HYDRAULIC MODELING OF DRINKING WATER DISTRIBUTION NETWORK OF SOUTH DUMDUM MUNICIPALITY IN WEST BENGAL USING EPANET AND WATERGEMS SOFTWARE

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Submitted by

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Dedicated to

Frontline Workers against COVID-19 Pandemic

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Declaration of Originality and Compliance of Academic Ethics

I hereby declare that this thesis contains literature survey and original research work by the undersigned candidate, as part of my Master of Engineering in Water Resources & Hydraulic Engineering in the Faculty Council of Interdisciplinary Studies, Law & Management, Jadavpur University during academic session 2021-22.

All information in this document have been obtained and presented in accordance with academic rules and ethical conduct.

I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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Certificate of Recommendation

This is to certify that the thesis entitled ***"Hydraulic Modelling of Drinking Water Distribution Network of South Dumdum Municipality in West Bengal Using EPANET and WaterGEMS Software."*** is a bonafide work carried out by Mr. Tanay Ghosh under my supervision and guidance for partial fulfilment of the requirement for the Post Graduate Degree of Master of Engineering in Water Resources & Hydraulic Engineering during the academic session 2021-2022.

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This foregoing thesis is hereby approved as a credible study of an engineering subject carriedout and presented in a manner satisfactorily to warranty its acceptance as a prerequisite tothe degree for which it has been submitted. It is understood that by this approval the undersigned do not endorse or approve any statement made or opinion expressed, or conclusion drawn therein but approve the thesis only for purpose for which it has been submitted.

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for the evaluation of the thesis.

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ABSTRACT

The importance of water supply is acknowledged by all governments in developing countries who have and are giving priority to this provision. The presence of iron and fluoride in theground water and the depleting level of the sub-surface water in the North 24 Parganas district has elevated a need to provide water for a community by tapping the most suitable source ofwater, i.e. surface water ensuring that it is safe for domestic consumption and then supplyingit in adequate quantities. This paper concerns for the design of 24 X 7 urban water distribution systems at Ward No. 30 area consisting of 5.94 km of Pipeline and serving to a design populationof 24871 within the South Dumdum Municipality in the North 24 Parganas district. This paper is helpful to have an insight for a comprehensive and diligent procedure for designing new distribution network in haphazard developed Municipality area. For designing of best economical hydraulic modeling of drinking water distribution system, EPANET & WaterGEMS software are used in this study. Designprocedure satisfied all parameters namely residual nodal pressure, velocity of flow in pipe, pipe material,unit head-loss, reservoir level, peak factor and available commercial pipe diameters in adherence to the CPHEEO manual clauses. The results obtained show that the procedure makes it possible to ensure the norms as per the standard and minimum pressures when the peak water demand conditions is prevailing. Both the software's EPANET & WaterGems yielded similar output.

Keywords: EPANET, Water-GEMS, Urban, Water, Distribution, Hydraulic, Modeling

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CHAPTER: 1

1.1 Introduction

Water is fundamentally the inherent necessity of mankind and essential resource of human civilization. Access to safe and clean water plays a vital role in social and economic development and public health for any society. Adequate availability of water is required for human life and hence the demand for fresh water resources is increasing the concern of planners and policy makers towards it. About 1.2 billion people in the world are without access to drinking water and about 2.4 billion are lacking basic sanitation and hence the symptoms of emerging global water crisis are too obvious. It has been projected that the population suffering from water scarcity will rise from 450 million at present to 2.7 billion by 2025 and the Indian subcontinent has already been classified as 'water stressed' which means that availability of water should be exceeded (Ministry of Urban Development, Government of India, 2013). India is highly dependent on groundwater resources for day to day survival. Right now, India is facing the challenge to fulfill its demand through the existing but depleting resources. To supplement its present resources, we haveto find unconventional solutions involving recycle and reuse of water, rainwater harvesting etc. It is possible to store 214 BCM of surplus monsoon runoff in ground water reservoir. We have to go back to the era when people valued and conserved each drop. India is home to ~17% of world's population but has only 4% of the world's freshwater resources *(Indian Institute of Tropical Meteorology, 2014)*. Managing these for a huge population is a mammothtask. It is estimated that about two lakh people die every year due to inadequate water sanitation and hygiene. In 2016, per person disease burden due to unsafe water and sanitation was 40 times higher in India than in China and 12 times higher than in Sri Lanka *(NITI Aayog, August 2019)*. With the country generating huge amounts of waste water annually, mismanagement of waste water, which also contaminates groundwater, lack of liquid waste management. Poor sanitation conditions and poor hygiene habits have contributed to a significant portion of population suffering from water—home diseases. Currently, nearly 820 million people in 12 major river basins of India are facing high to extreme water stress situation. Out of these, 495 million alone belong to Ganga river basin which generates nearly 40 percent of the country's GDP *(Aayog, August 2019)*. The scarcity of water resources also has many cascading effects including desertification risk to biodiversity, industry, energy sector and risk of exceeding the carrying capacity of urban hubs. It's a fact that water is a State subject and its optimal utilization and management lies within the domain of the States. Byconsidering all these, it is very clear that now the Nation has to come up with interventions which measure not only the outcomes but also the effort involved in achieving these outcomes. In addition to this, two new indicators related to 24 X 7 piped water supplies tovillages and villages having individual household water meters are included in the Index. These two indicators fulfill the mandate of Govt. of India to achieve the objective of 24 x 7 supply of piped water to not only urban areas but also rural areas.

