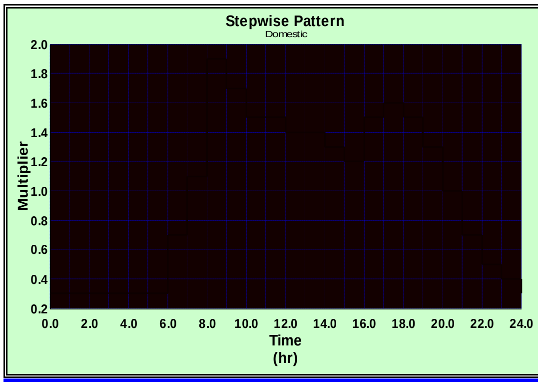
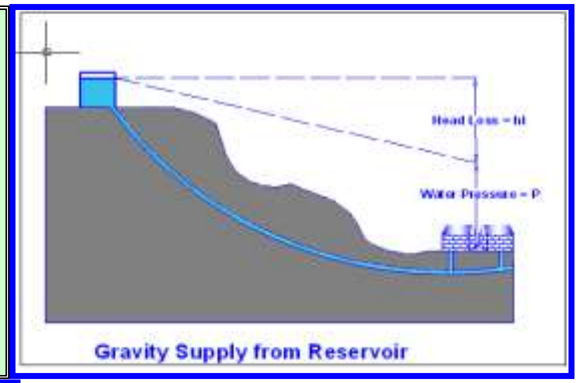
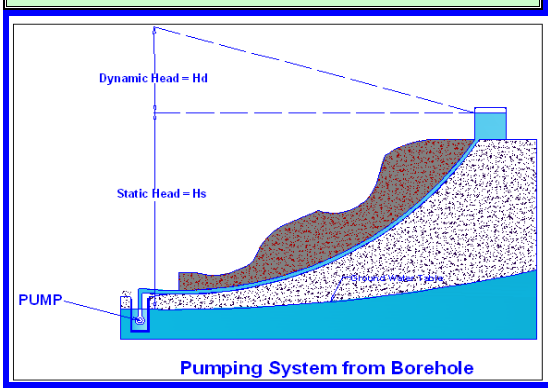
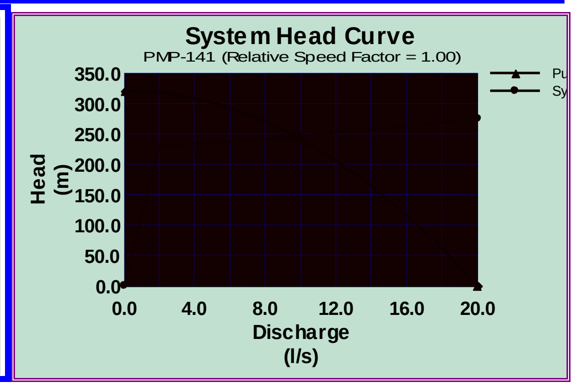
OROMIA WATER WORKS DESIGN AND SUPERVISION ENTERPRISE









DESIGN GUIDELINE FOR WATER SUPPLY PROJECTS

DECEMBER, 2008

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1 BACKGROUND

1.1 INTRODUCTION

This guide line has been prepared in response to the contract to train and build capacity of the client (OWDSE) engineers in the design and management of water supply projects. The guide line includes all procedures and design criteria required to design a water supply project in accordance with the terms of reference.

The design criteria and procedures contained in the guideline are a collection and compilation of predominantly National and Regional experiences in the design of rural and urban water supply projects. The designs reports of 12 Towns, 25 Towns, Environmental Support Project (ESP) Component3, Oromia Six Centers, Oromia Development Corridors, Mekele and Humera water supply projects have been reviewed in developing this guideline. The design and project manuals of ESRDF and World Bank prepared in context of Ethiopia have also been reviewed and taken into account. Lecture Notes prepared for higher level education courses in fields of applied hydraulics and design of water distribution system have been examined for developing conceptual and logical planning and design of water supply projects.

Definitions of project objectives and components, the development procedures and stages to be followed in the project preparation processes are outlined under section one (this Section).

The second section deals with planning and economic aspects of water supply projects. The recommended technical and economic design life of different components of a water supply project and the economic aspects to be considered in selecting alternative projects are addressed under this section.

The procedure and criteria to be followed to arrive at the water demands of consumers which are complex and basis for deciding project capacity and investments are described under section three.

The guideline and criteria to be used in designing different water supply system components are presented in Sections from four to seven. Concepts of hydraulics and engineering design guidelines are also incorporated for sizing, analyzing and implementation of water supply components.

Section eight elaborates how to estimate project costs i.e. both investment as well as operation costs and approaches to select the preferred project for detailed design stage.

Section nine outlines how to conduct detail design and prepare bid/construction documents for implementation of the project.

The basic design principles used for computer modeling of water distribution system are introduced in section ten. Further training materials used to model different components of water supply system are prepared in form of handouts and provided to trainees in due course as per the terms of reference.

The handouts to be prepared together with this guideline shall be used for practical on job design training and demonstration of Fedis-Midhega Lola water supply project. The latest version of WaterCAD which is the best and a powerful water distribution modeling software will be used for this training purpose. The contents of training courses to be prepared are mostly based on the design guideline of WaterCAD software itself. However the specific project database setup and distribution system schematization using other Softwares, such as Microsoft Excel, Digital Elevation Model (DEM), GIS and AutoCAD shall be organized separately within the handout.

The general procedures and contents of training courses will mainly focus on the following basic issues:

1. Orientation and introduction on WaterCAD software windows, training on how to build a water supply system network, steps for performing steady state and extended period simulation analyses and development of alternative scenario management systems.

2. Training on basic concepts of GIS, AutoCAD and DEM Softwares in the context of modeling water distribution network followed by project data collection and database creation using these Softwares and Topographic Maps.

3. Training on exporting and importing project data among different Softwares such as Microsoft Excel, AutoCAD and DEM formats for creation of water distribution network data to be used in WaterCAD software.

4. Training on data base connection from Microsoft Excel, AutoCAD and GIS files to WaterCAD files for efficient modeling of distribution network.

5. Training on data analysis problem analysis, rectification, validation and optimization of project designs

6. Report production and compilation using different methods of WaterCAD features

1.2 PROJECT OBJECTIVES AND COMPONENTS

The World Bank Technical Paper Number 12 defines the project and its objectives as follows:

a. The word ***Project*** refers to the entire set of actions to be taken to meet specific objectives. This involves the planning, design, construction and initial operation of physical facilities plus the provision of all other inputs needed to meet the objective of the project.

b. The ***Components*** are constituent parts of a project necessary to achieve its objectives. These components can be physical facilities (dams, treatment plants, pumping stations, pipe lines, etc.) and/or supporting activities (staff training, management assistance, etc.). Physical facilities are sometimes referred as "***Hardware***" supporting activities as ***"Software".***

In practice a project objective for urban centers is slightly different from rural due to differences in complexity, socio-economic conditions, standard of service levels and Government policies. The technical paper express Project objectives in two ways as follows:

1. ***General development objectives*** should include estimates of:

- Health improvements.
- Reduced burden in carrying water and expected impact, particularly on women and children (released time and energy).]
- Improved living standards.
- Pollution abatement.
- Staff development.
- Institutional improvements.

Another possible general objective for the project is to be used a model for replication by similar projects elsewhere in the country.

2. ***Operational objectives*** for the project concern improvements in service coverage and standards for water supply and sanitation systems. Each objectives should be quantified (to the extent practicable), and a schedule for achieving these objectives should prepared and implemented.

The Rural Water Supply Objective can be defined as follows:

- The objective of rural water supply project is to provide safe water, easily accessible, in quantities adequate for drinking, food preparation, personal hygiene, and sometimes small livestock, at a cost in keeping with the economic level of the communities and through facilities which can be easily operated and maintained at the local level.

1.3 OVER VIEW OF THE PROJECT PREPARATION PROCESS

The objectives stated under section 1.2 can only be achieved through careful planning and designing of water supply projects. All projects should then pass through a series of distinct stages between the initial idea and the time when the project is completed. However the number of stages, level of details and time required for preparation process may depend on the financing nature of the projects. For example projects which should seek funding from external agencies such as World Bank and African Development Bank are required to go through the following stages of development:

a. ***Identification*** (Awareness of Need for Improved Services and Assignment of Planning Responsibilities) and Preparation (Pre-Feasibility and Feasibility Studies) comprise the pre-investment planning stages which are discussed subsequently.

b. ***Approval*** (Appraisal and Investment Decision) is the stage where decision makers, including financiers, determine whether or not the project will be transformed from an idea into reality.

c. ***Implementation*** is the stage when detailed designs are completed and the project facilitates are built and commissioned. Supporting activities such as staff training are also underway.

d. ***Operation*** is when the project facilities are integrated with the existing system to provide improved services.

e. ***Evaluation***, the final stage, determines what lessons have been learned so that future projects can be improved accordingly.

Nationally and/or regionally funded Projects shall relatively require less number of stages and shorter period of planning process as the identification, screening, prioritizing and planning activities had already been done and included in the National and/or regional development plans. Once the projects had been included in the development plan, investments are secured on yearly bases and allotted for feasible projects. The need for identification and pre-feasibility stages of projects are then excluded to simplify and shorten the processes.

Practically all locally funded large scale water supply projects should pass thorough three important distinctive stages. Theses are

- The Feasibility and Preliminary Design Stage
- The Project Approval and Detailed Design Stage and
- The Project Implementation Stage

These three stages of project development are further summarized under the following series of activities:

1. Project Planning
2. Assessment of Physical Conditions in the Project Area
3. Assessment of Existing Water Supply and Population Served
4. The study of Social and Economic Conditions of the project Area
5. The Study of Water Resources in the project area
6. Assessment of Environmental and Social Impacts of the Project
7. Analysis of the Population and Water Demands
8. Formulating Design Criteria of the Project
9. Selection of Alternative Projects those could meet the Present and Future Water Requirements of the Project.
10. Preliminary Designing and Cost Estimation for Each of the Selected Alternatives.
11. Analysis of the Feasibility of each Alternative Project in Terms of Technical, Financial, Social, Economical And Environmental Factors, and Selecting the Most Preferred one for Final Design Stage
12. Carrying Out Details Designs and Preparation of Construction/Bid Documents for the Selected Water Supply Project.
13. Carrying Out Details Designs and Preparation of Construction/Bid Documents for the Selected Water Supply System.

Each of the above activities shall be more elaborated in the following sections. The Study and Design of Fedis-Midhega Lola Multi Village Water Supply Project shall be used as a case study and demonstration project to able to improve the designing skill of counterpart experts of the Lead Consultant.

2 PLANNING PHASE AND ECONOMIC ASPECT

2.1 GENERAL

This is a meaningful preparatory step before detailed consideration of design alternatives commence. A choice to commission water distribution systems is rather a political than the technical matter. It is therefore important that the people involved (decision makers, executers and users) do not have impression of careless investment. Assurance of this is the main purpose of planning.

The planning phase has to give answer to the following general questions.

a. Is the project feasible?

b. What is the best (global) approach, if more than one?

c. What are the estimated costs?

d. What is the required time for execution?

Looking for appropriate answer in this case is often a complex assignment in which experts of different profiles are included. Hence, a good organization of the work is an important initial element of planning.

The job normally starts by establishing the project management team with following main tasks.

a. To review the project

b. To survey the expertise and equipment required

c. To secure cooperation of involved organization

d. To formulate the objectives of the project with respect to the time, costs and quality

Conclusions are in the planning phase brought always with certain margin. This is obvious because 20-30 periods are sometimes quite unpredictable future (unexpected population growth and/or migration, natural disasters, wars, etc.). Due to that, it is wise to develop water distribution facilities in stages, following actual development of the area. This principle allows gradual accumulation of the funds for investment, as well as the intermediate evaluation and adaptation of the design where actual development deviates from the original planning. Thus, the planning phase is never fully completed before the design and execution will begin.

2.2 DESIGN PERIOD (PLANNING HORIZON)

The lifetime of a water supply system component are usually estimated in terms of its technical and economical period.

The ***technical lifetime*** of a system component represents the period of time it may operate satisfactorily in a technical sense.

Economic lifetime of a component refers to the predicted or expected time for which a particular part of equipment can be economically put into operation, which refers to the time until the cost of operating equipment exceeds what it can earn. It is usually much shorter than the technical lifetime. In practice, the economic lifetime is quite often used as the project design period.

The design period (planning horizon) is the length of time for which the system is expected to provide a community with good quality and sufficient quantity of water. This period should be neither too short, not less than ten years or too long because of economic reasons and the difficulty of predicting future water demand.

The design period also fixes the target date in the future for which a given project should serve before it is abandoned or expanded. The design period has a direct impact on the overall capacity, complexity as well as cost of water supply systems. It is the first task of the designer to fix the design period by considering the following issues:

a. Population growth rate

b. The economy of scale principle

c. Interest rates for the money to be borrowed

d. Lifetime water supply system components

In practice, the construction of urban water supply projects are mostly carried out in two phases in which each phase holds a deign period of 10 years and rural water supply systems are designed to accommodate the water requirements for a design period of 10 to 15 years depending on factors such as scope of the project, availability of fund and socio-economic condition of benefited communities.

