# DESIGN OF PRESSURIZED IRRIGATION SYSTEM TO IMPROVE WATER USE EFFICIENCY IN CANAL COMMAND

**A DISSERTATION**

**SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF THE DEGREE OF**

**MASTER OF TECHNOLOGY**

**IN**

**IRRIGATION WATER MANAGEMENT**

**BY**

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## Candidate's Declaration

I hereby declare that the work which is presented in this Dissertation paper, entitled, Design of Pressurized Irrigation system to Improve water use efficiency in Canal Command – a case study on Metahara Irrigation Scheme, Ethiopia, submitted in partial fulfilment for the requirement of the award of Master of Technology in "Irrigation Water Management", Water Resources Development and Management Department, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out under the Supervision of Dr. M.L. Kansal, IIT Roorkee.

I also declare that I have not submitted the matter embodied in this report for award of any other Degree.

Date: May 10, 2018

Place: Roorkee **Alemayehu Yadesa**

## Certificate

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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Abstract

The limited existence and the misuse of the available water resources have put pressure on the judicious use of water for irrigation. Likewise, Awash River is under increasing pressure due to the increase in demand and excessive exploitation of the available resource. The problem is escalating since this river is the only source of fresh water in the basin. Therefore, modernization of an existing irrigation system is the need of hour so that the scarcity of water resources can be addressed effectively. It is felt that the Pressurized Irrigation System (PIS) is one of the best ways to deliver water to the field preferably when the resource is scarce, the topography is undulating and/or rugged and source of water is from ground. Furthermore, this system is quit resilient and is preferred one as compared to canal irrigation system, particularly when there are complex issues of land acquisition, etc.

Though the most challenging problem is the financial doubts i.e. the fear of replacement works and energy cost, the design of cost effective network that satisfies hydraulic requirements in accordance with demand patterns is an equal important parameter. For this study, EPANET 2 software was used for hydraulic analysis of pipe network while the optimization of the pipe size was done by Labye's Iterative Dis-continues Method (LIDM). Most of the data used were from secondary sources. We could save 900,482 $m^3yr^{-1}$ volume of water for this study area if the system is converted to PIS. This volume of water has the potential to develop additional 91.8ha land per year. After optimizing the pipe size which was gained from the EPANET output using the LIDM, it was reached to the point that we could save 697,763 ETB for the pipe cost.

**Key words**: pressurized irrigation system, pipe network, EPANET, LIDM, water use efficiency, piezometric elevation head.

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## Abbreviations and Symbols

| Symbol | description |
|---|---|
| a.s.l | above sea level |
| $\beta$ | Greek letter beta |
| $\gamma$ | Greek letter gamma |
| CDN | Canal Distribution Network |
| CIN | Canal Irrigation Network |
| CV | Coefficient of variation |
| Dia. | Diameter |
| $D_{min}$ | the minimum commercially available diameter |
| ETB | Ethiopian Birr |
| FCV | flow control valve |
| FAO | Food and Agriculture Organization |
| GIR | Growth irrigation requirement |
| $H_{j\ min}$ | the minimum head required at the most unfavourable hydrant |
| Hr | hour |
| $J$ | friction loss per unit length of pipe |
| L | Length |
| LPS | litre per second |
| m | metre |
| NIR | net irrigation requirement |
| PDN | pressurized Distribution Network |
| PIN | Piped Irrigation Network |
| PIS | Pressurized Irrigation System |
| $C$ | pipe cost per unit length |
| sn | sub-network |
| e | section of the network |
| LIDM | Labye's Iterative Dis-continues Method |
| Q | discharge |
| UNESCO | United Nations Educational, Scientific and Cultural Organisation |
| WUE | Water Use Efficiency |
| Yr. | year |
| $Z_j$ | nodal elevation |
| $Z_{Z,\ in}$ | the initial piezometric elevation at unfavourable node |
| $Z_Z$ | the effective piezometric elevation at the upstream |

# CHAPTER 1

# INTRODUCTION

## 1.1. Background

The limited existence of water and higher demands for other usages has put pressure on the judicious use of water for irrigation. The scarcities of water resources and competition among water using sectors in addition with the overall low efficiency of conventional irrigation methods are the driving forces to think for the modernization of irrigation systems in order to allocate equitable resources to all beneficiaries. Problems in providing adequate irrigation water to the rapidly growing population, especially, that of the developing countries is increasing from time to time. Water supply systems for the agriculture is often unable to meet the existing demands and are not available to everyone uniformly rather, some consumers take disproportionate amounts of water and the other is the victim to the problem. The developing countries have great difficulties in supplying satisfactory volume of irrigation water in sustainable way to the population which is increasing very fast with time, due to financial or technical or even both and the limitation of the resources. As competition for water in different sectors is increasing daily, irrigation is being under increasing pressure to grant the required water amount. Both the reduction in water availability for irrigated agriculture and the recurrent seasonal available water fluctuation represent major difficulties for irrigation systems to perform better. However, the need to feed the population is on behave of this sector and this situation increases pressure on the scarce available water resources and lead to decreasing access by the water user groups. Beside to this, poor management of the existing infrastructural and its technology asset increases the level of water losses in irrigation water supply. Therefore, it is very essential to assess the irrigation water supply situation of the existing irrigation schemes.

According to (Rockström, Barron and Fox, 2003), irrigation accounts for about 72% of global and 90% of developing countries water withdrawals. Irrigation is globally critical to quality of life, providing at least 40% of the total worldwide food and fiber supply (Evans and Sadler, 2008)

(FAO, 1989) states that irrigated agriculture face a number of difficult problems in the future. One of the major concerns is the generally poor efficiency with which water resources have been used for irrigation stating that a relatively safe estimate is 40 percent or more of the water diverted for irrigation is wasted at the farm level through either deep percolation or surface runoff.

Assessing the performance and potential improvements of the existing conveyance and distribution systems are thus basic concerns and will most likely to get even greater attention in the future. Concerned stakeholders will be in fact more addressed to modernize poor-performing irrigation systems rather than to develop new irrigable areas or expand existing schemes. It is felt that the Pressurized Irrigation System (PIS) is one of the best ways to deliver water to the field preferably when the resource is scarce, topography is undulating/ rugged, source is from ground, need to improve efficiency, and etc. Furthermore, this system is quit resilient and is preferred one as compared to canal irrigation system, particularly when there are complex issues land acquisition.

An appropriately planned irrigation system addresses uniform water system application in an efficient way while limiting the losses. Well planned system matches soil and water qualities with water application rates to guarantee that water is available in the sum required at the perfect time.

Though the most challenging problem is the financial doubts i.e. the fear of replacement works and energy cost, the design of cost effective network that satisfies hydraulic requirements in accordance with demand patterns is an equal important parameter. The design of cost effective and a reliable irrigation water distribution system is of considerable importance because of the strong dependence of societies and irrigation sectors on these two main factors. In most Irrigation water distribution systems, problem is to modify and/or expand an existing network so that it is capable of fulfilling varying demand patterns of water at required pressure levels in a uniform manner. An efficient and suitable irrigation water supply can result in vast improvements in agricultural production and assure the economic vitality of any country. In many developing countries like Ethiopia, it is important to understand that the scope of irrigation science is not limited to diversion of available resource to the irrigated field; rather, better means of delivery with improved technical disciplines should get more emphasis. It is obvious that the benefit of irrigation projects may increase if such efficient and improved technologies are encouraged in order to use and manage the available resources properly.

### 1.2.Ethiopia’s Water Resources and agriculture scenario

According to (Dejen, 2015), nearly 66% of the total land area of the country is potentially suitable for agriculture, which is nearly equivalent to 72 million ha. Nevertheless, due to various factors including climatic, demographic, socio-economic, etc., only about 25% of the total cultivable land was being put under cultivation by the year 2010. Agriculture is by far the backbone of Ethiopia’s economy like for many least developed countries. It provides over 85% of the total employment, 43% of foreign exchange earnings, and approximately 50% of the GDP. The vast majority (95%) of the share of agriculture for the GDP is produced by smallholder farmers cultivating less than 1 ha of land mostly under rain fed agriculture. The Ethiopian highlands, constituting about 45% of the total land area, are regions facing high demographic pressure on land and water resources. The lowlands in the southern, south-eastern and south-western parts of the country are however, with sparse settlements, offer huge and unutilized land resources potentially suitable for agriculture. Actually, very little irrigation infrastructure has been so far developed in these areas to bring these vast areas under irrigation.

Ethiopia may have good opportunity in boosting the economy of the country, if focus on the modernization of existing irrigation infrastructures in addition to critical challenges in the planning, design, and maintenance of its new irrigation systems. Modernization of irrigation and agricultural water management plays great role to improve productivity and reduce vulnerability to climatic and soil conditions. Even though Ethiopia receives abundant rainfall and water resources during the rainy season, its agricultural system does not yet fully benefited from the technologies of water management and irrigation.

The main source of water for the country is annual precipitation. Although no one cannot be definite, preliminary studies indicate that the country get an annual surface run-off of about 122 billion cubic meters of water and between 2.6 to 13.5 billion cubic meters of ground water. Much of water however flows across the neighboring countries. Due to the rapid population increase, the per capita water availability in Ethiopia is definitely among the countries with serious economic water scarcity.

The annual per capita water availability including all the water resources draining out of the country less than 1,560 cubic metres, which falls lower than a threshold value for water scarcity of 1,700 cubic metres. Natural variability in rainfall patterns and distribution both in spatial and temporal punctuated by extreme climatic events has put regions of the country under conditions of extreme water scarcity, poor quality and chronic food insecurity.

The big and main water resources problem in Ethiopia is the uneven distribution and spatial occurrence. Between 80% - 90% of Ethiopia's water resources is found in the four Basins namely Abbay (Blue Nile), Omo-Gibe, Baro-Akobo and Tekeze. Contrarily, the population within these Basins is not more than 30%-40% of the total population. On the other hand, the water resources available within the rest of Central and Eastern River Basins is only 10% to 20%, where the population in these Basins is over 60%.

An estimated 93% of all water withdrawals in the country (surface and ground) is allocated for agriculture use, which is much higher than the global average of 70 per cent. However, water withdrawals for agricultural activities represent only an estimated 4-6% of the overall countries available renewable water resources. Due to limitations of topography, geology, physiography, quality and the present state of technology summed up with economic capability, only a part of available water resources can be utilised.

### 1.3. Necessity of Pipe Distribution Irrigation system

With the inexorably more noteworthy request on a constraint water supply in many parts of nation, there is a dire requirement for its productive usage by decreasing losses in the irrigation system framework. In the irrigation water delivery system, losses can be significantly decreased by having appropriate conveyance system for distribution of water within the system. Pipe distribution system offers the possibility of using the scarce/available resource efficiently by reducing the losses. In addition to the reduction of losses, this system is also advantageous in lifting water to higher elevation areas unlike the gravitational canal distribution system.

On the other hand, piped irrigation is better in an accommodation with the existing land ownership since the out let location is not limited under this system. No or less construction of drainage works and better option for flood prone areas. It also provides the opportunity to apply fertilizers and chemicals through the system.

Most scientists are stating that the scarcity of water is developing world wide and predict that water will be potential deterministic resource in the future. Contrarily, human beings are careless in managing the freely available water resources. As a result it is an alarming time to work on the ways we can use wisely the freely available water by reducing the wastage.

## 1.4.Research Gap

Many researchers have done very appreciable work on improving irrigation water use efficiency. However, the question of increasing the efficiency of the irrigation system is still not to its optimum point. Even though there are improvements in improving the irrigation water use efficiency, still there is no global solution to the problem. Particularly, the problem is series in developing countries where even lined canal irrigation systems are not well developed. Like many other developing countries, most of the irrigation water distribution system in Ethiopia is canal system. As witnessed by many professionals, the efficiency of irrigation water delivery and distribution by canal networks of Ethiopia is low.

(Bekele and Tilahun, 2006) showed that the irrigation water supplied to a farmer's field was generally more than the volume required per event. The irrigation method used was unlined canal network and the major cause of water loss was determined as due to deep percolation. They came up with idea that seepage loss varies with crop pattern i.e. 32% in sorghum, 57% in maize, and 70% in tomato and potato fields. (Mamo and Wolde, 2015) underwent 'Evaluation of Water Management in Irrigated Sugarcane Production' at Wondogenet, Ethiopia; and the evaluation result showed that the conveyance efficiency in 5km long canal was 64.25% while field application efficiency of the system was as low as 52.85%. (Awulachew and Ayana, 2011) stated that the conveyance efficiencies of the main systems vary between 58% in the Hare scheme and 88.7% in the Bilate scheme of Ethiopia. They witness that in the Hare scheme, the system consists of earthen channels and is poorly maintained leading to higher water losses while in Bilate scheme the conveyance system consists of a lined canal and closed pipes resulting in higher conveyance efficiency.

(Mohammed and Tefera, 2017) confirmed that the expected (designed) and the actual value of conveyance parameters obtained for the main canal of Fentale irrigation scheme(Oromia), Ethiopia show great difference; 77% during design against the 17% existing conveyance efficiency obtained during the study time. They witness that the huge variation in conveyance efficiency resulted from variation in the hydraulic parameters like manning's roughness coefficient and canal slope; the former being the main contributor to the decrease in conveyance efficiency.

(Hamdy, 2007) stated that in developing and developed countries of the Mediterranean area, despite the high priority and massive resources invested in the water resources development, the performance of large public irrigation systems has fallen short to expectation and affirms that competitive and inefficient use of limited regional water supplies by irrigated agriculture is a major threat to sustainability of water supplies. Very often the conveyance losses of conduits (unlined canals or leak pipes) are much too large; a 30% loss percentage of the available water is common in irrigation systems.

According to the World Bank Paper 205(1993), the percentage of allocation of available water resources for agriculture is an indicator for the level of development of any country. It states; the more the available water resources are allocated for agriculture, the lower the level of development and vice versa. Hence, it affirms that most countries where agriculture uses more than 90 percent of water are in Asia and Africa. Accordingly, records show that 93% of the available water resource of Ethiopia is also used for agriculture. As per the source stated above, countries which have allocated more than 60 percent of their water resources to agriculture are almost exclusively developing countries while developed countries typically use less than 50 percent of their water resources in the agricultural sector.

From the aforementioned research results, we might be easily convinced that the efficiency of canal irrigation water distribution is low. Hence, it needs system modernization. As a result, it is must to search for better performing system which improves the efficiency. Pressurized irrigation system is believed to perform better than canal irrigation network. However, this system is not widely prevalent in Ethiopia and is the area which needs more emphasis. Thus, this study emphasizes on the way to improve efficiency of irrigation system through design of pressurized irrigation pipe network system for an existing canal command which is believed to be effective in terms of cost and efficient in terms of water delivery.

## 1.5. Objectives of the study

The general objective of the study is to design pressurized irrigation system as alternative to existing canal irrigation system.

### *1.5.1. Specific Objectives*

- To determine minimum possible pipe diameter network for the system.
- To generate persuasive results regarding pipe distribution network over canal distribution network in a view to improve irrigation water use efficiency**.**
- To generate base line information and contribute for further expansion of the pressurized irrigation system in the country.

## 1.6. Organisation of the Study

This dissertation material has six chapters which in turn have sub-divisions within them except for chapter six. The introductory chapter portrays about the scarcity of water resources and competition among water using sectors. The pressure on irrigation and the low water use efficiency of existing canal irrigation system was also discussed. Chapter two gives a review of literatures on irrigation water use efficiency and optimization of pipe size through theoretical studies as building blocks.

The third chapter is all about the piped irrigation network and hydraulics of piped irrigation network. Chapter four is a brief description about the geographical location of the study area, the method of data collection and the methodology used to accomplish the study. Chapter five outlines the details of research results and discussion on the development of piped irrigation network by using EPANET2 software and later optimization of the pipe size by using the Labye's Iterative Dis- continues Method (LIDM). The sixth chapter provides summary and conclusions of the research outcomes derived from the preceding chapters.

The last part contains list of reference materials used and appendices of supplementary tables and figures.

# CHAPTER 2

# PIPED IRRIGATION NETWORK

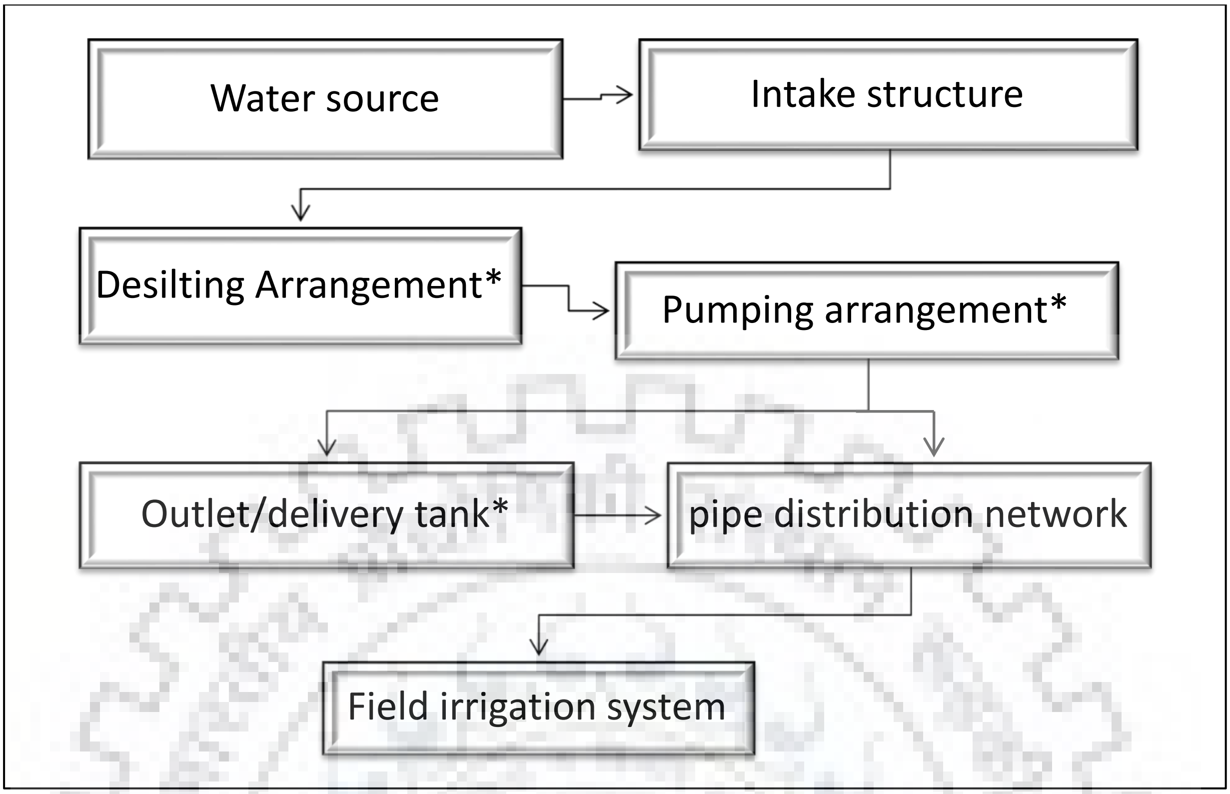
## 2.1. Introduction

In irrigation point of view, pressurized distribution network system is a network consisting of pipes, valves, fittings and other devices properly designed and installed to supply water under pressure from the source to the irrigable fields. Under the point of improving irrigation water efficiency, a well-designed and carefully laid distribution network is very important. Considering both open canals and pressurised pipe distribution systems, pressurised systems are getting more attention with considerable advantages. They guarantee better service to users because they provide better adaptation to crop needs and soil. The service provided by means of this system is more controllable and can be adjusted easily depending on the water need which may vary over time due to climatic, agronomic and other factors. The other advantage is, they overcome the topographic constraints and make it easier to supply water to higher elevation.

A well planned, designed and constructed distribution network for irrigation purposes should capable of delivering required quantity of water with the right pressure at the right time. To this end, the distribution system has to incorporate all necessary mechanical and hydraulic aspects such that all the requirements and any constraints imposed within the system are satisfied.

Pressurized irrigation systems specifically sprinklers are recommended and used on practically all types of soil, topographic conditions, and on almost all kinds of crops. (Li, 2018) States the importance of sprinkler and micro irrigation that the of use these systems allows to create more feasible environments in terms of water, nutrients, air, heat, etc. for crops in addition to the application of water and fertilizers to meet crop requirements in a timely and accurate way.

Its flexibility and efficient water control allows a wider area to be irrigated thereby allowing chance to more land. For instance, thousands of hectares of land in the United States and Israel, which was previously suitable only for dryland farming or as wasteland, is being irrigated today with high yield. The quantity of water required for pressurized irrigation is much less as compared to the surface irrigation. Therefore, smaller flow rates can be applied efficiently using this this system.




*optional

*Figure2. 1: Flow diagram of typical Piped Irrigation System*

Pressurized Distribution systems vary greatly in size and complexity, from spreading water over irrigable areas through open channels. The basic differences are summarized in the table below:

*Table2.1: PDN vs CDN*

| Parameter | CDN | PDN |
|---|---|---|
| Water Flow regime | The size of the stream should be large | very small flows required |
| The route direction of flow | water is conveyed from the source and distributed to the field through open canals and ditches by gravity alone | Irrigation water is conveyed and distributed through closed pipes by pressure and/or gravity |
| The area irrigated | water is applied in large volumes per unit area | distribute the water at small rates over a very large area |
| Pressure or energy required | Since water flow is controlled by gravity, no need of external energy | Require pressure which is provided from pumping unit or from a supply tank |

### 2.2. Classification of Piped Irrigations Network (PIN)

**a. Based on the driving force**

Gravity PIN: The piped irrigation network in which the driving force is completely provided by changing/falling topographical levels.

Pumped PIN: this is piped irrigation network in which the gravitational force is supplemented by external energy such as pumps.

**b. Based on the distribution network**

**Tree PIN**: In the Tree PIN, each outlet gets its supply from one and only one route.

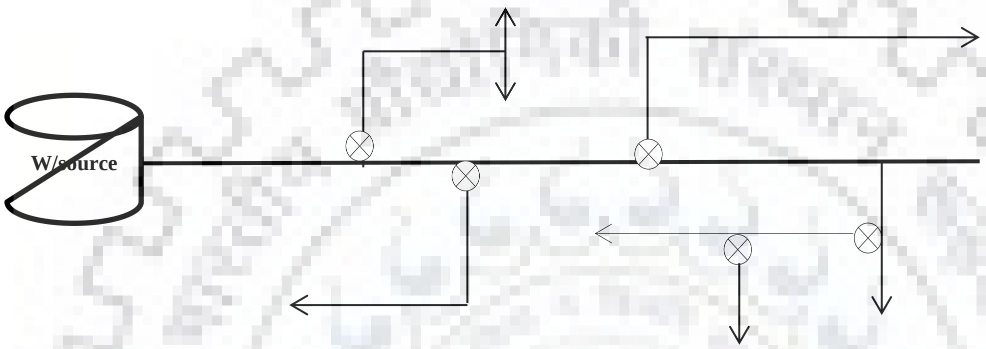



*Figure2.2: Example of Tree PIN*

**Loop PIN**: In the Loop PIN, each outlet gets its supply from more than one route.

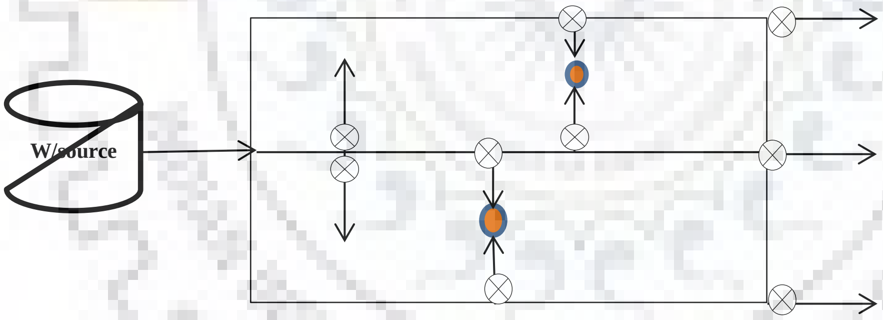



*Figure2.3: Example of Looped PIN*

**c. Based on the Movability**

Fixed/Stationary PIN: In this PIN, the entire supply lines up to outlets are fixed in location.

Semi- Fixed/Semi-Stationary: In this PIN, some the supply lines in the hierarchy for the outlet are fixed movable.

Mobile PIN: In this PIN, all the supply lines between the sources to outlets are movable. In special cases, if the entire lifting arrangement moves to another source to feed a different command such PINs are called Mobile Lift Irrigation Schemes.

**d. Based on the Operating pressure**

- Low Pressure PIN
- Medium Pressure PIN
- High Pressure PIN

**e. Based the Pressure Control**

Open PIN: In this type of PIN, the pressure is controlled by pressure regulating tanks/stand pipes open to atmosphere.