1.1.1 Present Status of Global Water Scarcity

Global water use has increased by a factor of six over the past 100 years and continues to grow steadily at a rate of about 1% per year as a result of increasing population, economic development and shifting consumption patterns *(The United Nations World Water Development Report 2020)*. Combined with a more erratic and uncertain supply, climate change will aggravate the situation of currently water-stressed regions and generate water stress in regions where water resources are still abundant today. Physical water scarcity is often a seasonal phenomenon, rather than a chronic one, and climate change is likely to cause shifts in seasonal water availability throughout the year in several places. Climate change manifests itself amongst others, in the increasing frequency and magnitude of extreme events such as heat waves, unprecedented rainfalls, thunderstorms and storm surge events.

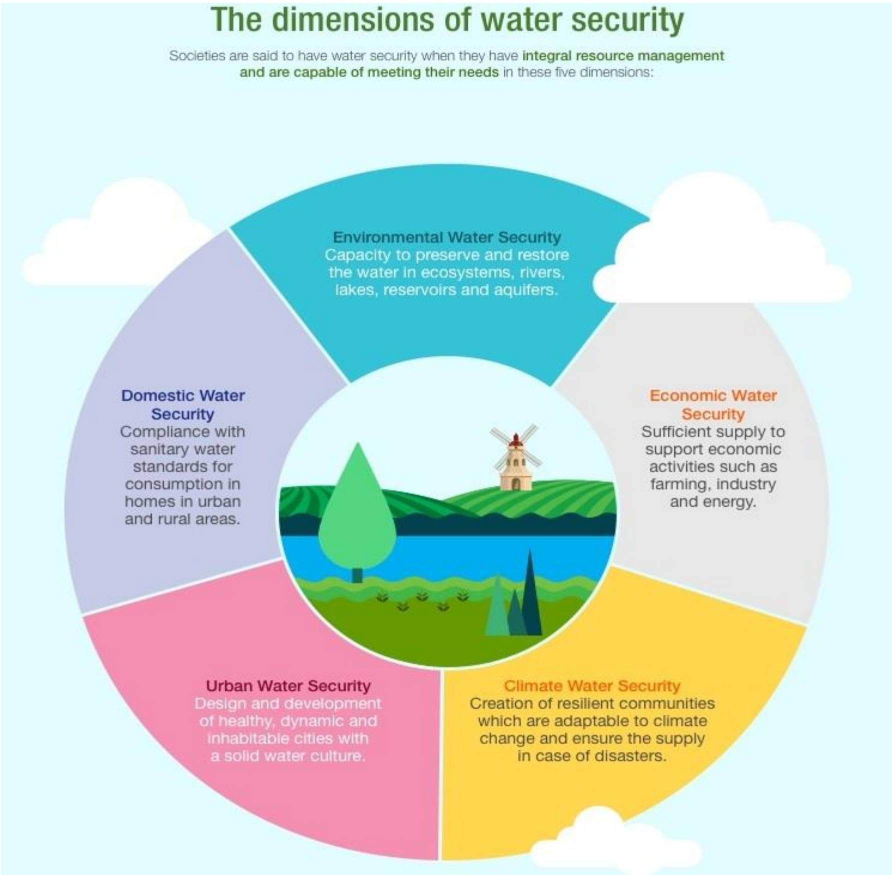

Figure 1: The dimensions of water security *(Source: Latin American Water Funds Partnership, Water security report.)*

Water quality will be adversely affected as a result of higher water temperatures, reduced dissolved oxygen and thus a reduced self-purifying capacity of freshwater bodies. There are further risks of water pollution and pathogenic contamination caused by flooding or by higher pollutant concentrations during drought. The physical infrastructure for delivery of water and sanitation facilities can also be disrupted, leading to contaminated water supplies and the discharge of untreated wastewater and storm water into living environments. Vector-borne diseases such as malaria, rift valley fever, leptospirosis and others are often observed after flooding events.

