and up to date information on the city water supply size and its undesirable impacts on the urban community due to shortage of water supply. It will also serve as a working document to policy makers in the water sector of oromia Regional state (OGRS), especially policy making bodies, and the Holeta town water supply and sanitation authority and the Non-governmental organizations (NGOs) which have interest in assisting Holeta town with financial and technical support in the area of urban water supply. Moreover, the finding will further serve as reference data and it opens avenue for any further investigation in the area, and as a useful material for academic purposes.

1.7 The scope of the study

The objective of this research is to present the fundamental concept of hydraulics applied to holeta town water supply network, in order for municipal officials of the town to a better evaluation and decision making of water distribution and delivery systems. Therefore, the research work was limited the design of water distribution network (from clear water well to distribution end point) of holeta town water supply system in West Shewa Zone of Oromia region of Ethiopia and it mainly focus on the determination of the population served, water demand assessment, distribution system design, selection of source which fulfill the demand of the town with borehole design and environmental impact assessment.

1.8 Limitation of the Study

Due to some major challenges associated with the collection of the data, the study was exposed for some limitations. The first limitation of the study was associated with availability of sufficient information regarding to the study area water supply distribution system network condition which has been designed for town. Secondly the study suffers from lack of sufficient secondary data related with urban water supply design due the inadequacy of works regarding the study area and the poor documentation of the water supply and sanitation authority offices.

1.9 Research Design and Data Collection

1.9.1 Research Design

Generally this study can be seen as a descriptive cross-sectional study with a central task of design of urban water supply system in Holeta town, Wolmera Woreda. The study used a mixed approach with a central premise of; the use of quantitative and qualitative approaches in combination provides a better understanding of research problems than either approach alone. Hence, the mixed approach that is used in this research employs strategies of inquiry that involve collection of qualitative and quantitative data simultaneously to best understand the research problem under investigation. The study was guided by the principles of multiple sources and subsequent crosschecking of information as well as by applying various data collection instrument and analysis techniques- both quantitative and qualitative.

1.9.2 Data collection

A combination of both quantitative and qualitative data from both primary and secondary sources was generated. The primary data was collected from residents of the sample Holeta town, officials of the water supply and sanitation authority and Wolmera woreda water bureau and from field visit. In an effort to supplement the primary data and make this research work more valid and worthy, relevant secondary sources pertinent to the study were consulted. Accordingly, official statistics and reports available in water projects implementing agencies' offices were the major sources of secondary data for this study. Moreover, different written documents both published and unpublished- books, CSA, government, non-government documents, journals and research works in relation to the issue under consideration; government policy and strategy were reviewed to supplement the study as well as to review the overall water supply situation in the study area .

2 RELATED LITERATURE REVIEW

2.1 Overview

A well planned water distribution network is very essential in the development of an area. The network is built to satisfy various consumer demands while meeting minimum pressure requirements at certain nodes. In the design stage it is of interest to arrive at the least-cost solutions that satisfy a set of constraints including demand and pressure requirements. Often it is also of interest to arrive at less expensive solutions that, however, violate slightly the constraints. Accordingly, research interests have been concentrating on the design of water distribution network by using EPANET to search for the optimal combination of decision variables (e.g. head loss and velocity) from a large number of solutions.

This chapter deals with the theoretical overview of potable water supply and distribution. It assesses the sources of water supply, urban water supply accessibility, major challenges of drinking water supply and distribution, potable water supply problems in developing countries in general and in Ethiopia in particular, benefits of access to safe, reliable, adequate and affordable potable water supply and impacts of inaccessibility of water supply and distribution facilities. In addition to these it assesses the Ethiopian government's water supply and sanitation policy, institutional arrangement and responsibilities at different levels.

2.2 Water and Civilization

Water has been an important factor to the development and survival of civilization. The first great civilization arose in the valleys of great rivers, the River Nile, valley of Egypt, the Tigris Euphrates valley of Mesopotamia, the Indus valley of India and Pakistan and Huang He valley of China. Through the ages people have been compelled to settle in region where water is not deficient in quantity, inferior in quality. Only when supplies failed or made useless by unbearable salt or pollution before them were centers of habitation abandoned. So, man's endeavors to achieve a more desirable relationship with the water of the earth have helped them mould his character and his outlook towards the world around him. People have preferred to meet their water troubles head on rather than to quit their places of abode and industry. So people have applied their creative imagination and utilized their skills and released heroic energy. The ancient well aqueducts and reservoirs of the old world, some still serviceable after thousands of years, at least to the capacity for constructive thinking and corporative ventures which had a part in human advancement. These aqueducts, canals, and reservoirs built by the ancient Romans turned regions along the coast of Northern Africa to be civilized. After Romans left, their water projects were abandoned (World Book, 1984).

2.3 The Water Supply and Demand Situation of the world

Our planet today is at the eve of accepting 7.355 Billion peoples which brings historically unprecedented pressure to the natural resource of the globe (*UNPD.WPP, Eurostat: Demographic Statistics,UNSD. Population and Vital Statistics Report (various years), .S. Census Bureau: International Database and Secretariat of the Pacific Community: SDP*). In this case Water, which is among the basic natural resource to the livelihood of the rural poor, is exposed to deterioration from time to time. As a result today bringing immediate remedy to the global water crisis became an agenda to achieve the target of the millennium development goals of halving the proportion of people without access to improved water.

Recently thought the global use of improved water sources showed progress from time to time but still 605 million people don't have access to safe drinking water which clearly shows the pressure of the population growth (WHO, 2012). Besides the population pressure of the globe, the water supply and demand gap are exacerbated by various factors of inequitable distribution of water rights, economic resources and uneven resource availabilities (White ford, 2005, cites in Wutich, and Ragsdale, 2008). Even though the problem of water supply is the fact for both urban and rural areas, the world is still predominated by the world rural population which lack access to improved water as compared to that of urban population. As shown in the Figure 2.2. It is only 3.55 % of the world urban populations are considered to be without access to improved water. Unfortunately this number is much higher for the rural population of the world for whom the population without accessibility to improved water source reached 15.423% which is five time higher than the urban population.

The world's water security situation is basically influenced by two grand driving forces: pressure on the supply of water and pressure on the demand for water. Pressures on water supply include; impact of climate change, multinational use of water basins and aquifers, poor water supply infrastructure and intermittency are just only listing some of the major once. On the other hand pressured on the demand side includes; population growth and distribution, agriculture (which currently accounts 70% of all water use), changes in diet and industry (20% of global water use) are the prime challenges for the spontaneous increment of water demand of the world today (REA, 2010). Hence identifying these two grand drivers of water supply and demand situation, the options for tackling these challenges will revolve around them. Therefore, integrating supply orientated and demand orientated measure through policy, governance and regulation, cultural change and institutional reform, as well as through better approaches to management and application of new technologies and techniques are promising measures if the two drivers are required to be tackled and the world water situation needed to be improved (ibid).

According to the World Health Organization and UNICEF, in 2010, 89% of the world's population used drinking water from improved sources (54% from a piped connection in their dwelling, plot or yard, and 35% from other improved drinking water sources), leaving 780 million people lacking access to an improved source of water (WHO/UNICEF, 2012).

The world met the United Nations' Millennium Development Goal (MDG) drinking water target to halve the proportion of people without sustainable access to safe drinking water by 2015 in 2010, 5 years ahead of schedule (WHO/UNICEF, 2012). More than 2 billion people gained access to improved water sources from 1990 to 2010. However if current trends continue, 605 million people will be without an improved drinking water source in 2015 (WHO/UNICEF, 2012).

Access to safe drinking water is measured by the percentage of the population having access to and using improved drinking water sources.

Improved drinking water sources should, but do not always, provide safe drinking water, and include:

- Piped household water connection
- Public standpipe
- Borehole
- Protected dug well
- Protected spring
- Rainwater collection

Unimproved drinking water sources include:

- Unprotected dug well
- Unprotected spring
- Surface water (river, dam, lake, pond, stream, canal, irrigation channel)

- ➢ Vendor-provided water (cart with small tank/drum, tanker truck)
- ➢ Bottled water*
- ➢ Tanker truck water

* Bottled water is not considered improved due to limitations in the potential quantity, not quality, of the water.

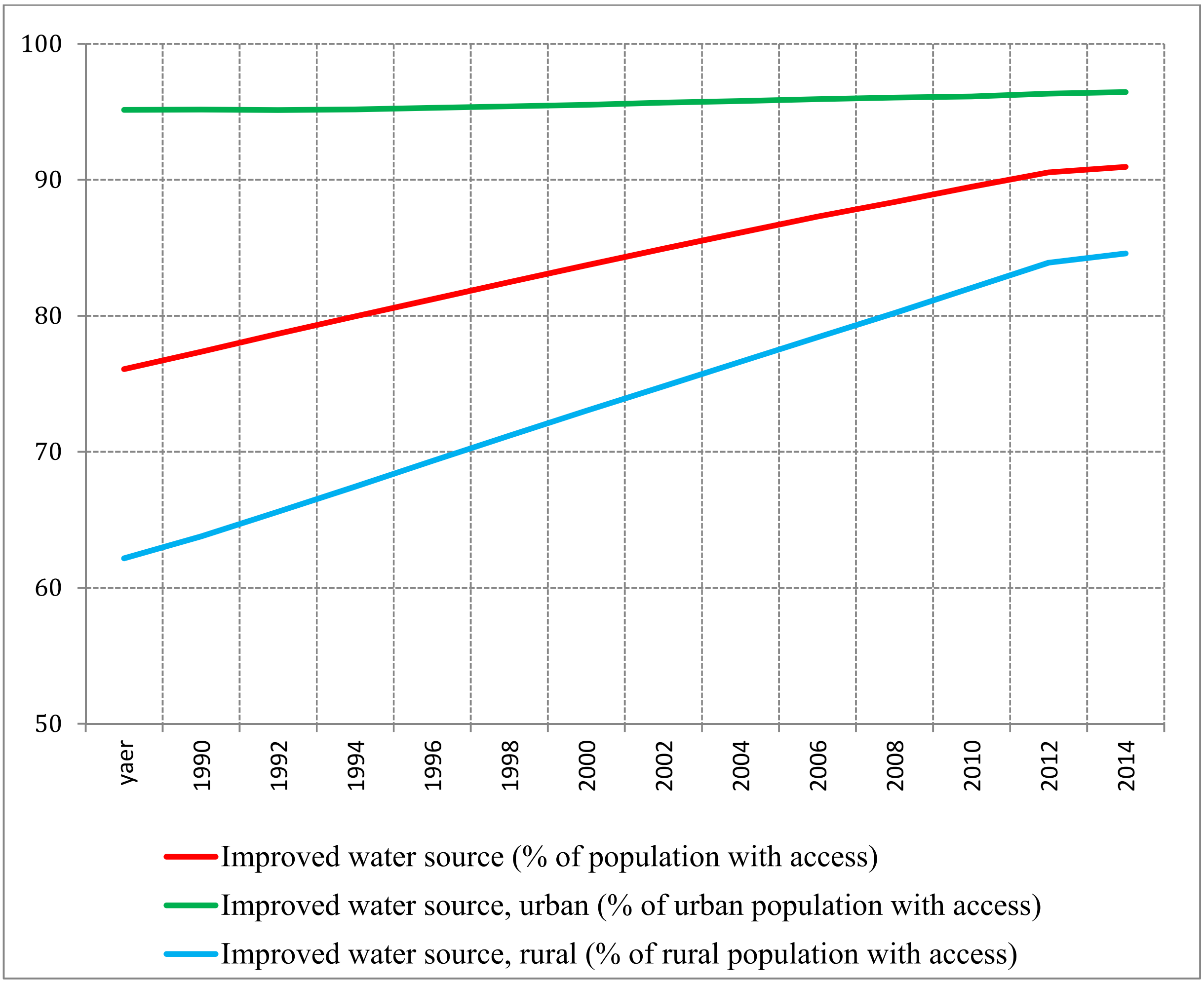

Figure 1: World Improved water source (% of population with access) in 2015

Source: computed from WHO/UNICEF (JMP) for Water Supply and Sanitation (wssinfo.org)

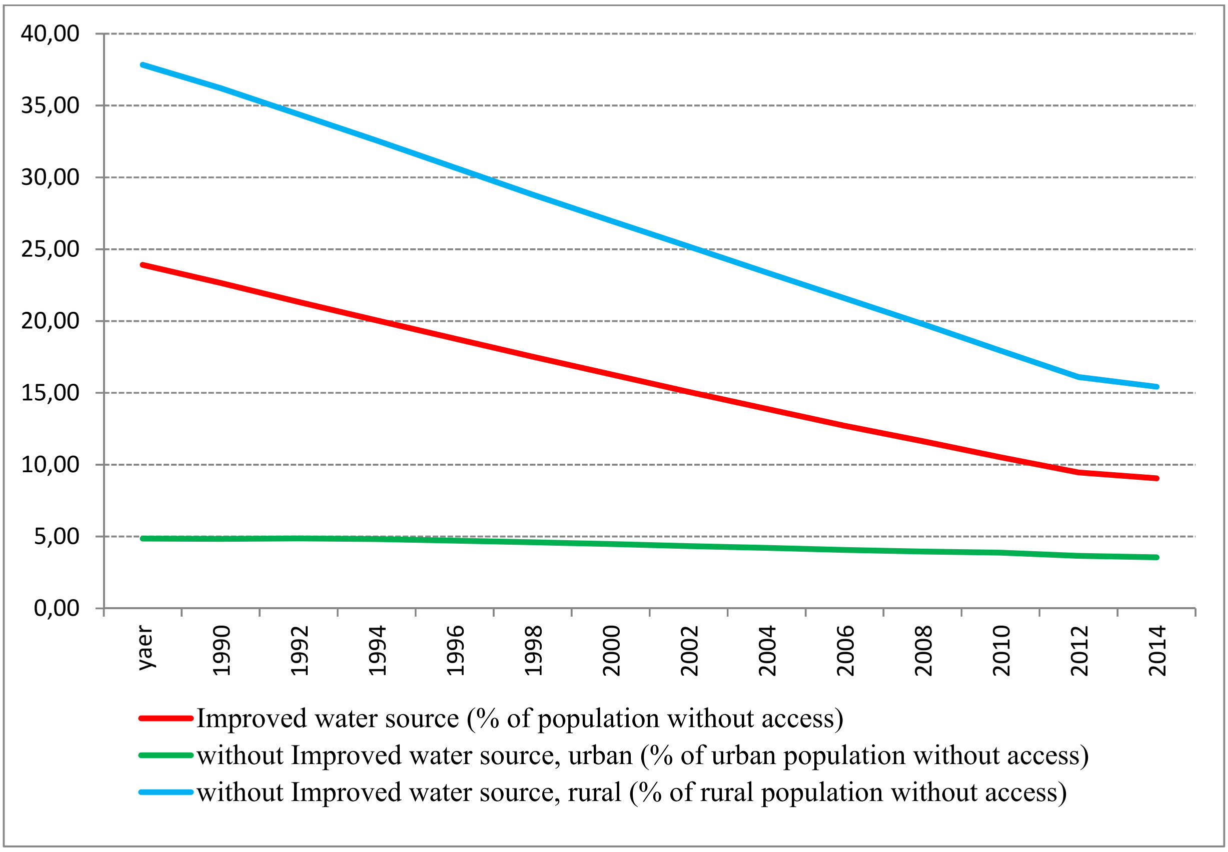

Figure.2: World without Improved water source (% of population without access) in 2015

Source: Source: Computed from WHO/UNICEF, 2015. (wssinfo.org)

2.4 The Rationales of Urban Water Supply in Africa

In the year 2000 all most all African countries were adopted the millennium development goals and seeks to "halve by 2015 the proportion of people without access to safe drinking water and sanitation" (Todaro and Smith, 2011). However in Sub-Saharan Africa it is anticipated to rich the target to the year 2040, after 25 year from the expected target (Sutton, 2008).That is why still, around 276.5 million of the people living in sub Saharan Africa are left without access to safe water with a majority of them being women and children living in rural households (*WHO/UNICEF (JMP) for WSS , 2015)*. SSA has the lowest drinking water coverage and the lowest sanitation coverage in the world (WHO, 2012).

With only 56 percent of the population enjoying access to safe water, Sub-Saharan Africa lags behind other regions in terms of access to improved water sources. Based on present trends, it appears that the region is unlikely to meet the target of 75 percent access to improved water by 2015, as specified in the Millennium Development Goals (MDG). The welfare implications of safe water cannot be overstated. The estimated health and time-saving benefits of meeting the MDG goal are about 11 times as high as the associated costs. Monitoring the progress of infrastructure sectors such as water supply has been a significant by-product of the MDG, and serious attention and funding have been devoted in recent years to developing systems for monitoring and evaluating in developing countries. Piped water reaches more urban Africans than any other form of water supply-but not as large a share as it did in the early 1990s. The most recent available data for 32 countries suggests that some 39 percent of the urban population of Sub-Saharan Africa is connected to a piped network, compared with 50 percent in the early 1990s. Analysis suggests that the majority of those who lack access to utility water live too far away from the distribution network, although some fail to connect even when they live close by. Water-sector institutions follow no consistent pattern in Sub-Saharan Africa. Where service is centralized, a significant minority has chosen to combine power and water services into a single national multi-utility urban water sector reforms were carried out in the 1990s, with the aim of creating commercially oriented utilities and bringing the sector under formal regulation. One goal of the reforms was to attract private participation in the sector.

In Africa despite there are recently positive trends regarding the water supply and coverage, still the problem is pervasive in the region and remains unsolved permanently. Even in the region for many of those who supposedly already enjoy an improved service, the reality is one of poor continuity, poor quality and premature failure. As a result Tens of millions of people face continuing problems with systems that fail prematurely, leading to wasted resources and false expectations (Lockwood, and Smits, 2011). According to the report of (WHO/UNICEF, 2011), 84% of people without access to improved drinking water sources live in rural areas of the region. In Africa the sustainability of water projects still remains the major challenge for continued provision of water to the rural population. The Water Supply Network indicates an average rate of non-functionality for hand-pumps in sub-Saharan Africa is 36% which is shameful wastage in the sector. Due to this fact huge amount money which estimated to be hundreds of millions of dollars over the last 20 years are wasted. Having recognizing such trends community managed projects has been envisaged but still the problem remains intact due to lack real participation of the community (RWSN, 2009, cited in Lockwood, and Smits, 2011).

Looking in to the trends of urban improved water coverage, in East Africa for instance the progress is still remained undone. As shown in the Figure 3 below only Djibouti reaches around 97.3% of urban improved water coverage, this percentage is even very high as compared to the urban provision of the other east African as well as sub Saharan African countries. Unfortunately among the East African countries, Ethiopia has the lowest improved water coverage estimation as compared to Uganda and Djibouti. Even though the prospects of urban water supply have shown some progress, still the trend fails to converge with the urban water supply.

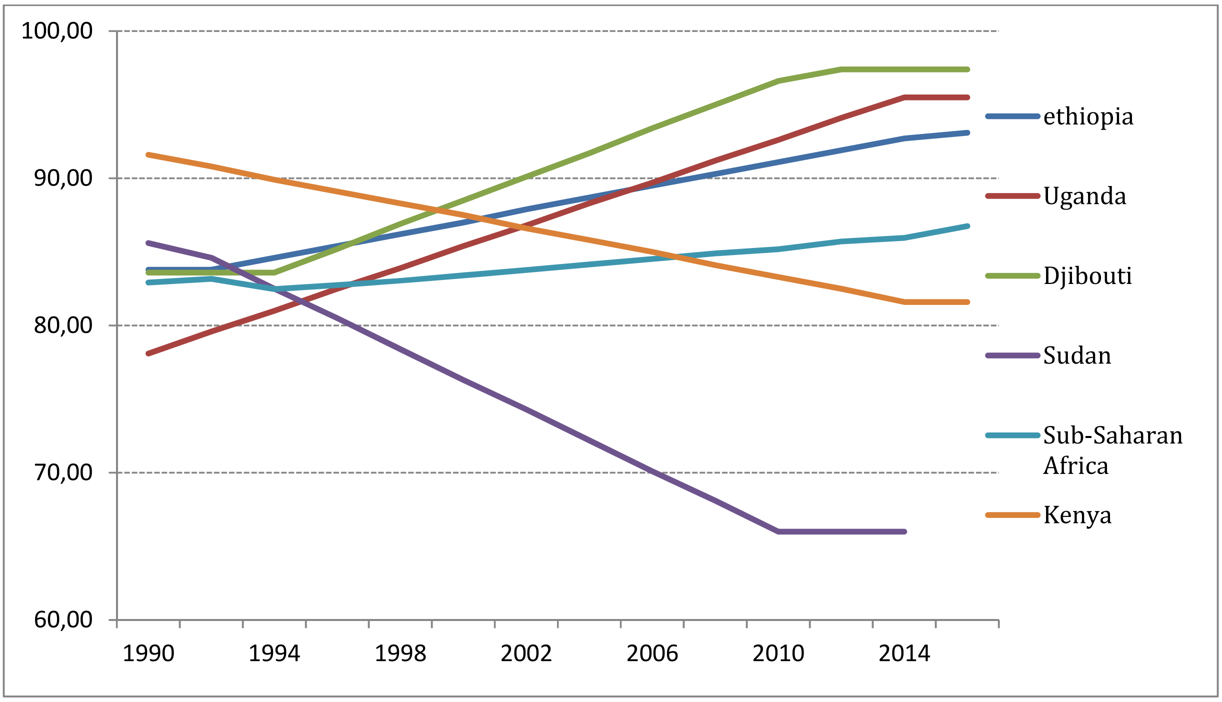

Figure 3: urban Improved Water Coverage in sub-Saharan African and East Africa
Source: Computed from WHO/UNICEF, 2015.

2.5 The state of urban water supply in Ethiopia

The water supply and sanitation sector in Ethiopia is one of the least developed and is mostly characterized by service deficiency of physical infrastructure as well as by inadequate management capacity to handle policy and regulatory issue and to plan, operate, and maintain the service.

Regarding this, World Bank Group (2005:2) stated that though Ethiopia is often referred to as the "water tower" of Africa, only a quarter of the country's population have improved access to water sources. Rushing streams from the Ethiopian highlands form tributaries of famous Blue Nile, Tekeze, Awash, Omo, Wabeshebele and Baro-Akobo-rivers which flow across borders to neighboring countries. Six billion cubic meters of water run out of Ethiopia as the Blue Nile River to the Sudan and Egypt. But as recurrent drought drives more and more rural people from their traditional farmlands to urban centers, Ethiopia faces growing urban water crises.

Ethiopia has one of the highest urbanization growth rates in the developing World. According to data obtained from the Central Statistical Authority, the country's urban population was growing at 4.8 per cent per annum between the 1995 to 2000. The urban population in Ethiopia in 1984, the first census period, was 4.3 million forming 11 per cent of the total population. In 1994, the second census period, the urban population was 7.4 million. Total urban population had increased by 12 per cent from that of 1984. In terms of urban centers, in 1984, Ethiopia had 312 urban centers with population of over 2000. In 1994, the second census period, the urban centers in the country grew to 534 registering an increase of 71 per cent over that of 1984 though the definitions of the two censuses are not the same (Tegegne, 2000:2).The growth has been much higher for some intermediate towns. In 2000 17.6% of Ethiopia's population or about 11 million people live in about 927 cities and towns of different sizes and categories. Currently, in 2005 about 20.1% of urban populations live in cities and towns of different sizes and categories.

The rapid growth of urban population has placed tremendous pressure on the management capacity of municipalities for service delivery and local economic development. This phenomenal growth has also burdened many municipalities with the problems of inadequate housing, poverty and unemployment, inadequate water and electricity supply, and poor sanitation systems. Available data also indicate that in the next 25 years (1994-2020), nearly 30 per cent of Ethiopia's population will live in cities. This kind of rapid urban population growth will inevitably call for huge investments in housing, urban infrastructure, water and electricity supply, sanitation systems and environmental protection programs and programs to alleviate poverty and unemployment in the cities. This implies that the challenge will require well trained municipal management and resource capacity, responsive urban governance and well trained and motivated personnel and sustaining services such as water, electricity supply, local revenue collection and administration to meet the ever growing demand for better and more quality services and infrastructures.

Because of this population pressure and other factors as per official statistics, coverage of water and sanitation service in Ethiopia is very poor, among the lowest in the world, especially for rural areas. Among the key indicators for International Development Goals, Ethiopia's performance on "sustained" access to safe water sources and sanitation services is one of the worst in the region.

According to the figures given by Tegegne (2000:16), the amount demanded is much higher than the supply. That is, in 1998 the amount supplied by Addis Ababa Authority was only 62 per cent of the amount demanded. With regard to the distribution of water, the Welfare Monitoring Survey of 1996 estimated that 36 per cent of the households use own tap while 61 per cent use public tap or "public fountain".

Berhanu and Said in Genenew (1999:8) also figured out that only 27 per cent of the populations of Ethiopia have access to safe water and 10 per cent have access to sanitation while these figures stand 71 per cent and 30 per cent for safe water and sanitation respectively for low income countries.

There is also regional variation both in rural and urban areas such as Addis Ababa, Dire Dawa and Harari in particular showing more per centage of population with access to safe source of water and sanitation. Afar, Benishangul-Gumuz and Gambella regions show the low per centage of population with access to safe water. For instance, in towns such as Mekele, Nazareth, Bahirdar and Harrar only 33.6, 38.6, 42.9 and 57.8 per cent of the housing units, respectively, had a private or shared water meter in 1994. The water supply in small towns is extremely low.

Regarding this, the World Bank group (2005:2) mentioned that towns in the 2,000 to 50,000 population range face special challenge in the provision of their WSS services. The demand for differentiated technologies-piped water supply in the core, alternative technologies in the fringe areas- and the often rapid unpredictable water demand and spatial growth require planning, design, and management skills that exceed community based management approaches. But unlike larger towns or cities, these smaller towns often lack the financial and human resources to independently plan, finance, manage and operate their WSS systems. This implies that a key challenge for Town WSS is to allocate limited government resources amongst a large number of dispersed towns. The following table 2.1shows the coverage for water supply and sanitation in Ethiopia.

Table 2.1: Coverage for Water Supply in Ethiopia

Region	**Per centage of population with access to a 'safe' source of water**			
	1994		**1998**	
	Rural	**Urban**	**Rural**	**Urban**
Addis Ababa	35.6	98.4	54.6	98.8
Affar	5.2	73.1	16.2	83.0
Amhara	15.2	80.2	13.0	85.0
Benishangul	14.8	55.0	14.1	61.00
Dire Dawa	23.6	98.1	84.6	99.6
Gambella	21.3	72.4	21.1	62.4
Harrari	11.2	98.2	37.7	97.3
Oromia	15.8	76.2	20.0	85.8
Semali	9.0	47.9	34.9	98.8
SNNPRS	15.5	71.1	21.4	73.8
Tigray	10.6	73.9	19.2	96.4
Total	14.8	81.0	18.3	86.0

There are also variations across urban areas. Based on the official statistics, conditions with access to safe water in urban areas is higher in terms of coverage, with about 84 per cent having access to safe water sources, though there are some variations across different town size classes. This, however, needs to be treated with caution as most households rely on shared services, consumption levels are very low, seasonal variability is very high and unscheduled disruptions to services are very common. Small towns with less than 2,000 populations have access levels of only 40 per cent and those with less than 10,000 populations have a level of around 60 per cent. Interestingly, except for the very small 'towns' with less than 2,000 population, most other towns have some form of piped systems, and access to piped systems is over 75 per cent in towns with more than 10,000 population.

MWR (2002:4) distinguished three categories of towns outside Addis Ababa. Rural towns: towns with less than 2,000 population where 60 per cent of towns have piped system, but coverage levels in terms of population with access to piped system is low at about 20 per cent. Small towns: towns with 2 to 10,000 population that mostly have piped systems but the access to piped system is only about 50 per cent and medium and large towns all have piped systems but do require some improvement in access. Even among these towns "access" is largely confined to yard taps or shared connections, with the resultant implications for cost recovery and financial viability. The following table 2.2 shows water supply status in urban areas.

Table 2.2: Water Supply Status in Urban Areas, 1994

Population size of towns	**Total pop. (in'000)**	**% of towns with piped system**		**% of housing units with**		
				Individual connection	**Piped supply**	**Safe water**
Lessan 2000	349	282	58	2	22	43
2-5,000	1093	338	90	3	43	61
5-10,000	1077	153	98	5	55	69
10-20,000	1192	85	100	10	73	82
20-30,000	754	30	100	13	74	83
30-80,000	1176	26	100	16	76	84
80-250,000	1334	10	100	19	89	94
Addis Ababa	2495	1	100	27	98	98
Total Urban	9470	925	83	15	76	84

Source: WSP, 2002:5

2.6 The challenges for urban water supply in Ethiopia

Ethiopia has plenty of water resources but the available water is not distributed evenly across the country and the amount varies with seasons and years. The challenge in any situation is to maintain a year-round supply that is adequate to meet people's needs. To ensure that supply meets demand the source of the water must be carefully chosen, taking into account present and future demand for water, and the costs. The cost of water supplies is heavily influenced by the distance of reliable water sources from towns. The challenge for many towns is finding nearby water sources.

Planning for present and future demand has to consider population growth. The demand for water is increasing in cities and towns due to an ever-growing population and the migration of people from rural areas to towns in search of jobs and a better life. There are also increasing demands from industrial and commercial development. The quantity of water required for domestic use depends not only on the number of people but also on their habits and culture, and on how accessible the water is. On average, Ethiopians in urban areas use only about 15 litres of water a day for their needs (MoH, 2001; Ali and Terfa, 2012).

There is a difference between the WHO estimate and the daily water consumption per person in Ethiopian towns. The shortfall is perhaps due to the shortage of private water taps, which means that people have to collect water from public taps. If people have a piped water supply in their home they are likely to wash and bathe more frequently, and some may have water-using appliances like washing machines. As water supply systems improve and access increases, the consumption of water will increase also. It is therefore important for water supply planners to consider the expected changes in society and in living standards. Planning of water supply projects should also consider the water requirements of schools, hospitals and other health facilities, churches and mosques, hotels, public washrooms, and other community facilities.

The government of Ethiopia has set targets of 100% coverage of safe water supply in urban areas and 98% coverage in rural areas. These targets originated from the Universal Access Plan of 2005 and the Growth and Transformation Plan of 2010, and have been adopted by the One WASH National Programme (OWNP), which is being implemented with major funding from government and international donors (FDRE, 2013). The planning criteria for water supply coverage in the OWNP are:

- rural water supply: 15 litres/person/day, within 1.5 km radius
- urban water supply: 20 litres/person/day, within 0.5 km radius (FDRE, 2013).