Closed PIN: In this type of PIN, the pressure is controlled by pressure regulating valves.

Semi-Closed PIN: In a closed PIN, if he pressure in part of the system is controlled by regulating tanks and part by pressure regulating valves, is called semi closed PIN.

**f. Economic Size of Pipe**

In this regard, selection of the pipe material becomes a significant part of the detailed project report (DPR). Selection of pipe material requires, taking into account life span, life cycle analysis, annual operational cost and other techno-economic considerations.

## 2.3. Hydraulics of Pipe Flow

### *2.3.1 Law of conservation of Mass*

Under steady flow, if it was assumed that water is incompressible, from the law of conservation of mass the continuity equation of flow can be given as:

Q = $A_1V_1$ = $A_2V_2$ =$A_3V_3$=……. =$A_iV_i$=constant

Where,

Q = Discharge

$A_i$ = Cross-sectional area

$V_i$= Velocity of flow

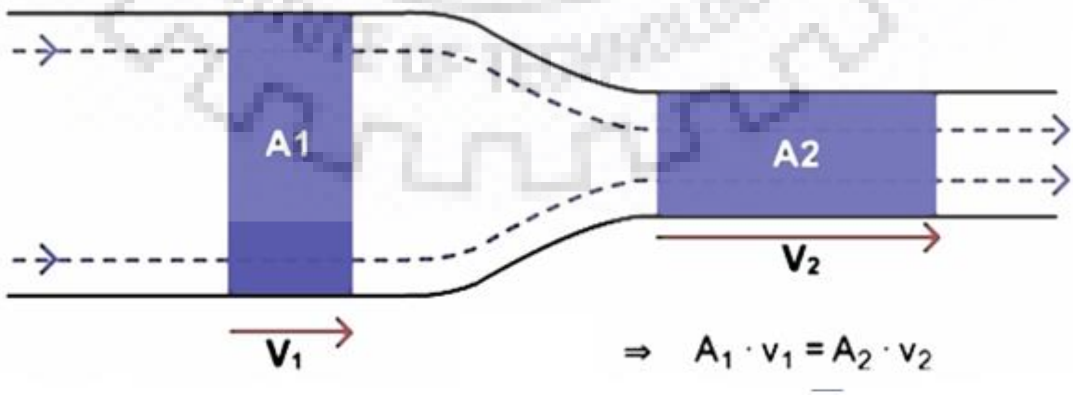



*Figure2.4: Flow through a pipe*

### *2.3.2. Law of Conservation of Energy*

The total energy of pipe flow consists of elevation head, pressure head and velocity head. From figure 5 below, between points (1) and (2), total energy may be conserved in prefect fluid.

However, if the water begins to move, head loss generated by friction will occur. So, the energy equation will be expressed as:

$Z_1 + {P_1}/{w} + {v_1}^2/{w} + \Delta E = Z_2 + {P_2}/{w} + {v_2}^2/{w} + \Delta H$ ………………………………………………...2.1

Where, $\Delta E$ is energy addition between to the system by pump

$\Delta H$ is the total head loss between points (1) and (2)

.

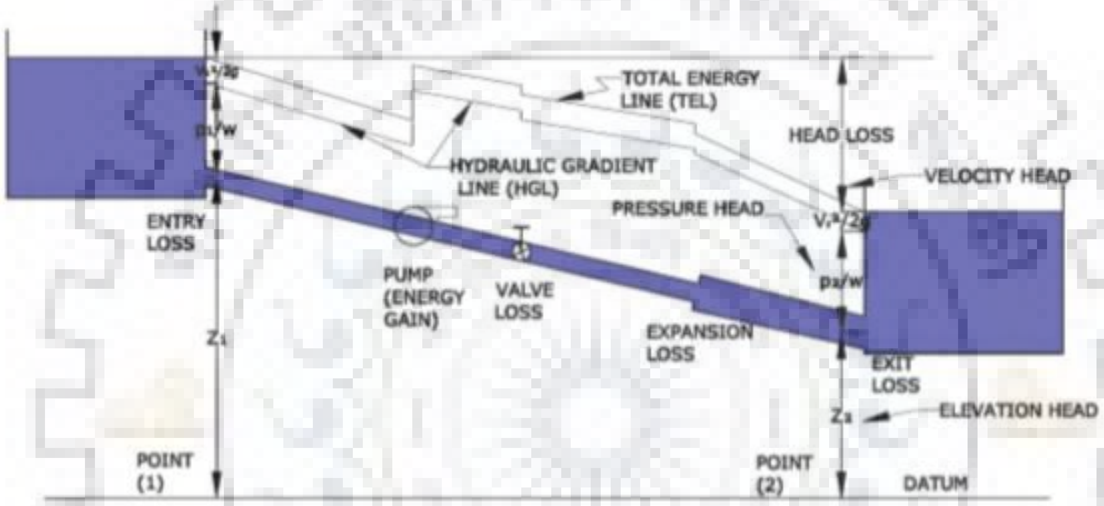



*Figure2.5: Law of conservation of energy*

### *2.3.3. Friction Loss (Major Loss)*

Since the flow used for the water supply will be turbulent in rough or transitional pipes, the friction factors depend upon the roughness of the pipe and upon the Reynold's number, which, in turn, depends in part upon the velocity in the pipe and its diameter; it is very difficult to estimate friction losses. Various pipe flow formulae like Hazen-William Formula, Darcy-Weisbach, Chezy's formula, Manning's formula, , etc. are used to predict head loss as a function of mean velocity in pipe.

It is said that Hazen-William's formula is the best of these formulae in case of pipes between hydraulically smooth and rough ones. This formula is most often used for designing water supply systems, because it is most likely that the pipe in the water supply system is transitional pipe

Generally head loss consists of friction loss and minor losses such as contraction, bend, enlargement, valve and so on. The friction loss in most cases was computed by different formulas.

**a) Darcy-Weisbach formula**

$h_f = \frac{flv^2}{2gd}$……………………………………………………… ……………………..…….2.2

where, = Coefficient of friction loss (Dimensionless)

$h_f$ = Head loss by friction (m)

l = Length of pipe (m)

d = Diameter of the pipe (m)

V = Mean Velocity ($ms^{-1}$)

g = Acceleration due to gravity ($ms^{-2}$)

k = Conveyance

In case of turbulent flow through pipes (R.K Rajput, 2010) it has been observed that the viscous friction effects associated with fluid are proportional to:

i. The length of the pipe, L

ii. The wetted perimeter, p

iii. $V^n$, where v is the average velocity of flow and n is an index varying from 1.5 to 2 depending on material type and nature of the pipe surface.

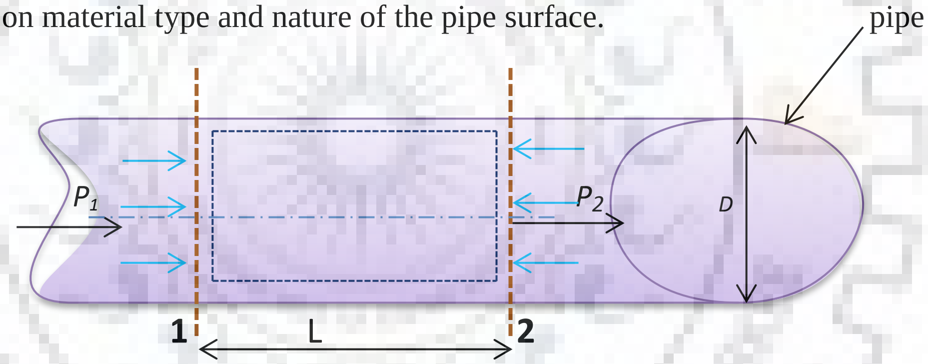



*Figure2.6: Forces on a control volume in a pipe flow*

where, $P_1$=Intensity of pressure at section1

$P_2$=intensity of pressure at section 2

L=the length of pipe

D=diameter of the pipe

Propelling force acting on the flowing fluid between the two sections is

= $(P_1\text{-}p_2)$ A……………………………………………..............2.3

where, A=cross section area of the pipe

Frictional resistance force=$f'PLV^2$……………………………………………….………2.4

where, p=wetted perimeter

V=average flow velocity

$f'$=non-dimensional factor, whose value depends on the type of material and nature of the pipe surface

The loss of head due to friction ($h_f$);

$h_f = \frac{fLv^2}{2Rg}$..........................................................................................2.5

where, R=hydraulic mean depth or hydraulic radius, $f$=Darcy-weisbach coefficient of friction

For circular pipes it is obvious that

$R=\frac{A}{P} \rightarrow R=\frac{D}{4}$

Therefore, $h_f = \frac{4fLv^2}{2Dg}$

where the term $4f=f_1$ and known as friction factor

$h_f = \frac{f_1 Lv^2}{2Dg}$

($P_1$-$p_2$) A= Propelling force acting on the flowing fluid=force due to stress at the pipe wall ( $\tau$ )* surface area

$= \tau * \pi * DL$

($P_1$-$p_2$) A= $\tau * \pi * D * L$..........................................................................................2.6

$(P_1-p_2) = \frac{4 * \tau * L}{D}$ ..........................................................................................2.7

Head loss, $h_f = \frac{4fLv^2}{2Dg} = \frac{P1-p2}{w}$..........................................................................................2.8

where, $w$=weight density=$\rho$g

From equations 2.7 & 2.8;

$$\frac{4 * \tau * L}{D} = \frac{4fLv^2}{2Dg}$$

$f = \frac{2 * \tau}{\rho v^2}$ ..........................................................................................2.9

**b) Hazen-William's formula**

A design criterion on pipeline for irrigation also recommends the Hazen- William's formula except the structures such as syphons which intervenes in Streams or Open Canals.

V=0.85$*$ $C_{HW}$$*$$R^{0.63}S^{0.54}$……………………………………………………………...2.10

V = Velocity in pipe (m/s)

R = Hydraulic Radius (m) = A/P = d/4

I = energy gradient

$h_f$ = Energy head loss (friction loss in m)

L =Length of pipe (m)

CHW = Hazen-William frictional coefficient of Velocity

On rewriting the above equation:

$h_f$=10.67$*\frac{1}{C_{HW}{}^{1.85}}\frac{L}{d^{4.87}}Q^{1.85}$…………………………………………………………...2.11

Or $h_f = kQ^{1.85}$………………………………………………………………………………...2.12

**c) Chezy's Formula**

It is given as: V=$\sqrt{\frac{w}{f'}} * \sqrt{\frac{A}{P} * \frac{h_f}{L}}$ ……………………………………………………….2.13

where; V=mean velocity

$\sqrt{\frac{w}{f'}}$ is the factor called the Chezy's constant, C

$\frac{A}{P}$ =R= the hydraulic mean depth

$\frac{h_f}{L}$ =s (slope), describes the loss of head per unit length of pipe

### *2.3.4. Minor losses*

The minor loss of head includes the following:

i. Loss of head due to bend in the pipe
ii. Loss of head due to an obstruction in pipe
iii. Loss of head in Pipe fittings
iv. Loss of due to Sudden enlargement of pipe
v. Loss of head due to Sudden contraction of pipe
vi. Loss of head at the entrance of pipe
vii. Loss of head at the exit of pipe

## 2.4. Design Standards of Pressurized Irrigation Network

### *2.4.1. Permissible Velocity*

**Maximum velocity**

The higher the velocity, the greater the risk of damage through surges and water hammer. This risk particularly applies to pipes subject to uncontrolled starting and stopping. Using larger pipe results in a smaller water velocity for a given flow rate, but smaller pipe is often preferred for cost reasons. For gravity PIN, design velocity should not exceed 2.5 m/s. For pumping PIN, design velocity should not exceed 2.0 m/s. When the water is silt rich, design velocity of PIN should not lower be than non-silting velocity while non-silting velocity should be determined by experiments.

**Minimum velocity**

Designers must specify pipe diameters and flow rates that allow for a minimum operational water velocity, especially for irrigation systems that utilize emitters with small apertures. This will ensure that any sediment or solids are flushed through the lines. Minimum velocity should not be below 0.6 m/s (Indian standard).

### *2.4.2. Permissible head*

Minimum driving head at intake should be 1.2m. To ensure equitable distribution, all outlets should have same residual head. In low lying areas if HG is more, it should be reduced by using lesser diameter pipes or valve. Outlet pressure for micro irrigation systems with design working pressure between 0.2 MPa and 0.4 MPa shall be used.

### *2.4.3. Selection of Pipe Materials*

The selection of pipe materials should be based on the following considerations;

a. The initial carrying capacity of the pipe and its reduction with use, defined, for example, by the Hazen Williams coefficient (C).
b. The strength of the pipe i.e. its ability to resist internal pressures and external loads.
c. The life and durability of pipe.
d. The ease or difficulty of transportation, handling and laying and jointing under different conditions of topography, geology and other prevailing local conditions.
e. The safety, economy and availability of manufactured sizes of pipes and specials.
f. The availability of skilled personnel in construction and commissioning of pipelines.
g. The ease of difficulty of operations and maintenance.
h. Nominal pressure of chosen pipe material should not be less than the sum of design working pressure and water hammer pressure.

i. Connection between pipe and pipes, fittings and accessories should be simple and reliable.
j. Nominal pressure of fittings and accessories should not be less than that of pipe; dimension and deviation should meet sealing requirements.
k. When the sulphate concentration in soil exceeds 1%, concrete pipes and metal pipes should not be used.

The cost of the pipe material and its durability or design life are the two major governing factors in the selection the pipe material.

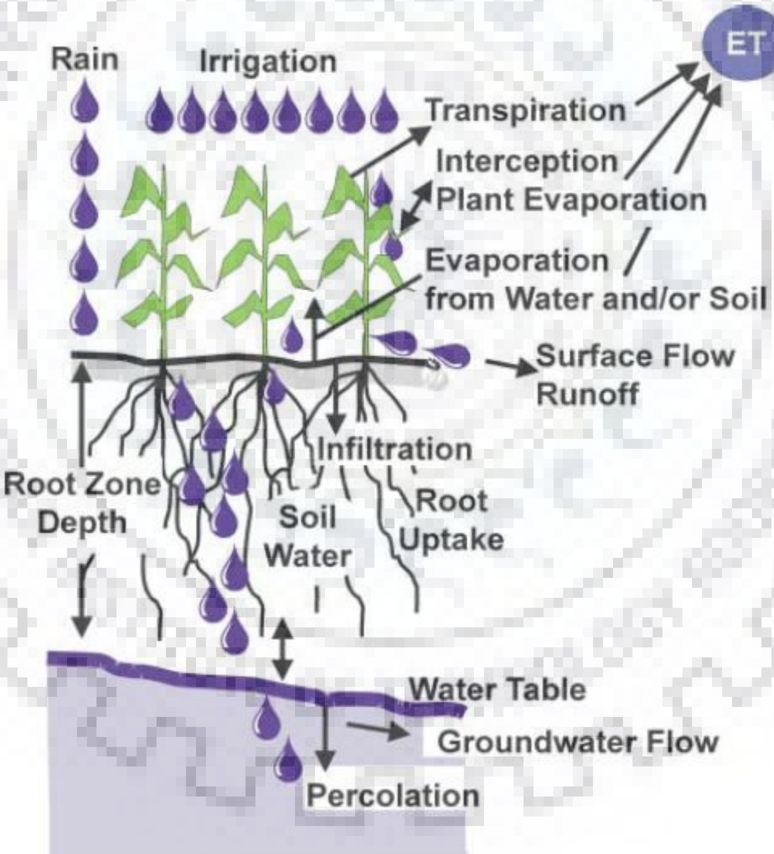
# CHAPTER 3

# LITERATURE REVIEW

## 3.1. Introduction

### *3.1.1. Irrigation water use efficiency*

The term ‘water use efficiency’ in irrigation system has been defined in different ways and has different meanings in different contexts. The definitions given mostly fall under two categories which are: outputs to inputs (kg/m$^3$) and the measure ratio (%) of the quantity of water effectively used at the target area to quantity of water released/diverted. The water released/diverted includes the water required to meet the crop evapotranspiration need, losses and salt balance. Water use efficiency is the ratio between effective water use and actual water withdrawal. It characterizes, in a specific process, how effective is the use of water.




*Source: Terry A. Howell, 2003*

*Table3. 1: Components of water transport system to characterize the irrigation efficiency*

(Evans and Sadler, 2008) tried to clarify the intention behind the term ‘‘water use efficiency’’ as ‘to do more with less water’ and recommend the term crop water productivity as water use efficiency.

(Howell, 2003) more clarified and defined the term irrigation efficiency with interrelated measures in terms of:

- the irrigation system performance,
- the uniformity of the water application, and
- the response of the crop to irrigation.

World Bank Technical Paper Number 205 (1993) given the definition as it is the relationship between the amount of water required for a particular purpose and the quantity of water delivered. It was explained in the paper that it is an important measure to guide conservation efforts for water resources. In addition, the effectiveness of water delivery can be another measure to evaluate the timeliness of supply, quantity, equity in allocation, and the quality of water. Similarly, field water use efficiency is a ratio between marketable crop yield and field water supply which includes water used by the plant in metabolic activities, evapotranspiration and deep percolation losses. Water Use efficiency at different levels in selected countries is presented below. This enables to compare the performance of irrigation projects 34 years ago and at the recent years.

*Table 3.2: Overall Level system Efficiencies*

| Country | % | Specification |
|---|---|---|
| Cyprus | 66 | Pipe conveyance systems with sprinkler and drip |
| Jordan | 53 | Open canals with rnanual control, on-farm storage & sprinkler/drip |
| Morocco | 42 | Open canal gravity systems with hydraulic control & surface irrigation |
| U.S. | 41 | Average |
| Turkey | 30 | Traditional open canal gravity systems with manual control |

*Table 3.3: On-farm Level Efficiencies*

| Country | % | Specification |
|---|---|---|
| Israel | 75-80 | nearly 100% by sprinkler irrigation |
| Cyprus | 70 | Pipe conveyance systems with sprinkler and drip |
| Jordan | 70 | Open canals with manual control, on-farm storage & sprinkler/drip |
| Morocco | 60 | Open canal gravity systems with hydraulic control & surface irrigation |
| U.S. | 53 | 50-85 % in intensively developed areas |
| Turkey | 50 | Traditional open canal gravity systems with manual control |
| Syria | 50 | Basin irrigation method used |
| Yemen | 40 | Large-scale gravity irrigation on the farm |

Source: 2, 3: World Bank Technical Paper Number 205, 1993

Hamdy A. et al. (2007), explained that water use efficiency includes any measure that reduces the amount of water used per unit of any given activity, consistent with the maintenance or enhancement of water quality.

(Michael, 1997) defined it as the ratio between the volume used by plants throughout the evapotranspiration process and the volume that reaches the irrigation plots and indicates how efficiently the available water supply is being used, based on different methods of evaluation.

While determining the system requirements, it is recommendable to consider efficiency factor to account for the losses that may incur during the conveyance and application of water to the field. This is because of that system efficiency is affected by different set of conditions.

(Attaher, Zaki and Karrou, 2013) used a "WP-optimizer" which is a computer modelling framework to analysis the integrated irrigation efficiency, water productivity and irrigation adequacy. It gives the numerical and spatial analysis of the system. In this system, there were on-farm irrigation management and irrigation distributary network routines linked by using Visual Basic Language with GIS.

The system efficiency is normally divided into three stages:

i. **Conveyance Efficiency**-Ratio between water received at inlet to a block of fields and that released at head.
ii. **Pipe line efficiency** - Ratio between water received at the field inlet and that received at the inlet of the block of fields.
iii. **Field application Efficiency -** Ratio between water directly available to the crop and that received at the field inlet.

Except some losses at the fittings, there will be neither evaporation losses nor seepage losses in piped networks as it is a closed system.

Therefore, design conveyance efficiency from Source to Minor should not be lower than 95% and with line efficiency of 95%, the project efficiency works out to be about 90%.

As per the Indian Central Water Commission guidelines for planning and design of piped irrigation network the overall irrigation efficiency for micro irrigation with sprinkler shall not be less than 68 per cent against the canal based conveyance system of 47 per cent. Similarly for drip irrigation, it shall not be less than 81.23 per cent.

In case of surface irrigation methods with ponding, the overall irrigation efficiency shall not be less than 72.2 per cent.

Water use efficiency measures the quantity of water taken up by the crop during its crop life to produce a unit quantity of the output/crop yield. In general, the lower the water resource input requirement per unit of crop yield produced, the higher the efficiency.

$$WUE=\frac{Y}{WR} \quad \ldots\ldots 3.1$$

Where,

WUE=Field water use efficiency, kg/ha-mm

Y=Crop yield, kg/ha

WR=Water used in metabolic activities, ET and deep percolation losses, mm.

### 3.2. Pipe size Optimization

In any design aspect, the cost of the project/activity is the crucial point and the deterministic factor for the acceptance of that project. Since the cost of the pipe increase with its diameter, the determination of pipe diameter needs great attention.

(Planells Alandi *et al.*, 2001) developed a procedure which takes into account both the network layout and the pipe size of an on-demand water branched network in order to obtain the lowest total cost (investment and energy cost) by considering the possible alternatives of branched irrigation networks. The Economic Series Methodology was used according to Clément's Methodology for optimization process that eliminates pipes progressively to obtain the most inexpensive branched irrigation network.

(Pereira *et al.*, 2003) states that results for the application of the iterative discontinuous method for the optimization of pipe size gives optimal pipe sizes having slightly lower cost and have higher performance than LP optimization methods in a case the number of the nodes of the system is lower than the number of flow regimes considered.

(Rajabpour R. and Naser T., 2014) underwent study on simultaneous layout and pipe size optimization of irrigation water distribution networks by using a method called "Hybrid approach''. In this approach, Labye's Iterative Dis-continues Method was used for the pipe size optimization and layout optimization algorithm is used for generation of set of tree layouts. The solution is reached by setting some termination criterion after iteratively optimizing the size and layout.

(Ostfeld *et al.*, 2014) suggested multi objective optimization model aimed at exploring the trade-off between least costs versus reliability for water distribution systems optimal design.

(Farmani, Abadia and Savic, 2007) developed rotation and on-demand delivery scheduling using a genetic algorithm for optimum design of a new irrigation system and better management of an existing irrigation system of pressurized irrigation systems based on performance criteria. They also witness that, the method performs better in satisfying the constraints than Linear Programming and said comparison between on-demand and rotation delivery scheduling shows that a greater than 50% saving in total cost by reducing flexibility in the irrigation time.

(M. Abunada, N. Trifunovića, M. Kennedy, M, Babel, 2014) developed a decision support tool called (Networks Optimization and Reliability Assessment Tool) and incorporated demand balancing tanks in network optimization and reliability assessment running extended period simulations. They witness that, NORAT better performs in determining the required balancing volume, optimizes pipe diameters and tank elevations, and finally calculates the total costs. In addition, NORAT further performs the analysis of the hydraulic reliability of the network since it is integrated with the EPANET programmer's toolkit functions for hydraulic analysis.

(Geem, 2006)used harmony search optimization method in order to find better design solutions i.e. pipe diameters in a water distribution network. The model also interfaces with EPANET to check the hydraulic constraints of the network.

(Lejano, 2006) expressed pipeline costs as a function of flow. He used the idea to attempt to simulate the planning and design process considering two heuristic design rules:

Rule (1): Pipes are sized to maintain a maximum nominal velocity at their respective design flows. This then allows one to relate pipe diameter and flow rate directly.

Rule (2): To allow for future development and changes in the customer base, agencies will commonly assume a minimum pipe diameter for installation.

(González-Cebollada, Macarulla and Sallán, 2011) developed a method called Recursive Design of Pressurized Branched Irrigation Networks for the optimum design of pressurized branched irrigation networks which is based on application of the problem-solving technique known as backtracking.

(Alperovits and Shamir, 1977) used a method called linear programing gradient (LPG) for optimal design of a water distribution system. Accordingly, the objective function, to be minimized, reflects the overall capital cost plus present value of operating costs. The constraints are that demands are to be met and pressures at selected nodes within the network are to be within specified limits. They stated that solution is obtained via a hierarchical decomposition of the optimization problem. The primary variables are the flows in the network. The other decision variables are optimized by linear programing and the post optimality analysis of the linear program provides the information necessary to determine the gradient of the total cost with respect to how distribution varies.

(Bhave and Sonak, 1992) used the linear programming gradient (LPG) method and proposed that it is better initially to suppress the minimum link flow constraint, necessary for reliability purposes, and introduce it later after obtaining the optimal solution for a branching configuration. They stated that the LPG method is inefficient, at least for optimization of the illustrative two-loop network, as compared to a heuristic method that initially identifies logically good branching configurations for obtaining the optimal solution.

(Sherali and Smith, 1997) addressed a global optimization approach to a water distribution network design problem with lower bounds derived through the design of appropriate Reformulation-Linearization Technique (RLT) that constructs a tight linear programming relaxation for the given problem.