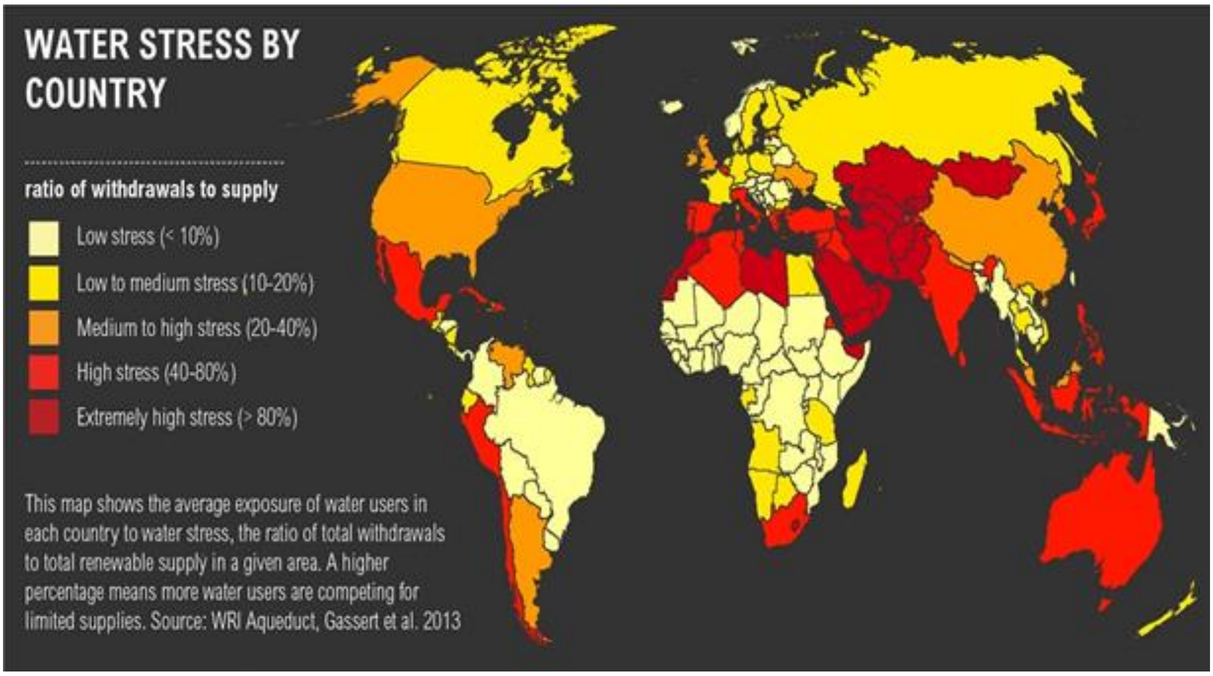

Figure 2: Countries those will be Water Scarce by 2040 *(Source: World Resources Institute, Gassert et al. 2013*)

Urban water resilience goes way beyond the traditional city boundaries. In cases where water supplies rely on distant watersheds, planning needs to look well beyond the city's boundaries and consider the long-term impacts of urban expansion on distant freshwater ecosystems and the local communities that also rely on them.

In small urban and rural settlements, use of water for agriculture and in some cases industrial applications results in reduced availability for domestic uses. Domestic supplies must be prioritized under the human rights to water and sanitation.

1.1.2 Water Scarcity in India

In spite of possessing surface water resources, India is highly dependent on groundwater resources for day to day survival. According to Central Groundwater Board (CGWB), contribution of groundwater is nearly 62% in irrigation, 85% in rural water supply and 45% in urban water supply *(CGWB Report,2017).* The entire green revolution in the country was based on the development of groundwater resources. There are over 20 million wells pumping water with power supply provided by the Government *(CWC Report,2017).* This has been depleting groundwater, while encouraging wastage of water in many states. A comparison of depth to water level of pre-monsoon 2018 with decadal mean pre-monsoon (2008-2017) reveals that about 52% wells are showing decline in water level.