As you can see, these figures are still below the WHO recommendation and are more than current usage, indicating the scale of the challenge ahead. The targets for Ethiopia are that 4.4 million urban inhabitants and 26.6 million rural inhabitants, nearly 30,000 schools, and more than 7500 health posts/centres will gain access to safe drinking water (FDRE, 2013).

Another key issue in urban areas is the reliability of the water supply. Consultations with the poor also highlighted this aspect vividly. Limited available information suggests that reliability of supply is likely to be quite poor, both in terms of quantity and frequency. Regarding access for the poor on the whole, relative level of access to water and sanitation in urban areas is estimated to be high in Ethiopia. However, in some larger urban centers the poor may lack access. The aforementioned information indicates that as a result of low level of development a significant proportion of the total urban population of Ethiopia in particular and total population of Ethiopia in general have no access to safe and adequate potable water supply. They still restrict themselves to use what nature has provided them with in the form of springs, rivers, lakes, ponds, traditional hand dug wells and rain water which are often unsafe, cause health hazards and are at considerable distance from households. Among the main reasons given for the slow pace of progress in water supply services in Ethiopia, the following are net worthy: lack of comprehensive legislation; inadequate investment resources; lack of a national water tariff policy and the absence of beneficiary participation and community management (Dessalegn, 1999:12). In relation to this, MWR (2002:13) stated that issues of poor sector capacity and low level of expenditures for WSS are interlinked and lead to a vicious circle – as low level of investments create low demand for technical and manpower inputs in WSS sector, the capacity remains underdeveloped. The resulting low sector capacity, means low allocations and expenditures are curtailed. The sustainability of water supply facilities mainly depends on a timely and regular maintenance and operation of the system. However, in most developing countries, including Ethiopia, it has been found out that operation and maintenance (O&M) of water supply facilities is in a poor state of condition and the sustainability of the scheme is at stake. Regarding this, MWR (2002:13) identified the following underlying problems:

- Inappropriate tariff setting without emphasis on full cost recovery;
- Lack of clear guidelines for urban tariff setting including issues related to fairness, and financial sustainability;
- Inappropriate or lack of institutional incentives for urban WSPs to achieve financial viability and improved operational performance;
- Poor technical and financial capacity among the urban service providers that leads to high levels of unaccounted for Water (UFW); and
- Poor or nonexistent consumer services and grievance handling system that leads to a lack of willingness to pay user charges.

According to the feedback gathered from the participants of the workshop conducted in Bahardar in April 1999, the following were pointed out to be the main causes or challenges for the O & M problems in Ethiopia in order of importance:

- Poor organizational setup in the sector coupled with the absence of trained manpower;
- Low community awareness regarding the importance of clean water;
- Absence of adequate repair parts, spare parts, and hand tools;
- Financial shortage to support O & M , and the limited funds that are available are used

for new installations;

- Low participation of the beneficiaries in the decision making process;
- Substandard designs, poor construction quality, and inappropriate technology;
- Absence of coordinated supervision and monitoring mechanisms;
- Unwillingness to pay for services;
- Low attention paid to local skills and minimal support to Artisans and private sector (Abay Engineering PLC, 2000).

There are still many challenges ahead but the following changes will all contribute to future success:

- an increase in funds for the expansion of water supply services to satisfy the demand of growing populations, particularly in small towns
- a reduction in bureaucracy to facilitate the spending of funds that are committed (currently only around 60% of budgeted finances are actually spent)
- a reduction in the turnover of personnel, and an increase in human resource capacity and expertise at different levels
- better coordination between the different stakeholders (for instance, there is lack of coordination between the water sector, telecommunication department and the road authority; because of this, water pipes are frequently damaged during activities such as laying down telephone and internet lines, and during road construction)
- the presence of more experts to monitor sector performance at all levels
- Better information management systems, giving early warning of requirements.

2.7 Policy Framework and Potable Water Supply

Before 1999, water resources development, in general, and the provision of potable water supply, in particular have been carried out without any policy framework and were not well coordinated in the country. However, since 1999, it seems due attention has been given by the Ethiopian government to alleviate the problem of access to safe water supply and achieve rapid socioeconomic development through better health care and productivity of its people by formulating the country's water resources management policy in 1999.

The water supply and sanitation policy is an integral part of the country's water management policy. According to the policy document (1999), the policy is believed to provide and impetus for the development of water supply for human and animal consumption. It focuses on increasing the coverage, quantity, reliability and acceptable quality, taking the existing and future realities of the country into consideration. Upon implementation, the policy is expected to achieve the objective of the Ethiopian people to attain adequate, reliable and clean water service that meets the water user's demand.

The policy of supplying free water to any group except for emergency, leads in practice to an unfair situation. Since there are no enough funds to provide such free services, the rural and urban poor are the first to suffer. A better and much more equitable way would be to collect water charges from consumers and then improve and expand the system. Accordingly, the policy envisages supplying improved potable water service for urban areas with tariff structures that are set based on "full cost recovery and self reliance".

Apropos this issue, Alebel (2004) stated that a full cost recovery program has the advantage of providing incentive for proper use; reduces waste and excessive consumption of water resources. Besides, it helps to release funds for other investment programs. The policy considers water as a social and economic good, and it is an integrated one. Full cost recovery requires charging consumers so as to cover the full cost of project construction as well as the operation and maintenance of providing the service. Water development investments by their nature require huge amounts of money.

This implies that charging consumers for water should be done carefully. If prices are set too low, revenues may not be sufficient to cover the full costs of supplying water. If, on the other hand, they are set too high, households may not be able to afford consuming the new 27 improved water, and again revenues will not be sufficient to cover the full cost. In relation to this, Alebel (2004) suggested that setting the required tariff, information on the ability and willingness of the consumers to pay for such services are essential. In other words, to cover the full costs and sustain the service, revenue should be collected from the sale of the water based on the tariff that considers the full recovery of the cost, on the one hand, and the fairness and willingness of the consumers that are supposed to be served, on the other.

Therefore, the policy for increasing the coverage as the proper use and sustainability of the service requires implementation of a cost recovery system, which can be either full or partial cost recovery. That is, in order to implement the existing policy for the provision of water supply in urban areas of the country fairness of the tariff, willingness to pay for the service and efficient management of the resources of the utility office need to be examined.

2.8 Institutional framework and organizational capacity

Although urban water supply services began during the Imperial regime, it was not until 1971 that a body responsible for all aspects of water use and development in the country, the Water Resources Commission, was established. The Awash Valley Authority was setup in 1962, but its duties were to plan and promote investment activities within the valley. The commission was given a wide mandate and entrusted with the responsibility of planning and utilizing the country's water resources including household consumption. In the early 1980, the government pledged to implement the UN initiated International Drinking Water Supply and Sanitation Decade, which in Ethiopia ran from 1984 to 1994, coinciding with the governments ten year plan, which set an ambitious target for the provision of safe water supply to the rural areas. At the beginning of the 1980, less than 6 per cent of the rural population and 19 per cent of the population in the twenty major towns had access to clean drinking water. At the end of the plan period, the coverage for rural areas was to reach 35 per cent and for the urban areas 85 per cent. While the record of achievement was not as high as planners had hoped for, considerable progress was made in 1980, (Dessalegn, 1999:11 cetid as; Assefa Dallecho).

The Water Supply and Sanitation Authority (WSSA), a division within the Water Resources Commission, was established in 1981. Between then and 1992, WSSA was the principal agency responsible for water development in the rural areas and all urban areas except Addis Ababa. By 1990, a total of 210 urban water systems serving about 3 million people came under WSSA's responsibility. Likewise, the authority was responsible for providing support and maintenance to cover 6000 rural water schemes serving over 4 million people throughout the country (Dessalegn, 1999:12).

With the establishment of regional administration under the Transitional Government of Ethiopia in 1992, Water development programmes became decentralized. At present, the Regional administrations are responsible for the development, operation and maintenance of rural and urban water supply systems in their regions. WSSA has also been absorbed into the ministry of water resources and become the Department of Water Supply and Sanitation (DWSS). However, the relationship between DWSS (or MWR) and the regions appear to be unclear and the way decentralization of water development will be carried out in practice needs to be spelt out in more detail. Within the emerging framework of demand responsive approaches, the role of government is changing from service provision to facilitating and providing an enabling environment.

Within the decentralization framework in Ethiopia, different responsibilities are emerging for different levels of government: policy and strategy development, project implementation and monitoring and evaluation. At the federal level the responsibility for the water sector is with the Ministry of Water Resources (MWR). Responsibility for ensuring the provision of these services is with the regions and will eventually be with woredas (MWR 2002:6).

At the regional level, Regional Water Bureaus (RWBs) along with their other responsibilities for water resources are also responsible for water and sanitation. In some of larger regions, woreda water offices with small staff of two persons or so have been established. This trend for the woreda level is intended to be strengthened in the coming years. Within the Ethiopian context, NGOs have been important players in the WSS sector. For rural water supply schemes (RWS) Ethiopia Social Rehabilitation and Development Fund (ESRDF) has also played a major role in recent years (WSP; 2002:7).

As MWR (2002:7) documented, in Ethiopia, a number of different forms of service providers exist with considerable inter and even intraregional variations, including: Addis Ababa water and sewerage Authority (AAWSA), Urban /Town Service Unit (TWSU), Some Scheme Water Boards (SWB) and at the very local level Water Board (WB) and Village Water and Sanitation Committee (VWSC). There has been limited involvement of the private sector to date, though there is an emerging interest. With regard to the financing issue, though the National Water Policy envisages financing from domestic financial institutions. So far sector financing has been largely through: budgetary allocations, external debt or grants from bilateral donors and international NGOs, sometimes provided either directly to communities or local levels of government and more recently other off-budget mechanisms such as ESRDF. MWR also proposes to establish a Water Resource Development Fund (WRDF). It is envisaged that the WRDF will pool the government and donor resources and channel in line with the overall sector policy. In the long-run it is visualized that WRDF will also mobilize additional resources (MWR, 2002:9).

With decentralization, a large share of federal resources is transferred to regional governments and regional and woreda governments allocate funds for the WSS sector from their own budgets. However, an effective decentralization process is constrained by: the lack of medium term federal subsidy estimates and donor practices that inhibit multi-year planning. WSS allocations within this emerging decentralization framework depend on the planning process at these levels and the issue of relative preparedness of the WSS sector at this level will be an important determinant (ibid).

Based on available information, preliminary and indicative estimates suggest that the current level of funding allocation to the sector is about 34 million USD per annum. Clearly, to achieve improvements in poverty reduction and other development goals 30 water supply and sanitation deserves an equal attention as other sectors such as education, health and roads. However, WSS allocations leave a great deal to be desired as compare to these sectors. This more likely reflects a lack of sector readiness to absorb resources rather than a low priority for water supply and sanitation. The priority actions and programs with in the sector will have to focus on strengthening overall sector capacity along with the specific investment strategies linked to coverage targets. (MWR; 2002:10). From this review of related literature we would understand the pertinence of and the different approaches to urban water supply.

3 DESCRIPTION OF THE STUDY AREA

3.1 Background

The Oromia regional state water resources development bureau is one of the governmental organizations which established to improve the water supply and sanitation situation of the region and the project area suited in one of the lack of potable drinking water of the region due to deprived and insufficient amount of clean water.Holeta water supply project is one of the projects aimed to achieve the millennium development goal of Holeta town, in the Oromia region of Ethiopia. The project aims to improve the living conditions of the town by rehabilitating existing systems and extending the water supply infrastructure as well as providing adequate training for Town Water & Sanitation Services (TWSS). The project will develop new water sources (deep and shallow wells and springs), increase the capacity of pipelines and reservoirs, extend the distribution system to those areas of towns not currently serviced, and provide technical assistance to the TWSS.

3.2 Location and Topography

Holeta is located in West Shewa Zone of Oromia National Regional State about 34km from Addis Ababa. The town is situated along the main Addis Ababa to Nekemete road. Geographically it is located at latitude $10^{0}02'92''$ north and longitude $44^{0}60'22''$ east. Presently, Holeta is the capital of Wolmera Administrative district. The total area of the town is estimated to be 5550 ha and is divided into four urban and five rural Kebeles. The town lies between elevations of 2320 and 2460 m.a.s.l.

Figure 1: Location of Holeta

Source : Holeta Town Feasibility Study

3.3 Climate

Holeta town is characterised by high mean annual rain fall of 1367 mm. The highest rainfall occurs in June, July, August and September with March and April being the driest months.

The mean monthly temperature of Holeta ranges from 12.3^0C- 15.9^0C. The minimum and maximum temperature varies with in 1.6^0C-8.9^0C and 19.6^0C-24.7^0C, respectively. The lowest temperature recorded during the months of December, January and February. The highest temperature recorded during the spring season (February, March, April and May). The variation of the temperatures is minimal, which is typical for the climatic region. According to the climatologically classification, Holeta town is climatically classified as" Dega Rainy" climate.

3.4 Demographic Conditions

3.4.1 Population

The 2007 national census reported a total population for Holeta of 25,593, of whom 12,605 were men and 12,988 were women. The majority of the inhabitants said they practiced Ethiopian Orthodox Christianity, with 73% of the population reporting they observed this belief, while 20.44% of the populations were Protestant, and 5.43% were Muslim.

According to the 1994 national census, this town has a population of 16,800. The 1994 census reported this town had a total population of 16,785 of whom 8,040 were males and 8,745 were females. Using this as a base, the 2008 population is estimated at 33,099.It is the largest of three towns in Wolmera Woreda.

3.4.2 Economic Situation

Holeta is the capital of Wolmera Woreda and, therefore, is an important administrative and communication centre with a population of more than 33,000 inhabitants. Holeta is a major transit centre and it is located on the main road from Addis Ababa to Nekemete. The town also serves as a major marketing centre with thousands of rural people flocking into the town. The major economic activities according to the town's administration office are trading, hotel services and small-scale industries.

Table 1: Commercial businesses in Holeta town

Type of Enterprise	Number
Hotels and bars	13
Metal and wood work	14
Garages	9
Retail &Wholesale trade	254
Grain Mills	19
Fuel Stations	1

Source: Holeta Town Municipality, 2007

According to the development plan of Holeta, social and personal services are the dominant employment sectors followed by trade and tourism. The status of Holeta as the capital of the district means that administrative and law enforcement institutions are centred in the town.

Manufacturing is the other dominant sector providing employment. It is mostly based on grain milling activities but the potential for diversification and growth is high. Construction is a relatively young sector linked to the growing demand for modern buildings.

3.4.3 Future Development of the Town

A Master Plan for the town was prepared in 2008. According to information from the town's administration office the economy of the town will improve significantly. It is expected that flower farming and small scale industries will grow substantially followed by the building sector and hotel industry. Trading and transport sectors are also predicted to grow significantly.

3.5 Basic Social Services

3.5.1 Education

Educational services in Holeta comprise kindergartens, first cycle schools, second cycle schools, high school, preparatory school, technical college and one private college.

Table 2: Distributions of Schools, Students, and Teachers

Level	no of Students	no. of Teachers	Student/ teacher ratio
Kindergarten	422	12	35
Elementary school(1-8)	9074	222	756
9-10	5173	66	431
11-12	1212	51	101
Total	15881	351	1323

Source: Holeta Town Education office, 2007

3.5.2 Health

One health centre and eight private clinics are found in Holeta town. The health centre has 14 beds.

3.6 Existing Water Supply and sanitary Service

3.6.1 Water Supply Service

The primary source of water is from three boreholes drilled in 1997, 2000 and 2007 with a cumulative output of 12.3l/s at the moment. Water is pumped from the boreholes to a 300 m3 and 50m3 reinforced concrete reservoirs from where it gravitates into the distribution system. The water is distributed to the consumers through a total of 2,529 private connections and 24 public fountains.

Water shortage is the major problem with the existing system. Accordingly domestic supplies are supplemented from secondary sources from the river, small springs and hand dug wells.

According to data from the Water Supply Service Office, the majority of households in Holeta are partially supplied with water from the town's water supply system. It is reported that 90% of householders collect part of their total water needs from the town's water supply system fed directly through private connections or public taps. However, there is insufficient water to meet all demands and the deficit is made up from other sources including wells and water vendors.

3.6.2 Sanitary Service

The overall sanitation of the town is poor and sanitation associated diseases are prevalent. There is no system for collecting, transporting, and dumping waste in the town.

- **Solid Waste Management**

The majority of households have no containers for storing garbage. There are few garbage collection facilities located in the community, therefore, residents of the town dispose of domestic waste in any open spaces especially on the road verge and in drainage ditches. There is a temporary sanitary land fill along main highway to Nekemte.

- **Liquid Waste Disposal**

There is no liquid waste disposal system in the town. Waste resulting from bathing and other domestic washing activities is almost entirely thrown out into the streets. There is no specific site for liquid waste disposal.

- **Toilet Facilities**

Most of the excreta disposal facilities in Holeta Town comprise pit latrines which are frequently poorly constructed, offensive and over filled. According to the town's municipality the majority of households use toilets in their own compound and the prevalence of open defecation is also significant and demands improvement.

- **Sludge Disposal Method**

The municipality does not own a vacuum truck for sludge disposal. However, according to the town's administration the municipality brings a vacuum truck from Addis Ababa and provides sludge disposal service for households by charging 310 Birr for a single service. According to the information from the municipality during the field visit by the Consultant, most households are unable to afford this facility and few households employ the service. Many households dig a new pit when the old one is filled. Currently there is no proper sludge disposal site and sludge is disposed in the farm land outside the town.

4. POPULATION FORECASTING & DESIGN PERIOD

4.1 Introduction

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 will be rendered absolute by technological advances. In order to design the parts of water system, the flow at the end of design period must be estimated.

The current development plan for Holeta Town was prepared in 2008 by Oromia Regional state Urban Planning Institute. The development plan shows that there are areas allocated for residential, commercial, industrial and service-giving institutions. With the growth of the private sector in the economic activity of the town, there will be a high demand for basic services among which water is the prime necessity.

The proposed town development plan supplemented with on-site observation, topographic maps and consultation with the local community, governmental and non-governmental organizations are among the basis for water demand computation and design of future water supply system.

It is necessary to fix the design period and forecast the population of the area in the design of any water supply scheme. Water supply projects are usually designed for a certain period after the completion of construction works in order to satisfy the population demand.

4.2 Design Period

Design period is the number of years for which the design of water works has been done. Before designing & construction of water supply scheme, it is necessary to assure that the water works have sufficient capacity to meet the future water demand of the town for the fixed design period. Therefore the number of years for which the design of the water works has been done is called design period. The design period, however, should neither too long or too short. Mostly water supply schemes have design period of 22-30years.

The different elements of the treatment & distribution systems may approximately be designed for different flow criteria as shown in the table below.

Table 3: Design periods for various units of water supply

system s.no	Name of units	Design period
1	Pump house	30
2	Pump	12
3	Generator	25
4	Water treatment unit	25
5	Distribution pipe	30
6	Service reservoir	50
7	Weir	50

Source (Dr.B.Cpunmia, water supply Engineering)

The design period of a water supply scheme can be limited by the following factors

1. Funds available for the completion of the project
2. Life of the pipe and other structural materials used in the water supply scheme.
3. Rate of interest on the loans taken to complete the project.
4. Anticipated expansion rate of the town.

Since Holeta is the capital of WolmeraWoreda it is expected to grow in the future and its water supply system should have a small design period and we adopted 25 years depending up on the life span of the material and anticipated expansion of the town.

4.3 Population Forecasting Approach

After the design period has been fixed, the population of the town in various periods has to be determined. As population of the area increases in the future, the correct present and past population data have to be taken form census office to determine design population of the area. 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 etc. These elements may produce sharp rises, slow growth, and stationery conditions or even decrease the populations. The populations are increased by births, decreased by deaths, increased or decreased by migration and increased by annexation. These all four factors affect the change in population. The correct present and past population can be obtained from census office. Knowing the present population from the recent census is possible to design or forecast future population of the town.

4.3.1 Methods of forecasting population

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
B. Geometric increase method
C. Incremental increase method
D. Method used by Ethiopian static authority

A. Arithmetic increase method

This method is based on the assumption that the population is increasing at constant rate, that is the rate of change of population with a time is constant. Generally, the method is applicable to large and old cities.

$$\frac{dp}{dt} = K$$

$$\int_{po}^{pn} dp = K\int_{0}^{n} dt \Rightarrow Pn = Po + Kn$$

Pn=Po + Kn

Where; Pn=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.

$P_1=P_0+K*P_0=P\ (1+K)$

$\mathbf{P_n=P_0\ (1+K)^n}$

Where P_0=initial population.

Pn=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. Hence it gives satisfactory results.

$P_n = Po + n * (K + r)$

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 $p_n=p_oe^{kn}$ for most water supply project in the country to project population at the end of required decade/year.

$$\mathbf{P_n=p_oe^{kn}}$$

Where P_n=population at n decades or 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 of Holeta town.

4.4 Population Data

The number of population of the town which is obtained from the census office is tabulated below.

Table 4: Given population

Year	1994	2008	2010	2015
Population	16785	33,099	36,288	45217

Source: from feasibility study, 2007

Table 5: Population growth rate

Year	Growth Rate (%)
2010 – 2015	4.4
2015 - 2020	4.2
2020 – 2025	4.0
2025 – 2030	3.8
2030 – 2035	3.6
2035 – 2040	3.4
2040 – 2045	3.2

Note: population Growth rate from 2030-2045 is found by extrapolation.

Table 6: Population increase

Year	Population	Increase in population	Incremental increase	Geometric increase
1994	16,785			
2008	33,099	1,165		0.0694
2010	36,288	1,595	429.2	0.0481
2015	45,217	1,786	191.3	0.0492
Total		4,546	620.5	0.166
Avg		1,515	310.25	0.0556

A. Arithmetic Increase Method

Sample calculation

$$K = (P_{2008} - P_{1994}) / (2008-1994)$$

$$= (33099-16785)/14 = 1165.28$$

Table 7: Calculation of the projected population by arithmetic method

Year	2015	2016	2021	2026	2031	2036	2041
Population	45217	46732	54308	61884	69460	77036	84612

Sample calculation

$P_n = P_o + kn$

$$P_{2016} = P_{2015} + kn$$

$$= 45217 + 1515*1 = 46732$$

B. Geometric Increase Method

The percentage growth rate (k) for this method is calculated as follows.

$P_n=P_0\ (1+k)^{n}$

k=5.56% - urban annual average growth rate

Table 8:Calculation of projected population by geometric method

Year	2015	2016	2021	2026	2031	2036	2041
Population	45217	47731	62561	81999	107476	140869	184637

Sample calculation

$$P_{2016}=p_{2015}\ (1+k)^{n}$$

$$=45217(1+5.56/100)^{(2016-2015)}$$

$$=47731$$

C. Incremental increase method

$P\ n = Po+ n * (k+ r)$

k= Arithmetic increase

r= Geometric increase

n= No of year

Table 9:Calculation of projected population by incremental increase method

Year	2015	2016	2021	2026	2031	2036	2041
Population	45217	47042	56169	65295	74421	83547	92674

Sample calculation

$$P_{2016}=p_{2015+}\ n^{*}\ (k+r)$$

$$= 45217+1*(1515+310.26)$$

$$=47042$$

D. Ethiopian statistic authority method

From population growth rates found from CSA, the growth rates of Holeta town is tabulated below.

Table 10: Population growth rate of urban population

Description	2015	2016	2021	2026	2031	2036	2041
Annual growth rate(urban) %	4.4	4.36	4.16	3.96	3.76	3.56	3.36

(Source: CSA, 1994 Population and housing census report)

Table 11:Calculation of projected population by Ethiopian census statistics (CSA) method

Year	2015	2016	2021	2026	2031	2036	2041
Population	45217	47232	58037	69901	82523	95495	105538

Sample calculation

$P_n = p_o e^{kn}$

$P_{2016} = p_{2015} e^{kn}$

$= 45217 * e^{(4.36/100)*(2016-2015)}$

$= 47232$

Table 12:Summary of projected total population by the four methods

Year	Population			
	Arithmetic	Geometric	Incremental	CSA
2015	45217	45217	45217	45217
2016	47731	46732	47042	47232
2021	62561	54308	56169	58037
2026	81999	61884	65295	69901
2031	107476	69460	74421	82523
2036	140869	77036	83547	95495
2041	184637	84612	92674	105538

Percentage error calculation

Those the above four formulas Arithmetic increase method, Geometric increase method, Incremental increase method and ESA method percentage error calculated blow in order to select the best fit formula to forecast the future Holeta town population and the town water demand.

Table 13: Percentage error calculation

Year	Arithmetic increase	Geometric increase	Incremental increase	CSA
Actual population 2015	45217	45217	45217	45217
Projected population 2015	18834	37340	46122	45218
N	0.5	0.5	0.5	5
% Error (rural)	58.35	17.42	-2.00	-0.001

Sample calculation

%error2015= (actual population2015-projected population2015)/(actual population2015)*100

= (45217-45218)/(45217)*100=-0.001%=0.00%

In general, we can simply observe that Holeta town is the city with vast opportunity of growth as stipulated in the preliminary/feasibility report and documents for the city municipality. In this regard one can select the geometric increase method. On the other, the above percentage error method shows the CSA is more reliable with less error. So we can finally take the CSA method for Holeta future population forecasting**.**

Table 14:Summary of population projection for Holeta Water supply project

Year	2015	2016	2021	2026	2031	2036	2041
Total population	45217	47232	58037	69901	82523	95495	105538

5. WATER DEMAND ASSESSMENT

5.1 General

Design of water systems require estimation of expected water demands applicable to size the pumping equipment, transmission and distribution pipe lines and storage facilities. 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. It varies according to the requirement of the domestic population, institutional, industrial and social establishments, etc. In addition to these, demand allowances need to be included for leakage, wastage, and operational requirements such as flushing of mains.

5.2 Water demand

Some of the factors that affect water demand are:-

Climatic condition size of the town, culture of people industries cost of wale, fault of water pressure in the distribution system, system of supply etc.

1. **Climatic condition: -** Water consumption during summer is more than winter. During summer everybody taxes both twice and thrice, clothes also become dirties, more water is used for drinking and more water s consumed, in running coolest. This is why we say water consumption is much more in summer than in winter.

2. **Size of the town: -** Generally, the demand of water per head will be more on big city than that in small city. In big cities lot of water is required for maintaining clean and health environments while in small towns more or less small.

3. **Culture of people: -** High class community uses more water due to their better standard of living and high economic status. Middle class people uses water at average rate and for poor people a single water tap may be sufficient for several families.

4. **Industries: -** more water is used in highly industrial city

5. **Cost of water: -** If cost of water is high, the water demand will be less .Hence the rate at which water is supplied to consumer may affect the rate of demand.

6. **Quality of water: -** A water work system having good facility and portable water supply will be more popular with consumers.

7. **Pressure in the distribution system: -** There would be of great importance in the case of localities having number of two or three storied buildings. Adequate pressure would mean an uninterrupted and constant supply of water.

8. **System of supply: -** The system of water may be continuous or intermittent. In continuous system water is supplied all 24 hours .while in the case of intermittent system water is supplied for hours of the day only results in some reduction in the consumption. This may be due to decrease in loss and other waste of full use.

9. **Method of charging: -** In a town where meters are used less quantity of water will be used than in towns without meters in their system. A metered supply ensures minimum of waste as the consumer then know that he was to pay.

Accordingly, the water demand of town is calculated with due consideration of actual conditions of the town and pertinent to available data. Where gaps are observed in acquiring of data, estimates are made from general experiences of the country utilised for similar towns.

The demand of water is divided under the following categories or types of water demand.

- ❖ Domestic water demand
- ❖ Non domestic demand
- ❖ Unaccounted for water

5.3 Domestic Water Demand

The water demand for actual household activity is known as domestic water demand. It includes water for drinking, cooking, bathing, washing flushing, toilet, etc. The demand will depend on many factors, the most important of which are economic, social and climatic.

Based on the available data obtained from the Holeta Water Supply Service has four major modes of service were identified for domestic water consumers. These are:

- ❖ House connections (HTC or HTU)
- ❖ Yard connections - private (YTO or YTU)
- ❖ Yard connections -shared (YTS), and
- ❖ Public taps (PT or PTU)

5.3.1 Population Distribution by Mode of Service

The percentage of population to be served by each mode of service for Holeta town is shown in the table below. Due to data limitation we adopt the calculation of extrapolation to know population percentage distribution of the remaining years. 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.

Therefore, the present and projected percentage of population served by each demand category is estimated by taking the above stated conditions and by assuming that the percentage for the house and yard tap users will increase gradually during the project service period while 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 come up.

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

In determining the future trends of the modes of service, factors that influence the growth rates were taken into account, these included:

- Willingness to pay and level of affordability of the community in relation to existing and planned water tariff levels;
- Ability to provide sufficient quantity and quality of water;

Table 15: Population percentage distributions by mode of service.

Year		2015	2016	2021	2026	2031	2036	2041
Connection type	House	5.7%	5.84%	6.58%	7.48%	8.38%	9.28%	10.18%
	Yard	24.6%	25.26%	28.64%	32.44%	36.64%	40.84%	45.04%
	Yard Shared	28.7%	29.44%	33.26%	37.68%	42.58%	47.48%	52.38%
	Public Tap	39.0%	37.54%	30.0%	21.40%	11.40%	1.40%	0.00%
	Non Domestic	30.0%	30.0%	30.0%	30.00%	30.00%	30.00%	30.00%
	Unaccounted For	30.7%	30.40%	28.9%	27.38%	26.78%	26.18%	25.58%

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

5.3.2 Per capita Water Demand

The per capita water demand for various demand categories varies depending on the size of the town, the level of development , the type of water supply schemes, the socio- economic conditions of the town, cost of water, system of sanitation and climatic condition of the area. The per capita water demand for adequate supply level has to be determined based on basic human water requirements for various activities of demand category. In Holeta water supply project we used projected per capita demand as follow.

Table 16: Projected per capita demand by mode of service (l/cap/day) (2015-2041)

Year		2015	2016	2021	2026	2031	2036	2041
Connection type	HTC	123	124.4	131.6	138	138	138	138
	YTU	34	34.4	36.4	38	38	38	38
	YTS	25	25.2	26.4	28	28	28	28
	PT	18	18.2	19.4	21	21	21	21

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

5.3.3 Domestic Water Demand Projection

In projecting the domestic water demand of Holeta the following procedures were followed:

- ❖ Determining population percentage distribution by mode of service and its future projection
- ❖ Establishment of per capita water demand by purpose for each mode of service;
  - Projected consumption by mode of service;
  - Adjustment for climate;
  - Adjustment due to socio-economic conditions
- ❖ **Adjustment for climate**

Climate condition is the main factor that affects water demand of the population under consideration. Therefore, the water demand should be adjust for climatic condition**.**

Table 17: Adjustment factors for climate

Group	Mean annual PPT(mm)	Factor
A	900 or less	1.1
B	900-1200	1.0
C	1200 or more	0.9

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

Holeta with a mean annual precipitation of 1367 mm belongs to Group C as per the design criteria. Thus, an adjustment factor of 0.9 was taken

- **Socio-economic adjustment factors**

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

Table 18: 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 for development but lower living standard at present	1.05
C	Town under normal condition	1.0
D	Advanced rural towns	0.9

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

Holeta is classified as a town of "Towns under normal Ethiopian conditions" and, therefore, categorized as a Group C town and was given an adjustment factor of 1.0.