The design lifetime of system components mostly adopted for financial/economic comparison of alternative projects are summarized in table 2.1 below.

TABLE 2.1 DESIGN LIFE OF WATER SUPPLY COMPONENTS

Items	Civil (years)	Mechanical and Electrical (years)
River Water Intakes	50	15
Boreholes	25	15
Impounding Dams	50	
Treatment Plant		
I. Clarifiers	50	15
II. Filtration	50	15
III. Chemical Dosing	50	10
Pumping Stations	50	15
Service Reservoirs		
I. Concrete	50	15
II. Pressed Steel	20	15
Pipelines		
I. Ductile iron pipes	40	
II. Steel pipes	40	
III. UPVC pipes	25	
Pipeline Fittings		25
Building	50	

(Source: Twelve Towns Water Supply and Sanitation Study Working Paper No 2 Design Criteria February 1994)

2.3 ECONOMICAL ASPECTS

Economic comparison of alternatives is very often decisive element for final choice. At the same time, this may be the most sensitive part of the whole project. All alternatives should be compared with in the same design period for all components, although the most economic design period for distinct component may be different. The important factors that influence the most economic design period are.

a. Interest Rate

b. Inflation Rate

c. Energy Prices

d. Water Demand Growth

e. The “Scale of Economy”

Economy of Scale in terms of business, it describes the possible or potential saving that can accrue to a company by increasing its size, its market share, diversifying, etc. for instance, in order to double its output without increasing its size or overhead, a company may utilize its equipment in a better manner or can buy better equipment, and thus, bring about a reduction in its unit cost

The "***Scale of Economy"*** in terms of engineering economy, it is a principle whereby with sufficient record of relevant component costs by a water supply company or a number of familiar companies, it is possible to find relation for the investment costs vs. Main characteristics of the component.

For instance, DHV (*ESP Component 3 Unit Costs)* has derived linear and exponential relations between Cost of Reservoir vs. Reservoir Volume and Cost of Pipes vs. Pipe diameters (*Table 2.2*) that can be used for comparison of options.

TABLE 2.2 UNIT COST OF RESERVOIR AND PIPES

$R_{Cost} = 918.8(Volume) + 109{,}501$	**Between 40 and 400 m3**
$R_{Cost} = 655.65(Volume) + 238{,}048$	Larger than 400 m3
$Cost/m = 0.0012(D)^2 + 0.627(D) + 23.677$	For uPVC
$Cost/m = 0.0012(D)^{1.3007}$	For DCI

(Source: ESP Component 3 Unit Costs)

A good comparison and selection of the most appropriate design period may be done using the present worth (value) or annual worth method.

By present worth method, all actual and future investments are calculated (back) to a reference year which is in general the year of the first investments. The alternative with the lowest present value is then the most economical solution. The basic parameter in calculation is single present worth factor ("$p_{n/r}$"):

$$P_{n/r} = \frac{1}{S_{n/r}} = \frac{1}{(1+r)^n} \qquad (2.1)$$

Where "$S_{n/r}$" is single compound amount factor and gives the growth of the present worth, "P_W", after "n" years with a compounded interest rate of "r". The present worth of future sum, "F", is obviously:

$$P_W = F \times P_{n/r} \qquad (2.2)$$

According to the annual worth method, a present principal sum, "P", is equivalent to a uniform series of "n" end-of- period sums, "A", whereby.

$$A = \frac{P \times r(1+r)^n}{(1+r)^n - 1} = P \times a_{n/r} \tag{2.3}$$

"$a_{n/r}$" is know as capital recovery factor or more simple annuity.

When present worth is calculated as PW = A/$a_{n/r}$ then "1/$a_{n/r}$" is called uniform present worth factor.

If instead of the interest rate per annum other interest periods are used, the correction should be introduced in the above equations. Furthermore, inclusion of so called ideal interest rate, "I", in Equ 2.1 and Equ 2.3 instead of true interest rate, "r", allows impact of inflation to be taken into account. Factor "f" in Equ.2.4 represents annual inflation rate.

$$I = \frac{1+r}{1+f} \tag{2.4}$$

Theory of engineering economy offers more sophisticated cost evaluation which cannot be further elaborated in this section. Hereby, a simplified principle of evaluation of the first investment and O&M costs for an alternative of transmission/Pumping main design is demonstrated. The present value includes investment (***First Cost, FC***). Thus:

$$PW = FC \tag{2.5}$$

The ***Annual Energy Cost*** required for pumping 1m^3/h of water against 1m head of H_2O through a unit m (1m) length of pipe is calculated as following

$$EC = \frac{\rho g Q H}{\eta} \times T \times e \tag{2.6}$$

$$EC = \frac{9.81 * Q * \Delta H}{\eta * 3600 * L} \times 24 * 365 \times e$$

Where "e" is unit price of energy, or per annum:

$$EC = 23.8 * Q * \frac{e}{\eta} * \frac{\Delta H}{L} \tag{2.7}$$

If "Q" is average flow in m3/h, and "η" is an average efficiency (energy cost in the equation is considered to be per meter length of the pipe).

By replacing hydraulic gradient "ΔH\L", from Equ.2.7 with e.g. Darcy-weisbach formula (assuming "λ" to be e.g. 0.010):

$$\frac{\Delta H}{L} = 0.010 * \frac{Q^2}{12.1 * D^5} \tag{2.8}$$

The energy cost per annum will be

$$EC = 32.5 * 10^{-10} * \frac{e}{\eta} * \frac{Q^3}{D^5} \tag{2.9}$$

And the total annual costs are then:

$$A = PW * a_{n/r} + EC = a * D * a_{n/r} + 32.5 * 10^{-10} * \frac{e}{\eta} \frac{Q^3}{D^5} \tag{2.10}$$

By adopting linear proportion between pipe diameter and its cost, Equ.2.9 has optimum solution if σA/σD = O, which results in the most economic diameter:

$$D = 0.05 * \sqrt{Q} * \sqrt[6]{\frac{e}{\eta * a_{n/r} * a}} \tag{2.11}$$

Equ.2.10 is simplified because it considers fixed energy cost and water demand over the design period. This hardly happens and the impact of increased water demand and energy cost has to be taken into account, which complicates further derivations.

3 POPULATION AND WATER DEMAND

3.1 POPULATION

Following the selection of proper design period, the design population shall be determined. The projected population growth during that time may be derived from available demographic data. Socio-economic factors should be taken into account when estimating the rate of population growth, such as family planning, migration and the level of economic prosperity.

In case of Ethiopia, the basic population data for the design has to be adopted from the census report of the Central Statistical Authority (CSA). Where possible the population data should incorporate classification based on the following issues.

a. Age and sex group

b. Economic activity

c. Religion

d. Education characteristics

Further, the average household size of the settlement should properly be estimated in order to determine the possible number of connections that the water supply service is expecting for different service levels. However, demographic data are not always reliable and should be checked against the actual population in the area to be served by the new system. If necessary, additional data should be collected.

It is essential that the water supply system is designed to meet the requirements of the population expected to be living in the community at the end of the design period by taking into account the design period and the annual growth rate (Table 3.1 and 3.2).

In the absence of CSA data, the design population can be calculated by multiplying the present population with a population growth factor as follows:

$$P_n = P_p(1+0.01r)^n \tag{3.1}$$

Where, P_n = Design population (after n years)

P_p = Present population (at the start of design period)

r = Annual population growth rate in %

n = Design period in years.

TABLE 3.1 URBAN POPULATION GROWTH RATES BY REGION

Region	Urban Population Growth Rate						
	2000	2005	2010	2015	2020	2025	2030
Addis Ababa	2.95%	2.96%	2.88%	2.64%	2.29%	1.90%	1.75%
AFFAR	3.84%	4.27%	3.94%	3.99%	3.62%	3.63%	3.48%
AMHARA	5.09%	4.53%	4.67%	4.41%	4.25%	4.05%	3.85%
B/GUMUZ	4.34%	4.65%	4.35%	4.51%	4.05%	3.99%	3.59%
DIRE DAWA	4.93%	4.40%	4.15%	3.78%	3.51%	3.27%	3.12%
GAMBELLA	5.15%	4.56%	4.10%	4.64%	4.01%	4.16%	3.78%
HARARI	4.19%	4.00%	3.63%	3.44%	3.25%	2.98%	2.99%
OROMIYA	5.29%	4.88%	4.74%	4.53%	4.32%	4.08%	3.84%
SNNPRS	5.50%	4.94%	4.70%	4.46%	4.25%	4.02%	3.77%
SOMALI	4.61%	4.65%	4.52%	4.39%	4.16%	3.90%	3.68%
TIGRAY	5.06%	4.63%	4.56%	4.41%	4.22%	4.04%	3.81%

(Source: ESP Component 3 Urban Planning Model December, 2001)

TABLE 3.2 RURAL POPULATION GROWTH RATES BY REGION

Region	Rural Population Growth Rate						
	2000	2005	2010	2015	2020	2025	2030
Addis Ababa	2.95%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
AFFAR	3.84%	2.06%	1.94%	1.72%	1.58%	1.39%	1.22%
AMHARA	5.09%	2.49%	2.33%	2.17%	1.98%	1.76%	1.52%
B/GUMUZ	4.34%	2.37%	2.26%	2.06%	1.95%	1.67%	1.50%
DIRE DAWA	4.93%	2.36%	1.92%	1.41%	1.00%	0.64%	0.31%
GAMBELLA	5.15%	2.19%	2.07%	1.79%	1.64%	1.37%	1.28%
HARARI	4.19%	2.71%	2.38%	2.13%	1.92%	1.57%	1.63%
OROMIYA	5.29%	2.65%	2.48%	2.24%	2.00%	1.72%	1.43%
SNNPRS	5.50%	2.80%	2.57%	2.31%	2.08%	1.82%	1.56%
SOMALI	4.61%	2.30%	2.21%	1.98%	1.69%	1.39%	1.10%
TIGRAY	5.06%	2.31%	2.14%	1.94%	1.71%	1.43%	1.14%

(Source: Ministry of Water Resources, ESP Component 3 Rural Planning Model January, 2001)

3.2 PHYSICAL DEVELOPMENT PLAN

In addition to the population figure the project area's physical development plan is the basis for water supply project design. For urban centers, a master plan or equivalent proposed plan of the project area is very important for demand forecasting as well as in laying out the different water supply components like the distribution line, service reservoirs, and pumping stations. A digitized map is the preferred option to implement computerized project design. In the absence of such digitized maps hard copies can also be digitized with the help of digitizers.

The master plan should represent the different land use plans of the project area and the proposed (expected) implementation periods. Specifically, the following land use types should clearly be demarcated for design of the water supply system.

- Road plans
- The residential quarters
- Industrial areas
- Commercial and public institution areas
- Green areas

For large scale and multi-village rural and/or town water supply projects, the following data can be used for the preliminary design and feasibility study of the project:

- Land use plan of the area if available
- Topographical maps
- Digital elevation model
- GIS data of the project area

The purposes and procedures how to use these data are further discussed in the later sections of water supply system configuration and schematization.

3.3 WATER DEMAND

3.3.1 GENERAL

Water is normally used for domestic purposes such as drinking, cooking, ablution, washing clothes and utensils and cleaning houses, and for non-domestic purposes such as public, commercial, industrial and fire fighting institutions, and livestock watering. In addition unaccounted for water should be considered while calculating the total water requirement of the system.

3.3.2 DOMESTIC WATER DEMAND

As earlier, domestic water demand includes water used for basic needs such as drinking, cooking, ablution, washing clothes and utensils and cleaning houses

The average amount of water used per person per day varies from country to country as well as from place to place within a country. The major important factors for these variations are:

a. Affordability and willingness of people to pay for water supply services

b. Level of water supply services to be provided

c. Cultural Practices

d. Climatic conditions

e. Level of socio economic development

f. Water Quality Standard and etc.

3.3.2.1 Level of Water Supply Services

As mentioned above the level of a water supply service greatly affects the water demand of the users. If the level of the service is excellent as house connection, the demand for water is also very high due to consumption for multipurpose such as toilet flushing, laundry machines and bathing rooms. The water demand of the users is decreasing as the level of the water supply decreases.

In general, the level of service to be provided to consumers depends on the social and economic development of the nation. Hence, host governments set a policy on what level of services to be provided to their people. For example, in our case, the government provides subsidies to communities who cannot afford to pay for basic services on capital costs.

Consequently, the following common three types of service levels have been adopted for urban centers of Ethiopia.

- House Connections (HC);
- Yard Connections (YC); And
- Public Fountains and Neighbor's Taps (PF)

Unlike the urban water supply service level, the rural water supply does not differentiate between service levels since there is only one type of service, i.e. individual water points; there are no private connections. However the number of users and distance to water points should be taken into account while deciding the service level of water supply for rural centers as shown in table below.