(Mulatu, 2017) underwent a research with the objective to evaluate the existing water pressure map and water demand for Shambu town, Ethiopia. He stated that pipe network and junction network system was simulated to understand its behaviour for different inputs using EPANET 2 and finally the results showed that the water pressure were not feasible enough to provide adequate water.

(Wenting Han et al. 2012) developed a system to reduce the length of lateral pipe requirement for sprinkler irrigation system by replacing alternatively half the original layout but keeping the heads of the original sprinkler unchanged. This was done by connecting the remained lateral pipes and heads of the sprinkler through short perpendicular pipes.

Dechmi et al. 2003, Underwent a study to analysis:

I. the variability of the water application depth in each irrigation event and in the seasonal irrigation and
II. to inter relate the spatial variability in crop yield to the variability of the applied irrigation and to the soil physical properties.

The result of the study came up with the conclusion that the grain yield variability was partly impacted by the water deficit resulting from the non-uniformity of water distribution during the crop period. The spatial variability of irrigation water depth when the wind speed was higher than 2m/s; was correlated with the spatial variability of grain yield, indicating that a proper selection of the wind conditions is required in order to attain high yield in sprinkler-irrigated maize.

# CHAPTER 4

## MATERIALS AND METHODOLOGY

### 4.1. Description of the study area

Metahara Irrigation Scheme is a state managed scheme which is found in Fentale District (Woreda), Oromia Regional Government, in the semi-arid Rift Valley region of Ethiopia. The study area is geographically found at latitude of $8^0$ 46' 00'' and longitude $39^0$54' 22'' and located about 200 km southeast of the capital city Addis Ababa. The area receives an annual average rain fall 509.5mm and the average annual temperature 25.0 °C.

The River basin in which this scheme found contributes too much to the economy of the country especially as a number of large irrigation scales of the country are found within this River basin. However, in addition to the lower annual rain fall it receives, it is also known for its high evaporation climatic condition and saline ground water. In addition, due to high salinity of groundwater and drainage water at Metahara Scheme, wastewater is hardly recoverable. The scheme is also found near Lake Beseka, which is known for its salty water and substantial increase in volume in recent years.

The Metahara irrigation system has two parts called the Main System and the Abadir system. Irrigation water is abstracted from the river through diversion weirs and intake headworks found at Metahara. The water delivery infrastructure consists mostly of unlined canals but with a little lined canal.

(Dejen, Schultz and Hayde, 2015) determined the five years (2006-2010) average annual water diversion at Metahara Irrigation scheme as 190 $Mm^3$. They affirmed that this diverted volume exceeds demand by 24 % or there was 37$Mm^3$ of excess water annually. The excess diversion was because of wastage in the conveyance, storage and distribution systems.

As per the paper, the flow in the system varies with time from the time of opening, and showed that there was excess flow for 2-3 hours and indicated that flows as high as 200% and as low as 40% of the design discharge was observed. Similarly, the spatial coefficient of variation (CV) 32% which is much higher than CV of 10% which is assumed to be practically acceptable in systems with good management and determined the non-uniformity of water distribution and losses by measuring the flow in the canal at different off take points.

This shows non-uniform water distribution within the system which in one case results in the loss of water in the distribution system due to excess flow of water for shorter period when flow is more than the design discharge and shortage in the system when the flow is less than what is required.

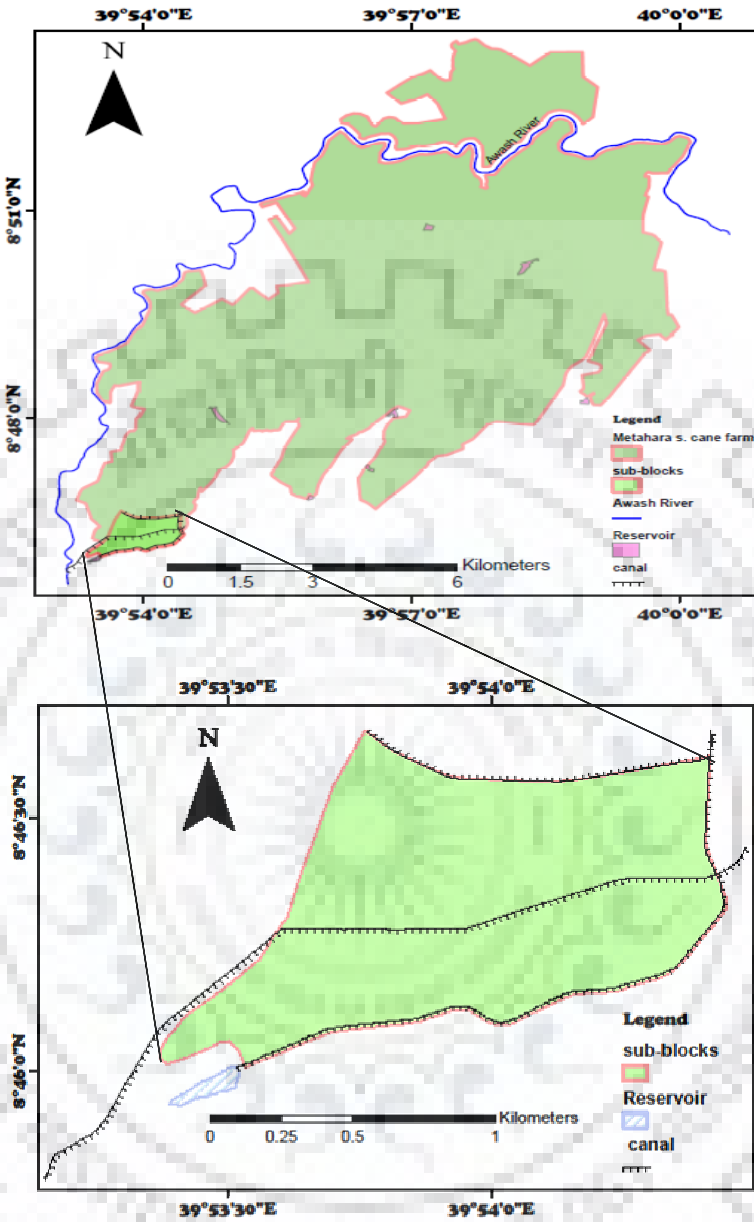



*Figure4. 1: Location of Metahara sugar cane farm (Sub-blocks)*

That is why this study comes up with the idea to manage and use the limited available water resources wisely in the study area. Though the study only considers two blocks that covers an area of 134.12 ha at the upper part of the main system, it may be considered as an indicator door for the modernization of the whole system.

*Figure4.2: Siphons feeding water to furrows*

### 4.1.1 Water losses under the existing operation

As stated earlier, due to high salinity of groundwater and drainage water at Metahara Scheme, wastewater is hardly recoverable. The scheme has also is found near Lake Beseka, which is known for its salty water and substantial increase in volume in recent years.

Z. A. Dejen et al., 2015 affirms that, of the total annual excess diversion of 41 Mm3 for Metahara Scheme, about 50% of the excess diversion is discharged at the tail end into shallow saline groundwater and waterlogged swampy fields.

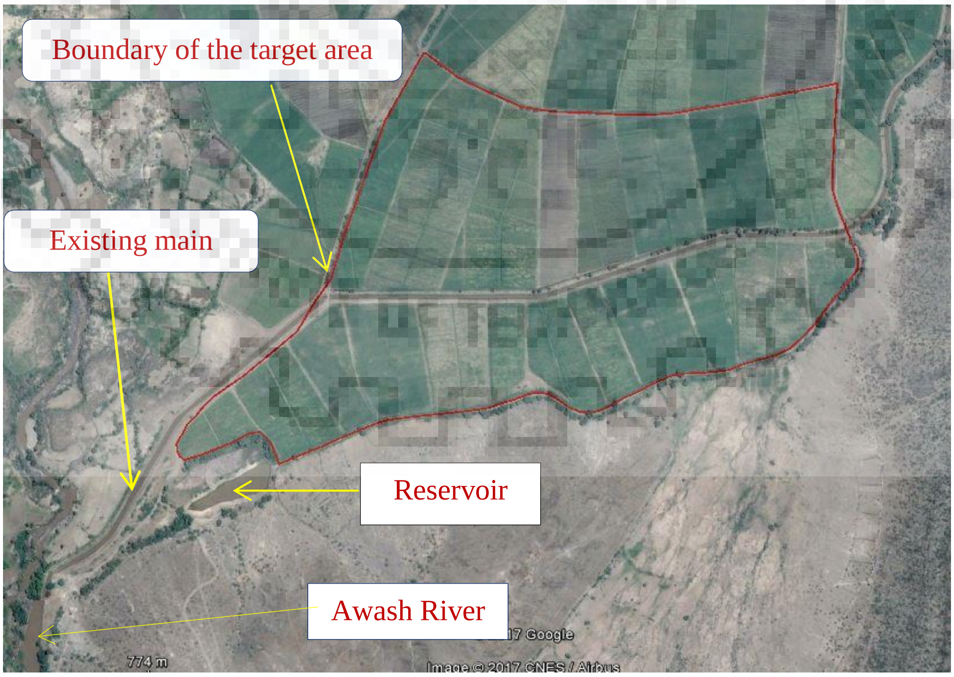



*Figure4.3: Existing system*

## 4.2.Source of Data

Secondary data from different sources which were believed useful for the study were collected, organised and used according to their necessities. Data from Ministry of Water, Irrigation and Electricity, National Meteorological Service Agency of Ethiopia and other sources publications were used.

## 4.3.Methodology

To undertake this study, two basic methods were used. The first method is to develop hydraulically efficient pipe network using EPANET 2 software for the system and secondly, optimization of the size of the pipe networks by Labye's Iterative Dis-continues Method (LIDM).

### *4.3.1. EPANET*

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks.

Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some applications of EPANET in distribution systems analysis.

EPANET contains following hydraulic and water quality modelling capabilities (Rossman, 2000):

- places no limit on the size of the network that can be analysed
- computes friction headloss using the Hazen-Williams, Darcy-Weisbach, or Chezy-Manning formulas
- Includes minor head losses for bends, fittings, etc.
- models constant or variable speed pumps
- computes pumping energy and cost
- models various types of valves including shutoff, check, pressure regulating, and flow control valves
- allows storage tanks to have any shape (i.e., diameter can vary with height)
- considers multiple demand categories at nodes, each with its own pattern of time variation
- models pressure-dependent flow issuing from emitters (sprinkler heads)
- Can base system operation on both simple tank level or timer controls and on complex rule-based controls.
- blending water from different sources

- age of water throughout a system
- loss of chlorine residuals
- growth of disinfection by-products
- tracking contaminant propagation events.

Steps in Using EPANET

i. Draw a network representation of our distribution system or import a basic description of the network placed in a text file.

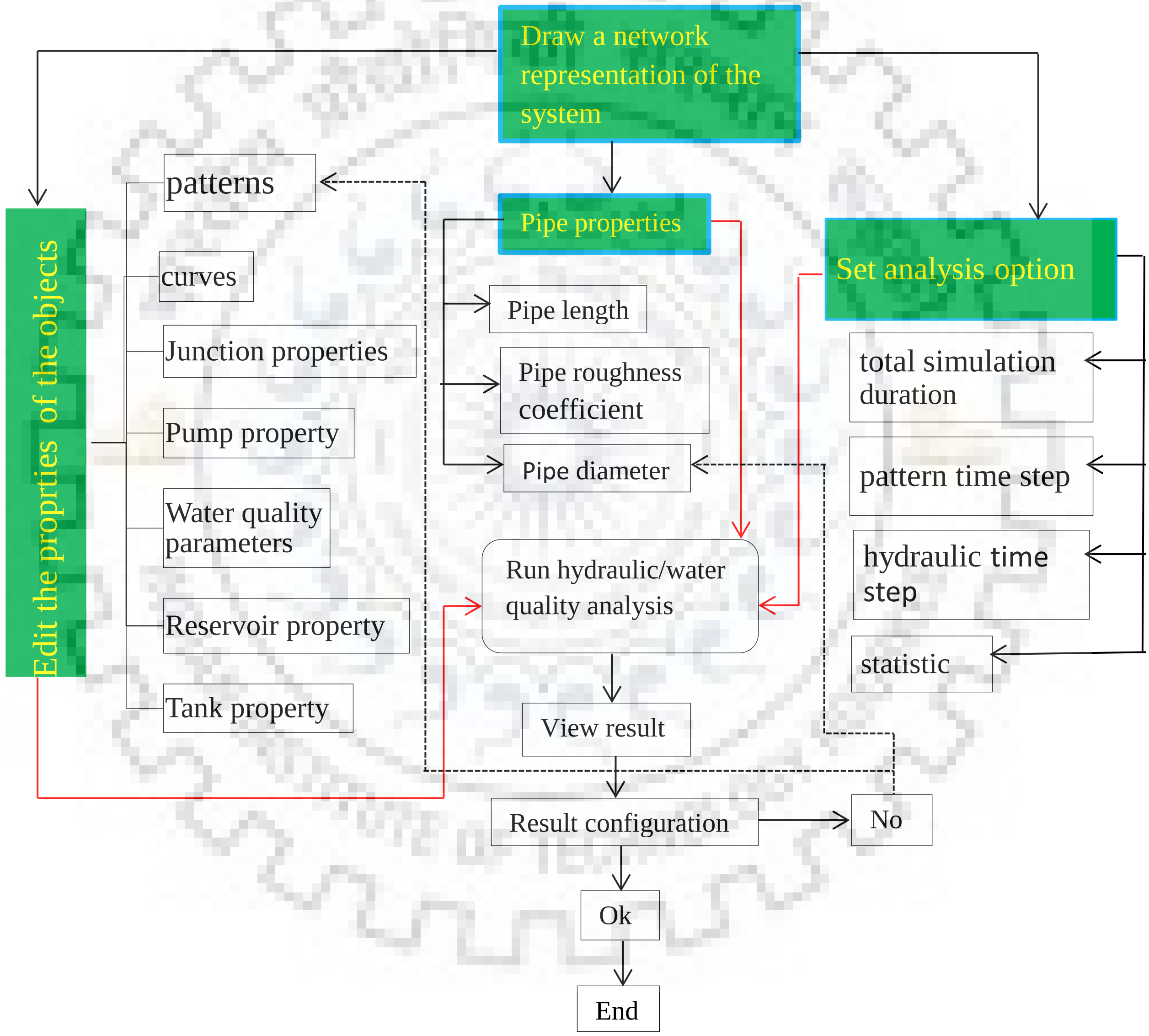



*Figure4.4: Components and Process utilized for computation of hydraulic simulation of the pressurized irrigation system using EPANET 2 software.*

ii. Edit the properties of the objects that make up the system.

iii. Describe how the system is operated.

iv. Select a set of analysis options.

v. Run a hydraulic/water quality analysis and

vi. View the results of the analysis

**The network model in EPANET**

**a. Physical Components**

EPANET models a water distribution system as a collection of links connected to nodes. The links represent pipes, pumps, and control valves. The nodes represent junctions, tanks, and reservoirs.

**b. Non-Physical Components**

EPANET employs three types of informational objects; curves, patterns, and controls that describe the behaviour and operational aspects of a distribution system.

**i. Curves**

Curves are objects that contain data pairs representing a relationship between two quantities. Two or more objects can share the same curve.

An EPANET model can utilize the following types of curves:

- Pump Curve
- Efficiency Curve
- Volume Curve
- Head Loss Curve

**Pump Curve**

A Pump Curve represents relation between the head and flow rate that a pump can convey at its nominal speed setting. Head is the head gain conferred to the water by the pump. A legitimate pump curve must have decreasing head with increasing flow.

A pump curve can be: single point curve, three point curve or multi-point curve

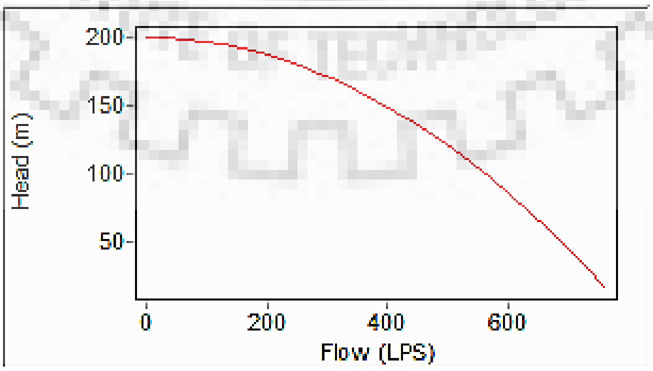



*Figure4.5: Example of Pump Curve*

**Efficiency Curve**

An Efficiency Curve decides pump effectiveness as a component of pump flow rate. Efficiency represent wire-to-water efficiency (a ratio between the electrical energy input to the pumps and the kinetic energy achieved by this input) that considers mechanical losses in the direct itself and also electrical losses in the pumps engine. The curve is used only for energy estimations.

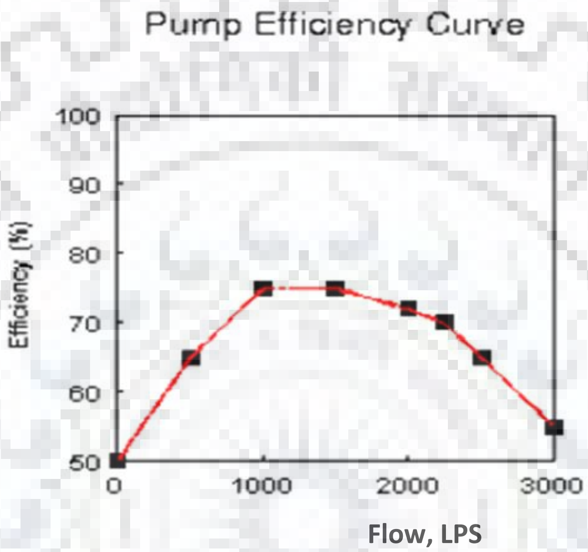



*Figure4.6: Example of efficiency curve*

**Volume Curve**

A volume curve decides how storage tank volume changes as a component of water level. It is utilized when it is important to precisely represent tanks whose cross-sectional area fluctuates with tallness. The lower and upper water levels provided for the curve must contain the lower and upper levels between which the tank works.

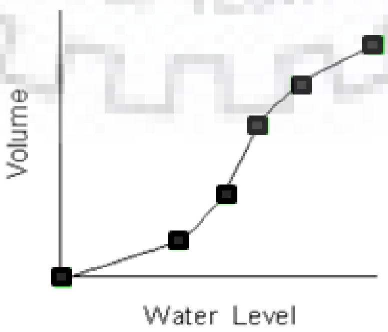



*Figure4.7: Example of volume Curve*

**ii. Time Patterns**

A time pattern is an accumulation of multipliers that can be connected to a quantity to enable it to fluctuate after some time. Nodal demands, reservoir heads, pump time tables, and water quality source information sources would all be able to have time patterns. The time interim utilized as a part of all patterns is a fixed value, adjusted with the project's time options. Inside this interim an amount stays at a consistent level, equivalent to the result of its nominal value and the pattern's multiplier for that time period.

**iii. Controls**

Controls decide how the system is operated after some time. They decide the status of chose interfaces as a component of time, tank water levels, and pressures at select focuses within the system. There are two classifications:

a) Simple Controls
b) Rule-Based Controls

**a. Simple Controls**

Simple controls change the status of a link based on:

- the water level in a tank,
- the pressure at a junction,
- the time into the simulation,
- the time of day

**b. Rule-Based Controls**

Rule-Based Controls permit connect status and settings to be founded on a blend of conditions that may exist in the system network after an initial hydraulic condition of the system is performed. This set of rules commands/ shuts down a pump and opens a by-pass pipe when the level in a tank exceeds a certain value and does the opposite when the level is below another value as an example.

***4.3.2. Labye's Iterative Dis-continues Method (LIDM)***

The method was proposed by Labye in 1981 for optimizing pipe sizes in an irrigation network. The method considers fixed discharges flowing into the pipes of the network. There were three basic steps that we should follow:

a. First, we construct initial solution by giving for section of the network the minimum commercially available pipe diameter according to the maximum valid flow velocity, when the pipe is capable of conveying the required discharge. Assigning $D_{min}$ to the minimum commercially available diameter, $V_{max}$ to maximum allowable flow velocity while the pipe is capable of supplying the calculated discharge, the diameter for any section can be calculated as:

$(D_{min})_e = \sqrt{\frac{4Q_e}{\pi v}}$ , in which $v \leq v_{max}$ ……………………………………………………4.1

b. Secondly, once we determine the initial diameter for each section of the network, it is possible to calculate the piezometric elevation at the upstream end of the network which satisfies the minimum head required at the most unfavourable hydrant.

$\boldsymbol{Z_{z,in}} = H_{\boldsymbol{j}\ \boldsymbol{min}} + \boldsymbol{Z_j} + \sum_{\mathbf{0}\to\mathbf{m_j}} \boldsymbol{Y_e}$ ……………………………………………………………...4.2

In which $\sum_{0\to M_j} Y_e$ are the head losses along the pathway '$m_j$' that connects the most unfavourable hydrant 'j' to the upstream end of the network.

$\boldsymbol{Z_j}$=elevation of node/hydrant j

$H_{j,min}$= the minimum head required at the most unfavourable hydrant j

c. Thirdly, the optimal solution is obtained by iteratively decreasing the piezometric elevation, at the most unfavourable hydrant $\boldsymbol{Z_{z,in}}$ until reaching the available effective upstream piezometric elevation $Z_z$. During this iteration process, for the section which we increase diameter, it produces the minimum increase in the network cost.

At any iteration i, the commercial pipe diameters $D_{x+1}$ and $D_x$ are known (in which $D_{x+1} > D_x$).

The values of coefficients $\beta_x$ are considered to identify the sections to be changed:

$\beta_x = \frac{C_{x+1} - C_x}{J_x - J_{x+1}} = \frac{\Delta c}{\Delta J}$ …………………………………………………………………………………………4.3

Where,

$P_x$=cost per unit length of pipe diameter, $D_x$ in metre

$J_x$=friction loss per unit length of pipe diameter, $D_x$ in $mm^{-1}$

$P_{x+1}$= cost per unit length of pipe diameter, $D_{x+1}$ in metre

$J_{x+1}$= friction loss per unit length of pipe diameter, $D_{x+1}$ in $mm^{-1}$

Again, the minimum cost variation, ΔC, of any sub-network, (sn), and a section (e) in series with (sn), for any given variation, ΔH', of the head H' (m), at the upstream end of (sn), is obtained by solving the following "local" linear programming((FAO, 2000); (Pereira *et al.*, 2003); (Rajabpour R. and Naser T., 2014)):

min. $\Delta C = -\beta_x, sn\ \Delta H - \beta_x, e\ \Delta Ye$……………………………………………...……………4.4

Subject to:

$\Delta H + \Delta Ye = \Delta H'$…………………………………………………………………………4.5

where;

$\Delta$H = the variation of the head at the upstream end of (sn), m.

$\Delta$Ye=the variation of the friction loss in section e, m. The optimal solution of the equations

$\Delta$H = $\Delta$H' and $\Delta Ye = 0$ if $\beta_s$,sn < $\beta_s$,e ……………………………………………….......4.6

$\Delta$H = 0 and $\Delta Ye = \Delta$H' if $\beta_s$,sn > $\beta_s$,e …………………………………………….........4.7

Therefore, the minimum cost variation, $\Delta$C, of 'sn' can be written as:

$\Delta$C = - $\beta$ * $\Delta$H'…...……………………......................................................................4.8

with $\beta$ = min ($\beta_x$, sn , $\beta_x$, e)…………………………………………………………........4.9

Proceeding from any terminal section of the pipe network, the equation.1.14 can be used to determine the section that will vary at each iteration.

In the case of a terminal section with a head in excess at its downstream end, the value of $\beta_x$, sn to be used in the process is equal to zero as long as the excess head exists.

The magnitude of $\Delta$H, for each iteration i, is determined as:

$\Delta H_i$ = min (EH $_i$,DY$_i$, DY$_i$)……………………………………………………………...4.10

Where:

EH $_i$ is the minimum value of the excess head prevailing in all the nodes where the head will change;

DY$_i$ is the minimum value of ($Ye,i$ - Y) for those sections which will change in diameters, with $Ye,i$ being the value of the head loss in the section 'e' at iteration i, and Y is, for this section, the value of the head loss corresponding to the largest diameter over its entire length if the section has two diameters, or the next greater diameter if the section has only one diameter. Note that for those terminal sections with head in excess, DYi is equal to the value of this excess (Hj - Hj, min).

DZ $_i$ is the difference between the upstream piezometric elevation, $(Z_Z)$i, at iteration i, and the piezometric elevation, $Z_Z$, effectively available at the upstream end of the network. The iterative process is continued until $Z_Z$ is reached, obtaining the optimal solution.