After considering changes in population and changes in the mode of service, per-capita demand and applying the adjustment factors, the domestic demands were calculated and are presented in table below.

Table 19: Projected domestic water demand

YEAR		2015	2016	2021	2026	2031	2036	2041
Total Population		45217	47232	58037	69901	82523	95495	105538
House Connected	% population	5.7%	5.84%	6.58%	7.48%	8.38%	9.28%	10.18%
	Population	2577	2758	3819	5229	6915	8862	10744
	PCD	123	124.4	131.6	138	138	138	138
	TPCD(l/day)	317016	343139	502555	721541	954325	1222946	1482643
	TPCD(m^3/day)	317.0	343.1	502.6	721.5	954.3	1222.9	1482.6
Yard Connected	% population	24.6%	25.26%	28.64%	32.47%	36.64%	40.84%	45.04%
	population	11123	11931	16622	22697	30236	39000	47534
	PCD	34	34.4	36.4	38	38	38	38
	TPCD(l/day)	378195	410420	605029	862475	1148980	1482005	1806307
	TPCD(m3/day)	378.19	410.42	605.03	862.47	1148.98	1482.00	1806.31
Yard Shared Connected	% population	28.7%	29.44%	33.26%	37.68%	42.58%	43.56%	44.54%
	Population	12977	13905	19303	26339	35138	41598	47007
	PCD	25	25.2	26.4	28	28	28	28
	TPCD(l/day)	324432	350409	509598	737478	983868	1164733	1316188
	TPCD(m3/day)	324.4	350.4	509.6	737.5	983.9	1164.7	1316.2
Public Tap Connected	% population	39.0%	37.54%	30.0%	21.40%	11.40%	1.40%	0.00%
	Population	17635	17731	17434	14959	9408	1337	0
	PCD	18	18.2	19.4	21	21	21	21
	TPCD(l/day)	317423	322703	338223	314133	197559	28076	0.0
	TPCD(m3/day)	317.4	322.7	338.2	314.1	197.6	28.1	0.0
Total domestic demand		1337.07	1426.67	1955.40	2635.63	3284.73	3897.76	4605.14
Socio-economic		1	1	1	1	1	1	1
Climatic Factor		0.9	0.9	0.9	0.9	0.9	0.9	0.9
Total domestic demand		1203.4	1284.0	1759.9	2372.1	2956.3	3508.0	4144.6

5.4 Non domestic water demand

Non-domestic water demand was also determined systematically. It can be broadly classified into the following major categories:

- ❖ Institutional water demand
- ❖ Industrial water demand.
- ❖ Commercial water demand

5.4.1 Commercial water demand

Commercial water demand are the water furnished to commercial establishments such as, hotels, bars, butchery, miscellaneous shops, metal works, video house, vegetable sells shops, grinding mills, beer and soft drinking distributers, cloth toilers, tea shops and restaurants etc. This quantity will vary considerably with the nature of the city and with the number and type of commercial establishments in it.

5.4.2 Industrial water demand

The demand for industrial water supply is generally assessed separately. In case of Holeta town some categories of industries will be included in domestic demand. Currently four small industries related to construction and flower culturing is utilizing about 30m^3/day from town's water supply service. The industrial water demand for Holeta town is 5% of the total domestic demand of the future year. But 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.

Table 20 Projected water demand for industrial

Year	2015	2016	2021	2026	2031	2036	2041
TDD(m3/d)	1203.36	1284.00	1759.86	2372.06	2956.26	3507.98	4144.62
% of TDD	5%	5%	5%	5%	5%	5%	5%
Industrial water demand(m^3/d)	60.17	64.20	87.99	118.60	147.81	175.40	207.23

5.4.3 Institutional Water Demand

The water required for schools, hospitals, health centre offices, government offices and services, religious institutions and other public facilities is classified as institutional water demand.

Summary of Total Non-Domestic Demand

Total non-domestic water demand of Holeta is calculated as institutional and commercial water demand is summarized in the below table.

Table 21: Summary for non-domestic water demand

Year	2015	2016	2021	2026	2031	2036	2041
TDD	1203.36	1284.00	1759.86	2372.06	2956.26	3507.98	4144.62
Commercial 10%TDD	120.34	128.40	175.99	237.21	295.63	350.80	414.46
Industrial 5%TDD	60.17	64.20	87.99	118.60	147.81	175.40	207.23
Institutional 15%	180.504	192.6	263.529	355.809	443.439	526.197	621.693
NDD(m^3/day)	361.01	385.20	527.96	711.62	886.88	1052.40	1243.39

5.4.4 Fire Fighting Demand

Fire fighting 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. Because the greater the population, the greater will be the number of buildings and hence greater risk of fire. By the minimum limit of fire demand is meant the amount and rate of supply required for extinguishing the largest possible fire that could be in the community. The required amount of water for firefighting will not be more than the amount of water distributed during the maximum day water demand.

The quantity of water needed to extinguish fire depends upon population, contents of Buildings, density of buildings and their resistance to life. In our case the fire fighting water requirement is taken care of by increasing 10 % of the volume of storage reservoir will be meet from the storage but not from the sources. Therefore, the water required for fire fighting shall be meet by stopping supply to consumer for required time and directly it for firefighting purposes.

5.4.5 Non-revenue water

Losses from a water distribution system consist of:

- ❖ Leakage and over flow from service reservoirs
- ❖ Leakage from main and service pipe connections,
- ❖ Leakage and losses on consumers premises when they get unmetered house hold supplies
- ❖ Under registration of supply meters and
- ❖ Large leakage or wastage from public taps
- ❖ Losses in the supply lines are mainly due to defective pipe joint, cracked pipe and loose valves and fittings.

Table 22: Percentage lost

Year	2015	2016	2021	2026	2031	2036	2041
Percentage %	30.70%	30.40%	28.86%	27.20%	26.78%	26.18%	25.58%
TDD m^3/d	1203.4	1284.0	1759.9	2372.1	2956.3	3508.0	4144.6
NDD m^3/d	361.01	385.20	527.96	711.62	886.88	1052.40	1243.39
UWD m^3/d	480.26	507.44	660.27	838.76	1029.19	1193.91	1378.25

5.5 Water demand variation

5.5.1 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 projects lifetime.

5.5.2 Maximum Day Water Demand

The water consumption varies from day to day. The maximum day water demand is 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.

Table 23: Maximum daily factor

Town population	MDF
0 to20000	1.3
20001 to 50000	1.25
50001 and above	1.2

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

The proposed maximum Day factor usually varies between 1.2 & 1.3 as per the design criteria. Hence, a maximum day factor of 1.2 is used for Stage I and for stage II design period. The maximum day demand is used to in infrastructure calculations such as for source pumping requirements.

Table 24: Recommended maximum daily demand

Year	2015	2016	2021	2026	2031	2036	2041
Total population	45217	47232	58037	69901	82523	95495	105538
ADD m^3/day	2044.63	2176.64	2948.09	3922.45	4872.33	5754.29	6766.27
MDF	1.25	1.25	1.2	1.2	1.2	1.2	1.2
MDD m^3/day	2555.79	2720.80	3537.71	4706.93	5846.80	6905.14	8119.52

5.5.3 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. It is the highest demand of any one-hour over the maximum day. It 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.

In our case the population ranges for phase Iis between 50,001 to 100,000 i.e. 82523 and for phase II greater than 100,000 i.e. 105538 so we have adopted peak hour factor of 1.8 for phase I and 1.6 for phase II respectively.

Table 25: Recommended peak hour Factors

Population Range	**Peak hour factor**
<20,000	2
20,001 to 50,000	1.9
50,001 to 100,000	1.8
>100,000	1.6

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

Table 26: Summary of water demand assessment

Year		2015	2016	2021	2026	2031	2036	2041
Tot population		45217	47232	58037	69901	82523	95495	105538
TDD	m^3/d	1203.36	1284.00	1759.86	2372.06	2956.26	3507.98	4144.62
NDD	m^3/d	361.01	385.20	527.96	711.62	886.88	1052.40	1243.39
UWD	m^3/d	480.26	507.44	660.27	838.76	1029.19	1193.91	1378.25
ADD	m^3/d	2044.63	2176.64	2948.09	3922.45	4872.33	5754.29	6766.27
	l/day	2044629	2176644	2948090	3922446	4872329	5754285	6766265
	l/sec	23.66	25.19	34.12	45.40	56.39	66.60	78.31
MDF		1.25	1.25	1.2	1.2	1.2	1.2	1.2
MDD	m^3/d	2555.79	2720.80	3537.71	4706.93	5846.80	6905.14	8119.52
	l/day	2555786	2720805	3537708	4706935	5846795	6905142	8119518
	l/sec	29.58	31.49	40.95	54.48	67.67	79.92	93.98
PHF		1.9	1.9	1.8	1.8	1.8	1.8	1.6
PHD	m^3/d	4855.99	5169.53	6367.87	8472.48	10524.23	12429.26	12991.23
	l/day	4855994	5169529	6367874	8472483	10524232	12429256	12991229
	l/sec	56.20	59.83	73.70	98.06	121.81	143.86	150.36

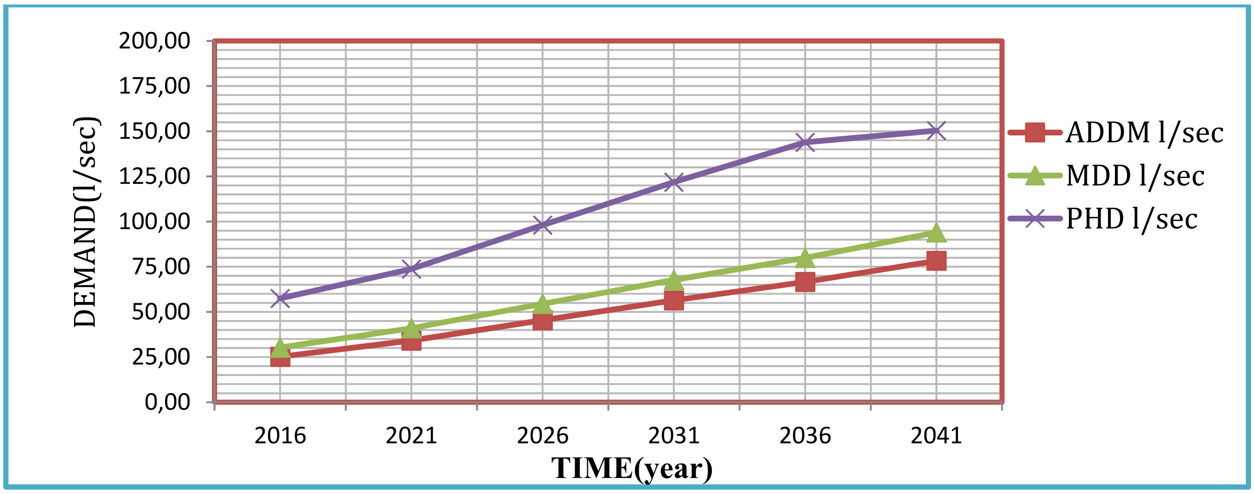

Figure 2: Summary of water demand assessment

6. WATER SOURCE AND WELL HYDRAULICS

6.1 General

Water source is the critical part of any water supply scheme. It's important that sources of supply be capable of providing service for both the short term and long term demands being projected. After deciding the water demand of the population at the design period, the next step is to search water source, which may be able to supply the required quantity of water.

There are mainly two aspects on which the success of a water supply scheme depends. These aspects are amount of available water from the source and the amount of water actually needed by the town. The source of water should be such that can provide adequate quantity of water. Availability of water from the source should at least be equal to the demand. Availability of water from a source which may be surface or ground ultimately depends upon rainfall. Rainfall is a natural feature, which may be more in one year and very slack in the next. In drought year, availability of water is minimum. The source of water for water supply schemes should be such which can provide adequate water even during severe drought conditions.

The existing water supply source of Holeta town is from three boreholes. The first borehole was drilled in 1997 by Oromia Water resources Bureau at the eastern end of the town near the bridge across the Holeta River, adjacent to Holeta-Wolmera road. According to the Town Water Supply and Enterprise/TWSSE/ head, the borehole has a production yield of 4l/s and depth of 90meters. The second borehole was drilled in 2000 as a test well during Feasibility Study and Engineering Design of Holeta Town water supply Development and Rehabilitation Project. It is in the east of the town at about 1,116m from the first borehole in the in the south. Currently, the borehole has a production yield of 3l/s with total depth of 147meters. The third borehole was drilled recently in 2007 by Oromia Water Resources Bureau, Holeta Town Administration and Holeta Water Supply and Sewerage Enterprise in the compound of Institute of Research for Agriculture /IRA/ in the east of town and commissioned service in April 2008. The borehole is reported to have a production yield of 5l/s and a total depth of 334m. Hence, the total yield of the existing water supply source is 12.3l/s.

Water shortage is the major problem with the existing system. Domestic supplies are supplemented from secondary sources from the river, small springs and hand dug wells.

6.2 Source of water

Determining the source of water is the main task in water supply scheme. Knowing the water demand of the population at the design period, the next step is to search water source, which may be able to supply the required quantity of water. The source of water can be either surface source of water or subsurface source of water (ground water).

This source should be capable of supplying enough water for the town population in a right quantity and quality. Availability of water from source, which may be surface or ground, ultimately depends up on rainfall. The sources of water supply schemes such which may provide adequate water during sever draught conditions.

Source Selection Criteria

Factors which are to be considered while selecting a source of water for a given water supply scheme are stated below.

- Location of the water source.
- Quantity of water.
- Quality of water.
- The cost of the water supply scheme.

1. **Location**

The source of water should be as near to the town as possible. If there are both surface and ground source available to the town selection should by considering other factors also. If there is no river, stream or reservoirs the city will have to depend on the underground source of water only.

2. **Quantity of Water**

The selected source should have sufficient quantity of water to meet up all the demand of city such as domestic demand, non-domestic demand and unaccounted for water throughout the year. There should be sufficient quantity of water to meet the demand dictated by future expansion. Source of water should be able to meet the maximum demand in dry weather also.

3. **Water Quality**

Water to be used for a public supply must be potable, i.e. drinkable. The water found in nature contains a number of impurities in varying amounts. The aim of water treatment is to produce and maintain water that is safe, aesthetically attractive and potable, in an economic manner the amount and type of treatment process will depend on the quality of raw water and standards of quality required after treatments. The water that is to be supplied to public consumption has to satisfy the standard of World Health Organization (WHO).

The water that removes its impurities only up to certain extent so that it may not be harmful to the public health is called wholesome water. Drinking water must be wholesome and potable.

Generally, the requirements of wholesome water are:-

- ❖ It should be free from disease producing organisms and poisonous
- ❖ It should be colorless, odorless and must be attractive
- ❖ It should be not corrode pipes
- ❖ It should be free from all objectionable matter
- ❖ It should have dissolved oxygen and free carbonic acid so that it remains fresh.

4. **Cost**

The cost of water supply project should also be taken in to account while selecting the source of water. The cost of water scheme depends on many factors as system of supply, ground levels of city, distance between source and distribution system etc. If the water flows under the gravitational force it will be cheap, but if it is to be pumped it will be costly. Similarly, the cost will directly depend on the distance between the source and city. If the distance is more it will be costly.

6.3 Existing source in Holeta

6.3.1 Surface source of water

The potentially available surface source for Holeta River is a good source of surface water close to Holeta town. It is a perennial river and the minimum 95% flow computed is about 150 l/s (i.e. the chance that less than this flow will occur is only 5%).This is sufficient to supply the town without a major storage requirement. Holeta River is only about 1.5 km from the town and is the most reliable surface water resource in the area for water supply development. This stream is not further considered for development of the town water supply both for Stage I and II since the water demand can be covered from groundwater which will be easier to manage.

6.3.2 Ground source of water

Ground water is the underground water that occurs in the saturated zone of variable thickness and depth, below the earth's surface. Cracks and pores in the existing rocks and unconsolidated crystal layers, make up a large underground reservoir, where a part of precipitation is stored.

The ground water is utilized through wells and tube wells. Various lifting devices, such as those using animal, manual, diesel or electric power, may be used, so as to bring the underground supplies to the surface.

The use of open wells is a traditional method of trapping ground water in where ground water table is high. The use of tube wells, however, is a subsequent development in the techniques of trapping ground water, certainly requires diesel or electric power.

Ground water sources include Springs Wells, Artesian wells, Infiltration galleries, Porous pipe galleries, Infiltration wells.

The recommended production rate from boreholes at Dobi near the Holeta–Mugger road was about 35 l/s (report by WWDSE, 2007). However, borehole yield is normally highly variable in fractured volcanic rock aquifers yet the assumed high production discharge rate is based on a single well test result. For similar borehole depths and well design, it is assumed that average, sustainable production discharges will be 8-10 l/s from the new boreholes in DobiKebele area.

6.4 Ground Water Recharge

Part of the rain water, that falls on the ground is infiltrated to the soil this infiltrated water is utilized partly in filling the soil moisture deficiency and part for it is percolated down reaching the water table. Recharge due to rainfall depends on various Hydro meteorological and topographical factors, soil characteristics and depth to the water table. To check the recharge of the borehole site, we used Krishna Reo empirical formula given as follows:

$$R = k\,(p - x)$$

Where - k, x, are constants according to annual rainfall

P- Rain fall in mm/year

R- Recharge in mm

Table 27: Values of k and x according to rainfall amount

K	X	P
0.2	400	400-600
0.25	400	600-1000
0.35	600	Above 2000

Source: Ray, K.Linsley, Water Resources Engineering 3rd edition

Since Holeta has an average precipitation of 1367 mm, to obtain the constant k and X values we must interpolate.

By interpolation for p= 1367 mm we have estimated,

$$k=0.2867$$
$$x=473.4$$
$$R=k\ (p-x)$$
$$R=0.2867(1367-473.4)$$
$$R=256.2m/year$$

Ground water recharges in terms of volume

$$V= R\ *\text{well site area}$$
$$V=0.256.2m/year*149km^2$$
$$= 38.17*10^3m^3/year$$
$$V=1210.5l/sec$$

In Holeta there are limited numbers of ground water sites. A bore hole drilled by WWDSE for exploration purpose at the vicinity of Holeta town is found to have attractive yield. In addition to the three existing boreholes and additional new borehole in Dobi site give the yield of 8 l/s.

Table 28: Existing borehole yield

Existing IAR BH (334m)	5.0 l/s,
HG1	3.3 l/s,
Wolmera bridge (BH I)	4.0 l/s

Source: from feasibility study

The Dobi test borehole indicates that the scoria aquifer at this location is of moderate transitivity and forms a potentially productive aquifer.

From this information we select the Dobi site for our project because the ground water yield is better rather than the existing site. After selecting the borehole site we have to know how many bore holes are enough to satisfy our peak hourly demand.

Determination of Number of Bore Holes

Number of bore hole at the end of design period i.e. at 2041

For economical purpose we have divided the design period into two phases

1st phase (2016 – 2031)

2nd phase (2031 – 2041)

(i) For 1st phase (2016-2031)

Number of bore holes $= \dfrac{\text{Max, day demand at 2031 - existing maximum daily demand}}{\text{safe yield}}$

Safe yield=8l/sec (from feasibility study)

$$= \frac{5846.8*10^{\wedge}3/(24*3600)l/s - 20.3l/s}{8l/s} = 5.92$$

= 5.92 bore holes say 6

As reported in feasibility study there are three existing bore holes having 5, 4 and 3.3 l/s each. So for first phase these existing boreholes can be included.

Therefore number of bore holes which have to be constructed in the 1st phase = 6-3= 3 boreholes.

(ii) For 2nd phase (2031-2041)

Number of bore holes $= \dfrac{\text{Max, day demand at 2041 - existing maximum daily demand at2031}}{\text{safe yield}}$

Safe yield=8l/sec (from feasibility study)

$$= \frac{5846.8*10^{\wedge}3/(24*3600)l/s - 20.3l/s}{8l/s} = 5.92$$

= 5.92 bore holes say 6

Therefore, keeping in mind the presence of three existing bore holes in phase **I**we will construct (4+3) =7 bore holes for this project to satisfy the water demand of the town up to 2041.

6.5 Well Hydraulics

6.5.1 General

Water well is a hole or shaft, in most cases, vertical excavated in the earth, or sunk in to the ground intercepting one or more water bearing strata, for bringing ground water to the surface.

The objective of water well is:-

- To provide water with good quality
- To provide sufficient quantity of water
- To provide water for long time
- To provide water at low cost

6.5.2 Factors Affecting the Quantity of Well Water

The quantity of water available from wells is affected by:-

- Porosity of different layers of earth
- Amount of water stored and absorbed in different layers
- Geological conditions indicating the slope of water bearing stratum

6.5.3 Well Development

The development of well have broad objectives:-

- Repairing of damage is done during the drilling operation, so that the natural hydraulic properties are restored.
- Altering the basic physical characteristics of the aquifer near the borehole, so that water will flow more freely to a well
- Well development is the most important for the wells where the formation material has been disturbed during the construction processes and highly affected by the drilling activity.

6.5.3.1 The Importance of Well Development

- Remove the filter cake or drill in fluid film that coats the borehole, and remove much or all of the drilling fluid and natural formation solid that have invaded the formation.
- Reduce the compaction and intermixing of grain size during drilling by removing fine material from the pore space.
- Increase the natural porosity and permeability of previously undisturbed formation near the well by selectively removing the finer fractional of an aquifer.
- Create a graded zone of sediment around the screen in a naturally development well, there by establishing the formation so that the well will yield sand

6.5.3.2 Methods of Well Development

In case of rocks the capacity of well can be increased by explosions in the wells which will increase the cracks and passage through which water in the wells. In the case of sandy stratum the yield can be increased by packing gravel around the well. In the beginning when new well is constructed the water which is drawn contains large quantity of sand. These sand particles will stick on the mesh of strainer pipe and will decrease the capacity of the well. Different well development methods have evolved in different areas, because of the difference of the physical characteristics of aquifer and the type of drilling methods used.

- **Back washing or back blowing**: - In this method water is forced in the reverse direction by means of compressed air pressure. All the sand, clay material which is stickled around the strainer pipe and chocked it is agitated and removed. These are then removed by means of pumping and bailing.
- **Surging:** is used to loosen sand and fine materials in the screen and filter zone. The surging action is created by lifting the water near to the surface by injecting air in to the well and then shut off the air to allow the water to flow back through the well and formation. Pumping water with air lift can be used for cleaning a well from sand and fine materials. Using the air lift means no water, as would be the case if a submersible or turbine pump is used to clean the well.
- **Over pumping**; - loose sand materials are removed by pumping the well at a higher rate than the well will be pumped when put in to service. It has advantage that much of the fine material brought into the borehole is pumped out immediately.
- **Water-jetting**; - maximum development efficiently is developed if water jetting is combined with simultaneous pumping with air lift, as the loosened material is not allowed to settle again.

Well development work must be done in a manner that does not cause under settlement and disturbance of the strata above the water bearing formation, not disturb the seal affected around the well casing and thereby reducing the sanitary protection otherwise afforded by such a seal.

6.6 Well Design

Well design is the process of specifying the physical material and dimension for a well. This includes the selection of a suitable material diameter and thickness of pipe. The choice of open wells or bore wells and the method of well design depend up on topography, geological conditions of the underlying strata, and depth of ground water table, rain fall climate and the quantity of water required. A well design involves selection of proper dimensions like the diameter of the well and that of the casing, length and location of the screen including slot size, shape and percentage open area whether the well naturally developed or gravel pack is necessary; design of gravel pack, selection of screen materials etc. Good water well design aims at ensuring an optimum combination of performance and long service life at reasonable cost. The use of correct sizes of well casing and well screen, choice of good quality materials, and strength and proper development of the well, will reduce long term power costs due to higher costs of pumping and maintenance costs and increases the useful life of the well.

Generally a well is completed to the bottom of the aquifer. This allows more of the aquifer to be utilized and ensures the highest possible production from the well.

The principal objective of good design should ensure:-

- To obtain highest yield with minimum draw down consistent and aquifer capability.
- Durability of the well so as to coincide with design period.
- Reduction of operation and maintenances cost to a certain extent.
- Good quality of water with proper protection.

As it is maintained above standard well design procedure involves choosing the casing diameter and material estimating the well depth, selecting the height diameter and material of screen.

6.7 Well Depth

The depth of a tube well depends up on the locations of water bearing formations, desired yields of the well and economic considerations.The well must be designed to penetrate the aquifer as deep as possible within the budgeted cost. During the test hole drilling, the drilling contractor will complete a formation log. Soil and rock samples are taken at various depths and the type of geologic material is recorded. This allows the driller to identify aquifers with the best potential for water supply. The well is usually drilled up to the bottom of the aquifer so that aquifer thickness is available, permitting greater well yield.

Generally a well is completed to the bottom of the aquifer. This allows more of the aquifer to be utilized and ensures the highest possible production from the well.

6.7.1 Design of Length and Size of Screen

Screen diameter is selected to satisfy a basic principle: enough open area must be provided so that the entrance velocity of the water generally not exceed the design standard of 3cm/sec. screen diameter can be adjusted with in rather narrow limits after the length of the screen and size of screen opening (slot size) have been selected . Well yields are affected by screen diameter, although increasing the screen diameter has much less in pelted on well yield than increasing the screen length.

The total length of the screen to be provided for a tube well shall be primarily controlled by the available thickness of the aquifers. In case of confined homogeneous aquifers about 80 to 90% of central portion of the aquifer is selected for screening.

Based on the above information the length of screen shall be taken 90% of the main aquifer depth. The main aquifer depth is from 236m – 300m, which is 250m and considered as confined aquifer. Hence 90% of the depth of the main aquifer should be screened which is 225m. There for our project the length of screen is 225m.

Table 29: Recommended value of screen diameter

Discharge l/min	Recommended screen diameter (cm)
0 to 475	10
475 to 1125	15
1125 to 3000	25
3000 to 5250	30
5250 to 9500	35
9500 to 13300	40

(Source:-Environmental Engineering vol.1, S.K.Garg)

The discharge of one well is 8l/sec that means 480 lpm. Therefore the recommended value of screen diameter will be 15cm.(Source:-Environmental Engineering vol.1, S.K.Garg)

Well screen slot openings

The size of the slot opening is determined by the size of gravel pack or aquifer material which the screen has to retain. The width of slot cut in iron pipes falls in the range of 1.6 to 2mm due to the limitations of the width of the cutting tool. Since gravel packed is used for our project, a slot size of 2mm will be used for both phase I and phase II.

6.7.2 Design of Well Screen

A discharge of 480 l/min the recommended screen diameter is 15 cm and also (15-20) % of opening area should be taken.

Equation

$$Q = A_o * v_e$$

Where A_o = area of openings π

Ve = entrance velocity

Q = well yield = 8 $\lambda/_{sec}$

$A_o = k * \pi * D_s * L_s$

Where k - % age of opening= 15%

D_s- Diameter of screen =15cm

L_s- Length of screen = 225 (calculated above)

Area of opening= 0.15*3.14*0.15m*225m

=15.89m^2

Therefore entrance velocity =well yield/area of opening

$$nV_e = \quad Q/A = \frac{0.008}{15.89}$$

$$= 0.000503 m/sec$$

This value is not safe because entrance velocity ranges 2-3cm/sec. Therefore, to get the permissible value we must decrease the length of the screen and / or also decrease diameter of screen, so decrease the length of the screen, take L_s = 40m, still not safe!

Using Continuity equation,

$A_1V_1 = A_2V_2 \qquad A_1V_e = AV$

Take length of screen=40m

Area of opening=0.15*0.15m*3.14*40m

=2.826m^2

Take V=3cm/sec

Ve=0.008/2.826

=0.283

Therefore;

2.826m^2*0.283cm/sec= A*3cm/sec

A = 0.266m^2

Use this area to get the length of the screen (Ls)

A=0.15*0.15*3.14*Ls

$Ls = 3.76m$, take 4m.

From the above calculation we conclude that the length of screen is 4m with 15 % opening area and diameter of 15 cm with entrance velocity of 3cm / sec are used for designing of well screen.

6.7.3 Diameter of the well pipe

The size of the well should be properly chosen since it significantly affects cost of well construction. It must be large enough to accommodate the pump required for the head and discharge with proper clearance of at least 5 cm around the maximum diameter of the bowl assembly for installation an efficiency operation

The diameter of the well pipe (or well tube) depends up on the discharge and Permissible velocity of flow through the perforation or slots of screen. The permissible velocity is usually limited b/n 1.5 to 3 cm /sec, based on the gross cross sectional area of the pipe.

The cross-sectional area of the pipe can be determined from the relation:

$$A= Q/v$$

where Q-yield of the well =8 l/sec

V-permissible velocity = take 2m/sec

$$A=\frac{8*10^{-3}}{2}$$

$$A =40cm^2$$

But, $A = \frac{\pi d^2}{4}$

$$\Rightarrow \quad D = \sqrt{\frac{4Q}{\pi V}}$$

$$D = \sqrt{\frac{4 * 0.008m3}{2m/s * \pi}}$$

$=0.071383061024825m$

$=7.14cm$ take 1”=2.54cm

$=2.81$inch takes 3 inch diameter of well pipe

6.7.4 Diameter of Bore Hole

The diameter of the bore hole is kept at least 5cm greater than the diameter of the well pipe, so that the pipe can be easily lowered into the bore hole and gravel packing can be done.

Diameter of the Bore Hole=Diameter of the well pipe + Allowance of 5cm + thickness of gravel packing.

D_b = 7.14cm + 5cm + 15cm

=27.14cm diameter of borehole.

Table 30:Recommended well diameter with varies yields

Anticipated Yield	nominal size pump bowl (cm)	size of well casing (cm)	
		Minimum	optimum
<400	10	12.5	15
400-600	12.5	15	20
600-1400	15	20	25
1400-2200	20	25	30
2200-3000	25	30	35
3000-4500	30	35	40
4500-6000	35	40	50
6000-10000	40	50	60

(Source:-Environmental Engineering vol.1, S.K.Garg)

From the given test, data the proposed site for development of well is 8/s, which is 480l/min. From the given well yield and casing diameter relationship (table) the recommended optimum size of casing hill is 20cm.

7. SERVICE RESERVOIR

7.1 General

Distribution reservoir is also called service reservoir, which are mainly provided for storing the treated water, for supplying water to the town or city. These reservoirs are provided for meeting the water demand during breakout of fires, breakdown of pumps, repair etc. The reservoirs avoid the hourly fluctuations in the water demand.