TABLE 3.3 WATER SUPPLY SERVICE LEVEL FOR RURAL CENTERS

No	Type of water supply scheme	Type of Water Point	No of users/WP	Max. Distance to Water Points (km)
1	Spring at point	Point	400	1.5
2	Spring by gravity	Public fountain (6faucets)	900	0.5
3	Spring motorized	Public fountain (6faucets)	900	0.5
4	Hand dug wells with hand pumps	Hand pump	300-350	1.0
5	Augur drilled wells with hand pump	Hand pump	300-350	1.0
6	Deep borehole with hand pump	Hand pump	300-350	1.0
7	Deep borehole with hand pump	Public fountain (6faucets)	900	0.5
8	Deep borehole with solar or wind pump	Public fountain (6faucets)	900	0.5
9	Infiltration galleries /sub surface dams	Public fountain (6faucets)	900	0.5
10	Difficult supply situation			1.5

Source: ESRDF Technical Design Manual December 1996:

3.3.2.2 Per Capita Water Demand

The amount of water used per person per day for daily life and activity is known as per capita water demand and it uses as a base for estimating the domestic water demand of communities. It is a function of daily basic needs listed under section 3.3.2 but should be adjusted by socioeconomic development and climatologic factors.

For urban centers, the design values obtained from past experience of 25 Towns and Oromia Six Centers Water Supply Projects can be adopted from the following tables.

TABLE 3.4 URBAN PER-CAPITA PER DAY WATER DEMAND IN LITERS (25 TOWNS WS PROJECT)

Activity	Year 2015			Year 2025		
	HC	YC	PF	HC	YC	PF
Drinking	1.5	1.5	1.5	1.5	1.5	1.5
Cooking	5.5	4	4	5.5	4	4
Ablution	12	9	7.5	14	10	9
Washing Dishes	6	5	2.5	8	5	4
Laundry	13	11	7	18	13	9
House cleaning	5	3.5	2.5	6	5	2.5
Bath or shower	20	12		24	14	
Toilet	12	4		12	6	
Other				1	1.5	
Total	**75**	**50**	**25**	**90**	**60**	**30**

TABLE 3.5 URBAN PER CAPITA PER DAY WATER DEMAND IN LITERS (OROMIA 6 CENTERS WSP)

Mode of Service	2004			2013			2023		
	HC	YC	PF	HC	YC	PF	HC	YC	PF
Drinking	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Cooking	5.5	3.5	3.5	5.5	4	3.5	5.5	4.5	3.5
Ablution	15	10	6	15	12	7	18	13	8
Washing Utensils	5	2	2	5.5	3	2	7	4	2.5
Laundry	15	8.5	7	16	12	7.5	18	15	8
House Cleaning	4	0.5	0	4.5	1	0.5	5	1.5	1.5
Bath or shower	20	4	0	27	10	0	32	13.5	0
Toilet	4	0	0	10	1.5	0	12	2	0
Other	0	0	0	0	0	0	1	0	0
Total	**70**	**30**	**20**	**85**	**45**	**22**	**100**	**55**	**25**

For rural water supply projects the following values of per capita per day water demand can be adopted taking into account the past experiences:

TABLE 3.6 RURAL PER CAPITA PER DAY WATER DEMAND IN LITERS WB (TECHNOLOGY OPTIONS)

Activity	Minimum	Average	Maximum
Drinking	1.5	1.5	1.5
Cooking	2.5	3.5	4.5
Ablutions	4	5	5
Washing Dishes	2	2	4
Laundry		3	5
House Cleaning			
Bath and Shower			
Toilets			
Total	**10**	**15**	**20**

Source: World Bank Technology Options and their Impact on Tariff

TABLE 3.7 RURAL PER CAPITA PER DAY WATER DEMAND IN LITERS (ESRDF)

Supply Type	Design Population	Average daily demand, l/c/day	
		Present	Projected to 15 years
Point springs/ hand pumps	All	15	20
Public fountains	Up to 5000	20	25
Public fountains	5001-10000	25	30

Source: ESRDF Technical Design Manual December 1996:

TABLE 3.8 RURAL PER CAPITA PER DAY WATER DEMAND IN LITERS (DEVELOPMENT CORRIDORS OF OROMIA)

Demand Category	Year				
	2007	2010	2013	2016	2020
YTU	15	17	19	21	25
PTU	10	11	12	13	15

Source: Development Corridors of Oromia East and West Hararge Zones Water Supply Project

3.3.2.3 Climatic and Socio- economic Adjustment Factors

As mentioned in the aforementioned sections the water used by a person depends on the climatic and socioeconomic conditions of the area. This means water is more used at hot area than cold ones. In addition reach people consume more water than poor ones. These are adjusted by multiplying the adjustment factors with total domestic demand. Climatic and Socio- economic Condition Adjustment Factors are presented in the following two tables respectively:

TABLE 3.9 CLIMATIC ADJUSTMENT FACTORS

Mean Annual Temp. (0C)	Description	Altitude	Factor	Examples
<10	Cool	>3,300	0.8	
10-15	Cool temperate	2,300-3,300	0.9	Goba
15-20	Temperate	1,500-2,300	1.0	Addis
20-25	Warm temperate	500-1,500	1.3	Metahara
25 and above	Hot	<500	1.5	Kebridehar

Source: Data Compilation and Analysis Project (1997)

TABLE 3.10 SOCIO-ECONOMIC ADJUSTMENT FACTORS

Group	Description	Factor
A	Towns enjoying high living standards and with high potential for development	1.10
B	Towns having a very high potential for development, but lower living standards at present	1.05
C	Towns under normal Ethiopian conditions	1.00
D	Advanced rural towns	0.90

Source: 9/25 Towns Water Supply Feasibility Study and Engineering Design Report (Oromia Towns)

3.3.2.4 Summary of Domestic Demand

Following the decision of per capita per day water demand, the percentage of population served by each mode of services i.e. HC, YC and PF has to be estimated for each respective year. The total domestic demand is then found by multiplying the per-capita per day demand with population served by each mode of service, adding the amount of water demand calculated for each mode of service and applying the socio-economic and climatic adjustment factors mentioned in tables 3.9 and 3.10 of sub section 3.3.2.3. The following table demonstrates how to arrive at a total domestic demand of a project.

TABLE 3.11 SAMPLE PROJECT FOR CALCULATION OF DOMESTIC DEMAND

Population/Service Levels	Unit	Year		
		2004	2013	2023
Population	No	46,101	69,398	105,411
Percentage of Population Served by:-				
HC	%	1.0	4.0	20.0
YC	%	24.0	30.0	40.0
PF	%	50.0	53.5	32.5
Population by Service Level				
HC	No	461	2,776	21,082
YC	No	11,064	20,819	42,164
PF	No	23,051	37,128	34,259
Per Capita Demand				
HC	lpcd	70	70	90
YC	lpcd	20	25	35
PF	lpcd	12	15	18
Demand by Service Standard				
HC	m3/d	32.27	194.32	1,897.38
YC	m3/d	221.28	520.48	1,475.74
PF	m3/d	276.61	556.92	616.66
Sub Total	m3/d	530.16	1,271.72	3,989.78
Climatic Factor (Say 1.3 for 20-250C) Socio-economic Factor (Say 1.05 for B)	Combined Factor 1.3*1.05=1.365	1.365	1.365	1.365
Total Domestic Water Demand	m3/d	723.67	1,735.89	5,446.05

3.3.3 NON DOMESTIC WATER DEMANDS

3.3.3.1 Industrial Water Demand

The development plans of many towns indicate that there are plans to establish some small to medium scale industries. In most cases big industries are assumed to have their own water supply systems. However, the following values can be taken for some industries.

TABLE 3.12 WATER DEMAND FOR SOME INDUSTRIES

Steel	**150 m^3/ton.**
Tannery	70 to 80 m^3/ton.
Garments, confectionery	50 m^3/ton
Biscuits, pasta, and similar	8 to 15 m^3/ton.
Rubber and synthetics	15 m^3/ton.
Concrete products	1 m^3/ton..
Soft drinks	15 litres/litre product.
Beer	10 litres/litre product.
Canned food	950 litres/can.

Source: Mekele Town Water Supply Development Project, Planning and Design Criteria

The following values can be used for the planning purpose in the absence of defined water demands for specified industries:

- 5m3 /ha/day for small industries
- 10m3 /ha/day for large industries

Where there is no land use plan for the project area, it is recommended to take 5 to 10% of the domestic water demand, depending on the size of the project.

3.3.3.2 Commercial and Public Water Demands

Commercial demand includes water requirement for restaurants, Cinema houses railways, bus stations, hoping centers, Local drinks (Teji, Areqe, Xela) etc where as institutional/public demand includes water required by schools, hospitals, public offices, military camps, public parks, dispensaries, day-care centers and so on.

In case where it is possible to estimate the number of public institutions and commercial services, the following water demand figures can be adopted:

TABLE 3.13 TYPICAL PUBLIC SPECIFIC WATER DEMAND

Category	Water Demand per Day
Day school	5 l/pupil
Boarding school	60 l/pupil
Hospitals	60 l/bed
Hostels	60 l/bed
Mosques and churches	5 l/visitor
Cinema Houses	5 l/seat
Public office	5 l/employee
Railway & Bus station	10 l/user
Public baths	60 l/visitor
Swimming Pool	500l/m3
Abattoir	100 l/cow
Military camps	60 l/person

Source: The World Bank Issue paper for project Design, Draft Final Report

In case where exhaustive estimation of the public and commercial institutions are not possible it is recommended to take 20 to 40% of the domestic water demand, depending on the size of population.

3.3.3.3 Livestock Water Demand

The livestock water demand is considered where there are no traditional sources such as rivers and streams available within a radius of 5 km from the area to be considered for water supply provision. Assumptions in estimating livestock water demand vary from consultant to consultant.

NOR consult (Feasibility Study and Engineering Design Report of 9 Towns) has determined the water requirements of in terms of a weight equivalent name called Tropical Livestock Unit (TLU). The average body weight of one TLU is 250 kg and on average, an animal consumes one liter of water per day for each 10 kg of body weight. Therefore, about 25 liters of water is required daily for each livestock unit. Hence, the daily water demand is estimated by calculating the equivalent TLU estimate of the domestic animals in the project area and multiplying the result with 25 liters.

In other cases the following figures are recommended for average per head per day water demands to be considered for different types of livestock:

TABLE 3.14 LIVESTOCK WATER DEMAND

Livestock Type	Consumption (Liter per animal per day)
Cow	60
Ox, Horse, Donkeys	40
Sheep, Goat	5
Chickens	0.2
Camel	2

Source: The World Bank Issue paper for project Design, Draft Final Report

3.3.3.4 Unaccounted For Water (UFW)

All water leakages in the system and unauthorized connections are categorized under UFW. This is usually taken as 15 to 25 % of the total demands which depends on the condition and management of the system.

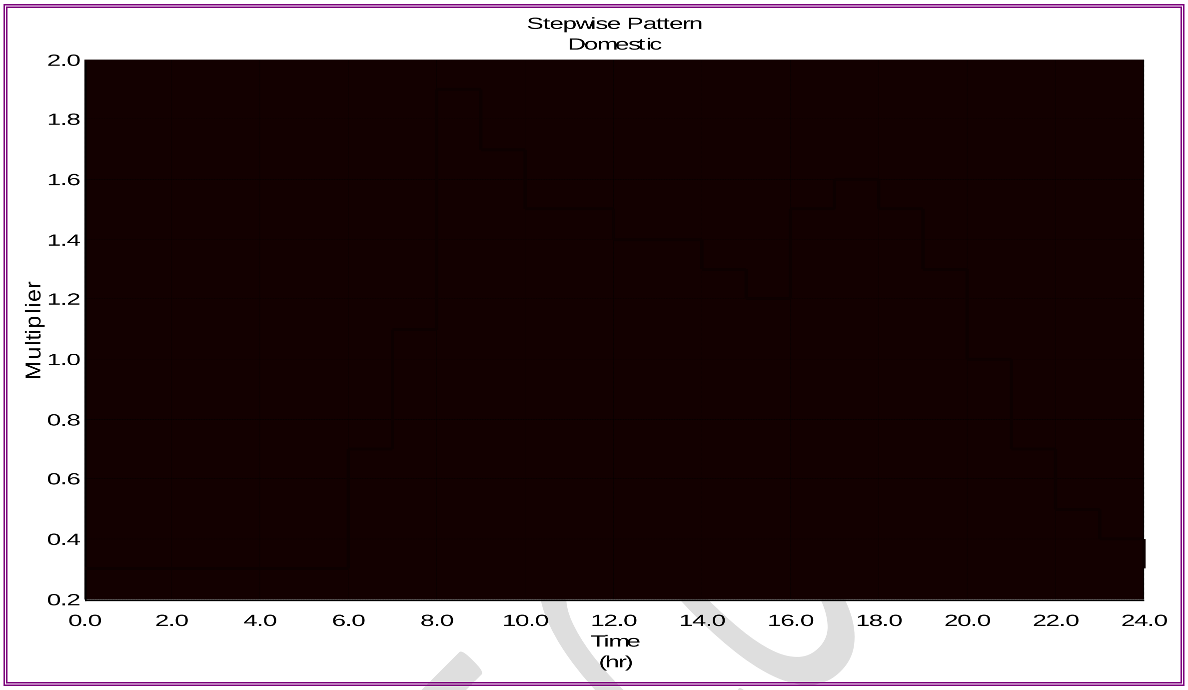
3.3.3.5 Water Demand for Fire Fighting

For assuring public safety, the provision of adequate fire demand is quite important. However unless there is a specific national and/or local regulation, water required for fire fighting shall be met by stopping supply to customers for the required time of fire suppression due to economic reasons. From past experiences, water demand for fire fighting is usually taken as 10 to 15% of the service reservoir volume.