# CHAPTER 5

# RESULTS AND DISCUSSION

## 5.1 Introduction

### *5.1.1. Net Irrigation Requirement*

Net irrigation requirement is the depth of irrigation water necessary to meet evapotranspiration need of the crop during a certain period and specific growing season of the crop. It may be computed by the relationship given below:

$$d=\sum_{k=1}^{n}\frac{(FC_l-S_l)}{100}*Ai*Di \qquad \ldots\ldots 5.1$$

In which,

d=net amount of water to be applied during an irrigation, cm

$FC_l$=field capacity moisture content in the $l^{th}$ layer of the soil in per cent

$S_l$=moisture content before irrigation in the $l^{th}$ layer of the soil in per cent

$A_l$=bulk density of the soil in the $i^{th}$ layer

$D_l$=depth of the soil layer within the root zone, cm

Net irrigation requirement is the amount of irrigation water required to bring the soil moisture level in the effective root zone to field capacity.

Referring back to previous studies, some professionals have determined the net Irrigation requirement of sugar cane which is the major crop pattern of the area.

Among those studies undertaken at this study area, the data given in the following table is selected for further reference:

*Table5.1: Monthly and annual irrigation demand for the main system (Metahara)*

| Month | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| NIR (mm/day) | 4.4 | 3.9 | 4 | 4.5 | 6.2 | 8 | 5 | 4.2 | 5.8 | 5.9 | 5.8 | 4.9 |
| NIR (mm/ month) | 136 | 110 | 123 | 134 | 193 | 241 | 157 | 132 | 174 | 182 | 175 | 152 |
| Irrig. Req. lps | 0.51 | 0.45 | 0.46 | 0.52 | 0.72 | 0.93 | 0.59 | 0.49 | 0.67 | 0.68 | 0.68 | 0.57 |

Source: Z. A. Dejen et.al, 2015

Considering the overall efficiency of 75 per cent, the growth irrigation requirement (GIR) can be determined.

$$\text{GIR}=\frac{NIR}{system\ efficiency}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots...5.2$$

Where,

GIR=net irrigation requirement of a given crop plus an operational losses of the system in mm/day

NIR= net irrigation requirement in mm/day

The computed daily GIR is summarized in the table below:

*Table5.2: Computed daily GIR*

| Month | NIR (mm/month) | GIR (mm/month) | GIR (mm/month) | Flow rate required when the system is assumed to run for 10 hours per day ( lps) |
|---|---|---|---|---|
| Jan | 4.4 | 5.9 | 181.9 | 218.4 |
| Feb | 3.9 | 5.2 | 145.6 | 193.6 |
| Mar | 4 | 5.3 | 165.3 | 198.7 |
| Apr | 4.5 | 6 | 180 | 223.3 |
| May | 6.2 | 8.3 | 256.3 | 307.9 |
| Jun | 8 | 10.7 | 320 | 397.5 |
| Jul | 5 | 6.7 | 206.7 | 248.5 |
| Aug | 4.2 | 5.6 | 173.6 | 208.6 |
| Sep | 5.8 | 7.7 | 232 | 288.1 |
| Oct | 5.9 | 7.9 | 243.9 | 293 |
| Nov | 5.8 | 7.7 | 232 | 288.1 |
| Dec | 4.9 | 6.5 | 202.5 | 243.5 |

Considering 50% irrigation water requirement during the rainy season of months July, August and September; the growth irrigation water requirement per year is 2234 mm. Higher and lower water requirement was seen in the months of June and February respectively. In relation with this it is obvious that we consider the worst condition in design activities. As a result, the peak water demand during the month of June is considered to determine the water requirement of the area and for the computation of various hydraulic and related parameters.

*Table5. 3: Water demand at each node when the system is assumed to run for 10 hours* $\boldsymbol{day^{-1}}$

| Node | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| O (8.56ha) | 13.9 | 12 | 12.7 | 14 | 19.7 | 25.4 | 15.9 | 13 | 18.4 | 19 | 18.4 | 15.5 |
| N (13.15 ha) | 21.4 | 19 | 19.5 | 22 | 30.2 | 39 | 24.4 | 21 | 28.2 | 29 | 28.2 | 23.9 |
| K (5.11 ha) | 8.3 | 7.4 | 7.6 | 8.5 | 11.7 | 15.1 | 9.5 | 7.9 | 11 | 11 | 11 | 9.3 |
| L (7.3ha) | 11.9 | 11 | 10.8 | 12 | 16.8 | 21.6 | 13.5 | 11 | 15.7 | 16 | 15.7 | 13.2 |
| E (7.25ha) | 11.8 | 11 | 10.7 | 12 | 16.6 | 21.5 | 13.4 | 11 | 15.6 | 16 | 15.6 | 13.2 |
| F (8.43ha) | 13.7 | 12 | 12.5 | 14 | 19.4 | 25 | 15.6 | 13 | 18.1 | 18 | 18.1 | 15.3 |
| G (8.47ha) | 13.8 | 12 | 12.5 | 14 | 19.4 | 25.1 | 15.7 | 13 | 18.2 | 19 | 18.2 | 15.4 |
| H (11.4ha) | 18.6 | 17 | 16.9 | 19 | 26.2 | 33.8 | 21.1 | 18 | 24.5 | 25 | 24.5 | 20.7 |
| I (7.44ha) | 12.1 | 11 | 11 | 12 | 17.1 | 22 | 13.8 | 12 | 16 | 16 | 16 | 13.5 |
| U (8.29ha) | 13.5 | 12 | 12.3 | 14 | 19 | 24.6 | 15.4 | 13 | 17.8 | 18 | 17.8 | 15 |
| W (8.41ha) | 13.7 | 12 | 12.5 | 14 | 19.3 | 24.9 | 15.6 | 13 | 18.1 | 18 | 18.1 | 15.3 |
| V (6.57ha) | 10.7 | 9.5 | 9.7 | 11 | 15.1 | 19.5 | 12.2 | 10 | 14.1 | 14 | 14.1 | 11.9 |
| X (5.88ha) | 9.6 | 8.5 | 8.7 | 9.8 | 13.5 | 17.4 | 10.9 | 9.1 | 12.6 | 13 | 12.6 | 10.7 |
| Y (3.91ha) | 6.4 | 5.6 | 5.8 | 6.5 | 9 | 11.6 | 7.2 | 6.1 | 8.4 | 8.5 | 8.4 | 7.1 |
| Z' (1.41ha) | 2.3 | 2 | 2.1 | 2.3 | 3.2 | 4.2 | 2.6 | 2.2 | 3 | 3.1 | 3 | 2.6 |
| A' 4.64ha | 7.6 | 6.7 | 6.9 | 7.7 | 10.7 | 13.7 | 8.6 | 7.2 | 10 | 10 | 10 | 8.4 |

| Node | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| B' (6.16ha) | 10 | 8.9 | 9.1 | 10 | 14.1 | 18.3 | 11.4 | 9.6 | 13.2 | 14 | 13.2 | 11.2 |
| C' (5.42ha) | 8.8 | 7.8 | 8 | 9 | 12.4 | 16.1 | 10 | 8.4 | 11.6 | 12 | 11.6 | 9.8 |
| D' (6.32ha) | 10.3 | 9.1 | 9.4 | 11 | 14.5 | 18.7 | 11.7 | 9.8 | 13.6 | 14 | 13.6 | 11.5 |
| Total(l/s) | 218 | 194 | 199 | 223 | 308 | 398.5 | 249 | 209 | 288 | 293 | 288 | 244 |

As we can analyse from the above table the water demand varies greatly season wise.

**5.2. Data Input**

For the hydraulic and water quality analyses using the EPANET 2 software, there were physical and non-physical data's that we should input to run the analysis. These data's are summarized under appendix table1 and 2.

In order to simplify the repetitive trial and error input of the initial pipe diameter for each link, the pipe diameter was computed manually depending upon the flow required in each pipe and the assumed flow velocity of 1.5 m/s for main and 1.25 m/s for the rest of the links. This does not mean that we manually compute the pipe diameters, rather when we calculate the pipe diameters for each link depending upon the flow that the pipe should deliver, we get values which approaches to the software result even though the manually computed was larger. This reduces the repetitions of input data by trial and error for the diameter.

**5.3. System flow**

After running the system analysis, we check for the properness of values computed. (Figure 5.1) shows the flow and pressure map of the proposed pipe network. The pressure variation within the system is between more than 15m to less than 45m while the flow in the network varies from less than 25 l/s to more than 100 l/s. In order to control the system flow, three one way flow control valves (FCV): V1, V2 and V3 were proposed on links 5, 4 and 28 respectively. It was supposed that valve V3 is kept closed if valve V1 is open and vice versa in line with valve V2 which is supposed to be open in the two case. In relation to this, when valve V1 is closed, pipe 5 and 27 were also kept closed and when valve V3 is closed, pipe 28 is kept closed. The nodal and link output values of the software were attached at the end of the material under appendix 4 and 5 for reference. The flow velocities gained after running system analysis was 0.52 m/s minimum and 1.41m/s maximum while the minimum and maximum gained pressure was 23m at nodes N and X and 33m at A respectively.

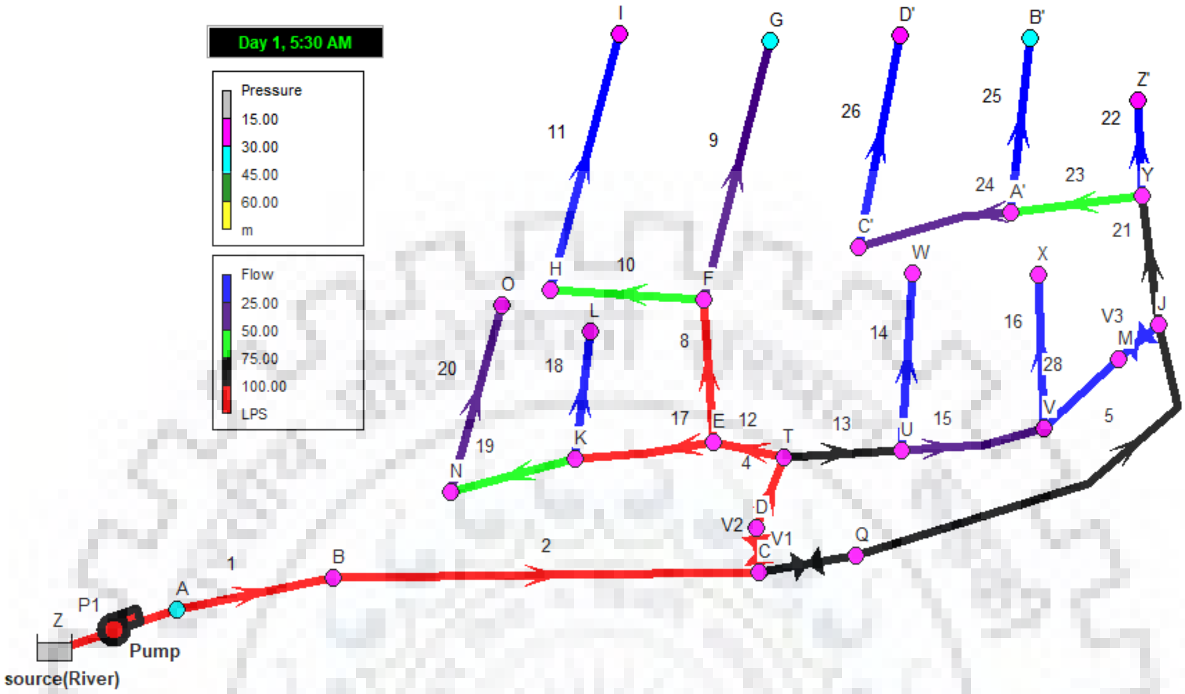



*Figure5.1: Flow and pressure map of the proposed Pipe Network (V3 close)*

### 5.4. Comparative view of water use under PIN and CIN

As discussed earlier, Z. D. Dejen et al. (2015) determined the five years (2006-2010) average annual water diversion at Metahara Irrigation scheme as 190 $Mm^3$ and affirmed that this diverted volume exceeds the annual demand by 24 %. The excess diversion was because of wastage in the conveyance, storage and distribution systems.

So the water supplied per unit ha area= $(\frac{190\text{Mm3}}{11{,}500\text{ha}})$/year = 16,522$m^3 ha^{-1} year^{-1}$.

In determining the water need for this study area, it was assumed that the demand may reduce by half for the three rainy months of the season.

The designed water need per unit ha area under the piped irrigation network by considering half demand during the three rainy months and nine days irrigation interval =daily demand * (365÷irrigation interval)÷(irrigable area under consideration)=36,537*$(\frac{275}{9}+\frac{90}{18})$÷134.12=9808 $m^3$ $ha^{-1}year^{-1}$. When we compare the water applied under canal network and designed water need under the pipe network; there was 6,714 $m^3$/ ha / year less water need for the pipe network. This implies that we could save 900,482 $M^3$ volume of water year for this study area. Hence, additional 91.8ha/year of land can be developed if the system is converted to PIS, with the same volume of water used under the canal irrigation network.

This implies that the total quantity of water that might be saved yearly and the total land that could be developed as much as the pipe network is serving properly is far better than the conventional method even though there could be running costs.

### 5.5. Pipe Cost Estimation

Depending upon the pipe diameters gained from the EPANET output (appendix table 4), the cost of the pipe network was determined as table 8 below.

The cost of the network was calculated using the formula:

$C_t = \sum D_x * L_x$ ………………………………………………………………………………5.3

Where;

$C_t$ is the total cost of the system in ETB

$D_x$ is the x$^{th}$ section pipe diameter cost per metre

$L_x$ is the length in meters of the x$^{th}$ section pipe

*Table5.4: Commercially available pipe cost*

| Diameter (mm) | | 75 | 80 | 100 | 150 | 200 | 250 | 300 | 350 | 400 | 450 | 500 | 550 | 600 | 630 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Basin, m$^{0.5}$ | | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 | 0.06 |
| cost per metre (ETB) | N10 | 91 | 95 | 110 | 135 | 148 | 178 | 210 | 258 | 264 | 291 | 318 | 350 | 372 | 388 |
| | PN16 | - | - | - | - | 580 | 875 | 1100 | 1450 | 1980 | 2252 | 2790 | 2954 | 3386 | 3872 |

N.B: 1ETB = 2.31 INR (₹) =0.036 US$.

*Table5.5: Estimated pipe cost*

| Pipe dia. from EPANET | | | |
|---|---|---|---|
| Dia. (mm) | Length (m) | Unit cost (ETB) | Sub-total (ETB) |
| 100 | 114 | 110 | 12,540 |
| 150 | 1190 | 135 | 160,650 |
| 200 | 2532 | 580 | 1,468,560 |
| 250 | 1185 | 875 | 1,036,875 |
| 300 | 734 | 1100 | 807,400 |
| 350 | 272 | 1450 | 394,400 |
| 400 | 163 | 1980 | 322,740 |
| 450 | 960 | 2252 | 2,161,920 |
| 500 | 42.5 | 2790 | 118,575 |
| 550 | 0 | 2954 | 0 |
| 600 | 1509 | 3386 | 5,109,474 |
| Total | | | 11,593,134 |

**5.6. Energy cost**

It is obvious that one of the drawbacks of pressurized irrigation system is its external energy requirement. As shown below, the energy requirement and cost per day in US$ extracted from the EPANET output is 0.13 KW-hr/m3 and 261.84 US$ per day respectively. This implies that an external cost of 104,390.25 ETB/yr. is required to run the system assuming 10 hours running time for the pump and 9 days irrigation interval.

```
Energy Usage:
------------------------------------------------------------------
                  Usage    Avg.      Kw-hr       Avg.      Peak
Cost
Pump/day          Factor   Effic.     /m3          Kw         Kw
-------------------------------------------------------------------
  P1
261.84             100.00   75.00      0.13     181.83     181.83
-------------------------------------------------------------------
                                                Demand Charge: 0.00
                                                 Total Cost: 261.84
```

## 5.7. Pipe size Optimization

### *5.7.1. Labye's Iterative Dis-continues Method (LIDM)*

In any designing aspect of projects, the cost of the project is the leading factor in line with the technical requirements. This implies that we should carefully determine the cost of the components of the project so that its economic return will be satisfactory. Keeping this view in mind, LIDM was selected to optimize the cost of the pipe network computed by the software in this case.

In this method, in order to decide the section for which we should change diameter first and also to know the next to come, the following points are given for clarification:

- $(\beta,_{\min})_x$ determines the section for which we are going to change diameter next depending on the minimum values among:
    i. excess available head (if exist) , m
    ii. Change in piezometric head (the difference in head between the available piezometric head at the most unfavourable head and the effective piezometric head found at the upstream of the sub-network/section.
    iii. change in head due to change in pipe diameter from the minimum existing to the next larger diameter for the path/section under consideration, m

    Hence, the minimum value among i, ii and iii, is the value which determines the section/s for which we change the diameter next. In addition, it also characterizes either to change to whole diameter of the section or to change part of it to the next larger diameter.

This method was selected because of that it considers pipe size in relation with the effective pressure required to run the system. To simplify the optimization process, the system network was represented by sections.

### *5.7.2. Sections of the network*

The term network 'section' in this context is to mean a link /group of links from one start node to other end node representing a link or successive links with equal pipe diameter/s. For instance, links 1 and 2 (Fig. 5.1) were represented by section 1 (Fig. 5.2). Similarly, link 5 was represented by section 17 and does for other links. This was done to simplify the process of optimization of pipe size by LIDM.

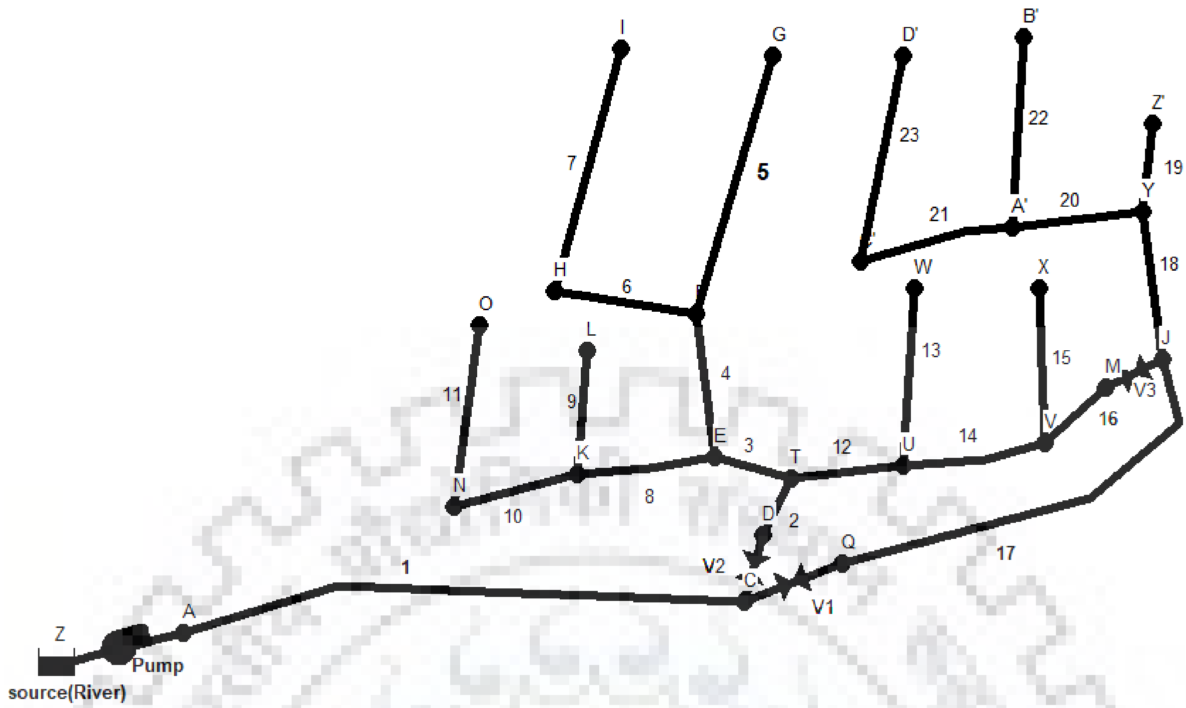



*Figure5.2: The network sections*

*Table5.6: path and sections of the network when V3 is close*

| s.no | Path | | Path sections | Path Length, m |
|---|---|---|---|---|
| 1 | Z-C-T-E-K-N-O | Z→O | 1-2-3-8-10-11 | 2741 |
| 2 | Z-C-T-E-K-L | Z→L | 1-2-3-8-9 | 2324 |
| 3 | Z-C-T-E-F-H-I | Z→I | 1-2-3-4-6-7 | 2942 |
| 4 | Z-C-T-E-F-G | Z→G | 1-2-3-4-5 | 2287 |
| 5 | Z-C-T-U-V-J | Z→J | 1-2-12-14-15 | 2231 |
| 6 | Z-C-T-U-V-X | Z→X | 1-2-12-14-16 | 2525 |
| 7 | Z-C-T-U-W | Z→W | 1-2-12-13 | 2160 |
| 8 | Z-C-T-U | Z→U | 1-2-12 | 1836 |
| 9 | Z-C-T-E | Z→E | 1-2-3 | 1715 |
| 8 | Z-C-J-Y-A’-C’-D’ | Z→D’ | 1-17-18-20-21-23 | 3601 |
| 9 | Z-C -J-Y-A’-B’ | Z→B’ | 1-17-18-20-22 | 3041 |
| 10 | Z-C -J-Y-Z’ | Z→Z’ | 1-17-18-19 | 2723 |

The first step in LIDM is to construct initial solution after we determine the flow velocities for each section depending on the commercially available pipe diameters. Once we determine the flow velocity, we select the valid flow velocities depending on the acceptable minimum and maximum flow velocities for each section as per our requirement. As discussed earlier 0.5 m/s minimum and 2.5 m/s flow velocity was considered in this case. For this study, the next step to flow velocity determination is computation of friction headloss for the minimum possible pipe diameter of each section. This was done to fix the maximum possible pressure required at each node if the minimum pipe diameter was used so that the system hydraulics is saved.

*Table5.7: valid flow velocity for each section when valve V3 is close*

| section | Equivalent link no | Diameter, $\emptyset(mm)$ | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | **75** | **80** | **100** | **150** | **200** | **250** | **300** | **350** | **400** | **450** | **500** | **550** | **600** | **630** |
| 1 | 1+2 | | | | | | | | | | | | 2.21 | 1.93 | 1.75 |
| 2 | 4 | | | | | | | | | 2.5 | 1.98 | 1.60 | 1.28 | 1.11 | 1.01 |
| 17 | 5 | | | | | | | | | | 2.28 | 1.85 | 1.47 | 1.28 | 1.16 |
| 4 | 8 | | | | | | 2.16 | 1.50 | 1.10 | 0.84 | 0.67 | 0.54 | | | |
| 5 | 9 | | | | 1.42 | 0.80 | 0.51 | | | | | | | | |
| 6 | 10 | | | | | 1.78 | 1.14 | 0.79 | 0.58 | | | | | | |
| 7 | 11 | | | | 1.25 | 0.70 | | | | | | | | | |
| 3 | 12 | | | | | | | | 2.38 | 1.82 | 1.44 | 1.16 | 0.93 | 0.81 | 0.73 |
| 12 | 13 | | | | | | 1.76 | 1.22 | 0.90 | 0.69 | 0.54 | | | | |
| 13 | 14 | | | | 1.41 | 0.79 | 0.51 | | | | | | | | |
| 14 | 15 | | | | 2.09 | 1.17 | 0.75 | 0.52 | | | | | | | |
| 15 | 16 | | | 2.22 | 0.98 | 0.55 | | | | | | | | | |
| 8 | 17 | | | | | | 2.06 | 1.43 | 1.05 | 0.80 | 0.64 | 0.51 | | | |
| 9 | 18 | | | | 1.22 | 0.69 | | | | | | | | | |
| 10 | 19 | | | | | 2.05 | 1.31 | 0.91 | 0.67 | 0.51 | | | | | |
| 11 | 20 | | | | 1.44 | 0.81 | 0.52 | | | | | | | | |
| 18 | 21 | | | | | | 1.68 | 1.17 | 0.86 | 0.66 | 0.52 | | | | |
| 19 | 22 | 0.95 | 0.84 | 0.53 | | | | | | | | | | | |
| 20 | 23 | | | | | 2.13 | 1.36 | 0.95 | 0.69 | 0.53 | | | | | |
| 22 | 24 | | | | 1.97 | 1.11 | 0.71 | | | | | | | | |
| 21 | 25 | | | 2.33 | 1.04 | 0.58 | | | | | | | | | |
| 23 | 26 | | | 2.38 | 1.06 | 0.60 | | | | | | | | | |

Hence, the list of the pipe diameters valid for each section according the allowable range of the flow velocities is as follows:

*Table5. 8: valid pipe diameters for section each section when Valve V3 is close*

| Section | Diameter in mm | | | | | |
|---|---|---|---|---|---|---|
| 1 | 550 | 600 | 630 | | | |
| 2 | 400 | 450 | 500 | 550 | 600 | 630 |
| 17 | 250 | 300 | 350 | 400 | 450 | |
| 4 | 250 | 300 | 350 | 400 | 450 | 500 |
| 5 | 150 | 200 | 250 | | | |
| 6 | 200 | 250 | 300 | 350 | | |
| 7 | 150 | 200 | | | | |
| 3 | 350 | 400 | 450 | 500 | 550 | 600 | 630 |
| 12 | 250 | 300 | 350 | 400 | 450 | | |
| 13 | 150 | 200 | 250 | | | | |
| 14 | 150 | 200 | 250 | 300 | | | |
| 15 | | | 100 | 150 | 200 | | |
| 8 | 250 | 300 | 350 | 400 | 450 | 500 | |
| 9 | 150 | 200 | | | | | |
| 10 | 200 | 250 | 300 | 350 | 400 | | |
| 11 | 150 | 200 | 250 | | | | |
| 18 | 250 | 300 | 350 | 400 | 450 | | |
| 19 | 75 | 80 | 100 | | | | |
| 20 | 200 | 250 | 300 | 350 | 400 | | |
| 22 | 150 | 200 | 250 | | | | |
| 21 | 100 | 150 | 200 | | | | |
| 23 | 100 | 150 | 200 | | | | |
| 16 | 250 | 300 | 350 | 400 | 450 | | |

To compute the head loss due to friction ‘J’ in the pipes, LIDM applies the Darcy equation which is:

$J=u*Q^2$……………………………………………………………5.4

where; $u=0.000857(1+2\gamma*D^{-0.5})^2*D^{-5}$…………………………………......5.5

Q is the discharge flowing in to the section in $M^3/s$

$\gamma$ is the roughness parameter of Basin in $m^{0.5}$

D is the diameter of the section in metre

By applying equation 5.5 the values of ‘u’ are computed and summarized as follows:

| Diameter(m) | u |
|---|---|
| 75 | 746.964 |
| 50 | 530.532 |
| 100 | 163.083 |
| 150 | 19.362 |
| 200 | 4.308 |
| 250 | 1.349 |
| 300 | 0.524 |
| 350 | 0.236 |
| 400 | 0.118 |
| 450 | 0.065 |
| 500 | 0.038 |
| 550 | 0.023 |
| 600 | 0.015 |
| 630 | 0.011 |

Once the acceptable flow velocity as per the requirement is determined, and the initial solution is constructed, the next step is to compute the friction head loss in each section by using the Darcy’s equation. The computed friction slope was summarized as next table.