Water storage requirement should take in to consideration the peak daily water uses and maximum hourly demand, the capacity of the normal and stand by pumping equipment, the availability and capacity of auxiliary power, the probable duration of power failure and promptness with which repairs can be made, and to furnish water for such emergencies as firefighting or accidental breakdowns. Additional considerations are land use, topography, pressure needs, distribution system capacity and demand.

7.2 Function of service reservoir

- To balance the hourly variation in water demand and to allow the treatment unit and pumps to work at average constant rate. This will reduce the running, maintenance and operation costs of the treatment units as well as improve their efficiency
- To maintain constant pressure in the distribution mains, because when the pressure in the pipeline decreases due to increase in demand at peak hours, the extra demand of water is fed by these reservoirs and the pumps continue their work at constant speed
- It reduces the necessary capacity of high rate pumping equipment
- It reduces the size of the transmission mains.
- It makes uniform pumping rate possible
- It reduces friction head losses.
- It provides uniform water pressure.

Water already pumped it to the elevated tank is more certain of availability than water at the lower level.

7.3 Accessories of Service Reservoir

The service reservoir has to be provided with the following accessories:

- **Storage tank**: -is the most important component. The site and shape of which depend on the design requirements.
- **Inlet pipe**: - pumped water under pressure is lifted through inlet pipe and this is kept above the highest water level stored in the tank. This is also known as rising main.
- **Man holes**: -for providing entry to the inside of reservoir for inspection and cleaning. Manholes are constructed on the roof slabs of the reservoir.
- **Out let Pipe**: -out let pipe is fitted at the lowest water level of the reservoir and used for supplying water to the community. This is operated with help of out- let control valve.

- **Wash out pipe (Drain off pipe):-** for removing water after cleaning of the reservoir. This is fitted at the lowest possible water level in the reservoir.
- **Float Gauge**: -This is a float arrangement fitted with a graduated scale, which indicates the water level in the tank at any time.
- **Over flow pipe**; - For some reason or other if the water raises above the full designed level of the tank it goes out of the tank through the over flow pipe.
- **Ladders**;-normally steel ladders are provided in the overhead reservoirs. They give facilities of climbing the top of the reservoir from ground level and also to get down inside the reservoir from reservoir roof.
- **Ventilators**: - This will allow fresh air to enter the reservoir which helps keeping the stored water under better condition

7.4 Sites of Distribution Reservoirs (proposed)

The distribution reservoirs are located near the central portion of distribution area. It is always better to construct on high ground of city or town, at such place it can be constructed economically.

7.5 Depth of Reservoir

There is an economical depth of service reservoir for any given site. For a given quantity of water either a shallow reservoir having long walls and a large floor area may be constructed or, alternatively. A deep reservoir may be constructed with high retaining walls and a smaller floor area. Depths most usually used are as follows:

Table 31: Depth of reservoir

Size (m3)	Depth of water (m)
Up to 3500	2.5 to 3.5
3500 to 15,000	3.5 to 5.0
Over 15,000	5.0 to 7.0

Source: Water Supply and Sanitation, ENE 301

These figures don't apply to water towers or pre-stressed concrete reservoirs. Factors influencing depth for a given storage are:

- Depth at which suitable foundation conditions are encountered

- ❖ Depth at which the out let main must be laid
- ❖ Slope of ground, nature and type of back fill
- ❖ The need to make the quantity of excavated material approximately equal to the amount required for backing, so as to reduce unnecessary carting of surplus material to tip.
- ❖ The shape and size of land available

7.6 Determination of Storage Capacity

Storage capacity of reservoir should be adequate for the demand for the period of two hours in small communities and 10 to 12 hours in the case of large communities. Demand of water always keeps on varying hour, but treated water continuous to come out of treatment plant of a constant rate. Balancing reserve is that quantity of water required storing for balancing the variable demand in the distribution system.

Reservoir capacity is determined on the following basis:

Computation of storage capacity of a reservoir can be obtained from one of the following methods.

a. Analytical method
b. Mass curve technique

Mass curve and analytical method are adopted for the case Holeta town water supply system.

The analysis of storage capacity can be calculated as follows.

i. ***Phase I (2016– 2031)***

a) By Analytical method

- ❖ Total demand of the town = 5846.80 m3/day
- ❖ Total demand of the town in liters per day = 5846800 lit/day
- ❖ Hourly demand of the town =5846.80/24= 243.62 m^3/hr.
- ❖ Pumping hours=20hr
- ❖ Hourly supply=hourly demand of the town/pumping hour
- ❖ Hourly supply = (1/20)* 5846.80 = 292.34m^3/hour

Table 32: Analytical calculation of storage capacity for phase I

Time (Hr)	Hourly Factor	Hourly Demand (m3)	Cumulative Hourly Demand (m3)	Hourly Pumping Rate/Hourly Supply(m3)	Cumulative Hourly Supply(m3)	Supply/ Surplus (m3)	Demand /Deficiency (m3)
1	0.25	60.90	60.90	0.00	0.00		60.90
2	0.25	60.90	121.81	0.00	0.00		121.81
3	0.25	60.90	182.71	292.34	292.34	109.63	
4	0.25	60.90	243.62	292.34	584.68	341.06	
5	0.5	121.81	365.42	292.34	877.02	511.59	
6	0.75	182.71	548.14	292.34	1169.36	621.22	
7	1	243.62	791.75	292.34	1461.70	669.95	
8	1.2	292.34	1084.09	292.34	1754.04	**669.95**	
9	1.5	365.42	1449.52	292.34	2046.38	596.86	
10	1.8	438.51	1888.03	292.34	2338.72	450.69	
11	1.7	414.15	2302.18	292.34	2631.06	328.88	
12	1.6	389.79	2691.96	292.34	2923.40	231.44	
13	1.5	365.42	3057.39	0.00	2923.40		133.99
14	1.4	341.06	3398.45	0.00	2923.40		475.05
15	1.32	321.57	3720.02	292.34	3215.74		504.29
16	1.32	321.57	4041.60	292.34	3508.08		533.52
17	1.4	341.06	4382.66	292.34	3800.42		582.24
18	1.4	341.06	4723.72	292.34	4092.76		630.97
19	1.2	292.34	5016.06	292.34	4385.10		**630.97**
20	1.06	258.23	5274.30	292.34	4677.44		596.86
21	0.9	219.25	5493.55	292.34	4969.78		523.78
22	0.7	170.53	5664.08	292.34	5262.12		401.97
23	0.5	121.81	5785.89	292.34	5554.46		231.44
24	0.25	60.90	5846.80	292.34	5846.80	0.00	

Thus the reservoir capacity for phase **I** from the above table will be

Maximum value of excess supply = 669.95m^3

Maximum value of excess demand = 630.97m^3

Capacity of reservoir =669.95 m^3+630.97m3= 1300.92m^3

For fire requirement (10%) = 130.092m^3

Accounting 5% for miscellaneous losses =65.046m^3

Total recommended reservoir capacity =1300.92m^3+130.092m^3+65.046 m^3 `

=1496.058m^3

For safety provide a reservoir with a capacity of =1500 m^3

Capacity of existing reservoir =300+50=350m^3

Capacity of new reservoir = 1500 m^3-350m^3 = 1150m^3

We will construct 1150m^3 standard reservoir.

b) Using mass curve method

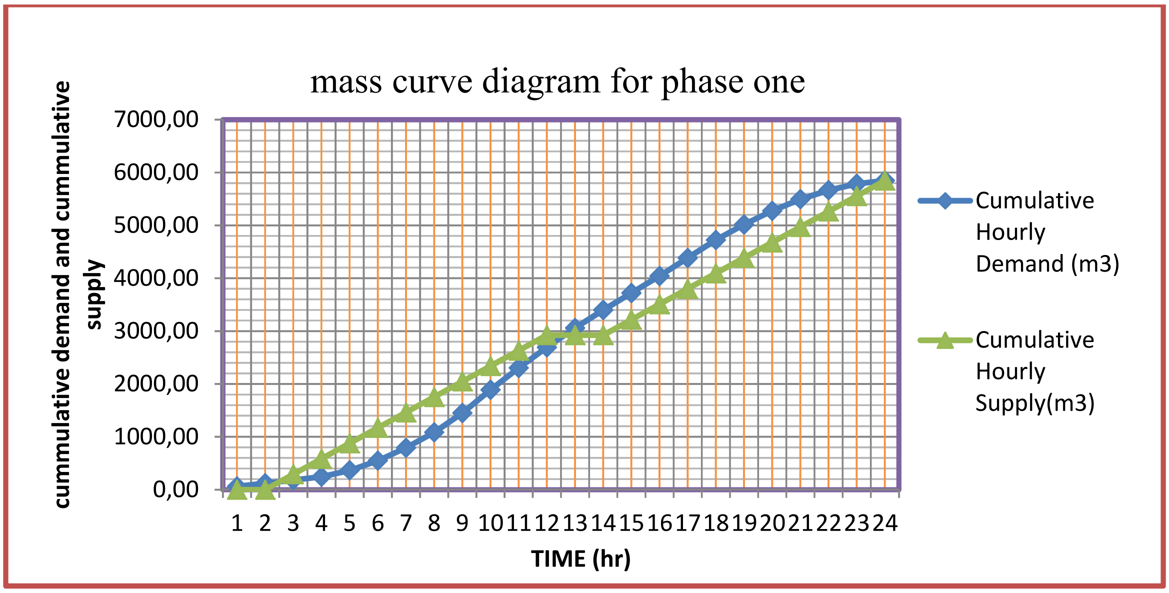

Figure 3: Mass curve graph for phase I

From the graph, Balancing storage= (1754.04-1084.09) + (5016.06-4385.10)

=1300.91m^3

The two methods are Almost they have equal result

Fire demand (10%) =130.091m^3

Accounting 5% for miscellaneous losses=65.0455m^3

Total capacity of reservoir=1495.2275 m^3

Capacity of new reservoir=1495.2275 -350=1145.2275m^3

Therefore, the capacity of reservoir =1150m^3

For design purpose we use the standard reservoir capacity of =**1150 m^3**

ii. ***Phase II (2031– 2041)***

a. By analytical method

Total demand of the town = 8119.52m3/day

Total demand of the town in liters per day = 8119520 /day

Hourly demand of the town = $\frac{8119.52}{24} m^3$=338.31m^3/hrs.

Pumping hours=20hr

Hourly supply/Pumping rate=$\frac{8119.52}{20}$ m^3 =405.976m^3/hrs.

Table 33: Analytical calculation of storage capacity for phase II

Time (Hr)	Hourly Factor	Hourly Demand (m3)	Cumulative Hourly Demand (m3)	Hourly Pumping Rate/Hourly Supply(m3)	Cumulative Hourly Supply(m3)	Supply/ Surplus (m3)	Demand/Deficiency (m3)
1	0.25	84.58	84.58	0.00	0.00		84.58
2	0.25	84.58	169.16	0.00	0.00		169.16
3	0.25	84.58	253.73	405.98	405.98	152.24	
4	0.25	84.58	338.31	405.98	811.95	473.64	
5	0.5	169.16	507.47	405.98	1217.93	710.46	
6	0.8	270.65	778.12	405.98	1623.90	845.78	
7	1.05	355.23	1133.35	405.98	2029.88	**896.53**	
8	1.35	456.72	1590.07	405.98	2435.86	845.78	
9	1.55	524.39	2114.46	405.98	2841.83	727.37	
10	1.6	541.30	2655.76	405.98	3247.81	592.05	
11	1.6	541.30	3197.06	405.98	3653.78	456.72	
12	1.45	490.55	3687.61	405.98	4059.76	372.14	
13	1.35	456.72	4144.34	0.00	4059.76		84.58
14	1.35	456.72	4601.06	0.00	4059.76		541.30
15	1.4	473.64	5074.70	405.98	4465.74		608.96
16	1.45	490.55	5565.25	405.98	4871.71		693.54
17	1.5	507.47	6072.72	405.98	5277.69		795.04
18	1.4	473.64	6546.36	405.98	5683.66		862.70
19	1.25	422.89	6969.25	405.98	6089.64		**879.61**
20	1.05	355.23	7324.48	405.98	6495.61		828.87
21	0.9	304.48	7628.96	405.98	6901.59		727.37
22	0.7	236.82	7865.78	405.98	7307.57		558.22
23	0.5	169.16	8034.94	405.98	7713.54		321.40
24	0.25	84.58	8119.52	405.98	8119.52	0.00	

Thus the reservoir capacity for phase **II** from the above table will be

Maximum value of excess supply =896.53m^3

Maximum value of excess demand = 879.61m^3

Capacity of reservoir =896.53m^3+879.61m^3=1776.14m^3

For fire requirement (10%) = 177.614m^3

Accounting 5% for miscellaneous losses =88.807m^3

Total recommended reservoir capacity = 2042.561m^3

For safety provide a reservoir with a capacity of =2050m^3

There is an existing reservoir with volume of 1550(i.e. 350+1150) m^3

Capacity of new reservoir =2042.561m^3-1496.058m^3=546.503m^3

Volume=**550 m^3**

b. Using mass curve method

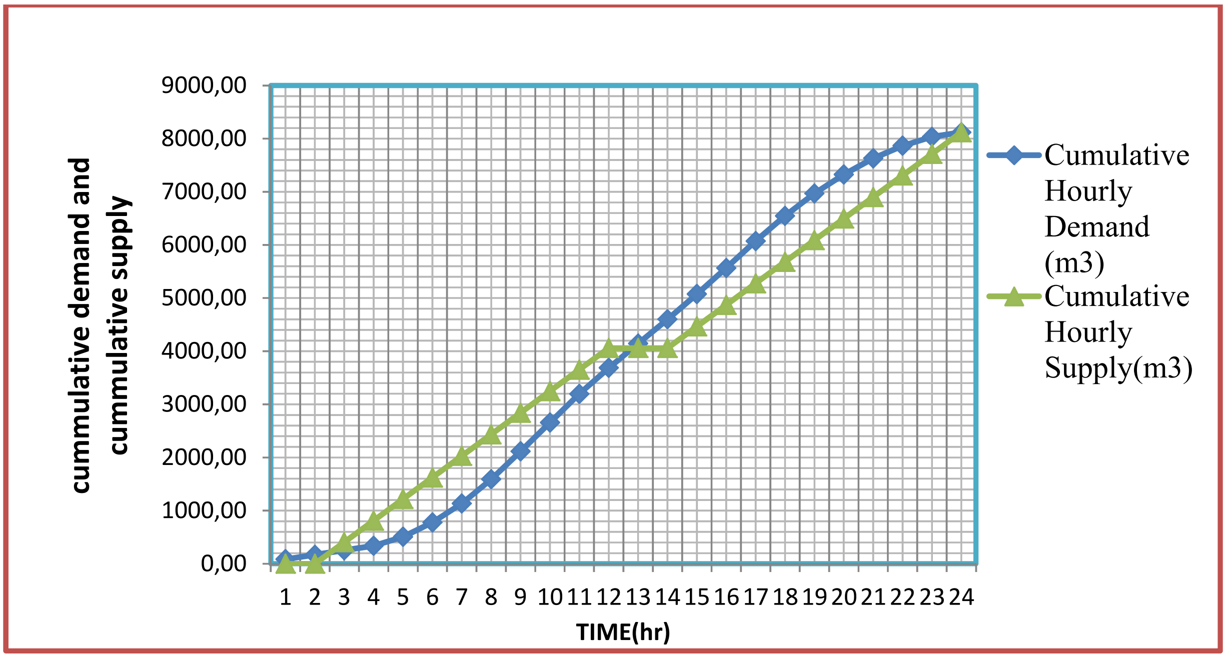

Figure 4: Mass curve graph for phase II

From the figure, Balancing storage = (2029.88-1133.35) + (6969.25-6089.64) =1776.26m^3

Fire demand (10%) =177.626 m^3

Accounting 5% for miscellaneous losses =88.813m^3

Total capacity of reservoir=2042.699m^3

Capacity of new reservoir=2042.699m^3-1495.2275 m^3=547.4715m^3

Therefore we take the maximum value from two methods. I.e. 550m^3

We use standard value of **550m^3**

7.7 Structural Design of service reservoir

7.7.1 Water Tank (reservoir) Design Consideration

The primary purpose of design is to achieve acceptable probabilities that the structure being designed will not become unfit in any way for the use it is intended. To accommodate the required amount of water and to ensure water tight structure, the reservoir must be designed using reinforced cement concrete that accounts for tensile forces as well as those due to bending.

The different types of reservoirs depending on the geometry (Circular and Rectangular), supports (resting on the ground, underground and elevated), and end restraints (free sliding, hinged and fixed at top and/or base) should be compared and selected based on their suitability and economic condition during the design of water containing reservoirs.

For small capacities rectangular tanks are usually used. And for bigger capacities circular tanks are generally used to ensure economical and efficient system of work.

Important considerations have to be given in limiting the size of crack (mostly with no cracks) so that leakage does not take place. The design generally governed by the requirements of the elastic design method, but stability considerations are particularly important. The design has to take careful account of the construction methods to be used.

Analysis and design of water tanks are based on two criteria i.e.strength design and resistanceto c racking. Important considerations have to be given in limiting the size of crack so that leakage does not take place. That means the tensile stress in steel will be limited by the avoidance of crack or limitation of crack width and it is related to the allowable tensile stress of concrete.

The design is generally governed by the requirements of the elastic design methodbut stability co nsiderations are particularly important. The design has to take careful account of the construction methods to be used.

The requirements for the elastic design method of water tanks are list as follows.

- Concrete grade C-30Mpa, (fck=24Mpa)
- Factor of safety for concrete, Yc=1.5
- $fcd = \frac{2}{3}\left(\frac{fck}{Yc}\right) = \frac{2}{3}\left(\frac{30}{1.5}\right) = 13.4N/mm^2$
- Deformed steel bar, S-300Mpa
- Factor of safety for steel, Ys = 1.15

- $f_{yd} = \left(\frac{fyk}{\gamma s}\right) = \frac{300}{1.15} = 260.8N/mm^2$
- Minimum cement content = 360kg/m^3
- Water cement ratio= 0.55
- Unit weight of concrete =25KN/m^3
- Unit weight of water =9.81KN/m^3
- The maximum crack width is limited to 0.2mm
- The minimum concrete cover should not less than 40 mm
- The minimum steel area in each of the two directions at the right angles is 0.3% of the concrete area (0.15% near each faces) for deformed bars.
- Maximum reinforcement spacing for wall section should not exceed 300 mm or thickness of the wall section.
- Assume exposure condition for wall as alternative wetting and drying
- The allowable tensile stresses in concrete than control cracks in concrete grade of C-30 shall be 1.44 N/mm^2 and 2.02 N/mm^2 due to direct tension and bending respectively.
- The allowable tensile stresses in steel for direct tension (fs, allow) taken as 130 N/mm^2 for deformed bars under alternate wetting and drying exposure conditions.
- The allowable compressive stress in concrete (fc, allow) is taking as 11N/mm^2.
- Modular ratio, n= 15
- Assume thickness of the wall 200 mm
- Assume thickness of the bottom slab 250mm thick
- Assume roof slab 150 mm thick
- Assume service live load=1KN/m^2 in addition to its own weight

There are different types of tanks in use depending on end restraints, flexible joints and rigid joints at top & base. Due to its effectiveness and less costly relative to the other types, fixed base type is select for the design of circular tank of reservoir.

7.7.2 Design of circular Reservoir with fixed base and Free at the top

At the top of the wall, shear force and bending moment are zero; and at the base of the wall slope and deflection is zero.

Design of circular reservoir Phase-I

- Base condition fixed
- Water depth in the reservoir 4.8m
- Free board 0.3m
- Grade of concrete-30mpa
- Volume of reservoir=1150m^3
- Clear diameter of reservoir (D) = $\sqrt{\frac{4*V}{\pi*h}}$

Since our reservoir capacity is 1150m^3 we Assumed, H = 4.8m

$$V = \pi\frac{D^2H}{4} \quad \text{where, V = Volume of reservoir}$$

D = Diameter of reservoir

H = Height of reservoir

$$D= \sqrt{\frac{4 \ * \ V}{\pi \ * \ h}}$$

$$1150 = \frac{\pi D^2 * 4.5}{4}$$

D= 18.038m≈18.5m

Design of Wall Section

Assume, t=250mm at bottom

Where, t =thickness of wall

t = 250 mm at top

$$t_{avg.} = \frac{250+200}{2} = 250 = \frac{h^2}{dtaverage} = \frac{4.5^2}{18.5*250*10^{-3}} = 4.3784$$

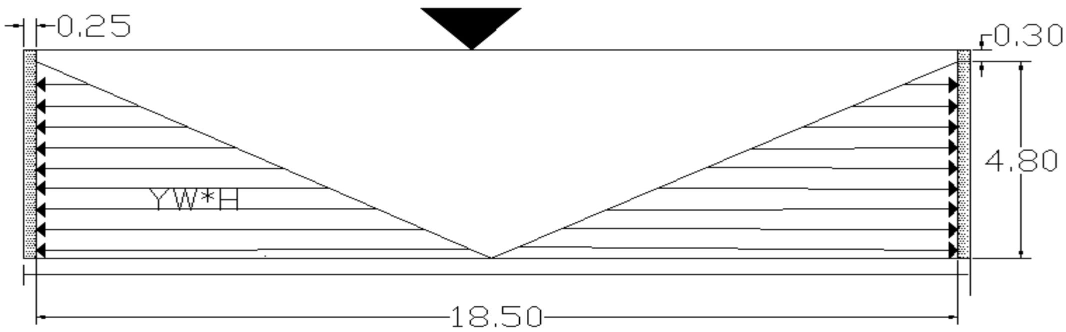

Figure 5: Pressure distribution on reservoir wall

Now using the value of $\frac{H^2}{D*t}$=5 and the corresponding depth from the top of the wall, the coefficient of vertical moment and hoop tension are determine from the table below.

Table 34: Coefficients for Hoop tension (Fixed at base and free at top).

$\frac{h^2}{dt}$	Coefficient at point									
	0.1h	0.2h	0.3h	0.4h	0.5h	0.6h	0.7h	0.8h	0.9h	1.0h
4	0.067	0.164	0.256	0.339	0.403	0.429	0.409	0.334	0.21	0.073
4.378	0.0511	0.1538	0.2518	0.3416	0.4124	**0.447**	0.4316	0.3548	0.228	0.080
5	0.025	0.137	0.245	0.346	0.428	0.477	0.469	0.389	0.259	0.092

Table 35: Coefficient for vertical moment (For fixed at base and free at top)

$\frac{h^2}{dt}$	Coefficient at point									
	0.1h	0.2h	0.3h	0.4h	0.5h	0.6h	0.7h	0.8h	0.9h	1.0h
4	0.0003	0.0015	0.0028	0.0047	0.0066	0.0077	0.0069	0.0023	-0.008	-0.0268
4.38	0.00026	0.00123	0.0023	0.0040	0.0058	**0.0070**	0.0065	0.0024	-0.00724	**-0.0249**
5	0.0002	0.0008	0.0016	0.0029	0.0046	0.0059	0.0059	0.0028	-0.006	-0.022

Source: *IS 3370-4 (1967): Code of practice for concrete structures for the storage of liquids, Part IV: Design tables [CED 2: Cement and Concrete]*

Horizontal reinforcement

a) For depth b/n 0.5h to 0.9h form max. Water level.

$$TH\ (max) = \alpha t + \gamma w * H * \frac{D}{2}$$

Where, H – Depth of water in the reservoir

D – Diameter of the reservoir

αt = Hoop tension coeff.

From the above table αt (coeff.) is max at 0.6h =2.7m from top

i.e. $\alpha t = T_H$ (max.) =00.447* 9.8*4.5*$\frac{18.5}{2}$

= 194.778KN

Area of reinforcement required along the hoop for unit strip of wall, A_s is

$$As = \frac{T_H}{fs, allow} = \frac{194.778 * 10^3 N}{100 N/mm^2} = 1947.78 mm^2$$

$$AS = 1947.78 mm^2 \geq \text{AS min} = 0.3\% \text{ of the concrete area}$$

$$\text{AS min} = \frac{0.3}{100} * 1000 * 250 = 750 \text{mm}^2$$

$$\text{AS} = 1947.78 mm^2 \geq \text{AS min} = 750 \text{mm}^2 \text{ ok!}$$

Check thickness of wall for no cracks

Thickness of wall is determined using the requirement of resistance to crack; therefore, thickness of wall is determined by limiting tensile stress in the concrete to allowable stress value as,

$$\text{fct} = \frac{\text{TH}}{(\text{Ac}+(\text{n}-1)*\text{As})} \leq \text{fct, allow} = 1.44 \text{N/mm}^2$$

$$\text{fct} = \frac{194.778 * 10^3 N}{(1000 * 250 + (15 - 1) * 1947.78)} \leq \text{fct, allow} = 1.44 \text{N/mm}^2$$

$$\text{fct} = 0.7025 \text{N}/mm^2 \leq \text{fct, allow} = 1.44 \text{N/mm}^2 \text{ok!}$$

Therefore, thickness is adequate for no crack!

Spacing of ring bars using Ø16

$$\text{s} = \frac{\text{as} * \text{b}}{\text{As}} \leq 300\text{mm or thickness of wall(250mm)}$$

$$\text{s} = \frac{\pi * 16^2 * 1000}{4 * 1947.78} = 103.23\text{mm} \leq 300\text{mm or thickness of wall(250mm)}\ ok!$$

Number of bar no bar=As/s=(1947.78)/(100)=19.4778=**20** bars

Therefore, provideØ16mm 20 bars @100mm c/c equally provided at a layer in each face.

Vertical reinforcement

Using the above table for H^2/Dt=4.378, the maximum value of moment coefficient=**-0.02498**, which occurs at 1.0H =1*4.8=4.8m from top.

The development of maximum positive and maximum negative flexure is calculating as follows.

$$\text{Mmax} = \text{max. coeff} * \gamma\text{w} * h^3$$

$$+\text{ve max . coeff } = 0.00701 \text{ at } 0.6\text{h}$$

$$-\text{ve max coeff } = -0.02498 \text{ at } 1.0\text{h}$$

Therefore,

$$(+\text{ve})\text{Mmax } = 0.00701 * 9.81 * 4.5^3 = 6.3469 KN.m$$

$$(-\text{ve})\text{Mmax } = -0.02498 * 9.81 * 4.5^3 = -22.330\text{KN}.m$$

Taking absolute value of the result, the maximum moment is therefore, $\text{Mmax } = -22.330\text{KN}.m = 22.330\text{KN}.m$

The minus sign indicates tension on inner face.

Check thickness of wall for flexure

fs, allow = 130N/mm^2, fc, allow = 11N/mm^2, n = 15

Design constants of balanced section are,

$$\mathbf{k} = \frac{\text{nfc}}{\text{fs} + \text{nfc}} = 15 * 11/(130 + 15 * 11)$$

k =0.5592

$$\boldsymbol{J} = \mathbf{1} - \frac{\boldsymbol{K}}{\mathbf{3}} = 0.8136$$

$$Rb = \frac{fc, allow}{2} * kb * jb = \frac{11}{2} * 0.559 * 0.8136 = 2.5028 N/mm^2$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{22.330 * 10^6\text{KN}.m}{2.5028N * 1000}} = 94.456mm$$

Hence, the already adopted thickness t=250mm is in order from the BM consideration.

Assuming cover 40mm up to center of main bars effective depth is;

d=250-40=210mm

$$Ast, M = \frac{M}{j*d*fs} = \frac{22.330*10^6}{0.813*210*130} = 1006.087mm^2$$

Spacing of Ø16 bars,

$$S = \frac{as * b}{As} = \frac{201 * 1000mm2}{1006.087mm2} = 199.784\text{mm}$$

Use S=195mm c/c

Number of bar $= \frac{As}{s} = \frac{1006.087}{195} = 5.16$ bars

Therefore, provide 6 bars with Ø16mm @195mm c/c inner face of the wall @ clear cover of 25mm.

Thickness of wall taking $\emptyset\ 16mm\ bars$ and 40 mm cover:

$$treq = dreq + cover + \frac{\emptyset}{2} = 94.456 + 40 + \frac{16}{2} = 142.456 < tasume = 250mm\ ok!$$

$$dreq = tassume - 40 - \frac{16}{2} = 202mm$$

Vertical reinforcement on inner face

$$(-ve)As = \frac{Mmax}{fs, allow * jb * dreq} = \frac{22.330 * 10^6}{130 * 0.813 * 202} = 1045.932mm^2$$

$$(-ve)As = 1045.932\ mm^2 > Asmin = 750mm^2\ ok!$$

Check thickness of wall for no crack

$$\bar{y} = \frac{\Sigma A * Y}{\Sigma A}$$

$$\bar{y} = \frac{(b * t) * \frac{t}{2} + (n * As) * d}{b * t + (n * As)}$$

$$\bar{y} = \frac{(1000 * 250) * \frac{250}{2} + (15 * 1045.932) * 202}{1000 * 250 + (15 * 1045.932)} = 129.547\text{mm}$$

y =250 – 130 =120 mm

x = 202 - 130= 72mm

$$\text{Ice} = \frac{b * \bar{y}^3}{3} + \frac{b * \boldsymbol{y}^3}{3} + n * As * x^2$$

$$I_{ce} = \frac{1000 * 130^3}{3} + \frac{1000 * 120^3}{3} + (15 - 1) * 1045.932 * 72^2 = 1540.576 * 10^6 mm^4$$

Then check for no crack:

$$f_{ct} = \frac{M_{max} * y}{Ice} < f_{c,allow} = 2.02mm^2$$

$f_{ct} = \frac{22.330*10^6*120}{1540.576*10^6}$= 2.02mm^2

=1.739mm^2< 2.02mm^2………………**ok**!

Therefore, thickness is adequate for no crack.

Spacing of Φ14 bars

$$S = \frac{as * b}{As} = \frac{153.94 * 1000}{1045.932} = 147.179mm < 300mm\ or\ 200mm\ ok!$$

Number of bar =$\frac{As}{s}$=$\frac{1045.932}{145}$=**7.2** bar/m

Therefore provide 8 bars with Φ14 bars at 145mm c/c vertically in the inner face.

Vertical reinforcements on outer face

$$(+ve)Mmax = 6.3469KNm$$

Using t=200 mm and $\emptyset 16mm$ bars $d = 250 - 40 - \frac{16}{2} = 202mm$

$$(+ve)As = \frac{(+ve)Mmax}{fs,allow * jb * d} = \frac{6.3469 * 10^6}{130 * 0.813 * 202} = 297.287mm^2$$

$$(+ve)As = 297.287mm^2 < As\ min = 750mm^2\ not\ \boldsymbol{ok}!$$

Therefore, take As_{min}=750 mm^2

Spacing of $\emptyset \boldsymbol{12mm}$ bars

$$s = \frac{(\pi * 12^2 * 1000)}{4 * 750} = 150.796mm < 300mm\ or\ 200mm$$

Number of bar $=\frac{As}{s}=\frac{750}{150}=$5 bar/m

So, provide 5 bar/m with $\emptyset 12mm$ @150 mm c/c in the outer face.