3.3.4 CONSUMPTION PATTERNS AND PEAK FACTORS

The consumption patterns of different users of water supply services vary on hourly, daily and annual basis. Keeping records of these variations can help in developing standard peak factors for a given locality, which is the basis for the design of different water supply components. Poor estimation of these factors can lead to under or over design of water supply systems.

FIGURE 3.1 SAMPLE OF HOURLY CONSUMPTION PATTERN



Source: Humera Town Water Supply Project Feasibility Study and Detail Engineering Design Report

To design the different elements of a water supply scheme, the following demand type have to be considered. These are:

a. ***Average day demand***: this is obtained by simply summing up the domestic and non-domestic demands as well as unaccounted for water (*UFW*).

b. ***Maximum day demand***: is the highest demand of any one 24-hour period over any specified year. It represents the changes in demand with season and some special events happening in any specified year. The maximum day demand is obtained by multiplying the average day demand with the maximum day peak factor.

c. ***Peak hour demand***: is the highest demand of any one-hour over the maximum day. It represents the diurnal variations in water demand resulting from the behavioral patterns of the local population. The peak hour demand is obtained by multiplying the maximum day demand with the peak hour factor.

As stated earlier, both the maximum day and peak hour factors greatly depend on the size of the population to be served. It is always advisable to adopt peak factors developed from locally recorded consumption data or from a water services having similar climatic, cultural and socio-economic characteristics. In the absence of such data, the designer can flexibly adopt the peak factors given in the following table considering the local situation.

TABLE 3.15 RECOMMENDED WATER DEMAND PEAK FACTORS

Population size	Maximum day factor	Peak hour factor
<2,000	1.3-1.5	2.6
2,000-10,000		2.4-2.2
10,000-50,000		2.2-1.8
50,000-80,000	1.2	1.8-1.7
>80,000		<1.7

Source: The World Bank Issue paper for project Design, Draft Final Report

3.3.5 SUMMARY OF WATER DEMANDS

The total water demand is obtained by adding all domestic and non-domestic demands, and unaccounted for water and applying all demand factors mentioned in the previous sections. This is more illustrated by completing the sample project started in Table 3.11 of section as follows:

TABLE 3.16 SAMPLE PROJECT FOR CALCULATION OF TOTAL WATER DEMAND

Item	Unit	Year 2004	Year 2013	Year 2023
Population	No	46,101	69,398	105,411
Percentage of Population Served by:-				
HC	%	1.0	4.0	20.0
YC	%	24.0	30.0	40.0
PF	%	50.0	53.5	32.5
Population by Service Level				
HC	No	461	2,776	21,082
YC	No	11,064	20,819	42,164
PF	No	23,051	37,128	34,259
Per Capita Demand				
HC	lpcd	70	70	90
YC	lpcd	20	25	35
PF	lpcd	12	15	18
Demand by Service Standard				
HC	m3/d	32.27	194.32	1,897.38
YC	m3/d	221.28	520.48	1,475.74
PF	m3/d	276.61	556.92	616.66
Sub Total	m3/d	530.16	1,271.72	3,989.78
Climatic Factor (Say 1.3 for 20-250C) Socio-economic Factor (Say 1.05 for B)	1.3*1.05 =1.365	1.365	1.365	1.365
Total Domestic Water Demand	m3/d	723.67	1,735.89	5,446.05
Industrial Demand (Say 10%)	m3/d	72.37	173.59	544.61
Commercial and Public Demand (30%)	m3/d	217.10	520.77	1,633.82
Total Water Demand	m3/d	1,013.14	2,430.25	7,624.47
Unaccounted For Water (UFW)	%	15%	20%	25%
Unaccounted For Water (UFW)	m3/d	151.97	486.05	1,906.12
Average Day Water Demand	m3/d	1,165.11	2,916.30	9,530.59
Maximum Day Factor		1.3	1.2	1.2
Peak Hour Factor		2.2	1.8	1.7
Maximum Day Water Demand	m3/d	1,514.64	3,499.56	11,436.71
Maximum Day Water Demand	l/s	17.53	40.50	132.37
Peak Hour Water Demand	m3/d	2,563.24	5,249.33	16,202.01
Peak Hour Water Demand	l/s	29.67	60.76	187.52

4 WATER SUPPLY SOURCE AND TREATMENT

4.1 WATER SUPPLY SOURCES

A public water supply system must have sufficient source capacity to meet reliably the maximum day demands of its customers. For ground water wells, estimates of the source reliability (well capacity) can be determined by pumping tests, whereas spring or surface water capacity is more often determined by flow measurement and/or hydrologic assessments including use of appropriate safety factors for design.

4.2 TREATMENT PLANT

4.2.1 PLANT CAPACITY

Treatment plants should also be designed to meet the maximum day demands of customers. The type of treatment plant required to meet acceptable water quality standards should also be selected carefully based on the raw water quality parameters in question.

Water required treatment plant operation such as backwashing filters, cleaning treatment plants, mixing chemicals and water supply required for operators has to be considered in addition to water demands described under section 3.3 while designing treatment plants. This taken as about 2% for groundwater sources (boreholes) and 5 to 10 % for surface water sources.

Treatment plant facilities should have a minimum of two operation lines with each line to be designed at full capacity for enabling the operation of one line at required capacity while the other one being cleaned and maintained reset to operation.

4.3 GROUND WATER TREATMENT

Groundwater would usually require a minimum treatment that might include the following:

- Aeration, when iron content exceeds the maximum recommended level, or when low pH value is assumed to be caused by high CO_2 content.
- PH correction, when the final value does not meet the recommended value.
- Disinfecting, mostly by chlorinating.

In addition, protection measures are to be taken to safeguard boreholes from surrounding contamination; these measures could be:

- Clay envelops and concrete block plug tightly capping the borehole head.
- Securing the borehole within a fenced perimeter.
- Designating the well field as protected area, where only limited activities may take place.

4.4 SURFACE WATER TREATMENT

4.4.1 GENERAL

Based on the raw water quality parameters, surface water treatment may require a combination of alternative treatment processes. One of the alternatives might be either a conventional or unconventional treatment plant. The most common surface water treatment processes are the following:

High Cost Technologies

- Screening
- Pre-chlorination.
- Coagulation with or without pH correcting.
- Flocculation, usually comprising baffled channels.
- Clarification, to reduce the turbidity by precipitating of flocks.
- Filtration.
- Disinfection (Post chlorination).

Low Cost Technologies

- Sedimentation
- Horizontal Roughing Filtration
- Slow Sand Filtration
- Disinfection (Post chlorination).

4.4.2 CONVENTIONAL SURFACE WATER TREATMENT

Conventional surface water treatment plant may encompass the following processes:

a. ***Screening*** will be considered at the water intake and/or the treatment plant headwork, where it might be incorporated in a common structure used also to accommodate aeration (if necessary) and chemicals injection functions.

b. ***Pre-chlorination*** is considered to prevent development of algae on concrete surfaces. It might be achieved in a similar manner as the disinfecting means, using calcium hypochlorite solution at concentration of around 10%, to achieve final chlorine dosage of about 3 mg/l. The dosing rate will be about 0.1 to 0.2 lit of solution per m3*.*

c. ***Aeration*** might be applied in cases of high iron or CO2 concentration in the raw water in conjunction with the coagulation mixing. Cascade aerator would usually be considered*.*

d. ***Coagulation***, utilizing rapid mixing device of coagulating agent. Normally, alum sulphate will be considered as coagulating agent, prepared in solution tanks at concentration of around 10% prepared in two tanks of 12 hours working time each to achieve final dosage of, say, 40 to 100 mg/l.

e. ***Flocculation*** is usually achieved by passing the water through baffled channels providing the required minimum detention time (usually around 3 min).

f. ***Clarifiers*** will be provided to reduce the turbidity by precipitating the flocs. The type would be selected according to local conditions, convenience and economic considerations. In case of horizontal basins the velocity rate would be around 1.0 to 1.2 m/h; in case of vertical flow devices the upward velocity would be around 2.0 to 2.5 m/h.

g. ***Filtration*** basins (normally of rapid sand type) will separate the remaining flocs after the clarifiers. The filtration rate is usually around 6 to 8 m3/h/m2. The accumulated impurities will be removed periodically by backwash with treated water and compressed air. Usual backwash rates are 18 to 50 m3/h/m2 and the air flow rate is about 15 to 25 lit/sec/m2.

h. ***PH correction*** is usually achieved by adding lime slurry or soda ash solution; it may be applied in the mixing chamber of the coagulating agent, in the clear water tank or in both places. In case of lime, slurry concentration is usually about 5 to 10% in order to achieve final dosage of 25 to 50 mg/l.

i. ***Disinfecting (chlorinating)*** is usually carried out by using commercial grade calcium hypochlorite (bleaching powder) in concentrations as for pre-chlorinating. Dosage rates will as deemed appropriate to meet specific site conditions in order to reach final residual concentration of 0.5 mg/l chlorine.

4.4.3 ANCILLARY TREATMENT PLANT FACILITIES

The water treatment plant will have the necessary auxiliary facilities to allow its continuous operation. These may include:

- All weather access road.
- Fenced compound with sufficient drainage system
- Chemicals store and chemicals preparation building.
- Operations and laboratory building.

- Workshop.
- Guard house and operators' Dwelling
- Internal water supply and sanitary provisions.

4.5 WATER QUALITY

The water quality standard is one the important factors to be determined while designing any water supply system. The quality of water to be provided to consumers should be within the limits of the standards set by national or international organizations. The most important parameters to be considered are physical, chemical and bacteriological constituents of water.

Nowadays there is a national water quality standard to be used for designing a water supply system. WHO water quality standard was mostly adopted before the national standard had been established. The standard of treated water quality should meet either the national or WHO drinking water quality standards. The following table shows sample of WHO drinking water quality standards.

TABLE 4.1 SAMPLE OF WHO DRINKING WATER QUALITY STANDARD

Constituent	***Unit***	***Guideline Value***
Chloride	*mg/l Cl*	*250*
Color	*True Color Units (TCU)*	*15*
Fluoride	*mg/l F*	*1.5-2*
Hardness	*mg/l $CaCO_3$*	*500*
Iron	*mg/l Fe*	*0.3*
Manganese	*mg/l Mn*	*0.1*
pH		*6.5 - 8.5*
Sodium	*mg/l Na*	*200*
Solids, total dissolved	*mg/l*	*1,000*
Sulphate	*mg/l SO_4*	*400*
Taste and Odour		*Inoffensive to most customers*
Temperature		*No guidelines value set*
Turbidity	*Nephelometric Turbidity Units (NTU)*	*5*

Source: Mekele Water Supply Development Project, Planning and Design Criteria (TAHAL CE)

5 TRANSMISSION AND DISTRIBUTION MAINS

5.1 TRANSMISSION MAIN

The transmission main, although it may have a small number of service connections on it, it is used to convey the majority of flow from the source, treatment plant, and/or storage facilities to the distribution system where the majority of service connections are located. It is designed to transport the maximum day demand of water to service reservoirs and for peak hour demand from reservoir to distribution network. It should also be designed for peak hour demands where direct pumping is required from source and/or treatment plant to distribution system.

5.2 DISTRIBUTION MAIN

5.2.1 COMPONENTS OF DISTRIBUTION SYSTEM

Basically distribution system is divided into primary, secondary and tertiary mains which are defined as follows:

Primary Main: that part of the system which conveys water from reservoirs to secondary distribution pipelines. The capacity of primary distribution main is determined by the peak hour demand. Generally, all pipelines of DN 250 mm and above which are not transmission mains will be considered as part of primary distribution main.

Secondary Main: that part of the distribution systems which is fed by the primary pipelines and conveys water to consumers, either directly or through a tertiary main, or that forms a cross-connection between two or more primary mains. The secondary distribution main is designed for peak hour demand. Generally, pipelines of DN 150 and 200 mm are considered part of the secondary distribution main.

Tertiary Main: that part of the distribution system which is fed by main or secondary pipelines and conveys water to consumers through service connections. The tertiary distribution main is also designed to meet the peak hour demand consumers.

5.2.2 TYPES OF DISTRIBUTION SYSTEM

5.2.2.1 Gravity Distribution Systems:

The principal idea of this system is to make use of the existing topography without pumping and nevertheless under acceptable pressure. The advantages of gravity system are

- No energy costs
- Simple operation
- Low maintenance costs
- No sudden pressure changes
- The disadvantages are
- Gravity system is less flexible for future extensions
- Small gradients are available for friction losses which require larger diameters within the whole or some parts of the system
- Longer pipelines are necessary for following terrain configuration

5.2.2.2 Pumped Distribution Systems

Pumped supply systems operate without or with limited water storage (water towers) in the distribution system. These systems are common for distribution system on flat topography. As mentioned before, with direct pumping they have to follow variations in water demand. Proper selections of units have to be done in order to optimize energy consumption, including reserve pumping capacity for irregular situations.

Advantage of pumped systems are opposite of those for gravity systems. However, they are systems with rather complicated operation and maintenance (O&M) and dependant on reliable power supply. To provide permanent supply, additional precautions have to be taken (stock with spare parts, alternative source of power supply, etc.).

5.2.2.3 Combined Distribution Systems

Combined distribution systems operate with reservoirs and pumping stations. Storage in this case has considerable volume provided for balancing of daily variations in consumption, and as a buffer for irregular situations. These are the most common systems for large distribution areas.