*Table5.9: The friction slope (J) in each section*

| Sections | Dia. (mm) | u | Flow, $m^3/s$ | length(m) | J (m) |
|---|---|---|---|---|---|
| section 1 | 550 | 0.023 | 0.545 | 1509 | 10.31 |
| | 600 | 0.015 | ’’ | ’’ | 6.59 |
| | 630 | 0.011 | ’’ | ’’ | 5.13 |
| section 2 | 400 | 0.118 | 0.315 | 42.5 | 0.5 |
| | 450 | 0.065 | ’’ | ’’ | 0.27 |
| | 500 | 0.038 | ’’ | ’’ | 0.16 |
| | 550 | 0.023 | ’’ | ’’ | 0.1 |
| | 600 | 0.015 | ’’ | ’’ | 0.06 |
| | 630 | 0.011 | ’’ | ’’ | 0.05 |

| Section | Dia. (mm) | u | Flow, $m^3/s$ | length(m) | J (m) |
|---|---|---|---|---|---|
| section 17 | 450 | 0.065 | 0.23 | 960 | 8.84 |
| | 500 | 0.038 | ’’ | ’’ | 3.43 |
| | 550 | 0.023 | ’’ | ’’ | 1.55 |
| | 600 | 0.015 | ’’ | ’’ | 0.78 |
| | 630 | 0.011 | ’’ | ’’ | 0.42 |
| section 4 | 250 | 1.349 | 0.106 | 272 | 4.12 |
| | 300 | 0.524 | ’’ | ’’ | 1.6 |
| | 350 | 0.236 | ’’ | ’’ | 0.72 |
| | 400 | 0.118 | ’’ | ’’ | 0.36 |
| | 450 | 0.065 | ’’ | ’’ | 0.2 |
| | 500 | 0.038 | ’’ | ’’ | 0.12 |
| section 5 | 150 | 19.362 | 0.025 | 572 | 6.98 |
| | 200 | 4.308 | ’’ | ’’ | 1.55 |
| | 250 | 1.349 | ’’ | ’’ | 0.49 |
| section 6 | 200 | 4.308 | 0.056 | 325 | 4.36 |
| | 250 | 1.349 | ’’ | ’’ | 1.37 |
| | 300 | 0.524 | ’’ | ’’ | 0.53 |
| | 350 | 0.236 | ’’ | ’’ | 0.24 |
| section 7 | 150 | 19.362 | 0.022 | 630 | 5.9 |
| | 200 | 4.308 | ’’ | ’’ | 1.31 |
| section 3 | 350 | 0.236 | 0.229 | 163 | 2.01 |
| | 400 | 0.118 | ’’ | ’’ | 1 |
| | 450 | 0.065 | ’’ | ’’ | 0.55 |
| | 500 | 0.038 | ’’ | ’’ | 0.32 |
| | 550 | 0.023 | ’’ | ’’ | 0.2 |
| | 600 | 0.015 | ’’ | ’’ | 0.13 |
| | 630 | 0.011 | ’’ | ’’ | 0.09 |
| section 12 | 250 | 1.349 | 0.086 | 284 | 2.86 |
| | 300 | 0.524 | ’’ | ’’ | 1.11 |
| | 350 | 0.236 | ’’ | ’’ | 0.5 |
| | 400 | 0.118 | ’’ | ’’ | 0.25 |
| | 450 | 0.065 | ’’ | ’’ | 0.14 |

| Section | Dia. (mm) | u | Flow, $m^3/s$ | length(m) | J (m) |
|---|---|---|---|---|---|
| section 13 | 150 | 19.36 | 0.025 | 324 | 3.89 |
| | 200 | 4.31 | ” | ” | 0.87 |
| | 250 | 1.35 | ” | ” | 0.27 |
| section 14 | 150 | 19.362 | 0.037 | 371 | 9.78 |
| | 200 | 4.308 | ” | ” | 2.18 |
| | 250 | 1.349 | ” | ” | 0.68 |
| | 300 | 0.524 | ” | ” | 0.26 |
| section 15 | 100 | 163.083 | ” | ” | 18.32 |
| | 150 | 19.362 | ” | ” | 2.17 |
| | 200 | 4.308 | ” | ” | 0.48 |
| section 8 | 250 | 1.349 | 0.101 | 286 | 3.94 |
| | 300 | 0.524 | ” | ” | 1.53 |
| | 350 | 0.236 | ” | ” | 0.69 |
| | 400 | 0.118 | ” | ” | 0.35 |
| | 450 | 0.065 | ” | ” | 0.19 |
| | 500 | 0.038 | ” | ” | 0.11 |
| section 9 | 150 | 19.362 | 0.022 | 323 | 2.92 |
| | 200 | 4.308 | ” | ” | 0.65 |
| section 10 | 200 | 4.308 | ” | 314 | 5.61 |
| | 250 | 1.349 | ” | ” | 1.76 |
| | 300 | 0.524 | ” | ” | 0.68 |
| | 350 | 0.236 | ” | ” | 0.31 |
| | 400 | 0.118 | ” | ” | 0.15 |
| section 11 | 150 | 19.362 | 0.025 | 426 | 5.32 |
| | 200 | 4.308 | ” | ” | 1.18 |
| | 250 | 1.349 | ” | ” | 0.37 |
| section 18 | 250 | 1.349 | 0.083 | 250 | 1.29 |
| | 300 | 0.524 | ” | ” | 0.5 |
| | 350 | 0.236 | ” | ” | 0.23 |
| | 400 | 0.118 | ” | ” | 0.11 |
| | 450 | 0.065 | ” | ” | 0.06 |

| Section | Dia. (mm) | u | Flow, $m^3/s$ | length(m) | J (m) |
|---|---|---|---|---|---|
| section 19 | 80 | 530.532 | 0.004 | 114 | 1.07 |
| | 100 | 163.083 | ’’ | ’’ | 0.33 |
| section 20 | 200 | 4.308 | 0.067 | ’’ | 2.31 |
| | 250 | 1.349 | ’’ | ’’ | 0.72 |
| | 300 | 0.524 | ’’ | ’’ | 0.28 |
| | 350 | 0.236 | ’’ | ’’ | 0.13 |
| | 400 | 0.118 | ’’ | ’’ | 0.06 |
| section 22 | 150 | 19.362 | 0.035 | 312 | 7.32 |
| | 200 | 4.308 | ’’ | ’’ | 1.63 |
| | 250 | 1.349 | ’’ | ’’ | 0.51 |
| section 21 | 100 | 163.083 | 0.018 | 417 | 22.77 |
| | 150 | 19.362 | ’’ | ’’ | 2.7 |
| | 200 | 4.308 | ’’ | ’’ | 0.6 |
| section 23 | 100 | 163.083 | 0.019 | 455 | 25.95 |
| | 150 | 19.362 | ’’ | ’’ | 3.08 |
| | 200 | 4.308 | ’’ | ’’ | 0.69 |
| section 16 | 250 | 1.349 | 0.083 | 24 | 0.22 |
| | 300 | 0.524 | ’’ | ’’ | 0.09 |
| | 350 | 0.236 | ’’ | ’’ | 0.04 |
| | 400 | 0.118 | ’’ | ’’ | 0.02 |
| | 450 | 0.065 | ’’ | ’’ | 0.01 |
| | 500 | 0.038 | ’’ | ’’ | 0.01 |

The coefficient $\beta_x$ (equation 4.3) which is the ratio of change in the cost (in ETB) per meter of the smaller and next larger pipe diameters valid for the section/sub-network to change in the friction head loss due to this change in pipe diameters of that section/sub-network is calculated and summarized as follows:

$$\beta_x = \frac{P_{x+1} - p_x}{J_x - J_{x+1}}$$

| Section | dia, mm | $J_x - J_{x+1}$ | $P_{x+1} - p_x$ | $\beta_x$ |
|---|---|---|---|---|
| section 1 | 550 | 3.58 | 432 | 121 |
| | 600 | 1.79 | 486 | 271 |
| | 630 | | | |
| section 2 | 400 | 0.22 | 272 | 1,218 |
| | 450 | 0.11 | 538 | 4,728 |
| | 500 | 0.06 | 164 | 2,594 |
| | 550 | 0.03 | 432 | 2,813 |
| | 600 | 0.02 | 486 | 8,830 |
| | 630 | | | |
| section 17 | 450 | 5.405 | 538 | 100 |
| | 500 | 1.887 | 164 | 87 |
| | 550 | 0.770 | 432 | 561 |
| | 600 | 0.353 | 486 | 1,376 |
| | 630 | | | |
| section 4 | 250 | 2.52 | 225 | 89 |
| | 300 | 0.88 | 350 | 398 |
| | 350 | 0.36 | 530 | 1,472 |
| | 400 | 0.16 | 272 | 1,682 |
| | 450 | 0.08 | 538 | 6,532 |
| | 500 | | | |
| section 5 | 150 | 5.42 | 445 | 82 |
| | 200 | 1.07 | 295 | 277 |
| | 250 | | | |
| section 6 | 200 | 2.99 | 295 | 99 |
| | 250 | 0.83 | 225 | 270 |
| | 300 | 0.29 | 350 | 1,201 |
| | 350 | | | |
| section 7 | 150 | 4.59 | 445 | 97 |
| | 200 | | | |

| Section | dia, mm | $J_x - J_{x+1}$ | $P_{x+1} - p_x$ | $\beta_x$ |
|---|---|---|---|---|
| section 3 | 350 | 1.00 | 530 | 528 |
| | 400 | 0.45 | 272 | 603 |
| | 450 | 0.23 | 538 | 2,341 |
| | 500 | 0.13 | 164 | 1,285 |
| | 550 | 0.07 | 432 | 6,345 |
| | 600 | 0.03 | 486 | 14,276 |
| | 630 | | | |
| section 12 | 250 | 1.75 | 225 | 129 |
| | 300 | 0.61 | 350 | 573 |
| | 350 | 0.25 | 530 | 2,119 |
| | 400 | 0.11 | 272 | 2,421 |
| | 450 | | | |
| section 13 | 150 | 3.02 | 445 | 147 |
| | 200 | 0.59 | 295 | 496 |
| | 250 | | | |
| section 14 | 150 | 7.60 | 445 | 59 |
| | 200 | 1.49 | 295 | 197 |
| | 250 | 0.42 | 225 | 540 |
| | 300 | | | |
| section 15 | 100 | 16.14 | 25 | 2 |
| | 150 | 1.69 | 445 | 263 |
| | 200 | | | |
| section 8 | 250 | 2.41 | 225 | 93 |
| | 300 | 0.84 | 350 | 416 |
| | 350 | 0.34 | 530 | 1,542 |
| | 400 | 0.16 | 272 | 1,726 |
| | 450 | 0.08 | 538 | 6,811 |
| | 500 | | | |
| section 9 | 150 | 2.27 | 445 | 196 |
| | 200 | | | |

| Section | dia, mm | $J_x - J_{x+1}$ | $P_{x+1} - p_x$ | $\beta_x$ |
|---|---|---|---|---|
| section 10 | 200 | 3.85 | 295 | 77 |
| | 250 | 1.07 | 225 | 209 |
| | 300 | 0.38 | 350 | 933 |
| | 350 | 0.15 | 530 | 3,460 |
| | 400 | | | |
| section 11 | 150 | 4.14 | 445 | 108 |
| | 200 | 0.81 | 295 | 363 |
| | 250 | | | |
| section 18 | 250 | 1.41 | 225 | 160 |
| | 300 | 0.49 | 350 | 712 |
| | 350 | 0.2 | 530 | 2,642 |
| | 400 | 0.09 | 272 | 2,958 |
| | 450 | | | |
| section 19 | 80 | 0.74 | 15 | 20 |
| | 100 | | | |
| section 20 | 200 | 1.58 | 295 | 186 |
| | 250 | 0.44 | 225 | 509 |
| | 300 | 0.15 | 350 | 2,269 |
| | 350 | 0.06 | 530 | 8,416 |
| | 400 | | | |
| section 22 | 150 | 5.69 | 445 | 78 |
| | 200 | 1.12 | 295 | 264 |
| | 250 | | | |
| section 21 | 100 | 20.07 | 44 | 2 |
| | 150 | 2.1 | 445 | 212 |
| | 200 | | | |
| section 23 | 100 | 22.87 | 25 | 1 |
| | 150 | 2.4 | 445 | 186 |
| | 200 | | | |
| Section 16 | 250 | 0.14 | 225 | 6,475 |
| | 300 | 0.05 | 350 | 23,320 |
| | 350 | 0.02 | 530 | 75,290 |
| | 400 | 0.01 | 272 | 224,264 |
| | 450 | 0 | 538 | 508,836 |
| | 500 | | | |

## 5.8. Initial solution

Considering the paths (Fig. 5.2) and taking the EPANET out put pressure as the minimum head required at each node, we can compute the excess piezometric elevation available at each node. When computing the pipe size by the LIDM method, the pipe size computed should preferably be either equal or smaller than the one gained from the software output. This is because of that our objective is to minimize the network cost. Since the pressures gained at each node is greater than the one computed manually it is expected that the system is safe if the pipe diameter which may be reached after optimization is one less the one gained from EPANET among the commercially available pipe diameters.

*Table5.10: Pressure at each node when valves V3 is close*

| Node ID | Elevation (m) | Base Demand (LPS) | H.G.L. (m) | Pressure (m) |
|---|---|---|---|---|
| A | 990 | 0 | 1023 | 33 |
| B | 996 | 0 | 1022 | 26 |
| C | 993 | 0 | 1018 | 25 |
| Q | 992 | 0 | 1018 | 26 |
| D | 993 | 0 | 1018 | 25 |
| E | 994 | 21.5 | 1018 | 24 |
| K | 993 | 15.1 | 1017 | 24 |
| N | 993 | 39 | 1016 | 23 |
| O | 990 | 25.4 | 1014 | 24 |
| F | 991 | 25 | 1017 | 26 |
| H | 988 | 33.8 | 1016 | 28 |
| I | 987 | 22 | 1014 | 27 |
| G | 984 | 25.1 | 1014 | 30 |
| U | 992 | 24.6 | 1018 | 26 |
| W | 991 | 24.9 | 1016 | 25 |
| V | 992 | 19.5 | 1017 | 25 |
| X | 991 | 17.4 | 1014 | 23 |
| Y | 991 | 11.6 | 1017 | 26 |
| Z' | 988 | 4.2 | 1017 | 29 |
| B' | 985 | 18.3 | 1015 | 30 |
| D' | 985 | 18.7 | 1014 | 29 |
| A' | 990 | 13.7 | 1016 | 26 |
| C' | 990 | 16.1 | 1015 | 25 |

| Node ID | Elevation (m) | Base Demand (LPS) | H.G.L. (m) | Pressure (m) |
|---|---|---|---|---|
| L | 989 | 21.6 | 1016 | 27 |
| J | 992 | 0 | 1017 | 25 |
| M | 992 | 0 | 1017 | 25 |
| T | 992 | 0 | 1018 | 26 |
| Resvr Z | 988 | #N/A | 988 | 0 |

The computed piezometric elevation (using equation 4.2) at selected nodes when valve V3 is close:

$Z_{Z\text{-}D}$'=1004+$J_{100}$, 23*L23+$J_{100}$, 21*L21+$J_{200}$, 20*L20+$J_{250}$, 18*L18+$J_{450}$, 17*L17+$J_{550}$, 1*L1

=1004+25.95+7.32+2.31+1.29+8.84+10.31=1060.02

$Z_{Z\text{-}B}$'=1007+$J_{150}$, 22*L22+$J_{200}$, 20*L20+$J_{250}$, 18*L18+$J_{450}$, 17*L17+$J_{550}$, 1*L1

=1007+7.32+2.31+1.29+8.84+10.31=1037.07

$Z_{Z\text{-}Z}$'=1012+$J_{80}$, 19* L19+$J_{250}$, 18*L18+$J_{450}$, 17L17+$J_{550}$, 1*L1

=1012+1.07+1.29+8.84+10.31=1033.51

$Z_{Z\text{-}I}$=1012+$J_{150}$, 7*$L_7$+ $J_{200}$, 6*$L_6$+$J_{250}$, 4*$L_4$+$J_{350}$, 3*$L_3$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1012+5.9+4.36+4.12+2.01+0.5+10.31=1039.2

$Z_{Z\text{-}G}$= 1013+$J_{150}$, 5*$L_5$+$J_{250}$, 4*$L_4$+$J_{350}$, 3*$L_3$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1013+6.98+4.12+2.01+0.5+10.31=1036.92

$Z_{Z\text{-}O}$=1012+$J_{150}$, 11*$L_{11}$+$J_{200}$, 10*$L_{10}$+$J_{250}$, 8* $L_8$+$J_{350}$, 3*$L_3$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1012+5.32+5.61+3.94+2.01+4.12+0.5+10.31=1043.81

$Z_{Z\text{-}L}$=1014+ $J_{150}$, 9*$L_9$+$J_{250}$, 8* $L_8$+$J_{350}$, 3*$L_3$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1014+2.92+3.94+2.01+0.5+10.31=1033.68

$Z_{Z\text{-}X}$=1010+$J_{100}$, 15*$L_{15}$+$J_{150}$, 14*$L_{14}$+$J_{250}$, 12*$L_{12}$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1010+18.32+9.78+2.86+0.5+10.31=1051.77

$Z_{Z\text{-}W}$=1015+$J_{150}$, 13*$L_{13}$+$J_{250}$, 12*$L_{12}$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1015+3.89+2.86+0.5+10.31=1032.56

During the iteration process of pipe size optimization under LIDM, we repeatedly work on the following constraints:

a. Excess head available at the sub-network (If exist)
b. Change in piezometric head due to the previous increased pipe diameter
c. Change in head due to the next change in pipe diameter for the section under consideration

For this study, the optimization of the pipe size was done by selecting paths (Table 8) of the network. As per the LIDM rules, the piezometric elevation available at two successive end nodes connected to or branched from same hydrant are considered since the piezometric available at one node has impact on the other node. The computation of the pipe size by using the LIDM was done as follows by pairing the successive end nodes.

***5.8.1. Paths(Z→D' and Z→B'***

By following the path of the network: among hydrants D' and B', the most unfavourable hydrant is D' and, therefore, the initial solution is represented by the upstream piezometric elevation $Z_{Z,in}$=1060.02m a.s.l.

$Z_{Z\text{-}D'}$=1004+$J_{100}$, 23*L23+$J_{100}$, 21*L21+$J_{200}$, 20*L20+$J_{250}$, 18*L18+$J_{450}$, 17*L17+$J_{550}$, 1*L1

=1004+25.95+7.32+2.31+1.29+8.84+10.31=1060.02

$Z_{Z\text{-}B}$'=1007+$J_{150}$, 22*L22+$J_{200}$, 20*L20+$J_{250}$, 18*L18+$J_{450}$, 17*L17+$J_{550}$, 1*L1

=1007+7.32+2.31+1.29+8.84+10.31=1037.07

the initial minimum diameters along the paths (Z-D') and (Z-B') are:

Path Z→ D'

| section | Diameters (mm) |
|---|---|
| 1 | 550 |
| 17 | 450 |
| 18 | 250 |
| 20 | 200 |
| 21 | 100 |
| 23 | 100 |

Path Z→ B'

| section | Diameters (mm) |
|---|---|
| 1 | 550 |
| 17 | 450 |
| 18 | 250 |
| 20 | 200 |
| 22 | 150 |

Starting from Z (the upstream node at which the effective head is 1023m a.s.l.) and following the paths, it is possible to compute excess head on either nodes (nodes D' and B'). Hence, there is excess head on node B' which is:

$\Delta H_{B'}$ = 1060.02-(985+22+7.32+2.31+1.29+8.84+10.31) =22.95m

LIDM theory states that in the case of a terminal section with a head in excess at its downstream end, the value of $\beta_x$, sn (the ratio of change in the cost (in ETB) per meter of the smaller and next larger pipe diameters valid for the sub-network to change in the friction head loss due to this change in the pipe diameters of that sub-network) to be used in the process is taken as zero, as long as the excess head prevails. This implies that $\beta_{22}$=0. And again, as per the theory of LIDM, we compare: $\beta_1$,$\beta_{17}$, $\beta_{18}$, $\beta_{20}$, $\beta_{21}$, and $\beta_{23}$ ($\beta_1$=121, $\beta_{17}$=100, $\beta_{18}$=285, $\beta_{20}$=186, $\beta_{21}$=2, $\beta_{23}$=1 ) in series.

The minimum value among the $\beta_x$ values is $\beta_{23}$, so we need to change the diameter of the section 23 from 100mm to 150mm. Next, we have to decrease $(Z_Z)_{in.}$ Hence, we select minimum value among;

- Excess head on node where the head change: excess head on node B'=22.95m
- $Z_{Z,in}$ -$Z_z$=1060.02-1023=37.02m

  ΔY→when changing the diameter of the section 23 from 100mm to 150mm;

  $\Delta Y_{23}$=25.95-3.08=22.87m

The minimum value is 22.87m.Therefore, we increase diameter of the section 23 from 100mm to 150mm

**Iteration 1**

$Z_{Z_1}$ =1060.02-22.87=1037.15m a.s.l.

We get the diameters shown below for this path:

| section | Diameters (mm) |
|---|---|
| 1 | 600 |
| 17 | 450 |
| 18 | 250 |
| 20 | 200 |
| 21 | 100 |
| 23 | 150* |

By changing the whole diameter of section 23 we would recover 22.87m. However, we need to decrease the piezometric elevation at the unfavourable hydrant by 37.02. This implies that the whole section 23 needs 150mm pipe diameter.

At this stage, there is still 0.08m excess head on the node B'. Hence, the values of coefficients β are considered to identify the sections to be changed.

- Section 22: $\beta_{22}$= 78
- Section 23: $\beta_{23}$ =186
- Section 21: $\beta_{21}$=2
- Section 20: $\beta_{20}$=186
- Section 18: $\beta_{18}$=160
- Section 17: $\beta_{17}$=132
- Section 1: $\beta_{1}$=175

The minimum value is $\beta_{21}$; as a result we increase the diameter of section 21 from 100mm to 150mm.

We select minimum value for $Z_{Z,21,}$ among:

- Excess head on the nodes where the head change: $\Delta H_{B'}$=22.95-22.87=0.08m
- $Z_{Z,21}$-$Z_{Z=}$1037.15m -1023=14.15m
- $\Delta Y$ →by changing the diameter of section 21 from100mm to 150mm:
  $\Delta Y_{21}$=22.77-2.7=20.07m

The minimum value is 0.08m. As a result we increase the whole diameter of section 21 from 100mm to 150mm.