Design of maximum hoop tension

T= 194.778KN

$A_{st,T}=\frac{\boldsymbol{T}}{\boldsymbol{fs}} = \frac{194.778*\mathbf{10^3}}{\mathbf{130}} = 1498.29mm^2$

Let us provide hoop tension at both faces.

Thus, Area of reinforcement on each face$=\frac{1498.29mm2}{\mathbf{2}} = 749.15mm^2$

Number of bar $=\frac{As}{\frac{\pi*d^2}{4}}=\frac{749.15}{\frac{\pi*12^2}{4}}$=6.63≈ 7 bar/m

Spacing using Ø 12 bars,

$S = \frac{113*1000}{749.15} = 150.837$mm

Provide Ø12mm7bars @150mm c/c in the outer face.

Curtailment of hoop reinforcement

i. @0.4hm below top

T=0.3416*9.81*4.5*$\frac{18.5}{2}$=139.489KN

$A_{st,2}=\frac{\boldsymbol{T}}{\boldsymbol{fs}} = \frac{139.489*\mathbf{10^3}}{\mathbf{130}} = 1072.99mm^2$

$A_{st,2}$, on each face$=\frac{1072.99mm2}{\mathbf{2}} = 536.497mm^2$

Spacing using Ø 12 bars,

$$S = \frac{113 * 1000}{536.497} = 210.625mm$$

Use Ø 12 bars,@210mm c/c

ii. @0.3h m below top

T=0.2518*9.81*4.5*9.25=102.82KN

$A_{st,1}$=$\frac{\boldsymbol{T}}{\boldsymbol{fs}} = \frac{102.82*\mathbf{10^3}}{\mathbf{130}} = 790.9mm^2$

$A_{st\ 1}$ =$\frac{\mathbf{790.9}}{\mathbf{2}} = 395.46mm^2$

Spacing using Ø 12 bars,

$$S = \frac{113 * 1000}{395.46} = 285.74mm$$

Number of bar= $\frac{As}{\frac{\pi*d^2}{4}}$=$\frac{395.46}{\frac{\pi*12^2}{4}}$=3.5 bars/m

Use Ø 12 bars @ 280mm c/c

Check for tensile stress in concrete

$$\text{fct} = \frac{\text{T}}{1000\text{t}+(\text{n}-1)*\text{Ast}} \leq \text{fct, allow} = 1.44$$

$$fct = \frac{194.778 * 10^3}{1000 * 250 + (15 - 1) * 1498.29} = 0.718MPa < 1.44MPa\boldsymbol{ok}!$$

Distribution reinforcement

% Dist. =0.3-0.1 $* \frac{(\boldsymbol{t}-\mathbf{100})}{\mathbf{350}}$

=0.3-0.1 $* \frac{(\boldsymbol{t}-\mathbf{100})}{\mathbf{350}}$

=0.3-0.1 $* \frac{(\mathbf{250}-\mathbf{100})}{\mathbf{350}}$=0.257%

Area of distribution bars= $A_{st}1$ =$\frac{0.257*250*1000}{100} = 642.5mm^2$

Number of bar =$\frac{As}{\frac{\pi*d^2}{4}}$=$\frac{642.5}{\frac{\pi*8^2}{4}}$=**12.78** bar

Spacing using Ø 8 bars,

$$S = \frac{50.26 * 1000}{642.5/2} = 156.47mm$$

Use **13** no bars with Ø 8 bars@150mm c/c

Design of base slab

Keeping in view of 250mm thick base slab, minimum reinforcement

$A_{st,min} = \frac{0.3*250*1000}{100} = 750mm^2$

Spacing using Ø10 bars

$$S = \frac{78.54 * 1000}{\frac{750}{2}} = 209.44mm$$

Number of bar $= \frac{As}{s} = \frac{750}{200} =$ **3.75**≈4bars

Use S=200mm

Design of Roof Slab

Cover slab of service reservoir is treats as roof slab, which is simply support by the circular wall.

Assume cover slab as freely support at edges and loaded uniformly.

Using strength limit state design for C-30Mpa,

$$fcd = \frac{2}{3}\left(\frac{fck}{Yc}\right) = \frac{2}{3}\left(\frac{30}{1.5}\right) = 13.4N/mm^2$$

And for S =300Mpa, $f_{yd} = \left(\frac{fyk}{Ys}\right) = \frac{300}{1.15} = 260.8N/mm^2$

Considering the tope thickness of slab as 200 mm and load on slab per m^2

$$\text{Dead load (DL) (own weight)} = t * \gamma c = 0.2 * 25 = 5KN/m^2$$

$$\text{Live load (LL)} = 0.5KN/m^2 \text{ (based on EBCS 1.1995)}$$

Therefore, the design load on the slab

$$Wd = 1.3DL + 1.6LL = 1.3 * 5 + 1.6 * 0.5 = 7.3KN/m^2$$

Design of moment

At centre of slab

$$Mr = M\theta = \frac{3}{16} * Wd * x^2, x = r + thicknes\ of\ wall = 9.25 + 0.25 = 9.5$$

$$Mr = M\theta = \frac{3}{16} * 7.3 * 9.5^2 = 123.529KNm$$

At the edge of the slab

$$Mr = 0, M\theta = \frac{Wd * x^2}{8} = \frac{7.3 * 9.5^2}{8} = 82.35KNm$$

Check thickness for flexure

$$\omega b = \frac{0.0028}{(0.0035 + \frac{fyd}{Es}} = \frac{0.0028}{0.0035 + \frac{260.8}{2*10^5}} = 0.5828$$

$$\omega max = 0.75\omega b = 0.75 * 0.5828 = 0.437$$

$$\mu max = \omega max - \frac{\omega max^2}{2} = 0.437 - \frac{0.437^2}{2} = 0.34$$

$$dreq = \sqrt{\frac{Mmax}{fcd * \mu max * b}} = \sqrt{\frac{82.35 * 10^6}{13.4 * 0.34 * 1000}} = 134.44mm$$

$$t_{req} = d_{req} + cover + \frac{16}{2} = 134.44 + 40 + 8 = 182.44mm < tass = 200mm\ ok!$$

$$d = tassum - cover - \frac{16}{2} = 200 - 40 - 8 = 152mm$$

Reinforcement

At centre of the slab

Mr=Mθ=123.529KNm, $\mu = \frac{M\theta}{fcd*b*d^2} = \frac{123.529*10^6}{13.4*1000*152^2} = 0.399$

$$\omega = 1 - \sqrt{(1 - 2\mu)} = 1$$

$$\omega = 1 - \sqrt{(1 - 2 * 0.399} = 0.55$$

Therefore $As = \frac{\omega*fcd*b*d}{fyd} = \frac{0.55*13.4*1000*152}{260.87} = 4294.246mm^2$

$$As = 4294.246mm^2 > Asmin = \frac{0.5 * b * d}{fyk} = \frac{0.5 * 1000 * 152}{300} = 253.33mm^2$$

Spacing using Ø$12mm$ bars $S = \frac{\pi*12^2*1000}{4*4294.246} = 26.34mm < 350mm\ or\ 2t = 400\ ok!$

Number of bar = $\frac{As}{\frac{\pi*d^2}{4}} = \frac{4294.246}{\frac{\pi*12^2}{4}}$=37.969 ≈**38** bars

Therefore, provide Ø12 mm @25 mm c/c in the form of mesh.

At the edge of slab

$$M\theta = 82.35KNm$$

$$\mu = \frac{M\theta}{fcd * b * d^2} = \frac{82.35 * 10^6}{13.4 * 1000 * 152^2} = 0.266$$

$$\omega = 1 - \sqrt{(1 - 2\mu)} = 1 - \sqrt{(1 - 2 * 0.266} = 0.316$$

Therefore $As\theta = \frac{\omega*fcd*b*d}{fyd} = \frac{0.316*13.4*1000*152}{260.87} = 2467.24mm^2$

$$As = 2467.24mm^2 > Asmin = \frac{0.5 * b * d}{fyk} = \frac{0.5 * 1000 * 152}{300} = 253.33mm^2$$

Number of bar = $\frac{As}{\frac{\pi*d^2}{4}} = \frac{2467.24}{\frac{\pi*12^2}{4}}$=21.815≈**22** bars

Spacing using $\emptyset 12mm$ bars $S = \frac{\pi * 12^2 * 1000}{4 * 2467.24} = 45.84mm < 350mm \; or \; 2t = 400 \; ok!$

Therefore, provide $\emptyset 12 \; mm$ bars @ 40 mm c/c in the form of ring at the edge of the slab just above mesh reinforcement.

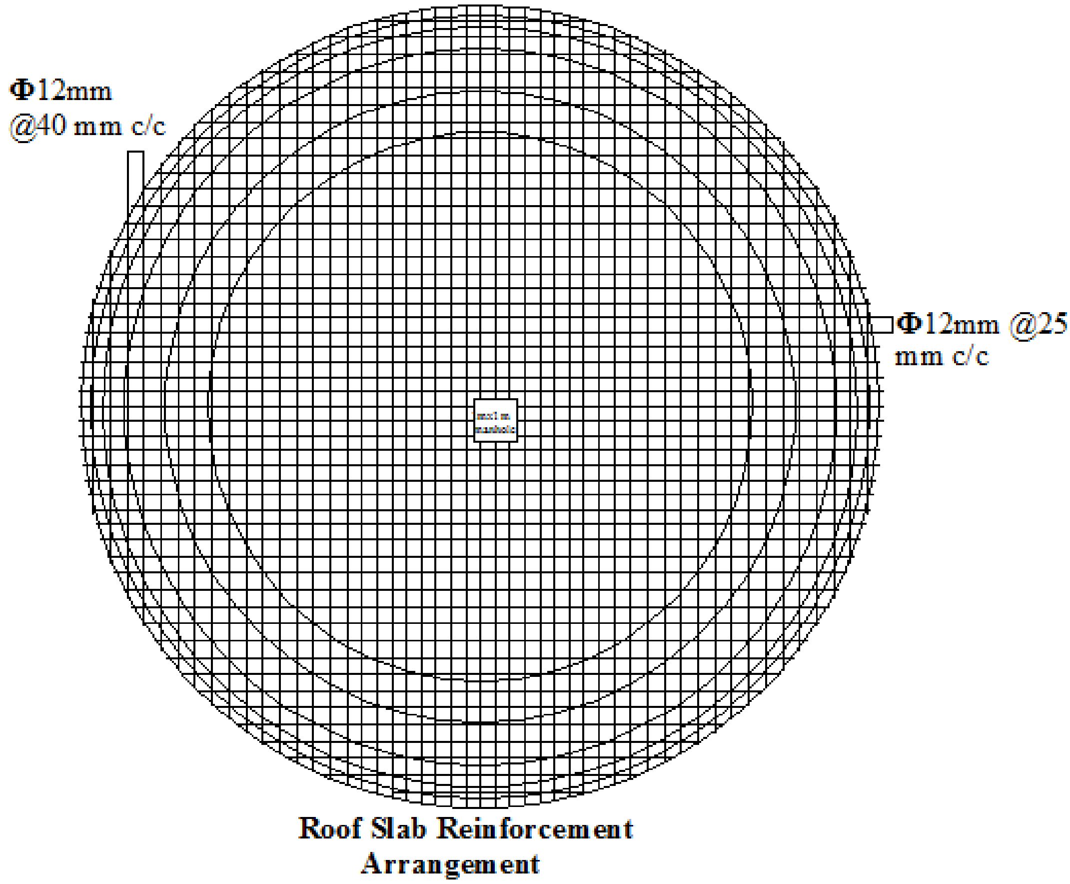

Figure 6: Roof slab reinforcement arrangement

Design of Base Slab

The bottom slab is resting on the ground (soil) and supports the water and the load of the wall and top slab. The weight of water is directly transferring to the soil; therefore, the bearing capacity of the soil should be adequate. The slab is design for tank empty condition.

The net bearing pressure on the soil for the tank empty condition is that due to the load from the roof slab and wall of the tank. That means the net earth pressure is caused by the wall of the tank and the load from the roof slab.

Load from roof slab = 7.3KN/m

Load from tank wall, $p = \frac{2N}{R} = \frac{2 * 1.4 * \gamma_c * t * H}{R} = \frac{2 * 1.4 * 25 * 250 * 4.5}{9.5} = 8.289 KN/m^2$

$\text{Pnet} = 7.3 + 8.289 = 15.589\text{KN/m}^2$.

For circular base with fixed supports around its edge and uniform net pressure, maximum positive radial and circumferential moments reach the same value at the centre and given by:

Mr = radial moment

MΘ = circumferential moment

$$\text{Mr, max} = \text{M}\theta\text{, max} = \frac{1+\mu}{16} * \text{Pnet} * \text{r}^2 = \frac{1+0.2}{16} * 15.589 * 9.5^2 = 105.5\text{KNm}$$

The value of μ for concrete is =0.2

The maximum negative moment occur at the fixed support given by:

$$(-\text{ve})\text{Mr, max} = -\frac{1}{8} * \text{Pnet} * \text{r}^2 = -\frac{1}{8} * 15.589 * 9.5^2 = -175.86\text{KNm}$$

$$(-ve)M\theta, max = -\frac{\mu}{8} * Pnet * r^2 = -\frac{0.2}{8} * 15.589 * 9.5^2 = -35.173\text{KNm}$$

The absolute maximum moment is 175.86 KNm/m.

Effective thickness of base slab, dreq, $dreq = \sqrt{\frac{Mmax}{Rb*b}} = \sqrt{\frac{175.86*10^6}{2.5028*1000}} = 265.076mm$

Using concrete cover of 40 mm and Ø20 mm bar,

$$\text{treq} = \text{dreq} + \text{cover} + \emptyset = 250 + 40 + 20 = 310m$$

Assuming thickness of slab t = 300 mm

$$d = 300 - 40 - \frac{20}{2} = 250\text{mm}$$

Reinforcement

1st At the edge supports

In radial reinforcement

$$(-ve)As = \frac{(-ve)Mr, max}{fs, allow * jb * d} = \frac{175.86 * 10^6}{130 * 0.8136 * 250} = 6655.691mm^2$$

Spacing of $\emptyset 20mm$ bars,

$$S = \frac{as * b}{As} = \frac{\pi * 20^2 * 1000}{4 * 6655.691} = 47.2\text{mm} < Smax = 300mm\ ok!$$

Number of bars $= \frac{As}{\frac{\pi * d^2}{4}} = \frac{6655.691}{\frac{\pi * 20^2}{4}} =$ **21.19 ≈22 bars**

ProvideØ20 mm bars @ 45 mm c/c at the top of slab in radial direction.

In circumferential direction (ring bars)

Ring bars for development length of bar:

Depth above the mesh reinforcement,d $= 250 - 20 = 230$mm

$$M\theta = -35.173\text{KNm/m}$$

$$(-ve)As\theta = \frac{(-ve)M\theta, max}{fs, allow * jb * d} = \frac{35.173 * 10^6}{130 * 0.8136 * 230} = 1446.93mm^2$$

Spacing of $\emptyset 20mm$ bars,

$$S = \frac{as * b}{As} = \frac{\pi * 20^2 * 1000}{4 * 1446.93} = 217.12\text{mm} < Smax = 300mm\ \ ok!$$

Therefore take, S=215 mm

Number of bars $=\frac{As}{\frac{\pi * d^2}{4}} = \frac{1446.93}{\frac{\pi * 20^2}{4}}$=**4.6≈5 bars**

Provide∅20 mm bars @ 215 mm c/c at the top of slab in circumferential direction.

2nd At the Centre of the Slab

$$M\theta, \text{max} = \text{Mr}, \text{max} = 105.5\text{KNm}$$

Using $\emptyset 20mm$ bars, and 40 mm cover, $d = 230mm$

$$(+ve)As = \frac{(+ve)Mmax}{fs, allow * jb * d} = \frac{105.5 * 10^6}{130 * 0.8136 * 230} = 4336.809mm^2$$

Spacing of $\emptyset 20mm$ bars,

$$S = \frac{as * b}{As} = \frac{\pi * 20^2 * 1000}{4 * 4336.809} = 72.44\text{mm} < Smax = 300mm\ ok!$$

Number of bars $=\frac{As}{\frac{\pi * d^2}{4}} = \frac{4336.809}{\frac{\pi * 20^2}{4}}$=**13.8 ≈14 bars**

Provide ∅20 mm14bars @70 mm c/c placed at the bottom directions in form of mesh.

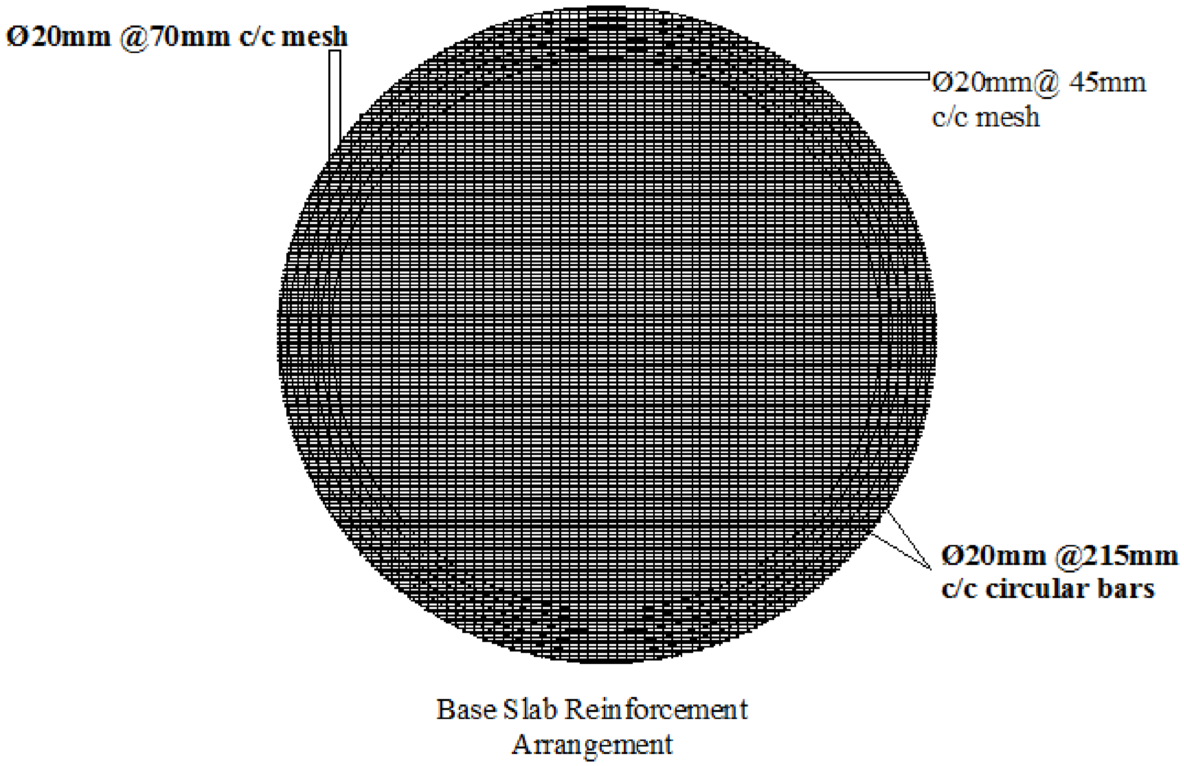

Figure 7: Base slab reinforcement arrangement

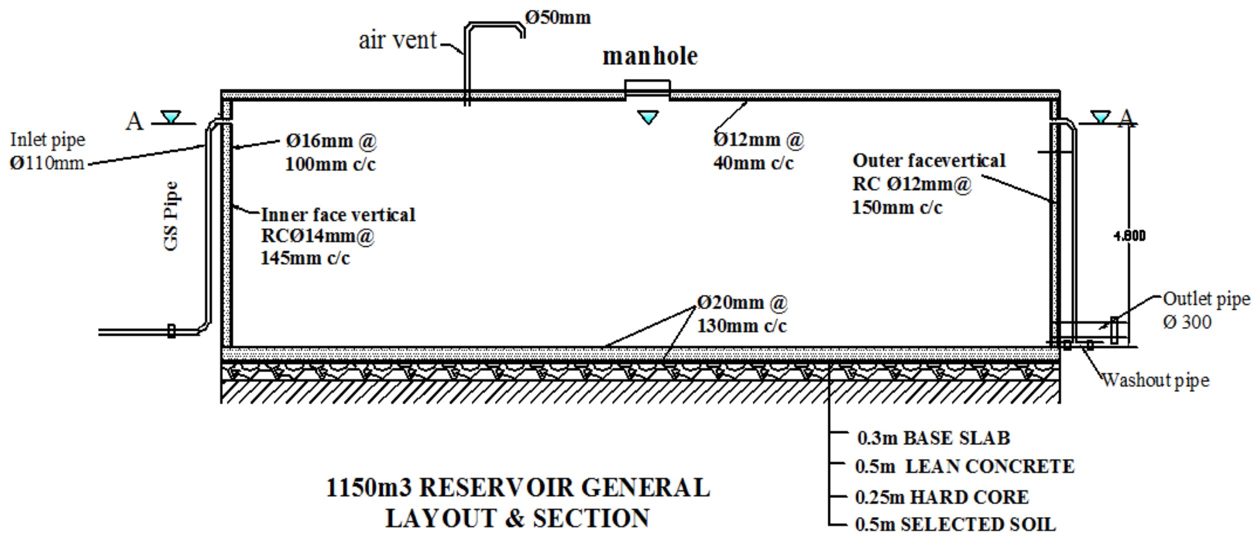

Figure 8: Sectional view of reservoir

Design of circular reservoir Phase-II

- Base condition fixed
- Water depth in the reservoir 3m
- Free board 0.3m
- Grade of concrete-30mpa
- Volume of reservoir=550m^3
- Clear diameter of reservoir (D) = $\sqrt{\frac{4*V}{\pi*h}}$

Since our reservoir capacity is 550m3 we Assumed, H = 4m

$V = \pi\frac{D^2H}{4}$ where, V = Volume of reservoir

D = Diameter of reservoir

H = Height of reservoir

$$550 = \frac{\pi D^2*4}{4}$$

$$D = \sqrt{\frac{4*V}{\pi*h}}$$

D= 13.5

Assume, t=250mm at bottom Where, t =thickness

t = 150 mm at top

$$tavg. = \frac{250+150}{2} = 200$$

$$\frac{h^2}{dtaverage} = \frac{4^2}{13.5*200*10^{-3}} = \underline{\mathbf{5.926}}$$

Table 36: Coefficients for Hoop tension (Fixed at base and free at top)

$\frac{h^2}{dt}$	Coefficient at point									
	0.1h	0.2h	0.3h	0.4h	0.5h	0.6h	0.7h	0.8h	0.9h	1.0h
5	0.025	0.137	0.245	0.346	0.428	0.477	0.469	0.398	0.259	0.092
5.92	0.0185	0.1203	0.2348	0.3441	0.4400	0.5020	**0.5107**	0.4711	0.2979	0.11052
6	0.018	0.119	0.234	0.344	0.441	0.504	0.514	0.477	0.301	0.112

Table 37: Coefficient for vertical moment (For fixed at base and free at top)

$\frac{h^2}{dt}$	Coefficient at point									
	0.1h	0.2h	0.3h	0.4h	0.5h	0.6h	0.7h	0.8h	0.9h	1.0h
5	0.0002	0.0008	0.0016	0.0029	0.0046	0.0059	0.0028	-0.0058	-0.0222	0.0002
5.92	0.00011	0.00034	0.00086	0.0032	0.0046	0.00516	0.0029	-0.0042	-0.0189	0.00011
6	0.0001	0.0003	0.0008	0.0032	0.0046	0.0051	0.0029	-0.0041	-0.0187	0.0001

Source: *IS 3370-4 (1967): Code of practice for concrete structures for the storage of liquids, Part4: Design tables [CED 2: Cement and Concrete]*

For strength

$$f_{c,allow} = 100\ N/mm^2$$

$$f_{c,allow} = 11N/mm^2 \& \ N = 15$$

For no concrete crack

$f_{ct,allow} = 1.44N/mm^2$--------------for direct tension

$f_{ct,allow} = 2.02\ N/mm^2$-------------for bending

Assume area of horizontal reinforcement for unit depth or unit width (BS-5337)

$$A_{s(\min)} = \frac{0.3}{100} * (b * t) = \frac{0.3}{100} * (1000 * 200) = 600mm$$

Detail reinforcement

Horizontal reinforcement

From the above table the maximum coefficient for hoop is equal to 0.51067 at 0.h.

$$\text{Hoop tention (TH)} = \text{max. coef} * \text{yw} * \text{h} * \text{r}$$

$$\text{Hoop tention (TH)} = 0.51067 * 9.81 * 4 * 6.75 = 135.266\text{KN/m}$$

Area of reinforcement required along the hoop for unit strip of wall, As is

$$As = \frac{T_H}{fs, allow} = \frac{135.266 * 10^3 N}{100N/mm^2} = 1352.66mm^2$$

$$\text{AS} = 223.24mm^2 \geq \text{AS min} = 0.3\% \text{ of the concrete area}$$

$$\text{AS min} = \frac{0.3}{100} * 1000 * 200 = 600\text{mm}^2$$

$$\text{AS} = 1352.66mm^2 \geq \text{AS min} = 600\text{mm}^2 \textbf{ok}!$$

Check Thickness of Wall for No Cracks

Thickness of wall is determined using the requirement of resistance to crack; therefore, thickness of wall is determined by limiting tensile stress in the concrete to allowable stress value as, $\mathrm{fct} = \frac{\mathrm{TH}}{(\mathrm{Ac}+(\mathrm{n}-1)*\mathrm{As})} \leq \mathrm{fct, allow} = 1.44\mathrm{N/mm^2}$

$$\mathrm{fct} = \frac{135.266 * 10^3}{(1000 * 200 + (15-1) * 1352.66)} \leq \mathrm{fct, allow} = 1.44\mathrm{N/mm^2}$$

$$\mathrm{fct} = 0.5029\mathrm{N}/mm^2 \leq \mathrm{fct, allow} = 1.44\mathrm{N/mm^2}\,\mathbf{ok}!$$

Therefore, thickness is adequate for no crack!

Spacing of ring bars using Ø16

$$\mathrm{s} = \frac{\mathrm{as} * \mathrm{b}}{\mathrm{As}} \leq 300\mathrm{mm\ or\ thickness\ of\ wall(200mm)}$$

$$\mathrm{s} = \frac{\pi * 16^2 * 1000}{4 * 1352.66} = 148.642\mathrm{mm} \leq 300\mathrm{mm\ or\ thickness\ of\ wall(200mm)}\ ok!$$

Number of bar$=\frac{As}{S}=\frac{1352.66}{145}=$**9.33≈10 bar**

Therefore, provide Ø16 ring bars @ 145mm c/c equally provided at a layer in each face.

Vertical reinforcement

The development of maximum positive and maximum negative flexure is calculating as follows.

$$\mathrm{Mmax} = \mathrm{max.coeff} * \gamma\mathrm{w} * h^3$$

$$+\mathrm{ve\ max.coeff} = 0.00516\mathrm{at}\ 0.6\mathrm{h}$$

$$-\mathrm{ve\ max\ coeff} = -0.01896\mathrm{at}\ 0.9\mathrm{h}$$

Therefore,

$$(+\mathrm{ve})\mathrm{Mmax} = 0.00516 * 9.81 * 4^3 = 3.239KN.m$$

$$(-\mathrm{ve})\mathrm{Mmax} = -0.01896 * 9.81 * 4^3 = -11.904KN.m$$

Taking absolute value of the result, the maximum moment is therefore,

$$\mathrm{Mmax} = -11.904KN.m = 11.904KN.m$$

Check Thickness of Wall for Flexure

fs, allow = 130N/mm^2, fc, allow = 11N/mm^2, n = 15

Design constants of balanced section are,

$$\mathrm{kb} = \frac{\mathrm{n}}{\mathrm{n}+\mathrm{r}}, \mathrm{r} = \frac{\mathrm{fs, allow}}{\mathrm{fc, allow}} = \frac{130}{11} = 11.82$$

$$kb = \frac{15}{15 + 11.82} = 0.559$$

$$jb = 1 - \frac{kb}{3} = 1 - \frac{0.559}{3} = 0.8136$$

$$Rb = \frac{fc, allow}{2} * kb * jb = \frac{11}{2} * 0.559 * 0.8136 = 2.5028 N/mm^2$$

The effective depth of section (dreq) is giving by

$$dreq = \sqrt{\frac{Mmax}{Rb * b}} = \sqrt{\frac{11.904 * 10^6}{2.5028 * 1000}} = 68.96mm$$

Thickness of wall taking $\emptyset\ 16mm\ bars$ and 40 mm cover:

$$treq = dreq + cover + \frac{\emptyset}{2} = 68.96 + 40 + \frac{16}{2} = 116.96 < tasume = 200mm\ ok!$$

$$dreq = tassume - 40 - \frac{16}{2} = 152mm$$

Use t assume=200 mm and dreq=152 mm

Vertical reinforcement on inner face

$$(-ve)As = \frac{Mmax}{fs, allow * jb * dreq} = \frac{11.904 * 10^6}{130 * 0.8136 * 152} = 727.544mm^2$$

$$(-ve)As = 727.544mm^2 > Asmin = 600mm^2 \boldsymbol{ok}!$$

Check thickness of wall for no crack

$$\bar{y} = \frac{\Sigma A * Y}{\Sigma A}$$

$$\bar{y} = \frac{(b * t) * \frac{t}{2} + (n * As) * d}{b * t + (n * As)}$$

$$\bar{y} = \frac{(1000 * 200) * \frac{200}{2} + (15 * 727.54) * 152}{1000 * 200 + (15 * 727.54)} = 103.05mm$$

y =200 – 103 = 97

x = 152 - 103= 49

$$I_{ce} = \frac{b * \bar{y}}{3} + \frac{b * y}{3} + n * A_s * x^2$$

$$I_{ce} = \frac{1000 * 103^3}{3} + \frac{1000 * 97^3}{3} + (15 - 1) * 727.54 * 49^2 = 668.966 * 10^6 mm^4$$

Then check for no crack:

$$f_{ct} = \frac{M_{max} * y}{Ice} < f_{c,allow} = 2.02mm^2$$

$$f_{ct} = \frac{11.904 * 10^6 * 97}{668.966 * 10^6 mm^4}$$

=1.726mm^2< 2.02mm^2………………OK!

Therefore, thickness is adequate for no crack.