5.2.2.4 Pressure Zones

The prevailing topography can lead to the use of so called pressure zones. These zones can be formed for economical and technical reasons. By establishing different pressure zone, saving can be obtained in supplying water to the various reservoirs (lower pumping costs) and in the application of lower-class piping due to the lower pressure.

Technically, pressure zones may be advantageous in preventing too high pressures in lower parts of the network (pressure reducing valves may be used) or providing sufficient pressures in higher parts (by pumping) when the source of supply is located in the lower zone.

5.3 HYDRAULIC DESIGN OF PIPES

5.3.1 DESIGN FORMULAE

5.3.1.1 Pressurized Flow in Pipes

Flow through pipes is generated by the energy difference between the two points or cross-sections (see figure 1 below). In hydraulic engineering practice, this flow is ***called pressure (closed conduit flow).*** Flow in the pressure pipes can be calculated from the Bernoulli equation. For steady and uniform flow conditions with negligible local losses, it can be written:

$$\Delta H = h_L \qquad (5.1)$$

Where: ΔH = Piezometric head difference between two cross-sections (mwc)

hL = Head loss (Friction) losses between two cross-sections (mwc)

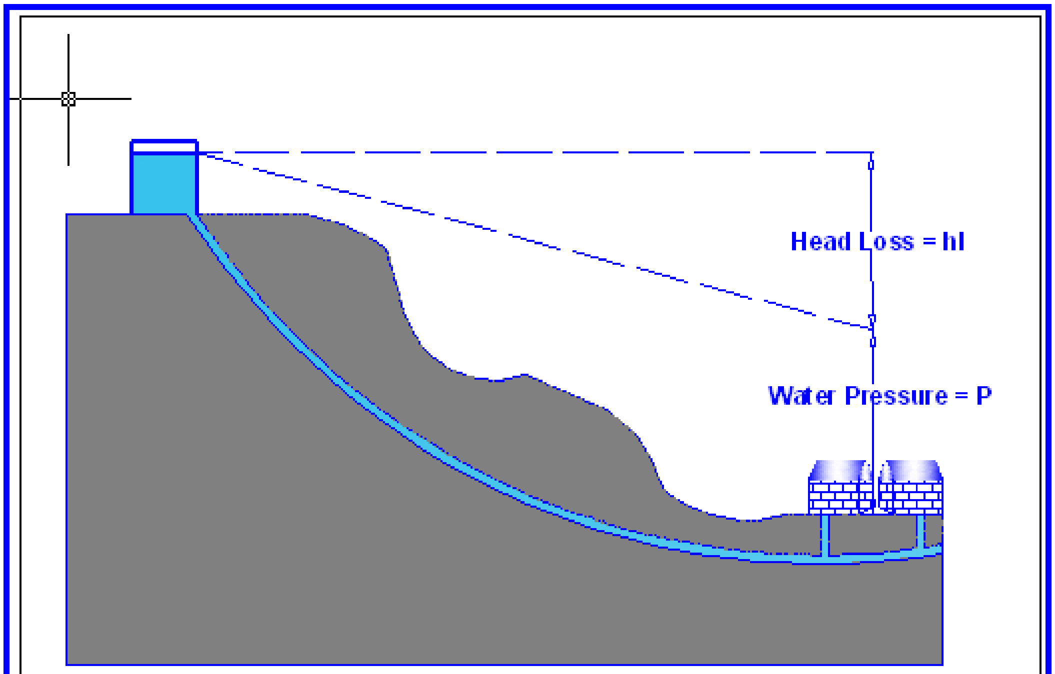
The magnitude of the friction loss in the above equation is proportional to the flow rate which is more elaborated in the following consecutive sections.

5.3.1.2 Head (Energy) Losses

A continuous resistance is exerted by the pipe walls during water flow. This resistance depends on the flow rate, pipe dimensions and internal roughness of the pipe material as well as from the fluid viscosity, and results in linear head degradation along the pipeline. A **head-loss (energy)** for a specified length is commonly referred to as **friction loss.**

There are several formulae for calculation of head losses. The most frequently used in the design of water supply system are Darcy-Weisbach and Hazen Wiliams formulae. The equations are stated as follows:

FIGURE 5.1 GRAVITY SUPPLY SYSTEMS



Gravity Supply from Reservoir

5.3.1.3 The DARCY-WEISBACH Formula

The formula is written as follows:

$$h_L = \lambda \frac{L}{D}\frac{V^2}{2g} \qquad (5.2)$$

Or

$$h_L = \lambda \frac{LQ^2}{12D^5} \qquad (5.3)$$

Where h_L		= Head loss in meter of water column, mwc
	l	= Darcy-Weisbach Friction factor (-)
	L	= Length of considered pipe section (m)
	D	= Diameter of pipe (m)
	V	= Velocity of flow through pipe (m/s)
	g	= Acceleration due to gravitational force (m/s^2)
	Q	= discharge (Flow rate) through pipe (m^3/s)

In the equation, h_L is the head (energy) loss due to friction in the length of pipe **L** of inside diameter **D** for average velocity **V**. The friction factor λ is a function of pipe roughness, velocity, pipe diameter and fluid viscosity. The Colebrook-White equation is used to iteratively calculate for the Darcy-Weisbach friction factor using the following formula:

$$\lambda = \frac{0.25}{Log^2\left(\frac{5.1286}{\mathrm{Re}^{0.89}} + \frac{k}{3.71D}\right)} \quad (5.4)$$

The dimensionless factor, "λ", is a representative of the pipe roughness. In practice it is usually in the range of ***0.01-0.04*** (generally smaller for larger diameters, new or /and smooth pipes).

For accurate calculation, "λ" is primarily determined by the type of flow condition, which is characterized by the **Reynolds number**, ***Re***:

$$\mathrm{Re} = \frac{VD}{\nu} \quad (5.5)$$

Where: n = Kinematics viscosity (m^2/s)

V = Velocity (m/s)

D = Pipe diameter (m)

The Velocity of flow through pipe is determined as follows:

$$Velocity = \frac{4Q}{D^2\pi} \quad (5.6)$$

Where Q = Discharge (Flow) m^3/s

V = Velocity (m/s)

D = Pipe diameter (m)

A good approximation in calculation of kinematic viscosity for water can be obtained by the following formula:

$$\nu = \frac{497*10^{-6}}{(T+42.5)^{1.5}} \quad (5.7)$$

Where T is the temperature of water in oC

As discussed above, the basic parameters involved in hydraulic calculation of pipes are:

- Length –L
- Diameter - D
- Absolute roughness – K
- Pipe discharge – Q
- Piezometric head difference, ΔH or head/friction loss, hL
- The water temperature, T

The derived parameters from a)-f) are the velocity, V(Q, D), the hydraulic gradient, S(ΔH, L), kinematic viscosity, ν(T), the Reynolds number, Re(V, D, ν), and the friction factor, λ(K, D, Re).

The problem of the above formula is, that "λ" is a function of the Reynolds number, i.e. fluid velocity and pipe diameter which (one of these two) is not known at the beginning of calculation. Hence, some iterative procedure has to be introduced by the assumption of unknown parameter (this can also be the λ-factor).

5.3.1.4 The HAZEN-WILLIAMS Formula

Probably the most popular formula in current use among water works engineers is the Hazen-Williams formula. The formula first published in 1904. (***AWWA M11 Steel Pipe a guide for Design and Installation).***

$$h_L = \frac{6.79L}{D^{1.16}}\left(\frac{V}{C}\right)^{1.85} \qquad (5.8)$$

The simplified form of the Formula by substituting the flow rate, Q is:

$$h_L = 10.67\frac{LQ^{1.85}}{C^{1.85}D^{4.87}} \qquad (5.9)$$

Where h_L		= Head loss in meter of water column
	V	= Average Velocity (m/s)
	C	= Hazen-Williams roughness coefficient (-)
	L	= Length of considered pipe section (m)
	D	= Diameter of pipe (m)
	Q	= discharge (Flow) through pipe (m^3/s)

Because of non-empirical origins, the Darcy-Weisbach equation is viewed by many engineers as the most accurate method for modeling friction losses. However, the Hazen Williams formula is widely used for manual design and analysis of a pipe line system for its less complexity than Darcy Weisbach.

In any case, the end result of any design is to select and quantify appropriate and economical pressure pipes and fittings that can deliver the required quantity of water and pressure to consumers provided that all other design criteria are met.

5.3.2 DESIGN CRITERIA

5.3.2.1 Typical Roughness Values

Typical pipe roughness values are shown below. These values may vary depending on the manufacturer, workmanship, age, and many other factors.

TABLE 5.1 COMPARATIVE PIPE ROUGHNESS VALUES

Material	Hazen William C	Darcy-Weisbach Roughness Height, k (mm)
Asbestos cement	140	0.0015
Brass	135	0.0015
Brick	100	0.6
Cast-iron, new	130	0.26
Concrete:		
Steel forms	140	0.18
Wooden forms	120	0.6
Centrifugally spun	135	0.36
Copper	135	0.0015
Corrugated metal	---	45
Galvanized iron	120	0.15
Glass	140	0.0015
Lead	135	0.0015
Plastic and uPVC	150	0.0015
Steel:		
Coal-tar enamel	148	0.0048
New unlined	145	0.045
Riveted	110	0.9
Wood stave	120	0.18

Source: 1995-2002 Haestad Methods. 1/14/02 (5.00032)

5.3.2.2 Pressure Requirements

Pressure in the distribution network to be as follows:

- As a rule, a minimum of 15 m manometric head is considered adequate during Peak Hour Demands. However, in exceptional and rural water supply cases, depending on the topography of the area, lower pressure levels may be permitted, but not less than 5 m.
- A maximum of 100 m manometric head, to avoid risking leaks and bursts in the distribution system, particularly during minimum flow conditions and when the static pressure would be dominant. If necessary, the distribution system is divided into separate pressure zones so that the maximum possible pressure does not exceed 100m.

Pipe pressure classes are chosen for the maximum pressure head that may occur under no or minimum consumption condition which is set at 10 percent of the average day demand and the service reservoir at maximum water level.

- Pipelines in the distribution system shall withstand a maximum operating pressure of 100 m manometric head and hence uPVC and/or PN10 class pipes can be used. In exceptional cases PN16 uPVC class pipes are used. These high pressure class pipes will only be applied where the water supply to certain areas require crossing of specific low lying valleys, etc. Any connection to these pipes will require pressure-reducing valves.
- Other pipe materials: GS pipes Class B and DCI pipes, Class K9 shall be used for transmission and distribution mains

5.3.2.3 Flow Velocities

Experience shows that in many cases pipes designed to flow velocities of, say, 0.8 to 1.2 m/sec are quite at optimum conditions for long lines; however, the following flow velocities will be acceptable:

- Minimum: 0.5 m/s.
- Maximum: 2 to 2.5 m/s.

Short sections, particularly at special cases, e.g. at inlet and outlet of pumps, may be designed for different values.

5.4 ENGINEERING DESIGN OF PIPE LINES

5.4.1 SELECTION OF PIPE MATERIALS

Pipes commonly used for water supply projects are Ductile Iron (DCI), Steel, uPVC, High Density Polyethylene (HDP) and Galvanized Iron (GI). The choice of pipe material is dependent on the following factors:

- Chemical nature of soil
- Chemical nature of water
- Comparative cost of alternatives pipes
- Weather conditions of the area
- Geologic formation of the pipe route
- Expected pressure in the pipeline

The following pipeline materials will be normally selected:

a. ***Metal pipes (DCI/Steel):*** will be laid where exposed above the ground and for special sections such as drain/stream crossings and/or other cases. The choice between DCI and Steel will be based on opportunity cost of the pipes and convenience of handling and pipeline formation.

b. ***Large diameter pipes*** (say, DN 400 mm and above) could be of metal (steel or DCI) depending on the opportunity cost of relevant pipes fabricated from the various materials.

c. ***uPVC and/or HDPE*** for the distribution system, with typical pipes diameters of DN 400 to 50 mm.

d. ***GI*** for service pipes of DN 2" to ¾" (50 mm to 20 mm).

5.4.2 ALIGNMENT AND LAYING OF PIPELINES

The following considerations will govern the alignment of pipelines within the supply area:

a. Transmission mains will follow the shortest route between the headwork and the supply area, allowing for deviations where necessitated by topographical conditions.

b. Wherever possible, pipelines will be laid at road sides and verges of footpaths, pavements or green strips. Pipes to be laid along roads will be located at a minimum distance of 900 mm outside from the edge of the road or the roadside drain.

c. Distribution system pipelines forming part of the main grid will follow the existing or planned roads, while observing the necessary requirements for hydraulic efficiency and economy.

d. Undesirable effects resulting from proximity of two metallic pipelines will be considered when routing new mains.

Laying depth of pipelines will be subject to the following criteria:

a. Depth of mains below ground will take into consideration ease of maintenance, avoidance of excessive earth pressure and protection from live load due to traffic.

b. Mains laid in trenches will have a minimum cover of 1.0 m for pipes of DN 400 mm and smaller, and 1.20 m for pipes of DN 400 mm and larger.

c. Mains laid under carriageways or road verges will have a minimum cover of 1.20 m.

d. The depth of cover will be increased as may be required where the ground level is to be changed in future for the construction of a road, where an increased depth is needed to maintain a minimum slope in the pipelines, where this will eliminate the need for an air valve, or where other special requirements call for greater depth.