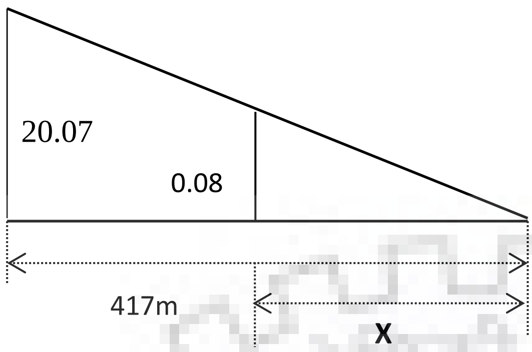
**Iteration 2**

$Z_{Z_2}$=1037.15-0.08=1037.07m a.s.l.

This gives the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 600 |
| 17 | 450 |
| 18 | 250 |
| 20 | 200 |
| 21 | 150* |
| 23 | 150* |

By changing the whole diameter of section 21 we would recover 20.07m, however, we only decrease the upstream piezometric elevation by 0.08m. This shows that we will have mixage of pipe length with the diameter size between 100mm and 150 for section 21.The length of such combination can be computed as:




$$\frac{417}{20.07} = \frac{x}{0.08}$$

x=3m

This implies that 3m of section 21 needs a pipe diameter 150mm and 413m needs 100mm pipe diameter. So, we need 100mm pipe diameter for section 21 by ignoring the 150mm pipe diameter with length 3m requirement for the section.

At this stage, there is no excess head on the nodes B' and D'. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed.

- For sections 22 and 23: $\beta = \beta_{22} + \beta_{23} = 212+186=398$
- Section 20: $\beta_{20}=186$
- Section 18: $\beta_{18}=285$
- Section 17: $\beta_{17}=100$
- Section 1: $\beta_1=121$

The minimum value is $\beta_{17}$; as a result we increase the diameter of section 17 from 450mm to 500mm. We select minimum value for $Z_{Z,17,}$ among:

- Excess head on the nodes where the head change: no head variations on the nodes B' and D' occur.
- $Z_{Z,17}$-$Z_{Z=}$1037.07-1023=14.07m
- $\Delta Y$ →by changing the diameter of section 17 from 450mm to 500mm:
  $\Delta Y_{17}$=8.84-3.43=5.41m

The minimum value is 5.41m. As a result we increase the whole diameter of section 17 from 450mm to 500mm. Therefore, the solution at the next iteration is:

**Iteration 3**

$Z_{\mathbf{Z_3}}$ = 1037.07– 5.41 = 1031.66 m a.s.l.

This gives the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 600 |
| 17 | 500* |
| 18 | 250 |
| 20 | 200 |
| 21 | 100* |
| 23 | 100* |

At this stage, there is no excess head on the nodes B' and D'. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed.

- For sections 22 and 23: $\beta = \beta_{22} + \beta_{23}$=212+186=398
- Section 20: $\beta_{20}$=186
- Section 18: $\beta_{18}$=285
- Section 1: $\beta_1$=121

The minimum value is $\beta_1$; as a result we increase the diameter of section 1 from 550mm to 600mm. We select minimum value for $Z_{\mathbf{Z},1,}$ among:

- Excess head on the nodes where the head change: no head variations on the nodes B' and D' occur.
- $Z_{Z,1}$-$Z_{Z=}$1031.66-1023=8.66m
- $\Delta Y$ →by changing the diameter of section 1 from 550mm to 600mm:

  $\Delta Y_1$=10.31-6.59=3.72m

The minimum value is 3.72m. As a result we increase the whole diameter of section 1 from 550mm to 600mm and proceed to the next iteration.

**Iteration 4**

$Z_{\mathbf{Z_4}}$=1031.66-3.72=1027.94m a.s.l.

This gives the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 600* |
| 17 | 500* |
| 18 | 250 |
| 20 | 200 |
| 21 | 100* |
| 23 | 100 |

At this stage, there is no excess head on the nodes B' and D'. Hence, the values of coefficients β are considered to identify the sections to be changed.

- For sections 22 and 23: $\beta = \beta_{22} + \beta_{23}$=78+186=264
- Section 20: $\beta_{20}$=186
- Section 18: $\beta_{18}$=285

The minimum value is $\beta_{20}$; as a result we increase the diameter of section 20 from 200mm to 250mm.

We select minimum value for $Z_{Z,20,}$ among:

- Excess head on the nodes where the head change: no head variations on the nodes B' and D' occur.
- $Z_{Z,20}$-$Z_{Z=}$1027.94.12-1023=4.94m
- $\Delta Y$ →by changing the diameter of section 20 from 200mm to 250mm:
  $\Delta Y_1$=2.31-0.72=1.59m

The minimum value is 1.59m. As a result we increase the whole diameter of section 20 from 200 to 250 and proceed to the next iteration.

**Iteration 5**

$Z_{Z_5}$=1027.94-1.59=1026.35m a.s.l. This gives the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 630* |
| 17 | 500* |
| 18 | 250 |
| 20 | 250* |
| 21 | 100* |
| 23 | 100 |

At this stage, again there is no excess head on the nodes B' and D'. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed.

- For sections 22 and 23: $\beta = \beta_{22} + \beta_{23}$=78+186=264
- Section 18: $\beta_{18}$=285

The minimum value is $\beta = \beta_{22} + \beta_{230}$; as a result we increase the diameter of sections 22 and 23. We select minimum value for $Z_{Z,\,22\,\&\,23,}$ among:

- Excess head on the nodes where the head change: no head variations on the nodes 22 and 23 occur.
- $Z_{Z,22\,\&23}$-$Z_{Z=}$1026.35-1023=3.35m
- $\Delta Y$ →by changing the diameter of section 22 from 150mm to 200mm:
  $\Delta Y_{22}$=7.32-1.63=5.69
- $\Delta Y$ →by changing the diameter of section 23 from 100mm to 150mm:
  $\Delta Y_{23}$=25.95-3.08=22.87

The minimum value is 3.35m. As a result we have a mixage on sections 22 and 23.

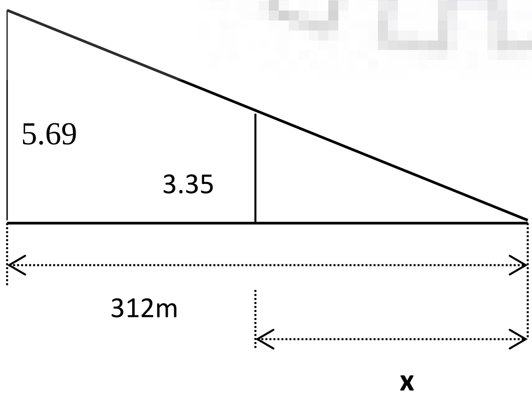
**Iteration 6**

$Z_{Z_6}$=1026.35-3.35=1023m a.s.l

This gives the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 630* |
| 17 | 500* |
| 18 | 250 |
| 20 | 250* |
| 21 | 100* |
| 23 | 150* |
| 22 | 200* |

Mixage on section 22:

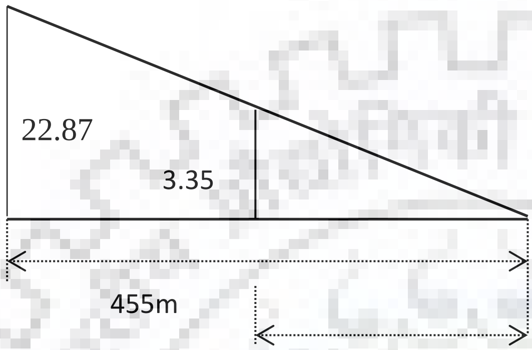


$$\frac{312}{5.69} = \frac{x}{3.35}$$

x=67m

This implies that 184m of section 22 needs a pipe diameter 200mm and 128m needs 150mm pipe diameter.

Similarly, mixage on section 23:




$$\frac{455}{22.87} = \frac{x}{3.35}$$

x=67m

This implies that 67m of section 23 needs a pipe diameter 150mm, so that 388m needs 100mm pipe diameter.

This gives the final solution diameters shown below:

| section | Diameter (mm) | Length (m) |
|---|---|---|
| 1 | 600 | 1509 |
| 17 | 500 | 960 |
| 18 | 250 | 140 |
| 20 | 250 | 120 |
| 21 | 100 | 312 |
| 22 | 200 | 184 |
| | 150 | 128 |
| 23 | 150 | 67 |
| | 100 | 388 |

### *5.8.2. Paths Z→B'and Z→Z'*

By following the paths of the network and Starting from Z along the path Z→ $B'$ and Z→ $Z'$, it is possible to compute excess head on either node (nodes B' & Z')

Among the paths of the network; $Z_{Z\text{-}B'}$ and $Z_{Z\text{-}Z'}$;

$Z_{Z\text{-}B}$'=1007+$J_{150}$, 22*L22+$J_{200}$, 20*L20+$J_{250}$, 18*L18+$J_{450}$, 17*L17+$J_{550}$, 1*L1

=1007+7.32+2.31+1.29+8.84+10.31=1037.07

$Z_{Z\text{-}Z}$'=1012+$J_{80}$, 19* L19+$J_{250}$, 18*L18+$J_{450}$, 17L17+$J_{550}$, 1*L1

=1012+1.07+1.29+8.84+10.31=1033.51

The most unfavourable hydrant in this case is B' and therefore, the upstream piezometric elevation $Z_{Z,in}$=1037.07m a.s.l. represents the initial solution.

Hence, there is excess head on node Z' which is:

$\Delta H_{Z'}$= 1037.07-(988+24+1.07+1.29+8.84+10.31) =3.56

As for the previous case, LIDM theory states that in the case of a terminal section with a head in excess at its downstream end, the value of $\beta_x$, sn (the ratio of change in the cost in ETB per meter of the smaller and next larger pipe diameters valid for the sub-network to change in the friction head loss due to this change in pipe diameters of that sub-network) to be used in the process is taken as zero, as long as the excess head prevails. This implies that $\beta_{19}$=0. And again, as per the theory of LIDM, we compare: $\beta_1$,$\beta_{17}$, $\beta_{18}$, $\beta_{20}$, and $\beta_{22}$ ($\beta_1$=121, $\beta_{17}$=100, $\beta_{18}$=285, $\beta_{20}$=186, $\beta_{22}$=78) in series.

The minimum value among the $\beta_x$ values is $\beta_{22}$, so we need to change the diameter of the section 22 from 150mm to 200mm. Next, we have to decrease $Z_{Z,\ in}$. Hence, we select minimum value among;

- Excess head on node where the head change: excess head on node Z'=3.56m
- $Z_{Z,in}$ -$Z_z$ =1037.07-1023=14.07m

  ΔY→when changing the diameter of the section 22 from 150mm to 200mm;

  $\Delta Y_{22}$=7.32-1.63=5.69m

The minimum value is 3.56m.Hence we increase part of the pipe diameter on section 22 from 150mm to 200mm.

**Iteration 7**

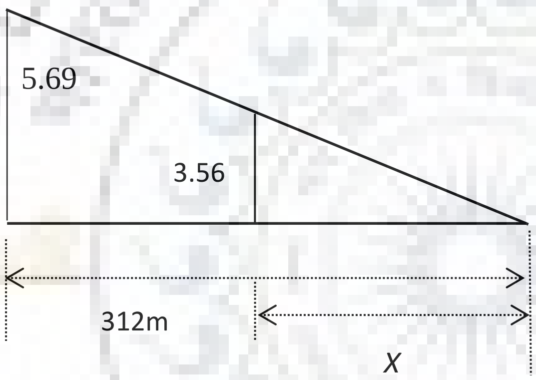
$Z_{Z_7}$=1037.07-3.56 =1033.51m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 550 |
| 17 | 450 |
| 18 | 250 |
| 20 | 200 |
| 22 | 200* |

By changing the whole diameter of section 22 we would recover 5.69m, however, we decrease the upstream piezometric elevation only 3.56m.This shows that we will have mixage of pipe length with the diameter size between 150mm and 200 for section 22.

The length of such combination can be computed as:




$$\frac{312}{5.69} = \frac{x}{3.56}$$

x=195m

This implies that 195m of section 22 needs a pipe diameter 200mm and 117m needs 150mm pipe diameter.

At this stage, there is no excess head on the nodes B' and Z'. Hence, sections 19 and 22 are now in parallel and using the values of the coefficients $\beta_x$ for identifying the sections to be changed:

- For sections 19 and 22: $\beta = \beta_{19} + \beta_{22} = 20 + 264 = 284$
- Section 20: $\beta_{20} = 186$
- Section 18: $\beta_{18} = 285$
- Section 17: $\beta_{17} = 100$
- Section 1: $\beta_1 = 121$

The minimum value among the coefficients $\beta_x$ is $\beta_{17}$, therefore we have to increase the diameter on the section 17.

We select the minimum value $Z_{Z,\,17,}$ among:

- Excess head on the nodes where the head change: no heads variations
- $Z_{Z,17}$-$Z_Z$=1033.51-1023=10.51m
- $\Delta Y$ → by changing the diameter of the section 17 from 450mm to 500 mm :

  $\Delta Y_{17}$=8.84-3.43=5.41m

  The minimum value is 5.41m; therefore, we have to change the whole diameter of section 17 from 450mm to 500mm.

**Iteration 8**

$Z_{Z_8}$ =1033.51-5.41=1028.1m a.s.l.

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 550 |
| 17 | 500* |
| 18 | 250 |
| 20 | 200 |
| 22 | 200-150 |

There is no excess head on the nodes B' and Z' as in the case of previous iteration. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed next.

- For sections 19 and 22: $\beta = \beta_{19} + \beta_{22}$=20+264=284
- Section 20: $\beta_{20}$=186
- Section 18: $\beta_{18}$=285
- Section 1: $\beta_1$=121

The minimum value is $\beta_1$, as a result we increase the diameter of section 1 from 550mm to 600mm.

We select minimum value $Z_{Z,\,17,}$ among:

- $Z_{Z,1}$-$Z_{Z}$=1028.1m -1023=5.1
- $\Delta Y$ →by changing the diameter of section 1from 450mm to 500mm:

  $\Delta Y_1$=10.31-6.59=4.62

The minimum value is 4.62m. As a result we increase the whole diameter of section 1 from 450mm to 500mm.

Hence, the next iteration solution is:

**Iteration 9**

$Z_{Z_9}$=1028.1-4.62=1023.48m a.s.l.

This gives the diameters:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 17 | 500* |
| 18 | 250 |
| 20 | 200 |
| 22 | 200-150 |

At this stage, there is no excess head on the nodes B' and Z'. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed.

- For sections 19 and 22: $\beta = \beta_{19} + \beta_{22}$=20+264=284
- Section 20: $\beta_{20}$=186
- Section 18: $\beta_{18}$=160

The minimum value is $\beta_{18}$, as a result we increase the diameter of section 18 from 250mm to 300mm.

We select the minimum value $Z_{Z,18}$ among:

- Excess head on the nodes where the head change: no heads variations
- $Z_{Z,18}$-$Z_Z$=1023.48-1023=0.48m
- $\Delta Y \rightarrow$ by changing the diameter of the section 18 from 250mm to 300 mm :
  $\Delta Y_{18}$=1.29-0.5=0.79

The minimum value is 0.48m, so we change part of diameter of section 18 from 250mm to 300mm.

**Iteration 10**

$Z_{Z_{10}}$ =1024.48-0.48=1023m a.s.l.

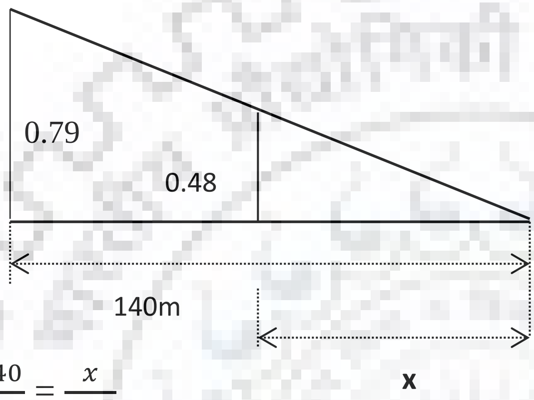
We get the diameters shown below.

| Section | Diameter (mm) |
|---|---|
| 1 | 600 |
| 17 | 500* |
| 18 | 300* |
| 20 | 200 |
| 22 | 200-150 |
| 19 | 100 |

At this stage, we have two options for the solution of pipe diameter on section 18:

- First, we can change the whole diameter from 250mm to 300mm since it fulfils the theory of LIDM and
- Secondly, we change part of the diameter of section 18 from 250mm to 300mm. Since our objective is to minimize the cost of the pipe network, it preferable to change part of the section 18 from250mm to 300mm.

The length of such mixage can be computed as:




$$\frac{140}{0.79} = \frac{x}{0.48}$$

x=85m

This means that 85m of section 18 needs 300mm pipe diameter and 55m needs 250mm pipe diameter.

The final solution for paths Z→ $B'$ and Z→ $Z'$:

| Section | Diameter (mm) | Length (m) |
|---|---|---|
| 1 | 600 | 1509 |
| 17 | 500 | 960 |
| 18 | 300 | 85 |
| | 250 | 55 |
| 20 | 200 | 120 |
| 22 | 200 | 195 |
| | 150 | 117 |
| 19 | 100 | 114 |

### *5.8.3. Paths Z→W and Z→X*

As for previous cases, it is possible to compute excess head on either node W or X.

Among the paths of the network; Z→ $W$ and Z→ $X$

$Z_{Z\text{-}X}$=1010+$J_{100}$, 15*$L_{15}$+$J_{150}$, 14*$L_{14}$+$J_{250}$, 12*$L_{12}$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1010+18.32+9.78+2.86+0.5+10.31=1051.77

$Z_{Z\text{-}W}$=1015+$J_{150}$, 13*$L_{13}$+$J_{250}$, 12*$L_{12}$+$J_{400}$, 2*$L_2$+$J_{550}$, 1*$L_1$

=1015+3.89+2.86+0.5+10.31=1032.56

The most unfavourable hydrant in this case is X and therefore, the upstream piezometric elevation $Z_{Z,in}$=1051.77 m a.s.l. represents the initial solution.

Hence, there is excess head on node W which is:

$\Delta H_W$= 1051.77-(991+24+3.89+2.86+0.5+10.31) =19.21 As for the previous case, LIDM theory states that in the case of a terminal section with a head in excess at its downstream end, the value of $\beta_x$, sn (the ratio of change in the cost in ETB per meter of the smaller and next larger pipe diameters valid for the sub-network to change in the friction head loss due to this change in pipe diameters of that sub-network) to be used in the process is taken as zero, as long as the excess head exists.

This implies that $\beta_{15}$=0. Keeping in mind the theory of LIDM as for the previous ones, we compare$\beta_1$,$\beta_2$, $\beta_{12}$, $\beta_{14}$, and $\beta_{13}$ ($\beta_1$=121, $\beta_2$=1197, $\beta_{12}$=129, $\beta_{14}$=59, $\beta_{13}$=147) in series.

The minimum value corresponds to$\beta_{14}$, so we change the diameter of section 14 from 150mm to 200mm. Next, we have to decrease $Z_{Z,\ in}$ so we select minimum value among:

- Excess head on node where the head change: excess head on node X=19.21m
- $Z_{Z,in}$ -$Z_z$ =1051.77-1023=28.77m

  ΔY→when changing the diameter of the section 14 from 150mm to 200mm;

  $\Delta Y_{14}$=9.78-2.18=7.60m

The minimum value is 7.6m. As a result we increase the whole diameter of section 14 from 150mm to 200mm.

**Iteration 11**

$Z_{Z\ \mathbf{11}}$=1051.77-7.6=1044.17m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 550 |
| 2 | 400 |
| 12 | 250 |
| 13 | 150 |
| 14 | 200* |
| 15 | 150 |

By changing the whole diameter of section 14 we would recover 7.6m, however, we need to decrease the upstream piezometric elevation by 28.77m. This shows there is excess pressure on node W and the excess pressure at this stage is:

$\Delta H_W$= 1051.77-(991+24+3.89+2.86+0.5+10.31)-7.6=11.61

As for the first case, we use the values of the coefficient $\beta_x$ to determine the link for which the diameter changes next. We compare; $\beta_1$,$\beta_2$, $\beta_{12}$, and $\beta_{13}$ ($\beta_1$=121, $\beta_2$=1197, $\beta_{12}$=129, $\beta_{13}$=147) in series.

The minimum value among the $\beta_x$ values is $\beta_{12}$, so we need to change the diameter of the section 12 from 550mm to 600mm. Next, we have to decrease $Z_{Z,\ in}$ so we select minimum value among;

- Excess head on node where the head change: excess head on node W=11.61m
- $Z_{Z,in}$ -$Z_z$ =1044.17-1023=21.17m

  ΔY→when changing the diameter of section 1 from 550mm to 600mm;

  $\Delta Y_1$=10.31-6.59=3.72m

The minimum value is 3.72m. As a result we increase the whole diameter of section 1 from 550mm to 600mm.

**Iteration 12**

$Z_{\mathbf{Z_{12}}}$=1044.17-3.72=1040.15m a.s.l. We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 12 | 250 |
| 13 | 150 |
| 14 | 200* |
| 15 | 150 |

By changing the whole diameter of sections 14 and 1 we would recover 11.32m, however, we decrease the upstream piezometric elevation 19.21m.This shows there is still excess pressure on node W and the excess pressure at this stage is: 19.21-11.32=7.89

We use the values of the coefficient $\beta_x$ to determine the section for which the diameter changes next. We compare,$\beta_2$, $\beta_{12}$, and $\beta_{13}$ ( $\beta_2$=1197, $\beta_{12}$=129, $\beta_{13}$=147).

The minimum value corresponds to$\beta_{12}$, so we change the diameter of section 12 from 250mm to 300mm. Next, we have to decrease $Z_{Z,\,in}$ so we select minimum value among;

- Excess head on node where the head change: excess head on node W=7.89m
- $Z_{Z,in}$ -$Z_z$ =1040.15-1023=17.15m

  $\Delta$Y→when changing the diameter of section 12 from 250mm to 300mm;

  $\Delta Y_{12}$=2.86-1.11=1.75m

The minimum value is 1.75m. As a result we increase the whole diameter of section 12 from 250mm to 300mm.

**Iteration 13**

$Z_{\mathbf{Z_{13}}}$=1040.15-1.75=1038.4, we get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 12 | 300* |
| 13 | 150 |
| 14 | 200* |
| 15 | 150 |

By changing the whole diameter of sections 14, 1 and 12 we would recover 13.07m, however, we decrease the upstream piezometric elevation 19.21m.This shows there is still excess pressure on node W and the excess pressure at this stage is: 19.21-13.07=6.14

We use the values of the coefficient $\beta_x$ to determine the link for which the diameter changes next. We compare, $\beta_2$, and $\beta_{13}$ ( $\beta_2$=1197and $\beta_{13}$=147).

The minimum value corresponds to $\beta_{13}$, so we change the diameter of section 13 from 150mm to 200mm. Next, we have to decrease $Z_{Z,\,in}$ so we select minimum value among;

- Excess head on node where the head change: excess head on node W=6.14m
- $Z_{Z,in}$ -$Z_z$ =1038.4-1023=15.4m

  $\Delta$Y→when changing the diameter of section 13 from 250mm to 300mm;

$\Delta Y_{13}$=3.89-0.87=3.02m

The minimum value is 3.02m. As a result we increase the whole diameter of section 13 from 150mm to 200mm.

**Iteration 14**

$Z_{\mathbf{Z_{14}}}$=1038.4-3.02=1035.38

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 12 | 300* |
| 13 | 200* |
| 14 | 200* |
| 15 | 150 |

By changing the whole diameter of sections 14, 1, 12 and 13 we would recover 16.09m, however, we decrease the upstream piezometric elevation 19.21m.This shows there is still excess pressure on node W and the excess pressure at this stage is: 19.21-16.09=3.12

We use the values of the coefficient $\beta_x$ to determine the section for which the diameter changes next. At this stage we left with $\beta_2$=1197 and therefore we change the diameter of section $\beta_2$ from 400mm to 450mm.

Next, we have to decrease $Z_{Z,\ in}$ so we select minimum value among;

- Excess head on node where the head change: excess head on node W=3.12m
- $Z_{Z,in}$ -$Z_z$ =1035.38-1023=15.4m

  ΔY→when changing the diameter of section 2 from 400mm to 450mm;

  $\Delta Y_2$=0.5-0.27=0.23m

The minimum value is 0.23m. As a result we increase the whole diameter of section 2 from 400mm to 450mm.

**Iteration 15**

$Z_{\mathbf{Z_{15}}}$=1035.38-0.23=1035.13

We get the diameters shown below:

| Section | Diameter (mm) |
| --- | --- |
| 1 | 600* |
| 2 | 450* |
| 12 | 300* |
| 13 | 200* |
| 14 | 200* |
| 15 | 150 |

Again, by changing the whole diameter of sections 14, 1, 12, 13 and 2 we would recover 16.32m; this shows there is still excess pressure on node W and the excess pressure at this stage is: 19.21-16.32=2.89

We use the values of the coefficient $\beta_x$ to determine the section for which the diameter changes next. At this stage we left with only $\beta_{15}$=2 and therefore we change the diameter of section $\beta_{15}$ from 100mm to 150mm.