Spacing of Φ12 bars

$$S = \frac{as * b}{As} = \frac{113.09 * 1000}{727.54} = 155.44mm < 300mm \; or \; 200mm \; \boldsymbol{ok}!$$

Number of bar =$\frac{As}{S}$=$\frac{727.54}{155}$=4.69≈**5** bars

Therefore provide Φ12 bars @ 155mm c/c vertically in the inner face.

Vertical reinforcements on outer face

$$(+ve)Mmax = 3.239KNm$$

Using t=200 mm and $\emptyset 16mm$ bars $d = 200 - 40 - \frac{16}{2} = 152mm$

$$(+ve)As = \frac{(+ve)Mmax}{fs, allow * jb * d} = \frac{3.239 * 10^6}{130 * 0.8136 * 152} = 201.47mm^2$$

$$(+ve)As = 201.47mm^2 < As\,min = 600mm^2 \boldsymbol{not\ ok}!$$

Therefore take As_{min}=600 mm^2

Spacing of $\boldsymbol{\emptyset 12mm}$ bars

$$s = \frac{(\pi * 12^2 * 1000)}{4 * 600} = 188.5mm < 300mm \; or \; 200mm$$

Number of bars =$\frac{As}{\frac{\pi * d^2}{4}}$=$\frac{600}{\frac{\pi * 12^2}{4}}$=5.3≈6 bars

So, provide $\emptyset 12mm$ 6bars @ 190 mm c/c in the outer face.

Design of roof slab for phase 2

Cover slab of service reservoir is treats as roof slab, which is simply support by the circular wall.

Assume cover slab as freely support at edges and loaded uniformly.

Using strength limit state design for C-30Mpa,

$$fcd = \frac{2}{3}\left(\frac{fck}{Yc}\right) = \frac{2}{3}\left(\frac{30}{1.5}\right) = 13.4N/mm^2$$

And for S =300Mpa, $f_{yd} = \left(\frac{fyk}{Ys}\right) = \frac{300}{1.15} = 260.8N/mm^2$

Considering the tope thickness of slab as 200 mm and load on slab per m^2

$$\text{Dead load (DL) (own weight)} = t * \gamma c = 0.2 * 25 = 5\text{KN}/m^2$$

$$\text{Live load (LL)} = 0.5\text{KN}/m^2 \text{ (based on EBCS 1.1995)}$$

Therefore, the design load on the slab

$$\text{Wd} = 1.3\text{DL} + 1.6\text{LL} = 1.3 * 5 + 1.6 * 0.5 = 7.3\text{KN}/m^2$$

Design Moment

At center of slab

$$Mr = M\theta = \frac{3}{16} * Wd * x^2, x = r + thicknes\ of\ wall = 6.75 + 0.2 = 6.95$$

$$Mr = M\theta = \frac{3}{16} * 7.3 * 6.95^2 = 66.114KNm$$

At the edge of the slab

$$\text{Mr} = 0, \text{M}\theta = \frac{\text{Wd} * \text{x}^2}{8} = \frac{7.3 * 6.95^2}{8} = 44.076\text{KNm}$$

Check thickness for flexure

$$\omega b = \frac{0.0028}{(0.0035 + \frac{fyd}{Es}} = \frac{0.0028}{0.0035 + \frac{260.87}{2*10^5}} = 0.5828$$

$$\omega max = 0.75\omega b = 0.75 * 0.5828 = 0.437$$

$$\mu max = \omega max - \frac{\omega max^2}{2} = 0.437 - \frac{0.437^2}{2} = 0.34$$

$$dreq = \sqrt{\frac{Mmax}{fcd * \mu max * b}} = \sqrt{\frac{66.114 * 10^6}{13.4 * 0.34 * 1000}} = 120.463mm$$

$$t_{req} = d_{req} + cover + \frac{16}{2} = 120.463 + 40 + 8 = 168.463mm < tass = 200mm\ \boldsymbol{ok}!$$

$$d = tassum - cover - \frac{16}{2} = 200 - 40 - 8 = 152mm$$

Reinforcement

At Centre of the slab

Mr=Mθ=66.114Nm, $\mu = \frac{\text{M}\theta}{\text{fcd}*\text{b}*\text{d}^2} = \frac{66.114*10^6}{13.4*1000*152^2} = 0.21355$

$$\omega = 1 - \sqrt{(1 - 2\mu)}$$

$$\omega = 1 - \sqrt{(1 - 2 * 0.21355} = 0.2431$$

Therefore $As = \frac{\omega * fcd * b * d}{fyd} = \frac{0.2431*13.4*1000*152}{260.87} = 1898.05mm^2$

$$As = 1898.05mm^2 > Asmin = \frac{0.5 * b * d}{fyk} = \frac{0.5 * 1000 * 152}{300} = 253.33mm^2$$

Spacing using $\emptyset 12mm$ bars

$$S = \frac{\pi * 12^2 * 1000}{4 * 1898.05mm^2} = 59.586mm < 350mm\ or\ 2t = 400\ ok!$$

Number of bars $=\frac{As}{S}=\frac{1898.05}{55}$=34.51≈35 bars

Therefore, provide $\emptyset 12\ mm$ bars @ 55 mm c/c in the form of mesh.

At the edge of slab

$$M\theta = 44.076KNm$$

$$\mu = \frac{M\theta}{fcd * b * d^2} = \frac{44.076 * 10^6}{13.4 * 1000 * 152^2} = 0.1424$$

$$\omega = 1 - \sqrt{(1 - 2\mu)} = 1 - \sqrt{(1 - 2 * 0.1424} = 0.1543$$

Therefore $As\theta = \frac{\omega * fcd * b * d}{fyd} = \frac{0.1543 * 13.4 * 1000 * 152}{260.87} = 1204.73mm^2$

$$As = 1204.73mm^2 > Asmin = \frac{0.5 * b * d}{fyk} = \frac{0.5 * 1000 * 152}{300} = 253.33mm^2$$

Spacing using $\emptyset 12mm$ bars

$$S = \frac{\pi * 12^2 * 1000}{4 * 1204.73} = 93.877mm < 350mm\ or\ 2t = 400\ \boldsymbol{ok}!$$

Number of bar= $\frac{As}{S}=\frac{1204.73}{90}$=13.338≈**14** bars

Therefore, provide $\emptyset 12\ mm$ bars @ 90mm c/c in the form of ring at the edge of the slab just above mesh reinforcement.

Design of base slab for Phase-II

The bottom slab is resting on the ground (soil) and supports the water and the load of the wall and top slab. The weight of water is directly transferring to the soil; therefore, the bearing capacity of the soil should be adequate. The slab is design for tank empty condition.

The net bearing pressure on the soil for the tank empty condition is that due to the load from the roof slab and wall of the tank. That means the net earth pressure is caused by the wall of the tank and the load from the roof slab.

Load from roof slab = 7.3KN/m

Load from tank wall, $p = \frac{2N}{R} = \frac{2 * 1.4 * \gamma_c * t * H}{R} = \frac{2 * 1.4 * 25 * 200 * 4}{6.75} = 8.296KN/m^2$

$Pnet = 7.3 + 8.296 = 15.596 KN/m^2$.

For circular base with fixed supports around its edge and uniform net pressure, maximum positive radial and circumferential moments reach the same value at the center and given by:

Mr = radial moment

MΘ = circumferential moment

$$Mr,max = M\theta,max = \frac{1+\mu}{16} * Pnet * r^2 = \frac{1+0.2}{16} * 15.596 * 6.75^2 = 45.3KNm$$

The value of μ for concrete is =0.2

The maximum negative moment occur at the fixed support given by:

$$(-ve)Mr,max = -\frac{1}{8} * Pnet * r^2 = -\frac{1}{8} * 15.596 * 6.75^2 = -88.824KNm$$

$$(-ve)M\theta,max = -\frac{\mu}{8} * Pnet * r^2 = -\frac{0.2}{8} * 15.596 * 6.75^2 = -17.765KNm$$

The absolute maximum moment is 88.824KNm/m.

Effective thickness of base slab, dreq, $dreq = \sqrt{\frac{Mmax}{Rb*b}} = \sqrt{\frac{88.824*10^6}{2.5028*1000}} = 188.387mm$

Using concrete cover of 40 mm and Ø18 mm bar,

$$\text{treq} = \text{dreq} + \text{cover} + \emptyset = 200 + 40 + 18 = 258m$$

Assuming thickness of slab t = 300 mm

$$d = 300 - 40 - \frac{18}{2} = 251\text{mm}$$

Reinforcement

At the edge supports.

In radial reinforcement

$$(-ve)As = \frac{(-ve)Mr,max}{fs,allow * jb * d} = \frac{88.824 * 10^6}{130 * 0.8136 * 251} = 5561.59mm^2$$

Spacing of Ø18mm bars,

$$S = \frac{as * b}{As} = \frac{\pi * 18^2 * 1000}{4 * 5561.59} = 45.755\text{mm} < Smax = 300mm\ \boldsymbol{ok}!$$

Number of bars =$\frac{As}{\frac{\pi*d^2}{4}}$=$\frac{5561.59}{\frac{\pi*18^2}{4}}$=21.86≈22 bars

Provide Ø18 mm bars @ 45 mm c/c at the top of slab in radial direction.

In circumferential direction (ring bars)

Ring bars for development length of bar:

Depth above the mesh reinforcement,d = 251 – 18 = 233mm

$$M\theta == -17.765KNm$$

$$(-ve)As\theta = \frac{(-ve)M\theta,max}{fs,allow * jb * d} = \frac{17.765 * 10^6}{130 * 0.8136 * 233} = 720.867mm^2$$

Spacing of $\emptyset 18mm$ bars,

$$S = \frac{as * b}{As} = \frac{\pi * 18^2 * 1000}{4 * 720.867} = 353\text{mm} > Smax = 300mm\ not\ ok!$$

Number of bars $=\frac{As}{\frac{\pi * d^2}{4}} = \frac{720.867}{\frac{\pi * 18^2}{4}}$ **=2.8≈3 bars**

Therefore take, S=300 mm

Provide∅18 mm bars @ 300 mm c/c at the top of slab in circumferential direction.

At The Centre of the Slab

$$M\theta, \max = Mr, \max = 45.3KNm$$

Using $\emptyset 18mm$ bars, and 40 mm cover, $d = 233mm$

$$(+ve)As = \frac{(+ve)Mmax}{fs,allow * jb * d} = \frac{45.3 * 10^6}{130 * 0.8136 * 233} = 1838.18mm^2$$

Spacing of $\emptyset 18mm$ bars,

$$S = \frac{as * b}{As} = \frac{\pi * 18^2 * 1000}{4 * 1838.18} = 138.435\text{mm} < Smax = 300mm\ \boldsymbol{ok}!$$

Number of bars $=\frac{As}{s} = \frac{1838.18}{135}$ **=13.616≈14 bars**

Provide∅18 mm bars @ 135mm c/c placed at the bottom directions in form of mesh.

8 COLLECTION CHAMBER AND PUMPS

8.1 Collection Chamber

Water from boreholes should be pumped and stored in temporary reservoir. Assuming for submersible pumping is working 20hour per day, hence the reservoir is needed to compensate the ideas hours of pump from 3 boreholes having design yield of 8l/s for each borehole in stage-I and 4 boreholes having design yield of 8l/s for each borehole in stage-II.

The collection works mainly consists of pumping works which are essential to convey water. The collection works are done to collect the raw water from the sources. The collection system mainly depends on:

- ❖ The topographical and hydrological features of the area.
- ❖ The location.
- ❖ The area to be served.

8.1.1 Determination of collection chamber capacity

The total yield of 24l/s and 32l/s is expected from the well field from five 5 boreholes and nine (9) boreholes for stage-I and stage-II respectively.

Water is assumed to be detained for some time in the collection chamber and the detention time in the collection chamber affects directly the size of reservoir i.e. the larger the detention time the larger the reservoir has. Thus, it is considered that the water will be detained for 10-30 minute (Urban water supply design criteria, 2006)

For phase-I

Taking discharge of 24l/s and detention time of 20minute,

Capacity of collection chamber = (24l/s*20*60) / 1000 = 28.8m^3

Adopt 30m3 reservoirs (collection chamber) to ensure good detention time and capacity during pump failure or any other fall. Also the detention time for this reservoir is expected to be around 20 minute.

For Phase-I

Taking discharge of 32l/s and detention time of 30 minute,

Capacity of collection chamber = (32l/s*30*60) / 1000 = 38.4m^3

For safety adopt 40m^3 collection chamber at detention time of 30 minute.

NB. Provide different capacities of collection chamber having a total capacity of 40m^3 and30m^3 for phase-I and phase-II respectively, according to the location of the service reservoirs.

8.1.2 Position of Collection system

The position of collection chamber, pumping station and service reservoir is very important for our design system to distribute water economically to the consumers. These positions are determined by observing the top map of the place and other geographical arrangement. But for our case the two positions are taken from feasibility study due to lake of important materials like the top map of the town. And their positions are such a way that the distribution system has to be supplied by gravity system. Each borehole will be equipped with a pump that pumps the raw water to the service reservoir located at 1002498 East and 443479 North at an elevation of 2441 masl. An intermediate collecting tank on a convenient high ground site is provided in the well field area of distribution reservoir. The collection tank and pumping station are situated at 1000767.94East and 444771.24 North and under constant elevation of 2359 mamsl.

8.2 Pumps

The primary function of a pump is to add hydraulic energy to certain values of fluid. This is accomplished which the mechanical energy imparted to the pump from a power source is transferred to the fluid, there by becoming hydraulic energy. Thus, a pump serves to transfer energy from a power source to a fluid, thereby creating flow or simply creating greater pressures on the fluid.

8.2.1 Purpose of Pump

Pumps are used in water works for the following reasons

- To lift raw water from a surface source of supply
- To lift raw water from wells (underground source of supply)
- To deliver treated water to consumer’s taper at desired pressure
- To fill elevated storage tanks (distribution reservoirs)
- To supply fire – pressure for fire hydrants
- To back wash filters
- To pump chemical solutions

8.2.2 Selection of a Pump

For proper selection of pump, it is necessary, to have certain essential data on the pump installation. The information should include:-

- Number of pump units required
- Nature of liquid to be pumped
- Capacity of pump
- Suction conditions
- Discharge conditions
- Total head
- Position of pump (i.e. horizontal or vertical)

The following points may be kept in mind while selecting any pumping machinery for water works.

- **Reliability of service:** It should be variable and should not fail suddenly and cause trouble.
- **Capacity:** It should be capable of pumping required quantity of water
- **Cost:** It should be cheap in initial cost
- **Power:** The power which is used for running pumps should be available easily at low cost.
- **Maintenance:** The maintenance cost of running pumps should be as less as possible.
- **Efficiency:** Pump should have higher efficiency
- **Deprecation:** pump should have long life and depreciation cost should be less.

8.2.3 Centrifugal Pumps

Centrifugal pumps are the most important types of variable displacement pump because of their wide use. It is capable of delivering large quantities of water, against high as well as low head condition, with good efficiency, combining those features with its other attributes such as simplicity, completeness, and adaptability to different methods of driving initiated us to select this type of pump.

The two most commonly used types of centrifugal pumps are:-

- Vertical spindle pump
- Submersible pump

I. Vertical spindle pump: It is frequently used for pumping water form a well. The driving motor is at the surface and the pump is immersed in water and it must, therefore, driven by a vertical spindle. This spindle rotates with in tube or sleeve which is held centrally in the raising main by spindle bearing the pumped water is delivered to surface via the annular space between the sleeve and the raising main.

II. Submersible pump: It is the modification on the deep well pump as the name indicates in this electric motor and pump both are submersed in the water. By submersing electric motor large economy can be made by avoiding long shaft, large number of bearing and large size rising main etc. Generally here in Holeta case both submersible and surface pump for source site are essential.

8.2.4 Determination of Pipe Size

Size of the pipe is determined by considering the discharge through the pipe and permissible velocity of the flow in the pipe.

- **Diameter of suction pipe for phase-I**

Velocity in suction pipe, permissible velocity of 0.6–1.5 m/s assumed v = 1.5 m/s for our project.

Q_{design} = 8 l/sec for single tube well

$$Q = AV,\ A = \frac{\pi D^2}{4}$$

$$D = \sqrt{\frac{4*Q}{\Pi * V}} = \sqrt{\frac{4*8*10^{-3}}{\Pi * 1.5}} = 0.082m = 90mm$$ But pipe size take, D = 90mm

Check velocity

$$Q = AV \Rightarrow 0.008m^3/sec = \frac{\pi * D^2 * V}{4}$$

$$V = \frac{8*10^{-3}}{\frac{\Pi * (0.09)^2}{4}} = 1.26m/s$$

- **Economical rising mains diameter**

For pumping a particular fixed discharge of water, it can be pumped through bigger diameter pipe at low velocity or through lesser diameter pipe at very high velocity. But, if the diameter of the pipe is increased, it will lead to the higher cost of the pie line. On the other hand if the diameter of the pipe is reduced, the increased velocity will lead to higher frictional head loss and will require more horse power for the required pumping, thereby increasing the cost of pumping. For obtaining the optimum conditions , it is at most necessary to design the diameter of the pumping main, which will be overall most economical in initial cost as well as maintenance cost for pumping the required quantity of water .

The following formula given by Lea is commonly used in determining the diameter of the pumping mains.

$$D = 0.97 \text{ to } 1.22\sqrt{Q}$$

Where D = Economical diameter of pipe in meters

Q = safe discharge of water to be pumped in m^3/sec.

This formula gives optimum velocity of water flow between 0.8 to 1.35 m/sec.

- **Calculation of economical diameter**

Based on the above economical size of pipe, from each bore hole to collection chamber is calculated as follows

For safety take the larger coefficient i.e. 1.22.

$$D = 1.22\sqrt{Q}$$

$$= 1.22\sqrt{8*10^{-3}} \quad =0.109\text{m}$$

Take D = 110mm

Check for velocity:-

$V = \dfrac{Q}{A}$ Where Q = required discharge of water to be pumped in m^3/s

A = area of pumping in m^2

$$V= \frac{8*10^{-3}}{\frac{\Pi*(0.11)^2}{4}} = 0.84m/s$$

Therefore, the velocity is OK since it is within the allowable limit i.e. 0.8 to 1.35m/s

For phase-I

In phase-1 there are a total number of 3 boreholes which feeds to reservoirs having capacity of $1150m^3$. Sample of calculation for the 3 boreholes which feeds to reservoir capacity of $1150m^3$

8.2.5 **Determination of Total Dynamic Head**

For the phase-I (2016-2031)

From borehole to collection chamber

Q =AV, A= $\frac{\pi D^2}{4}$ Q_{design} = 8 l/sec for single borehole

D= $\sqrt{\frac{4*Q}{\Pi*V}} = \sqrt{\frac{4*8*10^{-3}}{\Pi*1.5}} = 0.082m = 90mm$ But pipe size take, D = 90mm

Check velocity

$$Q = AV \Rightarrow 0.008m^3/sec = \frac{\pi * D^2 * V}{4}$$

$$V = \frac{8*10^{-3}}{\frac{\Pi*(0.09)^2}{4}} = 1.26\ m/s$$

- ❖ The total head against which the pump must operate including the total static head and total head losses.

TDH is represented by the following equation

$$TDH = H_{st} + H_{ls} + H_{ld}$$

Where H_{st} =total static head (i.e. elevation difference between the pumping source & reservoir)

H_{ls}= head loss in suction pipe

H_{ld} = head loss in raising main (i.e. friction, bend, valve & velocity head).

Sample calculation for one bore hole

Head loss from bore hole to collection chamber for BH-1

Total head loss due to friction loss

$$H_l = \frac{flv^2}{2gd} \ldots\ldots\ldots\ldots\ldots\ldots\ldots. \text{Darcy} - \text{Weisbach head loss equation}$$

Where h_f= head loss due to equation.

f= coefficient of friction.

l= length of the pipe in m.

v= average velocity of flow in m/s.

d = internal diameter of the pipe in m.

$H_l = \frac{flv^2}{2gd} = \frac{0.02*81.5*1.26^2}{2*9.81*0.08}$ = 1.65m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes =$\frac{kv^2}{2g} = \frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{1.26^2}{2*9.81}$ =0.0404m

THL= 1.65m +0.0809m +0.0404 +0.0404m

=1.812

H_{st}=Elevation difference between source and junction= 2359m – 2354m =5m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +10m =15m

Total head for source to junction =5m + 15m + 1.812m = 21.812m

Head loss from bore hole to collection chamber for BH-2

Total head loss due to friction loss =$H_l = \frac{flv^2}{2gd} = \frac{0.02*86.5*1.26^2}{2*9.81*0.08}$ = 1.75m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes =$\frac{kv^2}{2g} = \frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{1.26^2}{2*9.81}$ =0.0404

THL= 1.75m +0.0809m +0.0404 +0.0404m

=1.912m

H_{st}=Elevation difference between source and junction= 2359m – 2354m =5m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +10m =15m

Total head for source to junction =5m + 15m + 1.912m = 21.912

Head loss from bore hole to collection chamber for BH-3

Total head loss due to friction loss =$H_l = \frac{flv^2}{2gd} = \frac{0.02*84.7*1.26^2}{2*9.81*0.08}$ = 1.71m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes $=\frac{kv^2}{2g}=\frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e=\frac{1}{2}*\frac{1.26^2}{2*9.81}$ =0.0404

THL= 1.71m +0.0809m +0.0404 +0.0404m

=1.875m

H_{st}=Elevation difference between source and junction= 2359m – 2354m =5m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +10m =15m

Total head for source to junction =5m + 15m + 1.875m = 21.875 m

From collection chamber to service reservoir

$$D=\sqrt{\frac{4*Q}{\Pi*V}}=\sqrt{\frac{4*8*10^{-3}*2}{\Pi*1.5}}=0.116m=116mm$$

But pipe size available in market 110mm. So take, D = 110mm

Check velocity

$$Q = AV \Rightarrow 0.008m^3/sec = \frac{\pi*D^2*V}{4}$$

$$V=\frac{4*Q}{\pi*D^2}=\frac{4*2*8*10^{-3}}{\pi*0.15^2}=0.906m/sec$$

which lies b/n the range of permissible velocity ok!

Total head loss due to friction loss $=H_l=\frac{flv^2}{2gd}=\frac{0.02*3123.5*0.906^2}{2*9.81*0.11}$ = 23.75m, assume, f=0.02

Head loss due to velocity in pipe = $H_v=\frac{v^2}{2g}=\frac{0.906^2}{2*9.81}$ = 0.0418m

Head loss due to valve and bends of pipes $=\frac{kv^2}{2g}=\frac{0.5*0.906^2}{2*9.81}$ = 0.0209m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e=\frac{1}{2}*\frac{0.906^2}{2*9.81}$ =0.0209m

THL = 23.75m + 0.0418m + 0.0209m + 0.0209m

= 23.834m

H_{st}=Elevation difference between junction and reservoir = 2441m – 2359m =82m

TDH= THL +H_{st} = 23.834m + 82m = 105.834m

For the phase-II (2031-2041)

Head loss from bore hole to collection chamber for BH-1

Total head loss due to friction loss $=H_l = \frac{flv^2}{2gd} = \frac{0.02*286*1.26^2}{2*9.81*0.08}$ = 5.78m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes $=\frac{kv^2}{2g} = \frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{1.26^2}{2*9.81}$ =0.0404

THL=5.78m +0.0809m +0.0404 +0.0404m

=5.9417m

H_{st}=Elevation difference between source and junction= 2372m – 2367m =5m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +75m =80m

Total head for source to junction =5m + 80m + 5.9417m = 90.9417 m

Head loss from bore hole to collection chamber for BH-2

Total head loss due to friction loss $=H_l = \frac{flv^2}{2gd} = \frac{0.02*303*1.26^2}{2*9.81*0.08}$ = 6.129m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes $=\frac{kv^2}{2g} = \frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{1.26^2}{2*9.81}$ =0.0404

THL= 6.129m +0.0809m +0.0404 +0.0404m

=6.29m

H_{st}=Elevation difference between source and junction = 2372m – 2368m =4m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +75m =80m

Total head for source to junction =4m + 80m + 6.29m = 90.29 m

Head loss from bore hole to collection chamber for BH-3

Total head loss due to friction loss =$H_l = \frac{flv^2}{2gd} = \frac{0.02*306*1.26^2}{2*9.81*0.08}$ = 6.19m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes =$\frac{kv^2}{2g} = \frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{1.26^2}{2*9.81}$ =0.0404

THL= 6.19m +0.0809m +0.0404 +0.0404m

=6.35m

H_{st}=Elevation difference between source and junction = 2372m – 2365m =7m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +75m =80m

Total head for source to junction =7m + 80m + 6.35m = 91.35 m

Head loss from bore hole to collection chamber for BH-4

Total head loss due to friction loss =$H_l = \frac{flv^2}{2gd} = \frac{0.02*303*1.26^2}{2*9.81*0.08}$ = 6.129m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{1.26^2}{2*9.81}$ = 0.0809m

Head loss due to valve and bends of pipes =$\frac{kv^2}{2g} = \frac{0.5*1.26^2}{2*9.81}$ = 0.0404m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{1.26^2}{2*9.81}$ =0.0404

THL= 6.129m +0.0809m +0.0404 +0.0404m

=6.29m

H_{st}=Elevation difference between source and junction = 2372m – 2362m =10m

Assume, Head loss in suction pipe

= the depth of pump submerged + the depth from water level to the borehole top

= 5m +75m =80m

Total head for source to junction =10m + 80m + 6.29m = 96.29 m

From collection chamber to service reservoir

D=$\sqrt{\frac{4*Q}{\Pi*V}}=\sqrt{\frac{4*8*10^{-3}*2}{\Pi*1.5}}=0.116m=116mm$

But pipe size available in market 110mm. So take, D = 110mm

Check velocity

$Q = AV \Rightarrow 0.008m^3/sec = \frac{\pi * D^2 * V}{4}$

$V= \frac{4*Q}{\pi*D^2} = \frac{4*2*8*10^{-3}}{\pi*0.15^2}$ =0.906m/sec

which lies b/n the range of permissible velocity ok!

Total head loss due to friction loss =$H_l = \frac{flv^2}{2gd} = \frac{0.02*708*0.906^2}{2*9.81*0.11}$ = 5.38m, assume, f=0.02

Head loss due to velocity in pipe = $H_v = \frac{v^2}{2g} = \frac{0.906^2}{2*9.81}$ = 0.0418m

Head loss due to valve and bends of pipes =$\frac{kv^2}{2g} = \frac{0.5*0.906^2}{2*9.81}$ = 0.0209m where, k=0.5 for circular pipe.

Head loss due to entry = $h_e = \frac{1}{2} * \frac{0.906^2}{2*9.81}$ =0.0209m

THL = 5.38m + 0.0418m + 0.0209m + 0.0209m

= 5.464m

H_{st}=Elevation difference between junction and reservoir = 2441m – 2372m =69m

TDH= THL +H_{st} = 5.464m + 69m = 74.464m

Pump Power Requirement

This will depend on the head against which the pumping is to be done. Also it will depend on the energy losses due to friction, flow through valves, and fittings, rate of pumping etc. if the total head against pumping is to be done is H (including all losses) the rate of flow is Q and unit weight of water is w, and the efficiency of pumps and driving motors can be taken as 80 % and 90% respectively. Then water horse power required:

For phase-I

W.H.P-1= $\frac{Q*w*H}{75} = \frac{0.008*1000*21.812}{75}$ =2.3266 HP

And brake horse power will be:

B.H.P = $\frac{WHP}{efficiency} = \frac{2.3266}{0.9*0.8}$= 3.2314 HP

Power required = 3.2314 * 0.735KW/HP = 2.375KW.

W.H.P-2= $\frac{Q*w*H}{75} = \frac{0.008*1000*21.912}{75}$ =2.3373 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{2.3373}{0.9*0.8}$= 3.2462 HP

Power required = 3.2462 * 0.735KW/HP = 2.3856KW.

W.H.P-3= $\frac{Q*w*H}{75} = \frac{0.008*1000*21.875}{75} =$ 2.3333 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{2.33333}{0.9*0.8}$= 3.241 HP

Power required = 3.241HP * 0.735KW/HP = 2.382KW

From collection chamber to reservoir

W.H.P= $\frac{Q*w*H}{75} = \frac{0.008*1000*105.834}{75} =$ 11.289 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{11.289}{0.9*0.8}$= 15.67 HP

Power required = 15.67 HP * 0.735KW/HP = 11.24KW

The Power source for both stations (well & booster) is from the national grid of EEPCO through pole mounted transformers.

For Phase II

W.H.P-1= $\frac{Q*w*H}{75} = \frac{0.008*1000*90.9417}{75}$ =9.7 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{9.7}{0.9*0.8}$= 13.473 HP

Power required = 13.473 * 0.735KW/HP = 9.9025KW.

W.H.P-2= $\frac{Q*w*H}{75} = \frac{0.008*1000*90.29}{75}$ =9.63HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{9.63}{0.9*0.8}$= 13.375 HP

Power required 13.375 * 0.735KW/HP = 9.83KW.

W.H.P-3= $\frac{Q*w*H}{75} = \frac{0.008*1000*91.35}{75} =$ 9.744 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{9.744}{0.9*0.8}$= 13.53 HP

Power required = 13.53HP * 0.735KW/HP = 9.947KW

W.H.P-3= $\frac{Q*w*H}{75} = \frac{0.008*1000*96.29}{75} = $ 10.27 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{9.744}{0.9*0.8}$= 14.265 HP

Power required = 14.265 HP * 0.735KW/HP =10.485KW

From collection chamber to reservoir

W.H.P= $\frac{Q*w*H}{75} = \frac{0.008*1000*74.464}{75} = $ 7.94 HP

And brake horse power will be:

B.H.P = $\frac{\text{WHP}}{\text{efficiency}} = \frac{7.94}{0.9*0.8}$= 11.03 HP

Power required = 11.03 HP * 0.735KW/HP = 8.11KW

The Power source for both stations (well & booster) is from the national grid of EEPCO through pole mounted transformers.

9 DISTRIBUTION SYSTEM

The water distribution system is the part of the water work which receives the water from the pumping station or the conduit in several different ways, as local conditions or other considerations may dictate.

After the water is treated completely, it becomes necessary to distribute it to a number of houses, estates, industries, and public places by means of a network of distribution system. The distribution system consists of pipes of various sizes, valves, meters, pumps, distribution reservoirs, hydrants; stand posts etc. The pipelines carry the water to every street and road. Valves control the flow of water through the pipes. Pumps are provided to pump the water to the elevated service reservoirs or directly in the water mains to obtain the required pressure in the pipelines. Meters are provided to measure the quantity of water consumed by individual as well as by the town.