A minimum distance of 1.00 m will be maintained from fences and buildings to the verge of the trench for pipe laying. Where this can not be maintained, special arrangement will be made on ad-hoc basis.

5.4.3 COORDINATION WITH OTHER UTILITIES

In the absence of records on existing underground utilities and structures, the following procedure will be adopted, to be coordinated by Contractor during construction:

- Collection of relevant information prior to preparation of construction drawings.
- Coordination with the concerned authorities prior to commencement of construction.
- Continuous coordination with the concerned authorities during construction.

Underground Cables

- Mains running in parallel with underground cables will be located at a minimum distance of 900 mm away from the cable.
- Mains crossing underground cables will generally be laid at a minimum depth of 900 mm below such cables.

Overhead Lines

- Mains running in parallel with or near overhead lines will have a clearance of at least 1.5 m between the base of poles and the wall of the pipe trench.
- The position of the pipeline relative to overhead lines will allow easy access for maintenance and repair.

Coordination with and Crossing of Drains, Sewers, etc

- Whenever a main runs parallel to a drain or sewer, a minimum distance of 1.5 m will be provided between the adjacent walls of the two trenches.
- Pipes to be laid along open drains will be laid at a distance of at least 900 mm from the nearest side of the drain.
- A pipeline located above a drain, sewer, culvert, etc., will have the following minimum vertical clearances:
- 300 mm for pipe sewers, culverts, etc.
- 600 mm above maximum water level of open drains, channel, stream, etc.
- Whenever a main crosses in the form of an inverted siphon under the drain, sewer, pipe or culvert, it will be protected by a concrete surround which will extend at least 4.00 m at either side of the crossing. In addition, an impermeable layer will be formed above the water supply pipeline, such as 600 mm of compacted clay (if available) or similar.
- Where over-crossing of pipeline may occur, the exposed pipeline section will be of steel.

5.4.4 PIPELINE ACCESSORIES

5.4.4.1 Isolating Valves

Spacing of Valves

- Isolating valves on mains will be installed at intervals of about 1.5 km, their spacing being dictated also by such factors as washout requirements, connection to consumers, connections to other mains, etc.
- Wherever a secondary main runs alongside a tertiary main, isolating valves on the secondary main will be installed at intervals of not more than 0.5 km, this being necessary to reduce the number of consumers affected by any failure in the artery. Other factors governing the spacing of valves on arteries will be washout requirements, connection to consumers, connection to other arteries or consumer mains, crossing of streams, roads, etc.
- Isolating valves on consumer pipelines will be provided at every branch connection, every street junction, and where indicated by special requirements.

Number of Valves

- The number of isolating valves to be installed in an adequately looped grid at every intersection of secondary, tertiary or consumer mains will be n–1, where n is the number of arms at the intersection.
- One isolating valve should be provided at every one of the following points:
- Interconnecting pipe.
- Bypass.

- Hydrant connection.
- Washout.
- Air valve.
- Consumer connection.

Size of Valves

- Isolating valves on mains of DN 450 mm and smaller will be of the same size as the relevant main.
- Isolating valves on mains of DN 500 mm and larger may be of a smaller diameter than the respective mains as a means of cost economy and reduced stock range. Suitable sizes of isolating valves will be as follows:
- On mains of DN 450 to 500 mm: DN 450 mm valves.
- On mains > DN 600 mm: (DN – 100 mm) valves.
- Isolating valves installed in branches for air valves, hydrants, washouts and bypasses will be of the same size as the respective branch pipes.

5.4.4.2 Air Valves

Air vents (air valves) for ingress and release of air will generally be provided at the highest point of mains DN 350 mm and above (or on mains of smaller size if required) or on the downstream side near isolating valves.

Double orifice kinetic type air valve will usually be selected for installation in pipelines. All air vents will be assembled with isolating valves of the same size. The flanged end of air valves and of isolating valves shall be as per BS. The sizes of air valves to be installed on mains are to be as shown in table below.

TABLE 5.2 SIZES OF AIR VALVES TO BE INSTALLED ON MAINS

Main Diameter	Air Valve Diameter.
Up to 450 mm	80 mm
500 – 600 mm	100 mm
650 – 900 mm	150 mm
1,000 – 1,200 mm	2 x 100 mm

5.4.4.3 Washouts and Hydrants

Washouts will be located at the lowest points of transfer pipelines and transfer/distribution mains, near drains, streams, etc., wherever suitable.

Drain pipes discharging to the drainage or sewerage ditch will be provided with a flap check valve installed on its end. Washout valves to be installed on mains are to be as shown in table below.

TABLE 5.3 SIZES OF WASHOUT VALVES TO BE INSTALLED ON MAINS

Main Diameter	Washout Valve Diameter.
Up to 450 mm	80 mm
500 – 600 mm	100 mm
700 – 900 mm	150 mm
1,000 – 1,200 mm	200 mm

5.4.4.4 Pressure Regulating Facilities

Excessive pressure will be relieved in appropriate manner. As the case be, it could be obtained, e.g. in the following manners:

- Mechanical device (pressure relief/reducing valve).
- Orifice (for transient flows).
- Pressure breaking tanks.

5.4.4.5 Fittings

Pipeline fittings (such as bends, tees, etc.) will be as follows:

- To be appropriate for the pipeline configuration. Normally fittings will be of Cast Iron, GS, DCI, or of uPVC where such lines are installed, similar in size and class to the pipelines will be installed.
- Wherever fittings for assembling steel pipes will be required, they are to have the same design strength as that of the pipe.

5.4.5 PIPELINE APPURTENANT STRUCTURES

Valve Chamber

- Concrete manholes or chambers are designed for each valve location for protection and to provide easy access for different purposes.
- Thrust Blocks
- Whenever the pipeline changes direction horizontally or vertically or changes in size; concrete thrust blocks are designed to resist the thrust force in the piping system.

Pipe Support

- Concrete supports for pipes are designed whenever the pipe is laid above ground surface and also in situations where the foundation formations are not good.
- Lateral transverse anchors are designed for conditions where pipe is laid in steep slopes.

6 PUMPING STATIONS

6.1 DESIGN FORMULAE

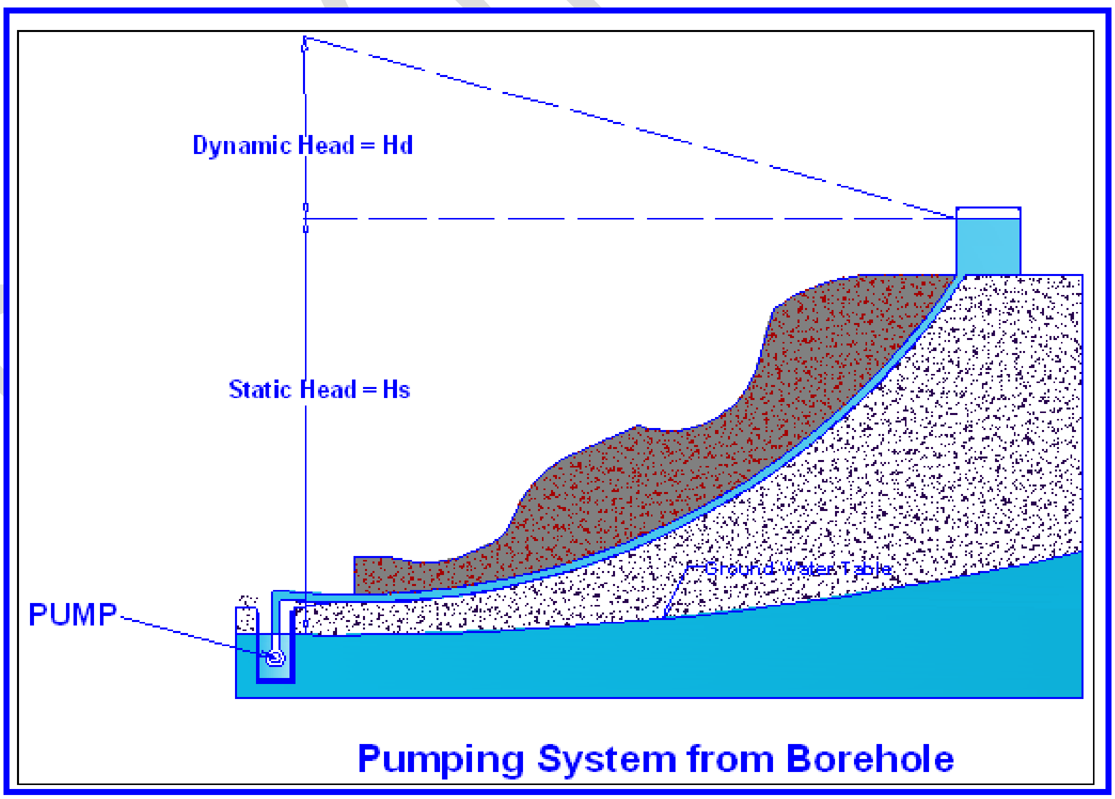
6.1.1 PUMPS

By installing pumps a certain amount of energy can be added to water flowing through pipes. This happens when mechanical energy of pump impeller is transformed to potential energy of flow. The amount of energy delivered is usually expressed as **head of water column** (**mwc**) called **Pumping Head** or **Pump Lift**.

It should be noticed that the pumping head is the difference of energy levels between the pump entrance (suction pipe) and the pump exit (discharge pipe). However, in many cases the pumping head is so high that the pump lift can be assumed to be equal to the pressure difference instead of energy difference ($H >> v^2/2g$).

When pumping water from one reservoir (Borehole, wet well, Booster station, etc.) to a higher elevated reservoir, the required pumping head, "**H**", consists of a **static head, "H_{st}"**, and the head of the transport system, also called **dynamic head, "H_{dyn}"** (**See Figure 2 below**).

FIGURE 6.1 PUMPING SYSTEM



The total pumping head, H some times called total manometer head is the sum of a static and dynamic heads as shown below:

$$H = H_{ST} + H_{DY} \tag{6.1}$$

The static head is the elevation difference between water levels in the reservoir and the section side of the pump.

The dynamic head is variable for variable flow, caused by the friction in the piping and the turbulence losses due to a change in velocities at valves bends, entrance, exit, etc. the dynamic head is calculated by head loss equation no 3.4 or 3.9.

The power of the water is:

$$N = \rho g Q H \tag{6.2}$$

Where N = Power of water (Watt)

ρ = Density of Water (Kg/m3)

g = Acceleration due to gravitational force (m/s2)

Q = Discharge (Flow rate) to be pumped (m3/s)

H = Total Pumping Head

The power required to lift the water is somewhat higher than the water due to energy losses in the pump:

$$N_P = \frac{\rho g Q H}{\eta_p} \tag{6.3}$$

Where "ηp" is pump efficiency dependant on pump design and working regime.

Finally, the power required to drive the pump or the motor power will be:

$$N_M = \frac{\rho g Q H}{\eta_p \eta_m} \tag{6.4}$$

Where "ηm" is motor efficiency

For Q in **l/s** and power in **KW**, the above equations can be simplified as follows:

$$N_P = \frac{QH}{102\eta_p} \quad \text{And} \quad N_M = \frac{QH}{102\eta_p\eta_m} \qquad (6.5)$$

The relation between the discharge and required head for a transport system is called system characteristics.

6.1.2 POWER SUPPLY

The power required to drive the pump is mostly delivered either from national grid system of electric power or from locally installed diesel driven generators. In case of generator, its capacity is determined by the power of pump motors. Local climatic factors such as temperature and altitude that have impacts on the operation of generators should also be taken into account while designing the components.

The power required to start a pump is assumed to be twice of the amount required for operation of the pump. Taking into account the efficiency of diesel engine to be η_d and power factor of **cosφ** the power of generator is calculated as follows:

$$N_d = 2\frac{QH}{102\eta_p\eta_m\eta_d} \qquad (6.6)$$

Where N_d = Power of Diesel Engine Generator (KVA)

6.2 DESIGN OF PUMPING STATIONS

6.2.1 DESIGN CAPACITY

Pumping stations are designed for the following capacities:

- It is designed to pump the maximum day demand of water when pumping is required from source and/or treatment plant to service reservoirs
- It is designed to pump the peak hour demand of water when pumping is required directing from source or treatment plant to the distribution system. In this case, it should also meet the required minimum pressure of 15m head within the distribution network during Peak Hour Demands.

6.2.2 BOREHOLE SUBMERSIBLE PUMPS

Deep boreholes will be provided with submersible pumps. In selecting pumps the following factors will be considered:

- Water quality.
- Hydraulic characteristics of the system.
- Groundwater drawdown parameters.
- Mechanical\electrical efficiency.
- Economic considerations.
- Standardization of equipment and familiarity by the operators.

The design working capacity of pumps (duty point) will be determined taking into account the system requirement, number of boreholes operated simultaneously, and a safety margin flow rate of 5 to 10 %. The pump characteristics will allow working in a range between 2/3 to 1.5 times of the nominal discharge at the design duty point. The duty point of pumps expected to work in parallel with others will take into account the effect of the combined pumps' duty on the resistance head of the system.

Electric motor ratings will take into account the said working range, and be at least 20% higher than the maximum calculated power requirement at any point in the said working range.