Next, we have to decrease $Z_{Z,\ in}$ so we select minimum value among;

- $Z_{Z,in}$ -$Z_z$ =1035.13-1023=12.13m

  ΔY→when changing the diameter of section 15 from 100mm to 150mm;

  $\Delta Y_{15}$=18.32-2.17=16.15m

The minimum value is 12.13m. Hence, we increase the part of diameter of section 15 from 100mm to 150mm.

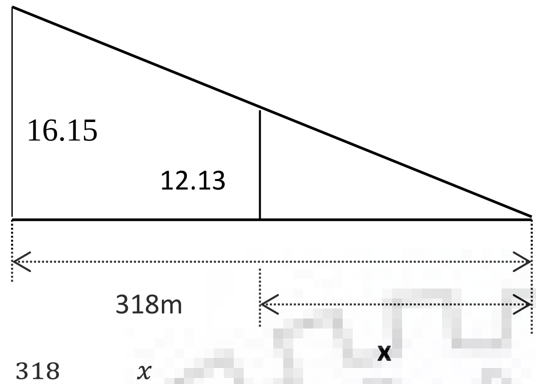
**Iteration 16**

$Z_{\mathbf{Z_{16}}}$=1035.13-12.13=1023

We get the diameters shown below:

| Section | Diameter (mm) |
| --- | --- |
| 1 | 600* |
| 2 | 450* |
| 12 | 300* |
| 13 | 200* |
| 14 | 200* |
| 15 | 100-150 |

The length of such combination can be computed as:




$$\frac{318}{16.15} = \frac{x}{12.13}$$

x=239m

This implies that 239 of section 15 need a pipe diameter 150mm and 79m needs 100mm pipe diameter.

At this stage, there is no excess head on the nodes W and X. Hence, sections 13 and 15 are now in parallel. The final solution for paths Z→ $W$ and Z→ $X$:

| Section | Diameter (mm) | Length (m) |
|---|---|---|
| 1 | 600 | 1509 |
| 2 | 450 | 42.5 |
| 12 | 300 | 284 |
| 13 | 200 | 324 |
| 14 | 200 | 371 |
| 15 | 150 | 239 |
| | 100 | 79 |

### *5.8.4. Path Z→I and Z→G*

By following the path of the network: among hydrants G and I, the most unfavourable hydrant is I and, therefore, the initial solution is represented by the upstream piezometric elevation $Z_{Z,in}$=1039.2m a.s.l.

$Z_{Z\text{-}I}=1012+J_{150,\,7}*L_7+J_{200,\,6}*L_6+J_{250,\,4}*L_4+J_{350,\,3}*L_3+J_{400,\,2}*L_2+J_{550,\,1}*L_1$

$=1012+5.9+4.36+4.12+2.01+0.5+10.31=1039.2$

$Z_{Z\text{-}G}= 1013+J_{150,\,5}*L_5+J_{250,\,4}*L_4+J_{350,\,3}*L_3+J_{400,\,2}*L_2+J_{550,\,1}*L_1$

$=1013+6.98+4.12+2.01+0.5+10.31=1036.92$

Starting from node Z along the path paths; it is possible to compute excess head on either node (node G & I). Hence, there is excess head on node G which is:

$\Delta H_G$= 1039.2-(1013+6.98+4.12+2.01+0.5+10.31) =2.28m

Again, as for the previous cases, LIDM theory states that in the case of a terminal section with a head in excess at its downstream end, the value of $\beta_x$, sn (the ratio of change in the cost in ETB per meter of the smaller and next larger pipe diameters valid for the sub-network to change in the friction head loss due to this change in pipe diameters of that sub-network) to be used in the process is taken as zero, as long as the excess head exists.

This implies that $\beta_5$=0. Keeping in mind the theory of LIDM, we compare $\beta_1$,$\beta_2$, $\beta_3$, $\beta_4$, $\beta_6$, and $\beta_7$ ($\beta_1$=121, $\beta_2$=1197, $\beta_3$=529, $\beta_4$=89,$\beta_6$=99, $\beta_7$=97) in series.

The minimum value corresponds to $\beta_4$, so we change the diameter of the section 4 from 250mm to 300mm. Next, we have to decrease $Z_{Z,\,in}$.

Hence, we select minimum value among;

- Excess head on node where the head change-excess head on node G=2.28m
- $Z_{Z,in}$-$Z_z$=1039.2-1023=16.2m

  ΔY→when changing the diameter of the section 4 from 250mm to 300mm;

  $\Delta Y_4$=4.12-1.6=2.52m

The minimum value is 2.28. As a result we increase the whole diameter of section 4 from 250mm to 300mm.

**Iteration 17**

$Z_{Z_{17}}$=1039.2-2.28=1036.92m

We get the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 550 |
| 2 | 400 |
| 3 | 350 |
| 4 | 300* |
| 6 | 200 |
| 7 | 150 |

By changing the whole diameter of section 4 we would recover 2.28m, and also we decrease the upstream piezometric elevation 2.28m.

This implies that at this stage, there is no excess head on the nodes G and I. Hence, sections 5 and 7 are now in parallel and, the values of coefficients β are considered to identify the sections to be changed next.

- For sections 5 and 7: $\beta = \beta_5 + \beta_7$=82+97=179
- Section 6: $\beta_6$=99
- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197
- Section 1: $\beta_1$=121

The minimum value among the coefficients $\beta_x$ is $\beta_6$, therefore we have to increase the diameter on the section 6 from 200mm to 250mm.

We select the minimum value for $Z_{Z,6}$ among:

- Excess head on the nodes where the head change: no heads variations on the nodes G and I
- $Z_{Z,6}$-$Z_Z$=1039.2m-1023=16.2m
- $\Delta Y$ → by changing the diameter of the section 6 from 200mm to 250 mm : $\Delta Y_6$=4.36-1.37=2.99m

The minimum value is 2.99m; therefore, we change the whole diameter of section 6 from 200mm to 250mm.

**Iteration 18**

$Z_{Z_{18}}$=1039.2-2.99=1036.21m a.s.l.

This gives the diameters shown below:

| section | Diameters (mm) |
|---|---|
| 1 | 550 |
| 2 | 400 |
| 3 | 350 |
| 4 | 300* |
| 6 | 250* |
| 7 | 150 |

As in the case of previous iterations, there is no excess head on the nodes G and I. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed.

- For sections 5 and 7: $\beta = \beta_5 + \beta_7$=82+97=179
- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197
- Section 1: $\beta_1$=121

The minimum value is $\beta_1$; as a result we increase the diameter of section 1.

We select minimum value $Z_{Z,\,1}$ among:

- $Z_{Z,1}$-$Z_{Z=}$1036.21-1023=13.21
- $\Delta Y$ → by changing the diameter of the section 1 from 550mm to 600 mm :

  $\Delta Y_1$=10.31-6.59=3.72m

  The minimum value is 3.72m, so we change the diameter on the whole section 1 from 550mm to 600mm.

**Iteration 19**

Therefore, the iteration solution is:

$Z_{\mathbf{Z_{19}}}$=1036.21-3.72=1032.49m a.s.l. and this gives the diameters:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 3 | 350 |
| 4 | 300* |
| 6 | 250* |
| 7 | 150 |

At this stage, there is no excess head on the nodes G and I. As a result, the values of coefficients $\beta_x$ for identifying the sections to be changed are:

- For sections 5 and 7: $\beta = \beta_5 + \beta_7$=82+97=179
- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197

The minimum value is$\beta = \beta_5 + \beta_7$, so we have to increase the diameter of section 5 and 7.

We select minimum $Z_{Z,\,5\,\&\,7}$ values among:

- $Z_{Z,5\&7}$ -$Z_Z$ =1032.49-1023=9.49 m
- $\Delta Y$ →by changing the diameter of section 5 from 150mm to 200mm:

  $\Delta Y_5$=6.98-1.55=5.43m
- ΔY →by changing the diameter of section 7 from 150mm to 200mm:

  $\Delta Y_7$=5.90-1.31=4.59m

The minimum value is 4.59m; so we increase the whole diameter of section 7 from 150mm to 200mm.

**Iteration 20**

$Z_{\mathbf{Z_{20}}}$ =1032.49-4.59=1027.9m a.s.l.

This gives the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 3 | 350 |
| 4 | 300* |
| 6 | 250* |
| 7 | 200* |

At this stage, there is no excess head on the nodes G and I. As a result, the values of coefficients $\beta_x$ for identifying the sections to be changed are:

- For sections 5 and 7= $\beta_5$+ $\beta_7$=82+97=179
- Section 3: $\beta_3$=528
- Section 2: $\beta_2$=1218

The minimum value is$\beta$= $\beta_5$+ $\beta_7$, so we have to increase the diameter of section 5 because the diameter of section 7 was already increased to its maximum possible.

We select minimum $Z_{Z,5}$ values among:

- $Z_{Z,5}$ -$Z_Z$ =1027.9-1023=4.9m a.s.l.
- $\Delta Y$ →by changing the diameter of section 5 from 150mm to 200mm:

  $\Delta Y_5$=6.98-1.55=5.43m

The minimum value is 4.9m; so we change part of diameter of section 5 from 150mm to 200mm.

Therefore the next Iteration solution is:

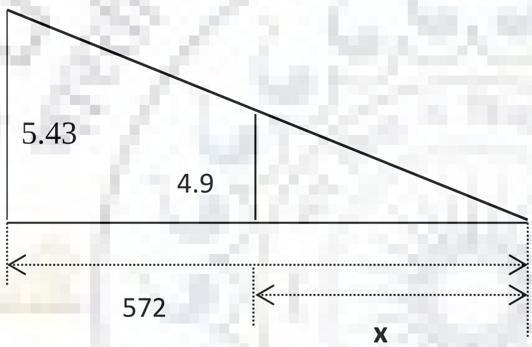
**Iteration 21**

$Z_{\mathbf{Z_{21}}}$=1027.9-4.9=1023m a.s.l.

This gives the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 3 | 350 |
| 4 | 300* |
| 6 | 250* |
| 7 | 200* |
| 5 | 200-150 |

The mixage on section 5:




$$\frac{572}{5.73} = \frac{x}{4.9}$$

x=516m

This implies that 516 of section 5 need a pipe diameter 200mm and 56m needs 150mm pipe diameter.

The final solution for Paths **Z→I and Z→G** are:

| Section | Diameter (mm) | Length(m) |
|---|---|---|
| 1 | 600 | 1509 |
| 2 | 400 | 42.5 |
| 3 | 350 | 163 |
| 4 | 300 | 272 |
| 6 | 250 | 325 |
| 7 | 200 | 630 |
| 5 | 200 | 516 |
| | 150 | 56 |

### *5.8.5. Paths Z→ $O$ and Z→ $L$*

By following the paths of the network and Starting from Z along the path Z→ $O$ and Z→ $L$, it is possible to compute excess head on either nodes (node L or O)

Among the paths of the network; $Z_{Z\text{-}O}$ and $Z_{Z\text{-}L}$;

$Z_{Z\text{-}O}$=1012+$J_{150,\,11}$*$L_{11}$+$J_{200,\,10}$*$L_{10}$+$J_{250,\,8}$* $L_8$+$J_{350,\,3}$*$L_3$+$J_{400,\,2}$*$L_2$+$J_{550,\,1}$*$L_1$

=1012+5.32+5.61+3.94+2.01+0.5+10.31=1039.69

$Z_{Z\text{-}L}$=1014+ $J_{150,\,9}$*$L_9$+$J_{250,\,8}$* $L_8$+$J_{350,\,3}$*$L_3$+$J_{400,\,2}$*$L_2$+$J_{550,\,1}$*$L_1$

=1014+2.92+3.94+2.01+0.5+10.31=1033.68

The most unfavourable hydrant in this case is O and therefore, the upstream piezometric elevation $Z_{Z,in}$=1039.69m a.s.l. represents the initial solution.

Hence, there is excess head on node L which is:

$\Delta H_L$= 1039.69-(989+25+2.92+3.94+2.01+0.5+10.31) =6.01m

As for the previous case, LIDM theory states that in the case of a terminal section with a head in excess at its downstream end, the value of $\beta_x$, sn (the ratio of change in the cost in ETB per meter of the smaller and next larger pipe diameters valid for the sub-network to change in the friction head loss due to this change in pipe diameters of that sub-network) to be used in the process is taken as zero, as long as the excess head exists.

This implies that $\beta_9$=0. Keeping in mind the theory of LIDM, we compare$\beta_1$,$\beta_2$, $\beta_3$, $\beta_8$, $\beta_{10}$ and $\beta_{11}$ ($\beta_1$=121, $\beta_2$=1197, $\beta_3$=529, $\beta_8$=93, $\beta_{10}$=77, $\beta_{11}$=108) in series.

The minimum value corresponds to $\beta_{10}$, so we change the diameter of the section 10 from 200mm to 250mm. Next, we have to decrease $Z_{Z,\,in}$ so we select minimum value among;

- Excess head on node where the head change: excess head on node L=6.01m
- $Z_{Z,in}$ -$Z_{\mathbf{z}}$ =1039.68-1023=16.69

  ΔY→when changing the diameter of the section 10 from 200mm to 250mm;

  $\Delta Y_{10}$=5.61-1.76=3.85m

The minimum value is 3.85m. So, we change the whole diameter of section 10 from 200mm to 250mm.

**Iteration 22**

$Z_{\mathbf{Z_{22}}}$=1039.69-3.85=1035.84m a.s.l

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 550 |
| 2 | 400 |
| 3 | 350 |
| 8 | 250 |
| 10 | 250* |
| 11 | 150 |

By changing the whole diameter of section 10 we would recover 3.85m, in fact, we decrease the upstream piezometric elevation 20.81m.

By changing the whole diameter of section 10 we would recover 3.85m, this implies that the whole section 10 needs 250mm pipe diameter.

At this stage, there is still 2.16m excess head on the node L. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed.

- Section 11: $\beta_{11}$=108
- Section 8: $\beta_8$=93
- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197
- Section 1: $\beta_1$=121

The minimum value is $\beta_8$; as a result we increase the diameter of section 8 from 250mm to 300mm.

We select minimum value for $Z_{Z,8,}$ among:

- Excess head on the nodes where the head change: $\Delta H_L$=6.01-3.85=2.16m
- $Z_{Z,8}$-$Z_{Z=}$1035.84m -1023=12.84m
- $\Delta Y$ →by changing the diameter of section 8 from 250mm to 300mm:

  $\Delta Y_8$=3.94-1.53=2.41m

The minimum value is 2.16m. As a result we have mixage on section 8 and increase part of diameter of section 8 from 250mm to 300mm

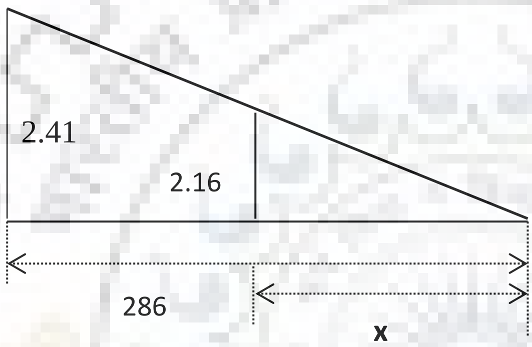
**Iteration 23**

$Z_{Z_{23}}$=1035.84-2.16=1033.68m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 550 |
| 2 | 400 |
| 3 | 350 |
| 8 | 300* |
| 10 | 250* |
| 11 | 150 |

The mixage on section 8:




$$\frac{286}{2.41} = \frac{x}{2.16}$$

x=256m

This implies that 256 of section 8 need a pipe diameter 200mm and 30m needs 150mm pipe diameter.

At this stage, there is no excess head on the nodes L and O. This implies that $\beta_9$=0. Hence, the values of coefficients $\beta_x$ are considered to identify the sections to be changed. Keeping in mind the theory of LIDM, we compare $\beta_1, \beta_2, \ \beta_3$ and $\beta_{11}$ in series.

- Section 11: $\beta_{11}$=108
- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197
- Section 1: $\beta_1$=121

The minimum value is $\beta_{11}$; as a result we increase the diameter of section 11 from 150mm to 250mm.

We select minimum value for $Z_{Z,11}$ among:

- $Z_{Z,11}-Z_{Z}=1033.68-1023=10.68$m
- $\Delta Y$ →by changing the diameter of section 11 from 150mm to 200mm:

$\Delta Y_{11}$=5.32-1.18=4.14m

The minimum value is 4.14m. As a result we increase the whole diameter of section 11 from 150mm to 200mm.

**Iteration 24**

$Z_{\mathbf{Z}_{24}}$=1033.68-4.14=1029.54m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600 |
| 2 | 400 |
| 3 | 350 |
| 8 | 300* |
| 10 | 250* |
| 11 | 200* |

As for the previous iteration, there is no excess head on the nodes G and I at this stage. As a result, the values of coefficients $\beta_x$ for identifying the sections to be changed are:

- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197
- Section 1: $\beta_1$=121

The minimum value is $\beta_1$; as a result we increase the diameter of section 1 from 550mm to 600mm.

We select minimum value for $Z_{\mathbf{Z},1}$ among:

- Excess head on the nodes where the head change: there is no excess head at this stage.
- $Z_{Z,1}$-$Z_{Z=}$1029.54m -1023=6.54m
- $\Delta Y$ →by changing the diameter of section 1 from 550mm to 600mm:

  $\Delta Y_1$=10.31-6.59=3.72m

The minimum value is 3.72m. As a result we increase the whole diameter of section 1 from 550mm to 600mm.

**Iteration 25**

$Z_{\mathbf{Z}_{25}}$=1029.54-3.72=1025.82m a.s.l

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 3 | 350 |
| 8 | 300* |
| 10 | 250* |
| 11 | 200* |

At this stage, there is no excess head on the nodes O and L. As a result, the values of coefficients $\beta_x$ for identifying the sections to be changed are:

- Section 3: $\beta_3$=529
- Section 2: $\beta_2$=1197

The minimum value is $\beta_3$; as a result we increase the diameter of section 3 from 350mm to 400mm.

We select minimum value for $Z_{Z,3}$ among:

- Excess head on the nodes where the head change: there is no excess head at this stage.
- $Z_{Z,11}$-$Z_{Z=}$1025.82m -1023=2.82m
- $\Delta Y$ →by changing the diameter of section 3 from 350mm to 400mm:

  $\Delta Y_3$=2.01-1.01=1m

The minimum value is 1m. As a result we increase the whole diameter of section 3 from 350mm to 400mm.

**Iteration 26**

$Z_{Z_{26}}$=1025.82-1=1024.82m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 400 |
| 3 | 400* |
| 8 | 300* |
| 10 | 250* |
| 11 | 200* |

Again, as in a case of the previous iteration, there is no excess head on the nodes O and L. As a result, the value of coefficient $\beta_2$=1197 is used for identifying the section to be changed. As a result we increase the diameter of section 2 from 400mm to 450mm.

We select minimum value for $Z_{\mathbf{Z},2}$ among:

- Excess head on the nodes where the head change: there is no excess head at this stage.
- $Z_{Z,2}$-$Z_{Z=}$1024.82m -1023=1.82m
- $\Delta Y$ →by changing diameter of section 2 from 400mm to 450mm:

  $\Delta Y_2$=0.5-0.27=0.23m

The minimum value is 0.23m. As a result we increase the whole diameter of section 2 from 400mm to 450mm.

**Iteration 27**

$Z_{\mathbf{Z_{27}}}$=1024.82-0.23=1024.59m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 450* |
| 3 | 400* |
| 8 | 300* |
| 10 | 250* |
| 11 | 200* |

It was supposed that we reduce the upstream piezometric elevation by 16.69m. However, we decreased the available piezometric elevation 15.1m. Hence, the prevailing 1.59m upstream piezometric elevation should be decreased. As a result we increase the diameter of section 9 from 150 to mm to 200mm.

We select minimum value for $Z_{\mathbf{Z},9}$ among:

- Excess head on the nodes where the head change: there is no excess head at this stage.
- $Z_{Z,9}$-$Z_{Z=}$1024.59m -1023=1.59m
- $\Delta Y$ →by changing diameter of section 9 from 150mm to 200mm:

  $\Delta Y_9$=2.92-0.65=2.27 m

The minimum value is 1.59m. As a result increase the whole diameter of section 9 from 150mm to 200mm.

**Iteration 28**

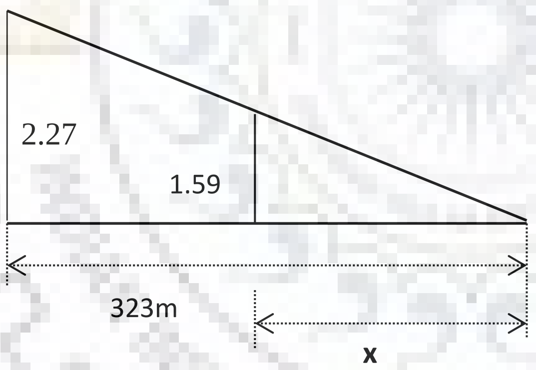
$Z_{Z_{28}}$=1024.59-1.59=1023m

We get the diameters shown below:

| Section | Diameter (mm) |
|---|---|
| 1 | 600* |
| 2 | 450* |
| 3 | 400* |
| 8 | 300* |
| 10 | 250* |
| 11 | 200* |
| 9 | 200* |

By changing the whole diameter on the section 9 we would recover 2.27 m and, in fact, we need to decrease the upstream piezometric elevation only 1.59 m at this stage. This implies that we have a mixage on section 9 between the diameters 150 mm and 200 mm.

The lengths of such mixage are:




$$\frac{323}{2.27} = \frac{x}{1.59}$$

x=226 m

This implies that 226m of section 10 needs a pipe diameter 200mm and 97m needs 150mm pipe diameter.

The Final Solution for paths **Z→ *L* and Z→ *O*** is:

| Section | Diameter (mm) | Length (m) |
|---|---|---|
| 1 | 600 | 1509 |
| 2 | 450 | 42.5 |
| 3 | 400 | 163 |
| 8 | 300 | 256 |
| | 250 | 30 |
| 10 | 250 | 314 |
| 11 | 200 | 426 |
| 9 | 200 | 226 |
| | 150 | 97 |

*Table5.11: Summary of the optimized pipe diameter solution path wise*

| Path Z→D' and Z→Z' | | Path Z→ $B'$ and Z→ $Z'$ | | Path Z→G and Z→I | | Path Z→L and Z→O | | Path Z→W and Z→X | |
|---|---|---|---|---|---|---|---|---|---|
| Section | dia. (mm) | Section | dia. (mm) | Section | dia. (mm) | Section | dia. (mm) | Section | dia. (mm) |
| 1 | 600 | 1 | 600 | 1 | 600 | 1 | 600 | 1 | 600 |
| 17 | 500 | 17 | 500 | 2 | 400 | 2 | 450 | 2 | 450 |
| 18 | 250 | 18 | 300 | 3 | 350 | 3 | 400 | 12 | 300 |
| 20 | 250 | | 250 | 4 | 300 | 8 | 300 | 13 | 200 |
| 21 | 100 | 20 | 200 | 6 | 250 | | 250 | 14 | 200 |
| 22 | 200 | 22 | 200 | 7 | 200 | 10 | 250 | 15 | 150 |
| | 150 | | 150 | 5 | 200 | 11 | 200 | | 100 |
| 23 | 150 | 19 | 100 | | 150 | 9 | 200 | | |
| | 100 | | | | | | 150 | | |

The highlighted section values were the minimum possible diameters selected for the network from the computed values under the LIDM.