9.1 Classification of distribution system

Depending on the method of distribution there are three types of distribution system

1. **Gravity system**: - This is the most reliable method of distributing water when there is some ground level sufficiently above the city. The adequate pressure of distribution is maintained by gravity force only.

2. **Pumping system: -** In this system the distribution pressure is maintained by direct pumping to the mains. This system has an increased maintenance cost for the pumps have to work at varying rates, their life is also reduced.

3. **Dual system.** In this system the pump is connected to the mains as well as to an elevated reservoir. The water to be supplied is pumped and stored in an elevated distribution reservoir from which it is supplied to the consumer under gravity. The surplus water is stored in the storage reservoir and may be supplied during maximum demand and emergency period. Therefore, this system is efficient and economical because the pumps are operated at constant speed which increases the efficiency and reduces wear and tear of the pumps.

Where some ground sufficiently high above the city area is available, gravity system can be used for distribution system. Depending upon these for Holeta town we adopt gravity system.

Requirement of good distribution system

Some of the requirements are:-

- ❖ It should convey the treated water up to consumers with the same degree of purity
- ❖ The system should be economical and easy to maintain and operate
- ❖ The diameter of pipes should be designed to meet demands
- ❖ It should safe against any future pollution.
- ❖ Water should be supplied without interruption even when repairs are undertaken

9.2 Layout of distribution system

Generally in practice there are four different systems of distribution, which are used depending up on their lay out and direction of supply.

1. Dead End or Tree System
2. Circular or ring system
3. Radial system
4. Grid iron system

From the above systems Grid iron 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 will be 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.

The main disadvantages of this system are increased in the overall cost, difficult design calculation & increase in the number of valves to be closed for repair work.

Based on the above criteria we use grid iron system for our project distribution system even though it may have relative disadvantages.

9.2.1 Selection of pipe material

Pipes are made from different materials like cast iron, wrought iron, asbestos, steel, plastic etc. so that selection of pipe material is based on the following methods.

- strength, durability & life of pipe
- carrying capacity of the pipe
- Type of water to be converged & its possible corrosive effect on the pipe
- Ease of transportation, handling & installation
- Tightness of joints & ease to tap for making connections
- Maintenance cost, repair etc.

Among different types of pipes, existing on the market PVC pipes is selected for the distribution system. This is because PVC pipes are light in weight, cheap, easy to join and install durable, good electric insulators and free from corrosion

9.2.2 Pipe Appurtenance

Appurtenances are very necessary materials required in the distribution network for good operation. Distribution system, pipes of various diameters, having many connections and branches are used.

i. **Valves and Fittings**: - In the water works valves are used to control the flow of water, to regulate pressures, to admit air, to prevent flow of water in opposite direction
ii. **Water meter**: used to measure volume of water. It also helps for billing purpose.
iii. **Valves:** are installed at necessary points so as to control the flow of water, for pressure regulations.
iv. **Fire hydrant**: This is installed on the main line and distribution system so as to prevent fire break at any instant. They are provided at places where fire break is expected.
v. **Valve chamber**: are provided for easy access and for safety of valves.

vi. **Flash-out valves**: Mostly located at low points in the pipe line for washing of the system.

vii. **Pressure-relief valve**: A safety valve designed to relief pressure in a pipe line.

viii. **Air release value**: A valve to release air or gas which tends to accumulate at high point on pipe lines.

9.2.3 Laying of Pipes

Since it passes mostly through farmer's private lands and main road side, it should be below ground. A trench about 60*80 cm size should be excavated along the alignment.

9.2.4 Flow Metering

Propeller or turbine types of flow meters calibrated in metric units will be provided at the following locations:

- ❖ Outlet from the boreholes;
- ❖ Outlet from the service reservoirs

The flow meter from outlet of boreholes indicates the volume of raw water produced, the flow meter from outlet of booster pumping station indicates volume water pumped to the storage and the flow meter from reservoir indicate the daily variation in water consumption including losses in the distribution system.

9.3 Analysis of water distribution network

We use the following basic elements for the analysis of the distribution system. These are,

- ❖ Elevation of the junctions using contour maps
- ❖ Length of pipes from one junction to another junction and from reservoir to the first node
- ❖ Reservoir elevation
- ❖ Peak hourly demand of the town

Then, having the above elements as input we analyse,

- ❖ Pressure of the distribution system
- ❖ Velocity of the flow
- ❖ Flow fluctuation within 24hours, etc...

And this analysis was done using the computer software of EPANET 2.0. The network is shown as follows:-

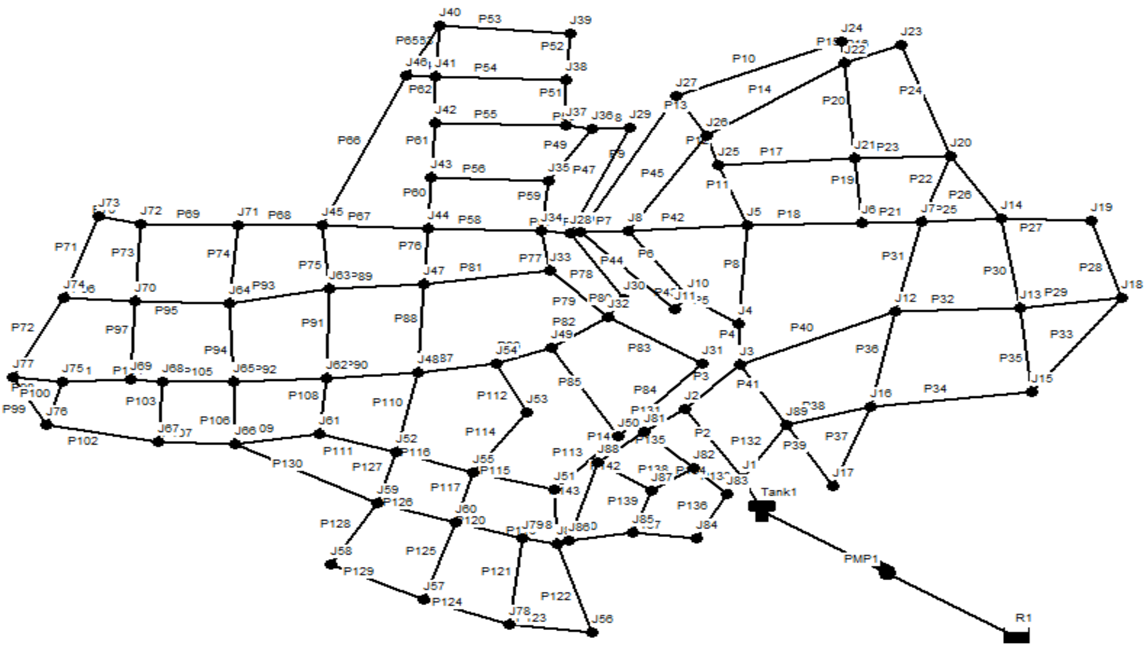

Figure 9: Epanetdistribution layout analysis

10 WATER QUALITY AND WATER TREATMENT

10.1 Ground water quality and treatment

For the most part ground water originates form infiltrated rainwater which after reaching the aquifer flows through the underground during infiltration, the water will pick up many impurities such as in organic and organic soil particles debris form plant and animal life micro-organisms natural or man – mode fertilizers pesticides etc. During its flow underground, however a great improving in water quality will occur suspended particles are removed by filtration, organic substances are degraded by oxidation, and micro – organisms die away because of lock of nutrients .The dissolved mineral content of the water can increase considerably through the leaching of salts form the underground layers.

The quality of ground water is influenced mainly by the quality of its source. Changes or the gradation in the quality of source water can have seriously affected the quality of ground water supply. Municipal and industrial waste seepage in to an aquifer is a major source of both organic and inorganic pollution.

According to the feasibility study of Holeta town water supply the standards are with in allowable limit of WHO guide line for potable water. Then our supply needs only disinfection.

Water samples were collected from Holeta Genet BH#1, BH#2 and BH#3 and sent to water works Design and Supervision Enterprise Laboratory for analysis. This groundwater is fresh and of low conductivity: (conductivity 295 μS/cm) and TDS (152 mg/l to 186 mg/l). The nitrate (3.5 to 7 mg/l), iron (trace amount) and fluoride (0.3 to 0.54 mg/l) contents are very low. The pH value (7.8 to 8.08) of the groundwater shows that it is to the alkaline side. Generally, the ground water is fresh, potable and within the acceptable limits of the WHO and Ethiopian drinking water guidelines.

Table 38: Selected water analysis results

SOURCE OF SAMPLE	BH	BH	BH	WHO maximum allowable Concentration (mg/l)
DATE RECEIVED	15/08/08	15/08/08	15/08/08	
CLIENTS ID.NO.	Holeta Genet BH#2	Holeta Genet BH #3	Holeta Genet BH #1	
LAB.ID NO.	378/2001	379/2001	377/2001	-
Colour (app)	-	-	-	-
Turbidity (NTU)	-	-	-	5.0
Total Solids 105˚C (mg/l)	-	-	-	-
T. Dissolved Solids 105˚C (mg/l)	186.0	192.0	152.0	1000.0
Electrical Conductivity (μS/cm)	284.0	295	215.0	-
P^H	7.91	8.08	7.81	6.5-8.5
Ammonia (mg/l NH3)	0.124	0.151	0.080	-
Sodium (mg/l Na)	38.0	35.0	40.0	200.0
Potassium (mg/l k)	1.4	1.5	0.6	-
Total Hardness (mg/l Ca Co3)	102.9	117.6	58.8	500.0
Calcium (mg/l Ca)	28.56	30.24	16.80	200.0
Magnesium (mg/l Mg)	7.65	10.2	4.08	150.0
Total Iron (mg/l Fe)	Trace	Trace	Trace	0.3
Manganese (mg/l Mn)	-	-	-	0.1

Fluoride (mg/l F)	0.5	0.54	0.32	1.5
Chloride (mg/l Cl)	11.33	9.27	10.3	250.0
Nitrite (mg/l NO2)	0.01	0.1	Trace	-
Nitrate (mg/l NO3)	5.09	3.49	7.2	45.0
Alkalinity (mg/l CaCO3)	144.9	153.3	111.3	-
Carbonate (mg/l CO3)	Nil	Nil	Nil	-
Bicarbonate (mg/l HCO3)	176.78	187.03	135.79	-
Sulphate (mg/l SO4)	3.32	6.33	1.24	400
Phosphate (mg/l PO4)	-	-	-	-

Source: feasibility study

10.2 Treatment processes

A number of treatment methods have been developed to produce water of requisite quality. The character and degree of treatment will depend up on the nature of the water and this depends largely up on its source. Surface waters are likely to be bacteriological contaminated and more or less turbid. They will thus generally require coagulation, sedimentation, filtration and disinfection. Whereas ground water is usually clear and therefore depends entirely up on disinfection. Since our source is ground water we concern only with disinfection.

10.2.1 Disinfection

The process of killing the pathogenic organisms from water and making it to the user is called disinfection; and the chemicals which are used for killing of the bacteria are known as disinfectant.

Requirements of disinfectant:-

- Destroy bacteria/pathogens within a practicable period of time, over an expected range of water temperature.
- Effective at variable compositions, concentration and conditions of water treated.
- Neither toxic to humans and domestic animals nor unpalatable or otherwise objectionable in required concentration.
- Not change water properties
- Have residual in a sufficient concentration to provide protection against recontamination
- Can be determined easily, quickly, and preferably automatically.

- Dispensable at reasonable cost
- Safe and easy to store, transport, handle and supply
- Not form toxic by-products due to their reactions with any naturally occurring materials in water.
- Readily available
- Cheap

10.2.2 Methods of Disinfection

The disinfection of water can be done by the following common methods:

a) **By boiling the water:-** The water can be disinfected by boiling for 15 to 20 minutes. All the pathogenic bacteria's can be killed by this method. This is very costly method and can not be used for water works, but it can be used in emergency by individuals during the break up of epidemics in the locality .

b) **By using ozone: -** Ozone is very efficient disinfectant. It is used in gaseous form. This method can be used only if electricity is easily and cheaply available at water works.

c) **By using excess lime:-**Lime is usually used for reducing hardness of water. It has been noted practically that if some additional quantity of lime is added than what it actually requires for removal of hardness, it will also disinfect the water while removing the hardness. The addition of excess lime increases the PH value of the water which may be harmful to human health.

d) **By Using ultra-violet rays: -** Ultra-violet rays are invisible light rays having wave lengths 1000 to 4000 $m\mu$. These rays are very effective disinfectant and kill all the disease producing. But this Process is costly and requires technical skill and costly equipment.This method is mainly used for disinfection of water in swimming Pool.

e) **By potassium Permanganate:** - Potassium permanganate ($KMnO_4$) is the most common disinfectant and used in the villages for disinfection of dug-well water, pond water or private source of water. In addition to the killing of bacterial, it also reduces the organic matters by oxidizing them. Since the efficiency of killing bacterial is 98% and not 100% and the colour of the water becomes light pink, it is not being used.

f) **By using iodine & Bromine:** - All the pathogenic bacteria can be killed with in 5 minutes contact period by adding Iodine and Bromine in water but their quantity should not exceed 8ppm. These disinfectants are easily available in the form of pills and also handy. Due to the high cost, they are not used in water works of public water supplies but they are used in individual dwellings.

g) **By using chlorine: -** when chlorine is added to water it produces nascent oxygen which kills the bacteria. This method is cheap and most reliable. Therefore, in the case of Holeta water supply system this method of disinfection is selected.

For our treatment process, chlorine, compounds (hypochlorite) are used as disinfectant because they are universally accepted. This is for the reasons that:-

- Quick and effective at killing micro-organisms
- Readily soluble at the concentration needed for disinfection
- Tasteless and odorless at the concentration required
- Non-toxic to human life at the concentration required
- Easy to handle, transport and apply
- Easy to detect and concentration easy to measure
- Capable of providing protection against later contamination.

10.2.2.1 Disinfection by Chlorination

When chlorine is added to water, it produces nascent oxygen which kills the bacteria .The method is cheap and most reliable .

The following are the types of chlorination depending up on the amount of chlorine added or the stage of treatment or the result of chlorination.

i. **Plain Chlorination:-** The plain chlorination is the process of chlorination in plain or raw water in the tanks or reservoirs By this method bacteria is removed from water and the growth or algae is controlled. This method also helps in removing color and organic matter from water. The amount of chlorine required is 0.5 ppm.

ii. **Pre chlorination: -** when chlorine is added to raw water before any treatment i.e. before sedimentations this type of chlorination is known as pre-chlorination .The dose of chlorine applied should be such that at least 0.2 to 0.5 ppm of residual chlorine comes to the filter plant. Pre-chlorination improves coagulation reducing the amount of coagulants and reduce the lead on filters there by increasing their efficiency.

iii. **Post chlorination: -** The addition of chlorine after all the treatment being applied to water is called post chlorination. This is done before the water enters the distribution system. The amount of chlorine added should be such that residual chlorine of about 0.22pm appears in water after a contact period of 20minutes.

iv. **Double chlorination**:- If chlorine isadded to water at move than one point the process is called double chlorination Both pre-chlorination and post chlorination are done when the water contains large number of bacteria's.

v. **Supper Chlorination:-**The amount of chlorine in excess of that necessary for adequate bacterial purification of water. This is done under certain circumstances such as epidemics of water born diseases. High dose of chlorine is added to water i.e. 2-3 ppm beyond break-point for safety of public. lt gives a strong odor and taste or chlorine in the treated water which is later can be removed by dechlorination.

vi. **Break-point chlorination:-**The chlorine when added in water removes the bacteria (disinfection) and oxidizes the organic matter .During disinfection the amount of residual chlorine will be less in beginning but will increase gradually as the demand for disinfection is satisfied. After this the oxidation of organic matter starts and chlorine again used and water contain less and less amount of residual chlorine as the process is continued. When this demand of chlorine is satisfied the amount of residual chlorine again increases. The stage at which both these demands are satisfied and residual chlorine tends to increase is known as break-point. Any further dose of chlorine applied will reappear as free chlorine. Application of chlorine up to the break-point is known as break-point chlorination.

vii. **Dechlorination:-**The process of partial or complete reduction of residual chlorine in water by chemical or physical treatment of residual is known as dechlorination. In this method some chemicals are added for the purpose of reducing the chlorine residual to a desired value in water.

viii. **Chlorine demand: -** chlorine demand is defined as the difference between the amount of chlorine added to water and the amount of chlorine (free available, and combined available) remaining at the end of a specified contact period.

The chlorine demand for a sample of water depends on:

a) Nature and concentration of chlorine consuming substances present in water

b) Time of contact

c) PH value of water

d) Temperature of water

e) Variable conditions in process of chlorination

❖ **Dosing plant and schedule of chlorine**

For sizing the container and dosing equipment, a dosing rate of 0.5 mg/l for post chlorination is used and to be utilized for 24hr. The day tank which has sufficient capacity for one day requirement is sized based average day demand using the formula presented below:

$$Ds = (Q*C_2) / (a*10^6)$$

$$Vw = (Q*C_2) / (a*S)$$

$$Vp = (Q* C_2) (a*d*10^6)$$

$$Vs = Vw + Vp$$

$$q= 0.694* (Vw+V_p)$$

$$T_s = \frac{V_s}{qml/\min} = \frac{V_s}{q*10^{-3}lit/\min} = \frac{V_s}{q*10^{-3}*60}(hr)$$

Where,

D_s = Mass of hypochlorite (Kg)

Q = Average day demand (l/day)

C= Active chlorine Concentration immediately after application (mg/l)

a= Active chlorine percentage (60-70%)

V_w= Volume of water required (l)

S = Solution concentration (900-11000(mg/l)

V_s = Volume of solution (l)

V_p= Volume of calcium hypochlorite (l)

d =Density of calcium hypochlorite (0.9 Kg/l)

q = Dosing rate ($ml/\min$)

Table 39: Dosing rate and chlorination schedule

year	flow rate (l/d)	C (mg/l)	a	D (Kg/l)	s(mg/l)	Ds (Kg)	Vw(l)	Vp(l)	Vs(l)	Q (ml/min)

2016	1283560	0.5	0.6	0.9	11000	1.070	97.239	1.189	98.428	68.309
2021	1364104	0.5	0.6	0.9	11000	1.137	103.341	1.263	104.604	72.595
2026	1444725	0.5	0.6	0.9	11000	1.204	109.449	1.338	110.787	76.886
2031	1525269	0.5	0.6	0.9	11000	1.271	115.551	1.412	116.963	81.172
2036	1605890	0.5	0.6	0.9	11000	1.338	121.658	1.487	123.145	85.463
2041	1705989	0.5	0.6	0.9	11000	1.422	129.242	1.580	130.821	90.790

11 COST ESTIMATION AND ANALYSIS

The water supply project is a public project and needs economic analysis in addition to financial analysis. The evaluation of the project involves analysis of capital out lays in terms initial investment cost, operation, maintenance cost and running cost.

Having completed the design of a given feasible project total investment cost must be estimated for financing purpose. The estimation should consider reliable cost data. However, in this case there is no available data for estimation. Therefore, the cost of the project is estimated roughly

11.1 Cost estimation for 1150 m^3service reservoir

For wall

Total volume of wall

Volume = perimeter*height*thickness

$= 2\pi r*5.1*250*10^{-3} = 74.065\ m^3$

Hoop reinforcement

Area of Φ16 mm ring bars

$$= \frac{\pi D^2}{4} = \frac{\pi * 16^2}{4} = 201.1\ mm^2$$

Number of bar used = 20

Total area of reinforcement in both side = $2*20*201.1\ mm^2$

$=8044\ mm^2$

Volume = $8044\ mm^2*12m*10^{-6}$

$=0.096528 m^3$

Length of bar =volume/area of bar=$0.096528/201.1*10^{-6}$=480m

Weight in kilogram=$\frac{0.222*d^2}{36}$*length=$\frac{0.222*16^2}{36}$*480=757.92kg

Vertical reinforcement

For Inner face reinforcement

Area of Φ14 mm bar

$$= \frac{\pi D^2}{4} = \frac{\pi*14^2}{4} = 153.93\text{mm}^2$$

Number of bar per meter width = 8 bar

Total area =8*3.14*14*153.93 mm^2

= 54134.1024mm^2

Volume =54134.1024*$10^{-6}$$m^2$*12m where the length of one bar is 12m

=0.06495m^3

Length of bar =volume/area of bar=0.06495/153.93*10^{-6}=421.92m

Weight in kilogram=$\frac{0.222*d^2}{36}$*length=$\frac{0.222*14^2}{36}$*421.92=509.96kg

Outer face reinforcement

Area of Area of Φ12 mm bar

$$= \frac{\pi D^2}{4} = \frac{\pi*12^2}{4} = 113.1\text{mm}^2$$

Number of bar per meter width = 5 bar

Total area = 5*3.14*12*113.1 mm^2

= 21308.04 mm^2

Volume = 21308.04 *$10^{-6}$$m^2$*12m where the length of one bar is 12m

=0.25569m^3

Length of bar =volume/area of bar=**0.**25569/113.1*10^{-6}=2260.7m

Weight in kilogram=$\frac{0.222*d^2}{36}$*length=$\frac{0.222*16^2}{36}$*2260.7=2007.54kg

Volume of concrete = Total volume of wall slab- Volume of steel

=74.065 m^3-(0.096528m^3+0.25569m^3+0.06495m^3) =73.647882m^3

For roof slab

Total volume of roof slab

Volume=area*thickness

$$V = A * T = \frac{\pi D^2 * T}{4}$$

$V = A * T = \frac{\pi*19^2*250*10^{-3}}{4}$ = 70.882 m^3

Volume of reinforcement

Area of Φ10 mm bar

$= \frac{\pi D^2}{4} = \frac{\pi*10^2}{4}$= 78.5398 mm^2

At centre of roof slab

Total area of steel bar=no of bar*diameter of roof slab*area of one Φ12 mm bar

=**38***14.4*78.5398mm^2=56705.7474mm^2

$\text{Total area at bottom and top reinforcement} = 56705.7474\text{m2} * 4$

$= 226822.99\text{mm}^2 \text{ in mesh}$

Volume for 12m long bar = 12m*226822.99*10^{-6} m^2= 2.722 m^3

Length of bar =volume/area of bar=2.722 m^3/78.5398mm^2*10^{-6}=34656m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*10^2}{36}$*34656=21371.204kg

At edge of roof slab

Total area of steel bar=no of bar*one meter width of roof slab including wall thickness*area of one Φ12 mm ring bar

=**22***1m*113.1mm^2=2488.2mm^3

$\text{Total area at bottom and top reinforcement} = 2488.2\text{mm3} * 2 = 4976.4\text{ mm}^2$

Volume for 12m long bar = 12m*4976.4 mm^2*10^{-6} m= 0.0597168 m^3

Length of bar =volume/area of bar=0.0597168 m^3/113.1mm^2*10^{-6}=528m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*12^2}{36}$*528=468.864kg

Total volume of reinforcement at roof slab= $\text{vc} + \text{ve} = 2.722\text{ m3} + 0.0597168\text{ m3}$

$= 2.7817\text{ m}^3$

$\text{Volume of concrete} = \text{total volume} - \text{total volume of reinforcement}$

$= 70.882\text{ m3} - 2.7817\text{ m}^3 = 68.1\text{m}^3$

For base slab

Total volume of base slab

Volume=area*thickness

$$V = A * T = \frac{\pi D^2 * T}{4}$$

$V = A * T = \frac{\pi * 14^2 * 300 * 10^{-3}}{4} = 46.18\ m^3$

Volume of reinforcement

Area of Φ20mm bar

$$= \frac{\pi D^2}{4} = \frac{\pi * 20^2}{4} = 314.159mm^2$$

At centre of base slab

Total area of steel bar

$$= \text{no of bar per width} * \text{diameter of base slab} * \text{area of one } \Phi 20 \text{ mm bar}$$

$$= 14 * 19 * 314.159mm^2 = 83566{,}294mm^2$$

Total area at bottom and top reinforcement $= 83566{,}294mm^2 * 4$

$$= 334265.176\ mm^2 \text{in mesh}$$

$$\text{Volume for 12m long bar} = 12m * 334265.176 \ * 10\text{^} - 6\ m2 = 4.0112m^3$$

Length of bar =volume/area of bar=$4.0112m^3$/314.159*10^{-6}=12768.056m

Weight in kilogram =$\frac{0.222 * d^2}{36}$*length=$\frac{0.222 * 20^2}{36}$*528=31494.4kg

In circumferential direction (ring bars)

Total area of ring bars

$$= \text{no of bar} * \ 0.75m \text{ inside and outside from center of wall}$$

$$* \text{ area of one } \Phi 20 \text{ mm bar}$$

$$= 5 * 0.75 * 314.159mm^2 = 1178.096mm^2$$

$$\text{Total area at bottom and top of ring reinforcement} = 1178.096mm^2 * 2$$

$$= 2356.1925mm^2 \text{in ring}$$

$$\text{Volume for 12m long bar} = 12m * 2356.1925 * 10 - 6m^2 = 0.0287431m^3$$

Length of bar =volume/area of bar=$0.0287431m^3$/314.159mm^2*10^{-6}=91.4922m

Weight in kilogram =$\frac{0.222 * d^2}{36}$*length=$\frac{0.222 * 20^2}{36}$*91.4922m =225.68kg

At edge of base slab

Total area of radial bar

$$= \text{no of bar} * \ 0.75m \text{ inside and outside from center of wall} \quad * \text{ area of one } \Phi 20 \text{ mm ba}$$

$$= 20 * 0.75 * 314.159mm^2 = 4712.385mm^2$$

$$\text{Total area at bottom and top of ring reinforcement} = 4712.385mm^2 * 2$$

$$= 9424.77mm^2 \text{in ring}$$

$$\text{Volume for 12m long bar } = 12\text{m} * 9424.77\text{mm}^2 * 10 - 6 \text{ m2} = 0.01131\text{m}^3$$

Length of bar =volume/area of bar=0.01131m^3/314.159mm^2*10^{-6}=36.00m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*20^2}{36}$*91.4922m =88.802kg

$$\text{Total Volume of steel} = \text{Vc} + \text{Ve} = 4.0112\text{m}^3 + (0.0287431 + 0.01131)\text{m}^3$$

$$= 4.0512531\text{m}^3$$

Volume of concrete = Total volume of base slab- Volume of steel

$$= 46.18\text{m}^3 - 4.0512531\text{m}^3 = 42.12875\text{m}^3$$

$$\text{For phase} - 1\text{: Total volume of concrete} = \text{vwc} + \text{vrc} + \text{vbc}$$

$$= 73.647882 + 68.1 + 42.12875 = 157.905\text{m}^3$$

9.2 Cost estimation for 550 m^3 service reservoir

For wall

Total volume of wall

Volume = perimeter*height*thickness

= 2πr*5.1*250*10^{-3} = 74.065 m^3

Hoop reinforcement

Area of Φ16 mm ring bars

$= \frac{\pi D^2}{4} = \frac{\pi*16^2}{4}$ = 201.1 mm^2

Number of bar used = 10

Total area of reinforcement in both side = 2*10*201.1 mm^2

=4022 mm^2

Volume = 4022 mm^2*12m*10^{-6}

=0.048264m^3

Length of bar =volume/area of bar=0.048264m^3/201.1*10^{-6}=240m

Weight in kilogram=$\frac{0.222*d^2}{36}$*length=$\frac{0.222*16^2}{36}$*240=375.46kg

Vertical reinforcement

For Inner face reinforcement

Area of Φ12 mm bar

$= \frac{\pi D^2}{4} = \frac{\pi*12^2}{4}$ = 113.1mm^2

Number of bar per meter width = 5 bar

Total area =5*3.14*12*113.1mm^2

= 21308.04mm^2

Volume =21308.04*10^{-6}m^2*12m where the length of one bar is 12m

=0.25569m^3

Length of bar =volume/area of bar=0.25569/113.1*10^{-6}=2260.8m

Weight in kilogram=$\frac{0.222*d^2}{36}$*length=$\frac{0.222*12^2}{36}$*2260.8=2007.6kg

Outer face reinforcement

Area of Area of Φ12 mm bar

$= \frac{\pi D^2}{4} = \frac{\pi*12^2}{4}$ = 113.1mm^2

Number of bar per meter width =6 bar

Total area = 6*3.14*12*113.1 mm^2

= 25569.6 mm^2

Volume = 25569.6 *10^{-6}m^2*12m where the length of one bar is 12m

=0.3068m^3

Length of bar =volume/area of bar=0.3068/113.1*10^{-6}=2712.96m

Weight in kilogram=$\frac{0.222*d^2}{36}$*length=$\frac{0.222*12^2}{36}$*2712.96=4082.1005kg

Volume of concrete =74.065 m^3-(0.3068m^3+0.25569m^3+0.048264m^3) =73.459m^3

For roof slab

Total volume of roof slab

Volume=area*thickness

$$V = A*T = \frac{\pi D^2 * T}{4}$$

$$V = A*T = \frac{\pi*14.4^2*200*10^{-3}}{4} = 32.55 \text{ m}^3$$

Area of Area of Φ12 mm bar $= \frac{\pi D^2}{4} = \frac{\pi*12^2}{4}$ = 113.1mm^2

At centre of roof slab

Total area of steel bar=no of bar*diameter of roof slab*area of one Φ12 mm bar

=35*14.4*113.1mm^2=57002.4mm^2

Total area at bottom and top reinforcement $= 57002.4\text{mm2} * 4 = 228009.6\text{mm}^2$ in mesh

Volume for 12m long bar = 12m*228009.6*10^{-6} m^2= 2.736 m^3

Length of bar =volume/area of bar=2.736 m^3/113.1*10^{-6}=24192m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*10^2}{36}$*24192 =14918.4kg

At edge of roof slab

Total area of steel bar=no of bar*one meter width of roof slab including wall thickness*area of one Φ12 mm ring bar.