The available system NPSH at the maximum flow rate will exceed by at least 1.0 m over the required pump manufacturer's NPSH.

The pumps will be provided with discharge valve, non-return valve, air vent and pressure gauge(s). It will have low level protection device or flow switch on the main circuit. Mechanical flow meters (water meters) will be installed in the borehole head installation. Borehole head installation will have arrangement for measuring the water level in the well.

6.2.3 SURFACE CENTRIFUGAL PUMPING STATIONS

6.2.3.1 Type of Pumps

All the pumps will have electrically driven motors and will be of end suction type (if available for the rated point) due to the simplicity of design and robustness. The recommended type will be with spacer coupling, enabling the dismantling of the pump impeller without dismantling the motor or the suction piping. The pumps and motors will be assembled on a common steel support, installed on a concrete platform.

6.2.3.2 Pump Station Piping

The internal pipe work of pump stations will be made of steel. The following design criteria will be followed in terms of velocities through the pipes:

- Suction manifold: 1 - 2.0 m/s
- Suction branch of pump: 1.2 - 2 m/s
- Discharge branch of pump: 2 - 3 m/s
- Discharge manifold: 2 - 3 m/s

6.2.3.3 Pump Station Fittings

The following types of fittings will be installed:

- Valves: Full bore type, gate type (for diameters of 0 - 300 mm) or butterfly type (for diameters above 350 mm).
- The valves will be manually actuated. Gate valves will be of flanged type, wedge type (without sealing), no rising stem. Butterfly valves will be flanged or wafer type, and they will include a worm gear type actuator.
- Non-return valves: Full bore type, flanged type, swing or tilting type including externally protruded shaft.
- Air valves: Flanged, double orifice type. Air valves will be of cast iron or bronze made, they shall be isolated using the same diameter (as the air valve) isolating gate valve.
- Pressure gages: Mechanical, Bourdon type elastic element. They will be installed by means of three way cocks, enabling the drainage of the pressure gauge.
- Flow meters: Mechanical turbine type, allowing the dismantling of the meter without the dismantling of the body for diameters up to DN 400 mm, and ultrasonic type for diameters above 400 mm.
- Mechanical joints: Flanged type, of the following sub-types:
  - Suction pipes: not withstanding axial thrust (flange adaptors type or equivalent).
  - Discharge pipes: withstanding axial thrust (self restrained dismantling joint or equivalent).

All the accessories installed outside the pump station structure will be located in concrete open chambers provided with suitable access and drainage.

6.2.3.4 Protection, Control, Automation

The following protections will be provided:

- Low level in the suction tank (level indicator/ transmitter).
- Low pressure in the suction main (pressure switch on suction main), to protect the pumps against running dry.
- High pressure in the rising main (pressure switch on discharge manifold), to stop the pumps when the float valve of the discharge tank has closed.
- Low pressure in the rising main (pressure switch on discharge manifold), to stop the pumps when the rising main is broken.
- Electric protections (including phase sequence, over/under voltage, over load, over heating, etc.).

6.2.3.5 Auxiliary Installations

Monorail

A monorail will be installed in each pump stations in order to enable easy installation /dismantling of the pump sets. The monorail will be installed over the centre of gravity of the pump + motor assembly. It will be designed at about 1.5 times the weight of the pump + motor assembly.

The monorail will have the following characteristics:

- Trolley: Manual, actuated by chain, operated from the pump station floor, traveling on I beam profile.
- Hoist (chain block): Manual, actuated by chain, operated from the pump station floor.

Bulk Diesel Fuel Tank

It will include:

- Bulk diesel fuel tank.
- Piping and ancillaries.

The bulk diesel fuel tank will have the following characteristics:

- Type: Steel horizontal cylinder with conical or torispherical ends.
- Installation: Above ground, inside reinforced concrete chamber.
- Capacity: Will be established in each specific case according to the following criteria:
    - Only one pump will be driven.

- Time: 5 days (120 hours) of full electric load of the generating set.
- Specific consumption: 0.25 l/KWh

The bulk diesel fuel tank will be provided with the following elements:

- DN 500 mm flanged manhole.
- DN 50 mm pipes for filling / return.
- DN 50 mm pipe for hand / electric transfer pump outlet.
- DN 50 mm pipe for level gauge.
- DN 50 mm vent.
- DN 50 mm drain pipe.

The piping and ancillaries shall include:

- 2 pipes between the generating set and the bulk fuel tank (supply and return).
- 1 manual or electrical transfer pump from the bulk diesel fuel tank to the daily tank of the generating set.

7 RESERVOIRS

7.1 GENERAL

Ground level reservoirs (GLRs) and elevated water tanks (EWTs) are used in water supply systems. Whenever the local topographical conditions permit, ground level reservoirs will be preferred.

Ground level reservoirs will be of cylindrical reinforced concrete or stone masonry structure.

Elevated water tanks could be:

- Cylindrical or conical reinforced concrete structure;
- Prefabricated or welded steel tank on a steel struts support structure.

A cost analysis will determine the type of tank recommended in each case. For equal (or almost equal) costs, reinforced concrete structures will be preferred.

Operational reservoirs are intended to command the distribution system of the town and will be located at elevations providing the required pressures for water flows within the system. They will also have sufficient storage to cover the difference between hourly peak demand and actual supply from the source, fire fighting demands and emergency volume in case of power breakdown, repairs or O&M activities.

7.2 STORAGE CAPACITY

The criteria for storage capacity sizing will be as follows:

7.2.1 SERVICE RESERVOIRS

As mentioned in section 7.1, the storage volume required for service reservoirs have to be large enough to accommodate the cumulative differences between water supply and demands plus 10% reserve for fire fighting and additional provision for emergency cases such as power interruptions and repair works. Accordingly, the capacity of service reservoirs are determined either by mass curve analysis for urban centers *(See Table 7.1)* or by calculating one third of maximum day demand plus the required provisions for fire and emergency cases.

Transfer Reservoirs

- Transfer reservoirs that are used for pumping station will be sized based for 1 hours of maximum day demand.
- Reservoirs that provide transfer via gravity (break pressure tanks), if necessary, will be sized for 30 minutes storage of the maximum day demand.

TABLE 7.1 COMPUTATION OF RESERVOIR CAPACITY BY MASS CURVE ANALYSIS

Time (Hour)	Consum Pattern	Water Consum (m3)	Cumulative Water Consum (m3)	Water Producti on (m3)	Cumulative Water Production (m3)	Difference (m3)
00-01	0.3	101	101	338	338	237
01-02	0.3	101	203	338	676	473
02-03	0.3	101	304	338	1,015	710
03-04	0.3	101	406	338	1,353	947
04-05	0.3	101	507	338	1,691	1,184
05-06	0.7	237	744	338	2,029	1,285
06-07	1.1	372	1,116	338	2,367	1,251
07-08	1.9	643	1,758	338	2,705	947
08-09	1.7	575	2,333	338	3,044	710
09-10	1.5	507	2,841	338	3,382	541
10-11	1.5	507	3,348	338	3,720	372
11-12	1.4	473	3,821	338	4,058	237
12-13	1.4	473	4,295	338	4,396	101
13-14	1.3	440	4,734	338	4,734	0
14-15	1.2	406	5,140	338	5,073	-68
15-16	1.5	507	5,647	338	5,411	-237
16-17	1.6	541	6,188	338	5,749	-440
17-18	1.5	507	6,696	338	6,087	-609
18-19	1.3	440	7,135	338	6,425	-710
19-20	1	338	7,474	338	6,763	-710
20-21	0.7	237	7,710	338	7,102	-609
21-22	0.5	169	7,879	338	7,440	-440
22-23	0.4	135	8,015	338	7,778	-237
23-24	0.3	101	8,116	338	8,116	0
A	Volume for Balancing Supply & Demand (Σ(Max_{5hr}+Min_{20hr}) Cum. Values)					1,995
B			Storage Required for Fire Fighting (10% of A)			200
C			Storage Required for Emergency Supply (15% of A)			299
D			Total Reservoir Capacity (A+B+C)			2,494
E			Recommended Reservoir Volume			2,500

Source: Humera Town Water Supply Project Feasibility Study and Detail Engineering Design Report

7.3 PIPE CONNECTIONS

7.3.1 OPERATING OBJECTIVES

The reservoir pipe work will be able to perform the following:

- Stop manually the flow upstream of the reservoir.
- Stop manually the flow downstream of the reservoir:
- Automatically in case of high water level in the downstream reservoir to avoid overflow.
- Manually in order to isolate the reservoir.
    - Measure the outlet flow from the reservoir.
    - Measure the water level inside the reservoir.
    - Limit and /or adjust the incoming flow.
    - Convey the overflow water at a convenient recipient through drain pipe downstream the isolating valve.

7.3.2 PIPE WORK CHARACTERISTICS

Inlet Pipe

- Water velocity through the pipe: 1-2 m/s.
- The water will be delivered at the top of the reservoir, approximately at the maximum water level (MWL).
- Accessories:
    - On the main supplying the reservoir: No accessories.
    - On the inlet pipe of each reservoir: Flow meter,
    - Isolating valve,
    - Control orifice, if required,
    - Bypass valve,
    - Float valve or control valve.

The float valve or control valve will have the role of automatically closing the water access to the reservoir in case of high water level, to avoid overflow.

The accessories will comprise:

- Float valve: mechanically actuated (by floating device), installed in the reservoir for inlet pipes up to DN 300 mm
- Control valve: hydraulically actuated, installed in a valve chamber for the inlet pipes above DN 350 mm.
- Isolating valve:
    - Gate type, manually actuated for inlet pipes up to DN 300 mm.
    - Butterfly type, manually actuated for inlet pipes above DN 350 mm.

Outlet Pipe

- Water velocity through the pipe: 1-2 m/s.
- To be located at the bottom of the reservoir, able to deliver water if the level in the reservoir is anywhere above the minimum water level (mWL).
- Accessories:
    - On the outlet pipe of each reservoir: Isolating valve.
    - On each the outlet pipe: Isolating valve.
    - Flow meter.

The accessories will be installed in chambers.

The isolating valves will be installed upstream the flow meter. The isolating valves will be of the types as described above.

Overflow Pipe

Bell mouthed pipe, with the upper lip located 50 mm above the MWL. The diameter of the pipe will be calculated in order to convey the maximum flow rate which can enter the reservoir in the most adverse hydraulic conditions, namely when:

- The other reservoirs are isolated.
- The by-pass valve is open.
- The float valve or control valve are not operable.

Drain Pipe

The drain pipe will be able to collect water being at any level, even between mWL and the bottom of the reservoir. The pipe will be connected to the overflow pipe and will be provided with an isolating valve similar to the above.

8 ESTIMATING COSTS OF WATER SUPPLY PROJECTS

8.1 GENERAL

Two major stages can be identified when designing a water supply for a particular location. In the first stage, decisions must be made about:

- System capacity of main components and sizing of the water supply system which has been described in the former sections.
- Estimated construction and operating costs which will be described in the following sections.

The outcome of the first stage may be used as a basis for fund raising, planning and organizational purposes. The second stage is the preparation of the system's structural design and a specification of the equipment and materials to be involved.

In general, two cost components have to be differentiated in any water supply project. These are the investment (capital) cost and the Operation & Maintenance (O & M) costs. Estimating the capital and O & M costs is the basis for the financial analysis. The level of detail of in a cost estimate depends on the planning stage of the scheme.

8.2 INVESTMENT (CAPITAL) COST

Investment costs are costs that are incurred for the design and construction of water supply projects. Based on the sizes and layout of different water supply components determined in the design, the amount of materials and equipment to be supplied as well as the volume of work to be undertaken should be clearly stated in the bill of quantity. Further the supply and installation work volumes and cost of the following components should be separately given:

8.2.1 ENGINEERING SERVICE

The cost required for engineering service includes:

- Costs associated with project identification, pre-feasibility and feasibility studies
- Costs required for project appraisal and detail engineering design and
- Costs for contract administration and supervision tasks

8.2.2 BOREHOLE DRILLING, TESTING AND COMMISSIONING

This includes all costs related to the following tasks:

- Mobilization of Drilling Rigs, Personnel and All Necessary Equipment and Materials
- Drilling of the Boreholes up to required depth
- Installation of Casings, Gravel Packing and Well Development
- Pumping and Water Quality Tests
- Wellhead Construction, Drilling Report Production and Commissioning

8.2.3 SUPPLY OF PIPES AND FITTINGS

It includes costs required to manufacture, supply, testing and commissioning of all Pipes and Fitting such as:

- All different types of pipes and fittings such as DCI, Steel, uPVC, Galvanized Iron, etc.
- Different types of Valves such as Air Release Valves, Pressure Reducing Valves, Gate Valves, Non-Return Valves and other valves which are mainly used for transmission and distribution networks
- All Water Meters including fitting and small size meters used for service connections

8.2.4 SUPPLY AND INSTALLATION OF ELECTRICAL AND MECHANICAL EQUIPMENT

It includes costs required to manufacture, supply, erection, testing and commissioning of all electrical and mechanical equipment such as:

- Pumps (Submersible, Surface Centrifugal and other Pumps)
- Generating Sets and/or Transformers
- All Electro-Mechanical Equipment and Fittings Required for Treatment Plants, Pumping Stations, Reservoirs, Dam and Intake works
- All Electrical and Mechanical Control Systems and etc.
- Equipment and Materials for Power Supply

8.2.5 CIVIL WORKS CONSTRUCTION

It includes the provision of all necessary materials, equipment and manpower, and the erection, testing and commissioning of all civil work structures such as:

- Dams, diversion weir and intake structures
- Pumping stations
- Treatment plants
- Laying of Transmission and Distribution pipes and fittings
- Water reservoirs, Water delivery points and all service connections
- Auxiliary buildings used for Operation and Management of the water supply system
- Access roads and other infrastructures

The unit rates to be used for cost estimation should be site specific and as much as possible has to reflect the current market price. Recently completed projects can also be adopted to get reasonable estimation of costs.