*Table5.12: Cost of the network (EPANET output diameter)*

| section | Link | length | dia.(mm) | unit cost | sub-total |
|---|---|---|---|---|---|
| 1 | 1+2 | 1509 | 600 | 3386 | 5,109,474 |
| 2 | 4 | 42.5 | 500 | 2790 | 118,575 |
| 17 | 5 | 960 | 450 | 2252 | 2,161,920 |
| 4 | 8 | 272 | 350 | 1450 | 394,400 |
| 5 | 9 | 572 | 200 | 580 | 331,760 |
| 6 | 10 | 325 | 250 | 875 | 284,375 |
| 7 | 11 | 630 | 200 | 580 | 365,400 |
| 3 | 12 | 163 | 400 | 1980 | 322,740 |
| 12 | 13 | 284 | 300 | 1100 | 312,400 |
| 13 | 14 | 324 | 200 | 580 | 187,920 |
| 14 | 15 | 371 | 200 | 580 | 215,180 |
| 15 | 16 | 318 | 150 | 135 | 42,930 |
| 8 | 17 | 286 | 300 | 1100 | 314,600 |
| 9 | 18 | 323 | 200 | 580 | 187,340 |
| 10 | 19 | 314 | 250 | 875 | 274,750 |
| 11 | 20 | 426 | 250 | 875 | 372,750 |
| 18 | 21 | 140 | 300 | 1100 | 154,000 |
| 19 | 22 | 114 | 100 | 110 | 12,540 |
| 20 | 23 | 120 | 250 | 875 | 105,000 |
| 22 | 24 | 312 | 200 | 580 | 180,960 |
| 21 | 25 | 417 | 150 | 135 | 56,295 |
| 23 | 26 | 455 | 150 | 135 | 61,425 |
| 16 | 28 | 24 | 300 | 1100 | 26,400 |
| Total cost(ETB) | | | | | 11,593,134 |

*Table5.13: Optimized Network cost*

| section | Equivalent Link | Dia.(mm) | Length(m) | unit cost(ETB) | sub-total cost(ETB) |
|---|---|---|---|---|---|
| 1 | 1+2 | 550 | 1509 | 2954 | 4,457,586 |
| 2 | 4 | 400 | 42.5 | 1980 | 84,150 |
| 17 | 5 | 500 | 960 | 2790 | 2,678,400 |
| 4 | 8 | 300 | 272 | 1100 | 299,200 |
| 5 | 9 | 200 | 516 | 580 | 299,280 |
| | | 150 | 56 | 135 | 7,560 |
| 6 | 10 | 250 | 325 | 875 | 284,375 |
| 7 | 11 | 200 | 630 | 580 | 365,400 |
| 3 | 12 | 350 | 163 | 1450 | 236,350 |
| 12 | 13 | 300 | 284 | 1100 | 312,400 |
| 13 | 14 | 200 | 324 | 580 | 187,920 |
| 14 | 15 | 200 | 371 | 580 | 215,180 |
| 15 | 16 | 150 | 244 | 135 | 32,940 |
| | | 100 | 74 | 110 | 8,140 |
| 8 | 17 | 300 | 256 | 1100 | 281,600 |
| | | 250 | 30 | 875 | 26,250 |
| 9 | 18 | 200 | 226 | 580 | 131,080 |
| | | 150 | 97 | 135 | 13,095 |
| 10 | 19 | 250 | 314 | 875 | 274,750 |
| 11 | 20 | 200 | 426 | 580 | 247,080 |
| 18 | 21 | 250 | 140 | 875 | 122,500 |
| 19 | 22 | 100 | 114 | 110 | 12,540 |
| 20 | 23 | 200 | 120 | 580 | 69,600 |
| 22 | 24 | 200 | 184 | 580 | 106,720 |
| | | 150 | 128 | 135 | 17,280 |
| 21 | 25 | 100 | 417 | 110 | 45,870 |
| 23 | 26 | 150 | 67 | 135 | 9,045 |
| | | 100 | 388 | 110 | 42,680 |
| 16 | 28 | 300* | 24 | 1100 | 26,400 |
| | | | | Total cost(ETB) | 10,895,371 |

*Table5.14: Comparison of the pipe cost gained from the EPANET output and after optimized by using LIDM*

| Pipe dia. from EPANET | | | | Pipe dia. optimized by LIDM | | | | Cost difference |
|---|---|---|---|---|---|---|---|---|
| Dia. (mm) | Length (m) | Unit cost ETB | Sub-total ETB | Dia. (mm) | Length (m) | Unit cost ETB | Sub-total ETB | (ETB) |
| 100 | 114 | 110 | 12,540 | 100 | 993 | 110 | 109,230 | 96,690 |
| 150 | 1190 | 135 | 160,650 | 150 | 592 | 135 | 79,920 | -80,730 |
| 200 | 2532 | 580 | 1,468,560 | 200 | 2797 | 580 | 1,622,260 | 153,700 |
| 250 | 1185 | 875 | 1,036,875 | 250 | 809 | 875 | 707,875 | -329,000 |
| 300 | 734 | 1100 | 807,400 | 300 | 836 | 1100 | 919,600 | 112,200 |
| 350 | 272 | 1450 | 394,400 | 350 | 163 | 1450 | 236,350 | -158,050 |
| 400 | 163 | 1980 | 322,740 | 400 | 42.5 | 1980 | 84,150 | -238,590 |
| 450 | 960 | 2252 | 2,161,920 | 450 | 0 | 2252 | 0 | -2,161,920 |
| 500 | 42.5 | 2790 | 118,575 | 500 | 960 | 2790 | 2,678,400 | 2,559,825 |
| 550 | 0 | 2954 | 0 | 550 | 1509 | 2954 | 4,457,586 | 4,457,586 |
| 600 | 1509 | 3386 | 5,109,474 | 600 | 0 | 3386 | 0 | -5,109,474 |
| | | | 11,593,134 | | | | 10,895,371 | -697,763 |

N.B: -negative sign shows decrease in cost

*Table5.15: Percentage of pipe length required*

| Dia.(mm) | Pipe dia. from EPANET | | Pipe dia. optimized by LIDM | |
|---|---|---|---|---|
| | length | % | length | % |
| 100 | 114 | 1.31 | 993 | 11.41 |
| 150 | 1190 | 13.68 | 592 | 6.80 |
| 200 | 2532 | 29.10 | 2797 | 32.14 |
| 250 | 1185 | 13.62 | 809 | 9.30 |
| 300 | 734 | 8.44 | 836 | 9.61 |
| 350 | 272 | 3.13 | 163 | 1.87 |
| 400 | 163 | 1.87 | 42.5 | 0.49 |
| 450 | 960 | 11.03 | 0 | 0.00 |
| 500 | 42.5 | 0.49 | 960 | 11.03 |
| 550 | 0 | 0 | 1509 | 17.34 |
| 600 | 1509 | 17.34 | 0 | 0.00 |

When we compare table 17 and 18, we can see a difference of the cost of network of the system which is 697,763 ETB when optimized.

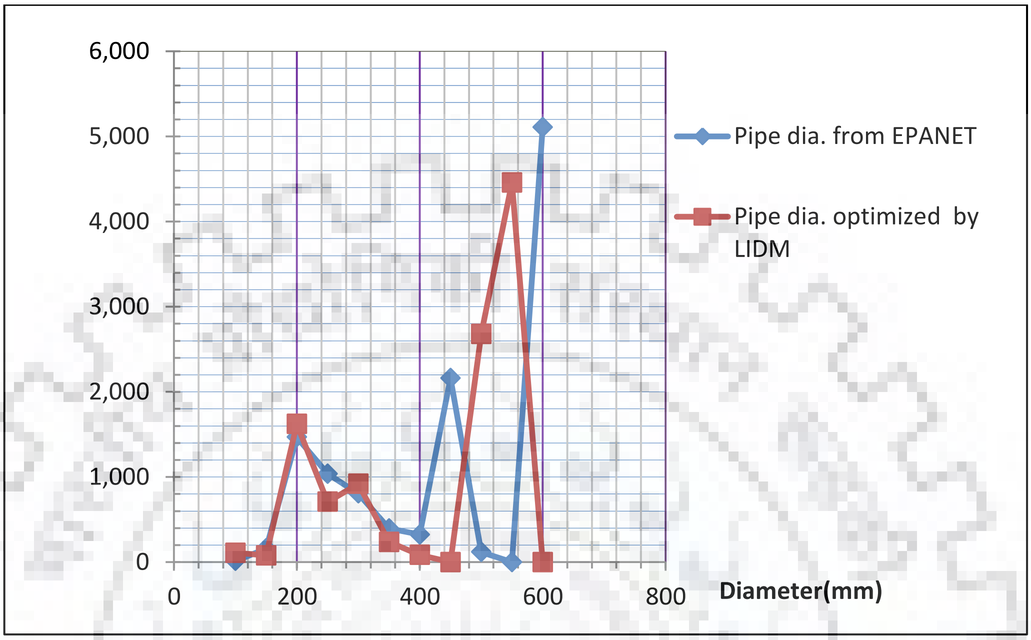



*Figure5.3: cost of the pipe diameter gained from the EPANET output vs the one optimized by LIDM*

# CHAPTER 6

# SUMMARY AND CONCLUSION

There is unbalanced exploitation of water and the output from the scheme. It was repeatedly seen that there was unequitable water use among the water using groups. Sometimes this is a cause of dis-agreement among the irrigation water user groups. The problem will continue even in the future as the source of water for the command area discussed is the only fresh source of water for various needs among different water using sectors. As it was already discussed, there was over exploitation of water due to; leakage from unlined canals, breakage from canals and over watering in flood irrigations at Metahara irrigation scheme while there were also other many agro-industries which need huge volume of water within the basin. A.M. Michael (2007) states that the net irrigated area of Ethiopia is mostly concentrated in the Awash River Valley; and, accounts about 48% of the total irrigated area of the country.

This implies that there is pressure on the available fresh water on the river aforementioned. Most of the time; the downstream of the basin face water shortage during the dry seasons. Hence, converting the canal network feed command areas to pressurised system greatly contributes in solving these issues because of its high efficiency. Even though the area considered for this study (134.12 ha); is small part of the total area under the canal command at Metahara state sugar cane farm, this study could be seen as an indicator and a door for improvement for whole the system.

In general, we have no option unless we improve the efficiency of the irrigation schemes through various interventions to use the scarce/available water resources for addressing the increased competition among water user groups. As it was stated earlier, we have an opportunity to save 900,482m$^3$ volume of water per year if the system is converted to pressurized irrigation system. This implies that, additional 91.8 ha$year^{-1}$ of land can be developed with the same volume of water used under the canal irrigation network. Hence, it offers an opportunity to irrigate more area with the same volume of water and allows other water user groups to share more water. The conversion of existing large scale sugarcane farm canal irrigation networks into pressurized irrigation particularly sprinkler irrigation system may contribute both in using the available water in an efficient and sustainable way. On the other hand, the low annual rain fall availability and high evapotranspiration at the study area may also enforce to search for other means of water saving technologies.

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# APPENDICES

Appendix Table 1: Nodal Inputs

| Node/hydrant | Elevation | Base demand, lps |
|---|---|---|
| A | 990 | 0 |
| B | 996 | 0 |
| C | 993 | 0 |
| Q | 992 | 0 |
| D | 993 | 0 |
| E | 994 | 21.5 |
| K | 993 | 15.1 |
| N | 993 | 39 |
| O | 990 | 25.4 |
| F | 991 | 25 |
| H | 988 | 33.8 |
| I | 987 | 22 |
| G | 984 | 25.1 |
| U | 992 | 24.6 |
| W | 991 | 24.9 |
| V | 992 | 19.5 |
| X | 991 | 17.4 |
| Y | 991 | 11.6 |
| Z' | 988 | 4.2 |
| B' | 985 | 18.3 |
| D' | 985 | 18.7 |
| A' | 990 | 13.7 |
| C' | 990 | 16.1 |
| L | 989 | 21.6 |
| J | 992 | 0 |
| M | 992 | 0 |
| T | 992 | 0 |

Appendix Table 2: Link Inputs

| Link Id | Start node | End node | Length, m | Roughness coefficient(C) |
|---|---|---|---|---|
| 1 | A | B | 400 | 120 |
| 2 | B | C | 1109 | 120 |
| 8 | E | F | 272 | 120 |
| 9 | F | G | 572 | 120 |
| 10 | F | H | 325 | 120 |
| 11 | H | I | 630 | 120 |
| 17 | E | K | 286 | 120 |
| 19 | K | N | 314 | 120 |
| 20 | N | O | 426 | 120 |
| 14 | U | W | 324 | 120 |
| 16 | V | X | 318 | 120 |
| 25 | A' | B' | 417 | 120 |
| 24 | A' | C' | 312 | 120 |
| 26 | C' | D' | 455 | 120 |
| 22 | Y | Z' | 114 | 120 |
| 21 | J | Y | 140 | 120 |
| 28 | V | M | 24 | 120 |
| 23 | Y | A' | 120 | 120 |
| 18 | K | L | 323 | 120 |
| 4 | D | T | 42.5 | 120 |
| 13 | T | U | 284 | 120 |
| 12 | T | E | 163 | 120 |
| 5 | Q | J | 960 | 120 |
| 15 | U | V | 371 | 120 |
| 1 | A | B | 400 | 120 |
| 2 | B | C | 1109 | 120 |
| 8 | E | F | 272 | 120 |

**Non-physical parameters**

**Analysis option**

Total Duration: 144:00

Hydraulic Time step: 1:00

Pattern Time step: 6:00

Pattern Start time: 0:00

Report Start time: 5:30

Statistic: None

**Patterns**

Appendix Table 3: Patterns

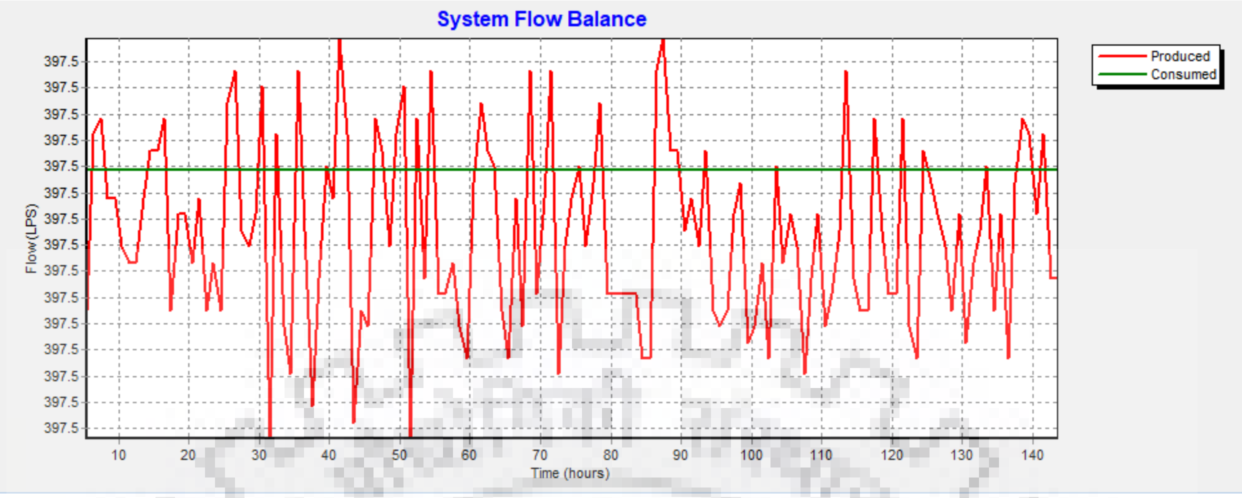
| Time period | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| 52multiplier | 1 | 1 | 1 | 1 |
| 1 Time period= 6 Hrs Time interval | | | | |

## EPANET 2 OUTPUT RESULTS

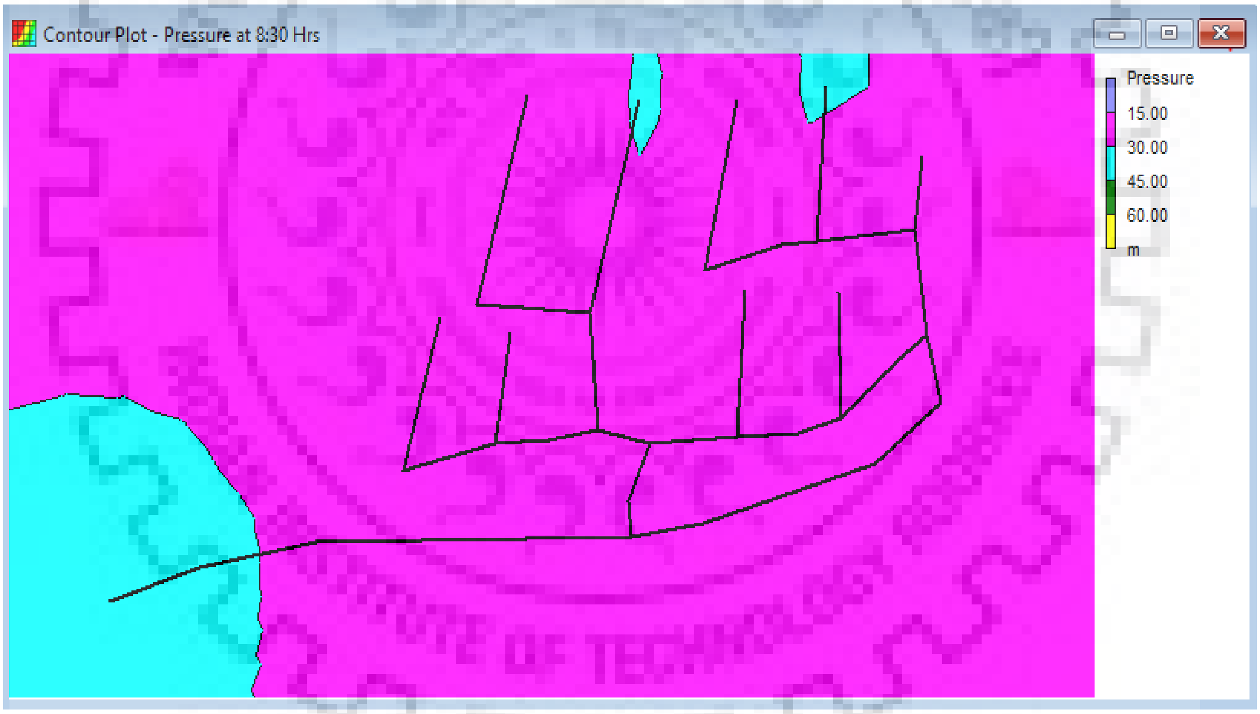
Appendix Table 4: Links at 8:30 Hrs

| Link ID | Length m | Diameter mm | Roughness | Flow LPS | Velocity m/s | Unit Headloss m/km | Status |
|---|---|---|---|---|---|---|---|
| 1 | 400 | 600 | 120 | 397.5 | 1.41 | 3.28 | Open |
| 2 | 1109 | 600 | 120 | 397.5 | 1.41 | 3.28 | Open |
| 8 | 272 | 350 | 120 | 105.9 | 1.1 | 3.91 | Open |
| 9 | 572 | 200 | 120 | 25.1 | 0.8 | 4.15 | Open |
| 10 | 325 | 300 | 120 | 55.8 | 0.79 | 2.53 | Open |
| 11 | 630 | 200 | 120 | 22 | 0.7 | 3.25 | Open |
| 17 | 286 | 400 | 120 | 101.1 | 0.8 | 1.87 | Open |
| 19 | 314 | 300 | 120 | 64.4 | 0.91 | 3.3 | Open |
| 20 | 426 | 200 | 120 | 25.4 | 0.81 | 4.24 | Open |
| 14 | 324 | 200 | 120 | 24.9 | 0.79 | 4.09 | Open |
| 16 | 318 | 150 | 120 | 17.4 | 0.98 | 8.55 | Open |
| 25 | 417 | 200 | 120 | 18.3 | 0.58 | 2.31 | Open |
| 24 | 312 | 250 | 120 | 34.8 | 0.71 | 2.56 | Open |
| 26 | 455 | 200 | 120 | 18.7 | 0.6 | 2.41 | Open |
| 22 | 114 | 100 | 120 | 4.2 | 0.53 | 4.43 | Open |
| 21 | 140 | 400 | 120 | 82.6 | 0.66 | 1.29 | Open |
| 28 | 24 | 400 | 120 | 0 | 0 | 0 | Closed |
| 23 | 120 | 250 | 120 | 66.8 | 1.36 | 8.58 | Open |
| 18 | 323 | 200 | 120 | 21.6 | 0.69 | 3.14 | Open |
| 4 | 42.5 | 600 | 120 | 314.9 | 1.11 | 2.13 | Open |
| 13 | 284 | 400 | 120 | 86.4 | 0.69 | 1.4 | Open |
| 12 | 163 | 600 | 120 | 228.5 | 0.81 | 1.18 | Open |
| 5 | 960 | 450 | 120 | 82.6 | 0.52 | 0.73 | Open |
| 15 | 371 | 250 | 120 | 36.9 | 0.75 | 2.86 | Open |
| Pump P1 | #N/A | #N/A | #N/A | 397.5 | 0 | -35 | Open |
| Valve V1 | #N/A | 450 | #N/A | 82.6 | 0.52 | 0 | Open |
| Valve V2 | #N/A | 600 | #N/A | 314.9 | 1.11 | 0 | Open |
| Valve V3 | #N/A | 400 | #N/A | 0 | 0 | 0 | Closed |

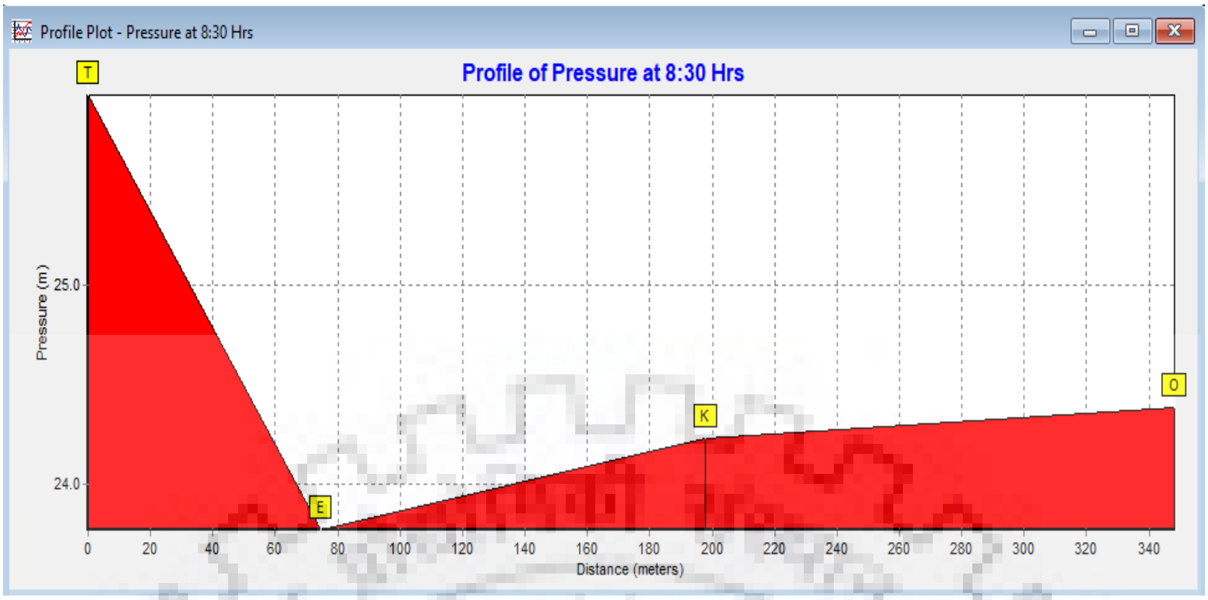
**Graphs from EPANET out put**



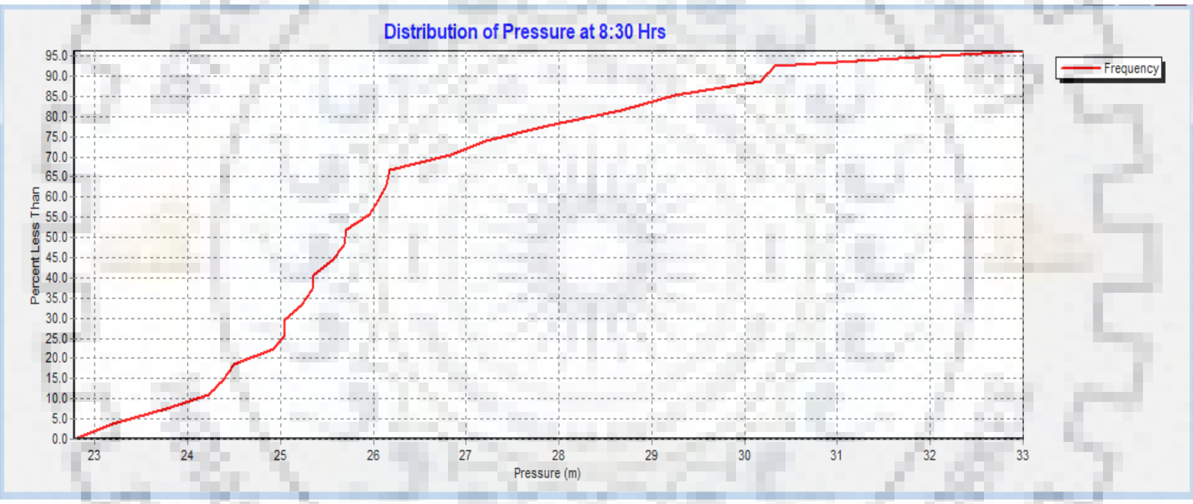

*Appendix figure 1: System Flow balance*




*Appendix figure 2: pressure contour plot at 8:30 Hrs*




*Appendix figure 3: Pressure profile of selected nodes at 8:30 Hrs*




*Appendix figure 4: Distribution of pressure at 8:30 Hrs*