=14*1m*113.1mm^2=1583.4mm^3

Total area at bottom and top reinforcement =1583.4$mm^3 * 2 = 3166.8\ mm^2$

Volume for 12m long bar = 12m*3166.8 mm^2*10^{-6} m= 0.038 m^3

Length of bar =volume/area of bar=0.038m^3/113.1mm^2*10^{-6}=336m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*12^2}{36}$*336=298.36kg

Total volume of reinforcement at roof slab= $vc + ve = 2.736\ m3 + 0.038\ m3m3$

$= 2.774\ m^3$

Volume of concrete = total volume − total volume of reinforcement

$= 32.55\ m3 - 2.774\ m^3 = 29.776 m^3$

For base slab

Total volume of base slab

Volume=area*thickness

$$V = A * T = \frac{\pi D^2 * T}{4}$$

$$V = A * T = \frac{\pi * 14^2 * 300 * 10^{-3}}{4} = 46.18\ m^3$$

Volume of reinforcement

Area of Φ18mm bar

$$= \frac{\pi D^2}{4} = \frac{\pi * 18^2}{4} = 254.34 mm^2$$

At centre of base slab

Total area of steel bar

$= \text{no of bar per width} * \text{diameter of base slab} * \text{area of one } \Phi 20\ \text{mm bar}$

$= 14 * 14.4 * 254.34 = 51274.944 mm^2$

Total area at bottom and top reinforcement $= 51274.944 mm^2 * 4$

$= 205099.776\ mm^2$in mesh

$$\text{Volume for 12m long bar} = 12\text{m} * 205099.776 \ * 10^\wedge - 6 \text{ m2} = 2.4612\text{m}^3$$

Length of bar =volume/area of bar=$2.4612\text{m}^3/254.34*10^{-6}$=9676.8m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*18^2}{36}$*9676.8=19334.2675kg

In circumferential direction (ring bars)

Total area of ring bars

$$= \text{no of bar} * \ 0.75\text{m inside and outside from center of wall}$$
$$* \text{ area of one } \Phi 20 \text{ mm bar}$$
$$= 3 * 0.5 * 254.34 = 381.51\text{mm}^2$$

$$\text{Total area at bottom and top of ring reinforcement} = 381.51\text{mm}^2 * 2 = 763.02\text{mm}^2 \text{in ring}$$

$$\text{Volume for 12m long bar} = 12\text{m} * 763.02 * 10^\wedge - 6\text{m}^2 = 0.0091524\text{m}^3$$

Length of bar =volume/area of bar=$0.0091524\text{m}^3/254.34\text{mm}^2*10^{-6}$=36m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*18^2}{36}$*36m =71.928kg

At edge of base slab

Total area of $\Phi 18$mm radial bar

$$= \text{no of bar} * \ 0.5\text{m inside and outside from center of wall} \quad * \text{ area of one } \Phi 18 \text{ mm ba}$$
$$= 22 * 0.5 * 254.34\text{mm}^2 = 2797.74\text{mm}^2$$
$$\text{Total area at bottom and top of ring reinforcement} = 2797.74\text{mm}^2 * 2$$
$$= 5595.48\text{mm}^2 \text{in ring}$$
$$\text{Volume for 12m long bar} = 12\text{m} * 5595.48\text{mm}^2 * 10^\wedge - 6 \text{ m2} = 0.06714\text{m}^3$$

Length of bar =volume/area of bar=$0.06714\text{m}^3/254.34\text{mm}^2*10^{-6}$=264.00m

Weight in kilogram =$\frac{0.222*d^2}{36}$*length=$\frac{0.222*18^2}{36}$*264m =527.47kg

Total Volume of steel $= \text{Vc} + \text{Ve} = 2.5375\text{m}^3$

Volume of concrete = Total volume of base slab- Volume of steel

$$= 46.18\text{m}^3 - 2.5375\text{m}^3 = 43.6425\text{m}^3$$

For phase-II: Total volume of concrete $(\text{V2}) = \text{vwc} + \text{vrc} + \text{vbc}$

$$= 73.459 + 29.3 + 43.6425 = 146.8775\text{m}^3$$

Total volume of concrete=V1+V2=$157.905\text{m}^3 + 146.8775\text{m}^3$=$304.7825\text{m}^3$

Total volume of concrete in dry volume=Vt*1.5=457.1738m^3

Therefore the concrete mix for C-30 is (1:1.5:3)

Assume fresh concrete is $1m^3$ and quantity of dry volume = $1.5*1m^3=1.5m^3$

Table 40:Concrete mix ratio for C-30

No	Description	Qty	Remark
1	Cement=$1/5.5*1.5=0.273m^3$	$0.273m^3$	
2	Sand=$1.5/5*1.5=0.409m^3$	$0.409m^3$	
3	Aggregate=$3/5.5*1.5=0.818m^3$	$0.818m^3$	

Density of cement is $3150kg/m^3$, Density of sand is $1840kg/m^3$ and Density of aggregate is $2250kg/m^3$

Calculate unit rate analysis for concrete

Cement = $\rho_c*Vc = 3150*0.273m^3=859.95kg$ or 8.5995Quintal

So for $457.1738m^3$ concrete we need 1440.0975Quintal

Sand =$Vt*V_{s.} = (457.1738m3*0.409m^3)/1.5= 124.6561m^3$

Aggregate =$Vt*V_a = (457.1738m3*0.818m^3)/1.5=249.3121m^3$

So for $304.7825m^3$ of freshconcrete we need dry quantity of $457.1738m^3$.

Calculate Unit Rate Analysis For Bar (Steel)

Density of steel is ranges between $7750kg/m^3$-$8500kg/m^3$ depeds upon allowable constraint. So we use $7750kg/m^3$ for our calculation.

Table 41: Analysis of reinforcement bar consumption

Diameter(mm)	Length(m)	Mass (kg)	Volume(m^3)
Φ10	58848	36289.604	5.458
Φ12	8098.36	8864.504	0.912897
Φ14	421.92	509.96	0.0649
Φ16	720	1133.38	0.14478
Φ18	9976.8	19933.6655	2.537492
Φ20	12895.5482	31808.882	4.051243

12 ENVIRONMENTAL IMPACT ASSESMENT (EIA)

12.1 Introduction

EIA is a process to improve decision –making and to ensure that the project or program option under consideration is environmentally sound. The EIA is concerned with identifying, predicting and evaluating the fore seeable environmental effects, alternatives and mitigating measures aiming at minimizing the adverse effects and maximizing the benefits obtained because of the project.

An EIA, in our case is concerned with impacts of water recourses development projects on the environment and with the sustainability of the projects themselves. Clearly, an EIA will not resolve all problems. There will be trade–offs between economic development and environmental protection as in all development activities. However, without consideration of basic concepts of EIA, the project will not be effective and economical as well as informed decision-making would be impossible.

12.2 Description of the potential impacts

An impact can be defined as any change in the physical, chemical, biological and socio-economic environmental system, which can be attributed to human activities relative to alternatives under study for meeting a project need. EIA cover both positive and negative impacts of the project and mitigation measures for the adverse effects.

12.2.1 Positive impact of the project

The significance of the Holeta town water supply project is mainly of socio –economic. Some, which are;

- ❖ Assuring an adequate supply of water in quality as well as quantity for the population.
- ❖ Time and energy saving in fetching of drinking water
- ❖ Improvements of life standard and water supply situation
- ❖ Control of fire damage
- ❖ The construction of the proposed water supply project will provide employment for a significant number of local skilled and unskilled workers.
- ❖ Generally, it supports the town development like establishment of different institutions, industries, health centers etc.

12.2.2 Negative Impacts of the project

The impacts are generally categorized in to two and these are;

I. Impacts during construction period or short term impact and
II. Long term impact

I. Short term impact

It is an impact that occurs during construction period of the project. Some of the short-term impacts are;

- ❖ Interruption of the existing water supply system
- ❖ Soil erosion from trench excavation(for pipe line)
- ❖ Air pollution during the construction activity
- ❖ Traffic movement is affected Spillage of chemical
- ❖ The pipes used for transmission and distribution can bring health hazards if they are supposed to corrode
- ❖ Access road change during construction especially in bore whole site.
- ❖ Sound pollution during well drilling

II. Long term impact

The impacts that has long lasting effects and may even bring irreversible environmental changes may include:

- ❖ Displacement of individual living near reservoir
- ❖ Pollution due to disposal of used oil during operation to natural water source.
- ❖ Deforestation for the construction of structures

12.2.3 Mitigation measures

For the negative impacts mentioned above, mitigation measures have to be applied before or during the commencement of the project as much as possible.

- ❖ During the excavation of trench for the new pipelines, the existing water supply will be interrupted. This can be avoided by excavation of the trench part by part
- ❖ During digging of trenches the disordered to should be prevented and restored and adequate precaution should be taken to prevent soil erosion.
- ❖ Because dust air that occur during construction, air pollution may be happen and this impact avoids by spraying water along the construction area.
- ❖ This happens when pipe trenches are excavated and minimized this by providing alternative routes until construction is completed.
- ❖ For the impacts of spillages. There should be a waste management plan. Accordingly, these waste materials could be removed through reaching, incarnating and disposing safely.

- ❖ The impact with related to pipe materials can be overcome or minimized by selecting appropriate pipe materials.

13 CONCLUSION AND RECOMMENDATION

13.1 Conclusion

The existing water supply system of Holeta town is not sufficient. Due to this reason and the alarmingly increasing population of the town, it is necessary to design and construct a new water supply scheme.

The project is designed for design period of 25 year for two phases. The first phase covers from 2016 to 2031 and, the second phase covers from 2031 to 2041.The method of CSA population forecasting is selected with inadequate data of central statically authority.

Selection of potential water sources are made to determine which sources should satisfy the respective quantity and quality of the demand though our design period. Ground water is selected as a potential water source because of its adequacy and closeness to the town.

The quality of water source at the borehole found to be potable without treatment. Thus, chlorination is provided to avoid contamination of water through transmission line and distribution network.

Finally this project improve the health of community by reducing water transmission diseases, save waste of time, supply adequate water to town and reduces shortage of water.

13.2 Recommendation

We observed that Holeta town is a city with high potential for growth. The city obviously needs well equipped distribution system, pollution free storage structure and consistent flow potable water. This needs commitment, hard work and dedication. We, as a team would like to recommend the following points to be understood and taken in to consideration for all decision makers, practitioners, client, designers and other stakeholders for further implementation.

- ❖ The existing distribution system and the entire system should be modified by the design in this paper for the design period horizon of the paper so as to answer the demand of the community in the city and within the vicinity.

- Chlorine dosage specified in this paper is more reliable which is calculated using the given data from the feasibility report. For further use chlorine dosage by jar test should be applied for more accuracy.
- For sustainable water use other perennial sources should be considered
- Fair and wise compensation should be given to those who lose their land due to the implementation of this project.
- It is also recommended that prior to the end of the Stage I design horizon, and prior to implementation of Stage II that population, town growth and the associated water demands are reassessed against the estimates contained in this report and that detailed design and implementation of Stage II is adjusted accordingly.

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19. Twort's Water Supply Seventh Edition

ANNEXES

Annex A. Epanet Analysis Report

Network Table - Nodes at 8:00 Hrs.

Node ID	X-Cord	Y-Cord	Elevation m	Base Demand LPS	Deman d LPS	Head m	Pressure m
J1	1002452	443573	2425	1.5929	2.39	2449.52	24.52
J2	1002315	443785	2417	1.39	2.09	2446.75	29.75
J3	1002447	443918	2421	1.331	2	2444.86	23.86
J4	1002443	444041	2421	0.7554	1.13	2443.75	22.75
J5	1002462	444335	2410	1.8801	2.82	2439.86	29.86
J6	1002732	444342	2415	0.5107	0.77	2439.85	24.85
J7	1002872	444346	2418	1.0308	1.55	2439.92	21.92
J8	1002183	444321	2400	2.3182	3.48	2437.79	37.79
J9	1002071	444315	2415	0.6618	0.99	2437.03	22.03
J10	1002323	444123	2420	0.6218	0.93	2440.51	20.51
J11	1002294	444083	2418	1.2834	1.93	2438.39	20.39
J12	1002811	444077	2422	2.648	3.97	2444.36	22.36
J13	1003103	444089	2420	1.9345	2.9	2441.43	21.43
J14	1003059	444357	2420	2.0194	3.03	2439.82	19.82
J15	1003132	443835	2420	3.6566	5.48	2441.6	21.6
J16	1002752	443790	2420	3.7657	5.65	2447.68	27.68
J17	1002664	443553	2422	1.2399	1.86	2447.94	25.94
J18	1003344	444118	2415	1.715	2.57	2440.23	25.23
J19	1003271	444350	2415	1.5471	2.32	2439.83	24.83
J20	1002940	444545	2410	1.3015	1.95	2438.86	28.86
J21	1002714	444539	2408	1.7451	2.62	2437.96	29.96
J22	1002692	444825	2415	0.8701	1.31	2437.94	22.94
J23	1002825	444878	2415	1.465	2.2	2438.05	23.05
J24	1002684	444889	2419	1.2702	1.91	2437.79	18.79

J25	1002394	444518	2410	1.5113	2.27	2437.16	27.16
J26	1002366	444605	2410	2.4849	3.73	2437.2	27.2
J27	1002297	444727	2410	1.4068	2.11	2437.15	27.15
J28	1002047	444314	2380	0.5883	0.88	2436.87	56.87
J29	1002188	444631	2376	0.9295	1.39	2436.68	60.68
J30	1002174	444114	2380	0.8576	1.29	2435.75	55.75
J31	1002356	443920	2373	0.7962	1.19	2432.35	59.35
J32	1002136	444060	2376	1.7054	2.56	2435.61	59.61
J33	1001998	444200	2380	1.6695	2.5	2435.81	55.81
J34	1001980	444319	2373	0.7615	1.14	2436.58	63.58
J35	1001997	444470	2371	0.5258	0.79	2436.27	65.27
J36	1002099	444628	2375	0.5404	0.81	2436.52	61.52
J37	1002037	444637	2374	1.1307	1.7	2436.44	62.44
J38	1002038	444774	2376	1.1827	1.77	2436.35	60.35
J39	1002048	444912	2375	1.0644	1.6	2436.33	61.33
J40	1001740	444938	2373	1.1659	1.75	2434.09	61.09
J41	1001730	444785	2375	1.0775	1.62	2434.08	59.08
J42	1001731	444642	2372	1.0887	1.63	2434.5	62.5
J43	1001721	444481	2374	1.0229	1.53	2434.51	60.51
J44	1001711	444328	2379	1.1999	1.8	2433.19	54.19
J45	1001465	444338	2376	1.8335	2.75	2430.58	54.58
J46	1001662	444788	2375	1.7041	2.56	2433.84	58.84
J47	1001704	444159	2380	1.7341	2.6	2430.18	50.18
J48	1001690	443893	2375	2.5683	3.85	2422.18	47.18
J49	1002002	443970	2390	1.425	2.14	2413.83	23.83
J50	1002161	443703	2375	1.3	1.95	2411.21	36.21
J51	1002010	443541	2380	1.564	2.35	2401.38	21.38
J52	1001636	443655	2385	2.9487	4.42	2406.99	21.99
J53	1001943	443773	2380	1.479	2.22	2402.01	22.01
J54	1001873	443919	2390	1.2272	1.84	2411.16	21.16

J55	1001817	443594	2380	2.5264	3.79	2401.88	21.88
J56	1002097	443112	2375	1.1457	1.72	2398.86	23.86
J57	1001704	443212	2370	1.1173	1.68	2399.11	29.11
J58	1001483	443316	2374	1.3495	2.02	2406.22	32.22
J59	1001592	443502	2378	1.9744	2.96	2406.88	28.87
J60	1001776	443442	2380	2.0302	3.05	2403.82	23.82
J61	1001456	443708	2387	2.0859	3.13	2420.16	33.16
J62	1001475	443876	2373	1.7646	2.65	2422.35	49.35
J63	1001483	444145	2380	3.3577	5.04	2426.05	46.05
J64	1001245	444101	2375	2.909	4.36	2425.91	50.91
J65	1001256	443865	2389	0.7719	1.16	2422.81	33.81
J66	1001258	443678	2382	0.9101	1.37	2417.84	35.84
J67	1001078	443686	2385	1.5488	2.32	2422.4	37.4
J68	1001089	443867	2389	1.2136	1.82	2424.48	35.48
J69	1001014	443872	2385	1.258	1.89	2426.7	41.7
J70	1001023	444107	2375	0.884	1.33	2426.84	51.84
J71	1001267	444337	2375	1.0162	1.52	2428.94	53.94
J72	1001036	444340	2375	0.9428	1.41	2427.8	52.8
J73	1000937	444363	2375	1.7655	2.65	2427.6	52.6
J74	1000856	444117	2375	0.882	1.32	2426.48	51.48
J75	1000852	443867	2384	0.9379	1.41	2425.83	41.83
J76	1000815	443737	2384	0.8351	1.25	2425.74	41.74
J77	1000737	443880	2389	1.6943	2.54	2426.35	37.35
J78	1001903	443138	2369	1.6279	2.44	2398.96	29.96
J79	1001935	443397	2376	1.4806	2.22	2400.36	24.36
J80	1002017	443378	2370	0.7971	1.2	2400.29	30.29
J81	1002221	443717	2420	1.04	1.56	2445.17	25.17
J82	1002337	443607	2435	1.75	2.63	2446	11
J83	1002416	443529	2425	0.6053	0.91	2447.68	22.68
J84	1002345	443396	2424	1	1.5	2445.14	21.14

J85	1002192	443411	2430	0.6251	0.94	2443.17	13.17
J86	1002042	443389	2430	1.0465	1.57	2443.22	13.22
J87	1002237	443540	2420	1	1.5	2443.57	23.57
J88	1002112	443625	2410	1.474	2.21	2443.7	33.7
J89	1002555	443736	2425	2.3549	3.53	2448.2	23.2
R1	1002498	443479	2359	#N/A	-35.75	2359	0
Tank1	1000768	444771	2441	#N/A	-157.41	2450.47	2.5

Network Table - Links at 8:00 Hrs

Link ID	Start Node	End Node	Length m	Diameter mm	Roughness	Flow LPS	Velocity m/s	Unit Head loss m/km
P1	R1	J1	106.1	350	140	193.16	1.95	8.95
P2	J1	J2	251.2	300	140	144.28	1.94	11.04
P3	J2	J3	187.4	300	140	137.2	1.94	10.06
P4	J3	J4	123.1	300	140	129.48	1.83	9.04
P5	J4	J10	242.5	80	140	4.94	0.98	13.34
P6	J10	J8	242.5	50	140	1.31	0.67	11.25
P7	J8	J9	112.2	300	140	110.5	1.56	6.74
P8	J4	J5	469.9	300	140	123.41	1.75	8.27
P9	J9	J27	469.9	80	140	-0.59	0.32	0.26
P10	J27	J24	419.5	80	140	-1.53	0.3	1.52
P11	J5	J25	195.2	50	140	1.46	0.75	13.85
P12	J25	J26	91.39	50	140	-0.22	0.31	0.42
P13	J26	J27	140.2	100	140	1.18	0.35	0.31
P14	J26	J22	64.5	50	140	-1.32	0.67	11.47
P15	J22	J24	64.5	100	140	3.43	0.44	2.29
P16	J22	J23	320.7	100	140	-1.25	0.36	0.35
P17	J25	J21	320.7	50	140	-0.58	0.3	2.49

P18	J5	J6	197.8	180	140	2.65	0.41	0.08
P19	J6	J21	286.8	80	140	3.37	0.67	6.58
P20	J21	J22	140.1	200	140	4.81	0.35	0.15
P21	J6	J7	140.1	100	140	-1.49	0.29	0.49
P22	J7	J20	210.3	100	140	5.23	0.67	5
P23	J21	J20	226.1	100	140	-4.64	0.59	4
P24	J20	J23	352.3	100	140	3.44	0.44	2.31
P25	J7	J14	187.3	100	140	1.56	0.28	0.53
P26	J20	J14	222.5	100	140	-4.81	0.61	4.28
P27	J14	J19	212.1	100	140	-0.53	0.37	0.07
P28	J19	J18	243.2	100	140	-2.85	0.36	1.63
P29	J18	J13	242.7	80	140	-2.89	0.58	4.95
P30	J14	J13	271.6	100	140	-5.74	0.73	5.94
P31	J7	J12	275.8	100	140	-9.83	1.25	16.1
P32	J12	J13	292.3	110	140	9.78	1.03	10.02
P33	J18	J15	353.6	80	140	-2.53	0.5	3.87
P34	J15	J16	382.7	100	140	-9.77	1.24	15.91
P35	J13	J15	255.7	100	140	-1.75	0.22	0.66
P36	J16	J12	293	100	140	8.14	1.04	11.35
P37	J16	J17	252.8	80	140	-1.24	0.25	1.03
P38	J16	J89	204.3	200	140	-22.32	0.71	2.51
P39	J17	J89	213	110	140	-3.1	0.33	1.19
P40	J3	J12	397.2	200	140	15.44	0.49	1.27
P41	J89	J3	211.6	100	140	9.72	1.24	15.76
P42	J8	J5	279.4	300	140	-116.48	1.65	7.43
P43	J10	J11	49.41	50	140	2.7	1.37	42.93
P44	J11	J9	321.8	50	140	0.77	0.39	4.23
P45	J8	J26	337.9	110	140	3.8	0.4	1.74
P46	J9	J28	24.02	300	140	110.88	1.57	6.78
P47	J28	J29	89.05	200	140	20.27	0.65	2.1

P48	J29	J36	89.05	200	140	18.88	0.6	1.84
P49	J36	J35	188.1	100	140	2.57	0.33	1.34
P50	J36	J37	62.65	200	140	15.5	0.49	1.28
P51	J37	J38	137	200	140	10.5	0.33	0.62
P52	J38	J39	138.4	200	140	5.15	0.16	0.17
P53	J39	J40	309.1	80	140	3.55	0.71	7.23
P54	J38	J41	308.2	80	140	3.58	0.71	7.36
P55	J37	J42	306	80	140	3.3	0.66	6.33
P56	J35	J43	276.2	100	140	5.95	0.76	6.35
P57	J28	J34	67.19	300	140	86.61	1.23	4.29
P58	J34	J44	269.2	200	140	53.35	1.7	12.61
P59	J34	J35	152	110	140	4.17	0.44	2.07
P60	J44	J43	153.3	80	140	-3.91	0.78	8.65
P61	J43	J42	161.3	100	140	0.51	0.06	0.07
P62	J42	J41	143	80	140	2.17	0.43	2.92
P63	J41	J40	153.3	80	140	-0.27	0.25	0.06
P64	J41	J46	68.07	100	140	4.41	0.56	3.65
P65	J40	J46	169.1	80	140	1.53	0.3	1.52
P66	J46	J45	491.2	80	140	3.39	0.67	6.63
P67	J44	J45	246.2	200	140	48.53	1.54	10.58
P68	J45	J71	198	200	140	42.46	1.35	8.26
P69	J71	J72	231	200	140	32.25	1.03	4.97
P70	J72	J73	259	180	140	8.79	0.35	0.75
P71	J73	J74	265.2	110	140	6.14	0.65	4.24
P72	J74	J77	265.2	200	140	9.09	0.29	0.48
P73	J72	J70	233.4	180	140	22.05	0.87	4.1
P74	J71	J64	237	100	140	8.69	1.11	12.79
P75	J45	J63	193.2	80	140	6.7	1.33	23.47
P76	J44	J47	120.4	80	140	6.93	1.38	24.97
P77	J34	J33	120.4	180	140	27.94	1.1	6.36

P78	J28	J30	196.6	80	140	3.12	0.62	5.69
P79	J33	J32	196.6	180	140	10.5	0.41	1.04
P80	J30	J32	66.03	80	140	1.83	0.36	2.12
P81	J33	J47	161.4	100	140	14.93	1.9	34.9
P82	J32	J49	161.4	50	140	5.01	1.55	134.93
P83	J32	J31	260.8	80	140	4.77	0.95	12.51
P84	J31	J50	291.7	50	140	3.58	1.82	72.44
P85	J50	J49	310.8	50	140	-1.12	0.57	8.42
P86	J49	J54	138.7	50	140	1.75	0.89	19.24
P87	J54	J48	184.8	50	140	-3.22	1.64	59.63
P88	J47	J48	221.4	80	140	8.46	1.68	36.11
P89	J47	J63	215.7	100	140	10.8	1.38	19.17
P90	J48	J62	215.7	100	140	-1.91	0.24	0.78
P91	J63	J62	269.7	110	140	11.58	1.22	13.7
P92	J62	J65	219.3	110	140	-4.21	0.44	2.1
P93	J63	J64	241.4	80	140	0.89	0.28	0.56
P94	J64	J65	236.3	110	140	11.31	1.29	13.12
P95	J64	J70	222.1	110	140	6.1	0.64	4.18
P96	J70	J74	167.3	110	140	4.26	0.45	2.15
P97	J70	J69	235.2	200	140	10.35	0.33	0.61
P98	J77	J75	115.7	50	140	0.8	0.41	4.54
P99	J77	J76	162.9	110	140	5.74	0.36	3.74
P100	J75	J76	135.2	50	140	0.27	0.34	0.62
P101	J75	J69	162.1	50	140	-0.88	0.45	5.37
P102	J76	J67	267.9	80	140	4.76	0.95	12.47
P103	J67	J68	181.3	80	140	-4.54	0.29	11.43
P104	J68	J69	75.17	80	140	-7.59	1.51	29.56
P105	J65	J68	167	50	140	-1.23	0.62	9.96
P106	J65	J66	187	80	140	7.17	1.43	26.6
P107	J66	J67	180.2	80	140	-6.98	1.39	25.34

P108	J62	J61	169.1	110	140	11.23	1.18	12.94
P109	J61	J66	200.3	80	140	4.58	0.91	11.61
P110	J48	J52	244.1	50	140	3.3	1.68	62.25
P111	J61	J52	187.6	50	140	3.52	1.79	70.19
P112	J54	J53	161.9	50	140	3.13	1.59	56.51
P113	J50	J51	221.5	50	140	2.75	1.4	44.41
P114	J53	J55	218.9	80	140	0.91	0.18	0.58
P115	J51	J55	200.1	50	140	-0.58	0.3	2.53
P116	J55	J52	191	50	140	-2.09	1.06	26.75
P117	J55	J60	157.4	50	140	-1.37	0.7	12.32
P118	J51	J80	163.2	50	140	0.99	0.5	6.65
P119	J80	J79	84.17	80	140	-1.07	0.31	0.78
P120	J79	J60	165.3	80	140	-6.31	1.25	20.97
P121	J79	J78	261	80	140	3.02	0.6	5.35
P122	J80	J56	277.8	50	140	0.86	0.44	5.16
P123	J56	J78	195.7	80	140	-0.86	0.17	0.52
P124	J78	J57	212.3	50	140	-0.29	0.15	0.67
P125	J60	J57	241	50	140	1.77	0.29	19.58
P126	J60	J59	193.5	110	140	-12.49	1.31	15.76
P127	J52	J59	159.2	50	140	0.3	0.15	0.74
P128	J59	J58	215.6	80	140	2.22	0.44	3.03
P129	J57	J58	244.3	20	140	-0.2	0.63	29.13
P130	J59	J66	377.5	110	140	-17.37	1.83	29.04
P131	J2	J81	116	80	140	4.99	0.99	13.6
P132	J1	J89	190.7	200	140	38.67	1.23	6.95
P133	J1	J83	58.96	80	140	7.82	1.56	31.23
P134	J83	J82	111	80	140	5.29	1.05	15.12
P135	J82	J81	159.9	50	140	0.86	0.44	5.21
P136	J83	J84	150.8	50	140	1.63	0.83	16.82
P137	J84	J85	153.7	20	140	0.13	0.26	12.83

P138	J82	J87	120.4	50	140	1.8	0.92	20.23
P139	J87	J85	136.6	50	140	0.63	0.32	2.88
P140	J85	J86	151.6	50	140	-0.18	0.29	0.3
P141	J81	J88	142.6	80	140	4.3	0.85	10.3
P142	J87	J88	151.2	50	140	-0.33	0.27	0.88
P143	J88	J86	246.2	80	140	1.75	0.35	1.96
PMP1	R1	Tank1	#N/A	#N/A	#N/A	35.75	0	-91.47

Annex B. Summary of Cost Estimation

S.N	General Items Description	Unit	Qty	Unit rate (Br)	Total cost(ETB)
1	Earth work				
1.1	Site clearing	m^2	625	50	31250
1.2	Trench Excavation				
A	0.6mX0.9mX1m	m^3	16561	10	165610
B	0.6mX1mX1m	m^3	1236.6	12	14839.2
Sub Total					190449.2
1.3	Backfill with excavated native soil, material				
A	0.6mX0.9mX1m	m^3	16561	5	82805
B	0.6mX1mX1m	m^3	1236.6	6	7419.6
Sub Total					90224.6
2	Supplying of pipes				
2.1	HDPE				
A	DN110 PN10	m	3278	79.8	261584.4
B	DN100 PN10	m	5168	76.73	396540.6
C	DN80 PN10	m	8508	57.6	490060.8
D	DN50 PN10	m	6240	50.23	313435.2
E	DN20 PN10	m	400	44.25	17700
Sub Total					1479321

2.2	UPVC				
A	DN100 PN10	m	365	32.5	11862.5
B	DN180 PN10	m	5410.39	33.2	179624.9
C	DN200 PN10	m	1205	33.4	40247
Sub Total					231734.4
2.3	DCI				
A	DN100	m	605.34	80.7	48850.94
B	DN180	m	235.61	85.9	20238.9
C	DN200	m	519.08	87.5	45419.5
Sub Total					114509.3
2.4	Cast Iron				
	DN100	m	135.64	60.5	8206.22
	DN110	m	297.48	65.2	19395.696
Sub Total					27601.916
3	Fittings ,valves& installation works				
A	HDPE				608179.83
B	UPVC				177539.745
C	DCI				320953.54
D	GS				115588.58
sub total					1222261.695
3.1	Surveying of pipeline	km	505.44	340	171849.6
Total					3372467.821
4	Boreholes				
4.1	Site surveying				50000
4.2	Mobilization of man power and materials				500000
4.3	Drilling up to finishing				1000000
4.4	Pumping test				72000
Total					7672000
5	Reservoirs				
5.1	Earth workReservoirs 1				

5.1.1	Site clearing to a depth of 30 cm	m^2	400	50	20000
5.1.2	Foundation excavation to depth not greater 1.5m	m^3	600	70	42000
5.2	Earth workReservoirs 2				
5.2.1	Site clearing to a depth of 30 cm	m^2	225	50	11250
5.2.2	Foundation excavation to depth not greater 1.5m	m^3	337.5	70	23625
Total					96875
5.3	Concrete material				
A	cement	Qntl	1440.1	250	3600024.375
B	sand	m^3	124.656	281.25	35059.528
C	aggregate	m^3	249.312	525	130888.8525
Sub total					3765972.756
5.4	Reinforcement bar				
5.4.1	Bar Ø10 mm	kg	36289.6	23	834660.892
5.4.2	Bar Ø12 mm	kg	8864.504	23	203883.592
5.4.3	Bar Ø14 mm	kg	509.96	23	11729.08
5.4.4	Bar Ø16 mm	kg	1133.38	23	26067.74
5.4.5	Bar Ø18 mm	kg	19933.67	23	458474.3065
5.4.6	Bar Ø20 mm	kg	31808.88	23	731604.286
Sub Total					2266419.897
6	Pump				
6.1	Submersible	No	7	300000	2100000
	Booster pump	No	2	240900	481800
sub total					2581800
7	Generator				
7.1	standby generator	No	2	250000	500000
sub total					500000
Total					18693412.99
others 15%					2804011.948

Ground total	**27,497,424.93**