8.3 OPERATION AND MAINTENANCE COSTS

The operation and Maintenance costs of a water supply system are summarized in the following sections.

8.3.1 OPERATION

Operation costs are costs associated with water production, distribution, bill preparation, revenue collection etc. These costs are accounted as follows:

- Chemical costs
- Energy costs
- Personnel and other administrative costs

8.3.2 MAINTENANCE, REPAIR AND CONTINGENT COSTS

It comprises of all repairs, preventive maintenance and contingent costs such as:

- Civil work structures
- Electrical and mechanical equipment

- Pipes and fittings and etc

8.3.3 FUNDS FOR RECOVERY OF INVESTMENT (CAPITAL) COSTS

It includes all costs associated with expansion or/and depreciation of all assets or/and repayment of debt services etc.

Finally all costs required for investment, operation and maintenance of the water supply system should be done as described above, for each of the identified alternative water supply systems for feasibility analysis of the project.

8.4 FEASIBILITY ANALYSIS OF THE WATER SUPPLY PROJECT

Feasibility study of water supply projects should be conducted preferably by considering different alternative options having different source types and arrangement of water supply system components. For each proposed alternative option the following tasks have to be accomplished:

- Describe the technical appropriateness
- The investment an O & M costs have to be computed
- Financial analysis should be done
- Environmental and Social impact assessment has to be conducted. Form these studies, the environmental and social impact of each alternative option should clearly be stated

By giving appropriate weight for the technical, financial, environment and social aspect of each alternative, the option to be selected is the one having the maximum from the cumulative of all. It has to be recalled that in multi-criteria selection of alternative options, the option having the least cost might not be the best option. Option having higher cost can be selected if the designer can justify the technical, environmental and social appropriateness of the system over the cost.

9 DETAIL DESIGN AND BID/CONSTRUCTION DOCUMENT

Having selected the preferred technology of the water supply system, the final stage of the study and design of water supply is to conduct detail design, preparation of drawings, specifications, bill of quantities and Bid/construction documents.

The purpose of this document is to show how the selected project will be constructed. The process ad contents of the project will be outlined in the following sections.

9.1 DETAIL DESIGN OF THE WATER SUPPLY SYSTEM

System capacity of main components and sizing of the water supply system has already been determined in the former sections in order to compare and select the most feasible project. Activities to be carried at this stage are thus to prepare the system's structural design, drawings and a specification of the equipment and materials to be involved. However all hydraulic and electrical and mechanical equipments design are also revised respective drawings are prepared.

In general, the following components are designed and documents are prepared for respective system components:

- Hydraulic design
- Structural design
- Electrical design
- Mechanical design
- Geo-technical design
- Drawings
- Bill of quantities
- Specifications

9.2 BID DOCUMENTS

The Bid document is prepared to invite and select potential contractors for construction and commissioning of the water supply components. It is also the basis for making a contract between the client and the contactor for implementation of the project.

For systematic arrangement and simplicity of implementation, the works to be Bided should be grouped under the following contracts:

- Contract for the supply of pipes, fittings, valves, water meters and appurtenants
- Contract for civil works construction
- Contract for supply and installation of Electrical and Mechanical Equipment
- Contract for contract administration and supervision of water supply works
- Each of the Bid documents to be prepared should have the following contents:
- Invitation for Bids
- Instruction to Bidders
- Form of Bid, Qualification information, Letter of Acceptance, and Agreement
- Condition of Contract
- Contract Data
- Specifications
- Drawings
- Bill of Quantities
- Security Forms (Bid, Performance, Advance, etc.)

10 COMPUTER MODELING OF WATER DISTRIBUTION SYSTEM

10.1 BASIC DESIGN PRINCIPLES

After the inventory of present situation is ready, Goals of the design are clear, design parameters are determined, the "simple" question is: how to start?

Initially known data are locations and quantities of supply and demand points in the area. Allocation of the demand to a node can be done in different ways, but if proper metering does not exist, preliminary assumption of homogeneous dispersion of connections around the node may be accepted. For instance, the border of the area supplied from one particular node can be assumed to be at the half of the distance towards the surrounding nodes Adjustment of this principle has to be done in every single case taking into account the existing knowledge about the area (local population densities, coverage, existence, of different demand categories, concentration of the demand due to large consumers, etc.).

The logical start in the design phase is definition of the main pipe routes. There, several circumstances have to be taken into account (e.g. Topographic conditions, etc.). The first solution is usually with lower investment costs, providing necessary supply for each point, i.e. from one side only

After the network is spread over the area, pipe sizing and analysis of hydraulic behavior are the following steps. Despite low investment, hydraulic considerations may indicate pressure and operational problems that make the solution unacceptable. In practice the first alternative is rarely the best, thus several others options have to be proposed. Further development of the alternatives leads to extension of the network by laying additional pipelines or/and potential inclusion of installations (reservoirs, pumping stations, pressure reducing valves, etc.), in order of optimizing hydraulic performance and reliability of the system This requires deeper analyses where proper cost benefit analysis of the alternatives has to indicate the best solution. The final stage then is the design of local parts of the network.

10.2 NETWORK CONFIGURATIONS

In laying the pipes through distribution area the following configurations can be distinguished

- Serial
- Branched
- Grid
- Combined

The branched system can be designed with trees, parallels or combined of both. The grid system can be developed from parallel branched system by adding extra connections, or designed as a looped system. The combined system comprises any combination of types a)-C).

10.2.1 SERIAL NETWORK

This is a network without branches or loops, the simplest of all configurations. Generally, it has one source, one (dead) end and a couple of intermediate demand points (nodes). All intermediate nodes are connected by two pipes: a supply link at the upstream and a distribution link downstream. The direction of flow is fixed from source to the end of the system.

Design of this type of network is based on the principles of single pipe calculation. Flows in the pipes are based on nodal demand, and they can be easily determined from the continuity equation for each point. The pressures in the system can be calculated starting from known value at the source point or from minimum required value at the end or some of the delivery points.

These networks characterize very small (rural) distribution areas and although rather cheap they are not common due to extremely low reliability and quality problems caused by water stagnation at the end of the system. When implemented in water transport systems, they are even more simple but not cheap any more.

10.2.2 BRANCHED NETWORK

The branched network is combination of serial networks. It usually consists of one supply source and several (dead) ends. The intermediate nodes in the system are connected by one supply link upstream and one more distribution links downstream. Fixed flow direction is generated by the distribution from the source to the ends of the system.

The principles of hydraulic calculation are the same as for serial network. Pressure gradient in each pipe can be determined from its main characteristics (length, diameter and roughness description), knowing flow and pressure at one side. The calculation proceeds starting from the source or minimum required pressure somewhere in the system.

The branched networks are adequate for small communities having in mind acceptable investment costs. However, main disadvantages still remain:

- Low reliability,
- Potential danger of contamination caused by large part of network without water, during irregular situations,
- Accumulation of sediments due to stagnation of the water at the dead ends, occasionally resulting in taste and odour problems,
- Future extensions, that may cause pressure problems,
- A fluctuating water demand producing rather high pressure oscillations.

10.2.3 GRID NETWORK

The grid systems consist of demand points that are supplied from more than one pipe. This is a consequence of looped structure of the network formed in order to eliminate the disadvantages of branched systems. These network can be derived from the branched system by connecting parallels at later stages of system design by connecting parallels at later stages of system design or/and extension, or simply developed initially as a group of loops.

The grid systems are hydraulically far more complicated than serial or branched networks. Flow direction there, is predominantly determined by the system operation and much less by its layout, thus it is not fixed which means that the location of critical points (e.g. low pressure) may vary in time. Apart from that, these systems are very often supplied from more than one point which makes analysis even more complex. Therefore, single pipe calculation procedure can be used here only as a basis for more complicated (iterative) procedures, discussed in further paragraphs.

The problems of branched system operation are eliminated in this case by the fact that.

- The water in the system flows in more than one direction, and long time stagnation does not occur easily any more,
- During pipe bursts subsequent repairs, the area concerned will continue to be supplied by water flowing from other direction (in case of pumped system, a pressure increase caused by restricted supply can even promote this),

- Water demand fluctuations will not produce significant effect in pressure fluctuations.

However, these systems are more expensive in investments and in costs of operation. Therefore, they are appropriate exclusively for large (urban or industrial) distribution areas that require high reliability of supply.

10.2.4 COMBINED NETWORK

This is in fact the most common type of network for urban areas, where by looped structure is a central part of the system and supply of localities on the outskirts of the area is provided through several serial or/and branched sub-systems. For more simply analyses, these systems can be "hydraulically dismantled" if the parts of the system, removed from the analysis, can be properly represented at the point of interconnection (e.g. this applies for all branched parts in the system).

All advantage and disadvantage of these systems are as discussed previously for branched and looped systems.

10.3 NETWORK SCHEMATIZATION

Procedures for hydraulic calculation of looped networks are based on systems of equations with the complexly directly proportional to the size of the system. Thus, some schematization (skeletonization) of grid is necessary up to the level where accuracy of the results will not be substantially affected by the simplification, enabling quicker calculation.

This was particularly important in former times when computers were not extensively used and calculation of properly schematized network could save a few days (or even weeks) of work. Eventually, this has become a minor problem and current development of network modeling goes into the opposite direction. More and more detailed information of the system is including in analysis, specifically for monitoring its operation by computer (in particular the operation of storage and pumping stations). However, the schematization of distribution network is still attractive for design purposes, because.

- It saves computer time
- It provides easier analysis of various alternatives, and more clear picture about global operation of the system,
- Sophisticated analysis of overall network operation is not crucial for the pipeline design; improperly interpreted it can even to the false design of smaller pipes.

The last point may be understood keeping in mind relation between the shape of peak factor curve and size of the observed area, creating winder fluctuations for smaller number of consumers. Hence, the correct calculation of peak water demand in one node will depend on number of consumer served from that node. Implementation of the same peak factor diagram regardless to the size of the distribution area can be tolerated only whilst analyzing network of main pipes, (above 100-150 mm in diameter).

The most common means of network schematization are:

- Combination of a few close demand points into one node with cumulative demand,
- Exclusion of hydraulically irrelevant part of the network such as branches and ends at the border of the system (the exclusion of the pipes and nodes but not of the consumption in them),
- Neglecting of smaller pipe diameters,
- Introduction of equivalent pipe diameters

The decision on how to treat the network during the process of schematization is based largely on personal experience. Sometimes, it is desirable to include all main pipes (if the system is not extremely large), in other situation the pipes under a specific size can be excluded (e.g. below 200mm). In any case the basic structure of the system should remain coherent, without removing the pipes that form major loops.

The effects of schematization were analyzed in some literatures. An example from USA is shown how the part of the existing network ranging in diameters 50-500 mm, which serves app. 11000 people was schematized with the removal of the particular diameter in following percentage: 50 mm -100%, 100 mm- 77.3 %, 150 mm – 20.1 %, 200 mm -8.6 %, 250 mm 6.1 % reduced from 43.5 km to 29.6 km. The observed results of the schematized and full model were fairly close for different supply conditions (deviation of about 7%).

Conclusions of the study are that small diameter pipes can be omitted during schematization, especially when such pipes are perpendicular to the usual direction of flow or located near large pipes. On the contrary smaller pipes laying close to supply points large water users or not in the vicinity of large diameter pipes, should be included in the schematics, either fully or replaced through equivalent pipes.

11 REFERENCES

Twelve Towns Water Supply and Sanitation Study Working Paper No 2 Design Criteria February 1994)

Environmental Support Project (ESP) Component 3 Unit Costs December 2001

Environmental Support Project (ESP) Component3 Urban Planning Model December, 2001

Environmental Support Project (ESP) Component3 Rural Planning Model January, 2001

The World Bank Issue paper for project Design, Technical Financial and Economic Feasibility Draft Final Report, November 2002

ESRDF Technical Design Manual December 1996:

World Bank Technology Options and their Impact on Tariff Draft Report, June, 2001

Development Corridors of Oromia, East and West Hararge Zones Water Supply Project design Report

Oromia Six Centers Water Supply Project Design Review Report

Data Compilation and Analysis Project (1997)

9/25 Towns Water Supply Feasibility Study and Engineering Design Report (Oromia Towns)

Mekele Town Water Supply Development Project, Planning and Design Criteria November, 2008

Humera Town Water Supply Project Feasibility Study and Detail Engineering Design Report

World Bank Technical Paper Number 12, Water Supply Project Preparation Guide Line

Water Transport and Distribution, IHE Lecture Notes

Applied Hydraulics in Sanitary Engineering, IHE Lecture Notes

Haestad Methods. 1/14/02 (5.00032) 1995-2002