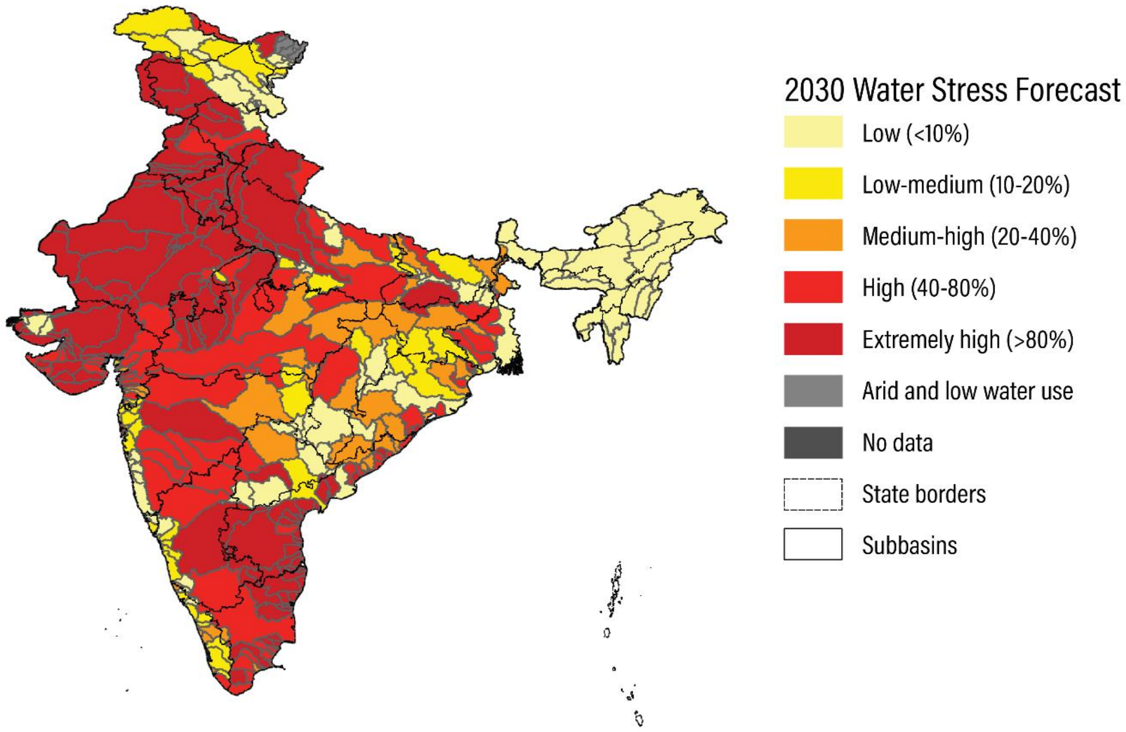

Figure 3: Water Stress forcast for year 2030 in India (*Source: World Resources Institute Aqueduct Water Risk Atlas Report*)

Water scarcity can seem difficult to full grasp, given the dichotomous ways in which water is affecting habitations. On the one hand, the low-lying areas are getting submerged due to risein ocean water and on the other hand, droughts are becoming a common phenomenon inhighly populated regions. In India, during 1996-2015, nearly 19 million and 17.5 millionpeople annually were simultaneously affected by floods and droughts, respectively *(UNESCO,2019)*. In a curious irony, both scarcity and excess of water are affecting habitations.

1.1.3 Present Scenario in West Bengal

West Bengal is the one of the most densely populated state of India at 1000 persons per square km (Govt. of West Bengal, 2010). Its average urban density is much higher at around 7500 persons per square km. West Bengal has liberal water availability as a natural resource that supports intensive rain-fed agriculture. However, the pressure on urban water resources has been increasing over some years due to increasing population, low investment in supply augmentation and dilapidating state of existing systems.

The State of West Bengal comprises of three distinct regions in terms of water resources - North Bengal, Western Rarh and Eastern Bhagirathi. The basins of north Bengal consist of 63 per cent of state water resources while the Rarh and eastern plains carry 22 per cent and 15 percent respectively (Rudra). In terms of urban water delivery, WB can be divided into two separate areas – Kolkata Metropolitan Area (KMA) and non- KMA area. The KMA area consists of 41 urban local bodies (Three Municipal Corporations including Kolkata Municipal Corporation and 38 Municipalities) out of 127 in WB, the rest being under the non-KMA area. The urban water supply in the KMA area has been the responsibility of Kolkata Metropolitan Development Authority (KMDA) while the supply in the non-KMA area was handled by the public health engineering department (PHED) till recently before the devolution of power to the local authorities. Following the 74th constitutional amendment in 1992 and financing schemes like Jawaharlal Nehru National Urban Renewal Mission (JNNURM) from the central government, a slow devolution of authority has been taking place to the local governments. According to the JNNURM report for West Bengal, the responsibility for operation and maintenance for water supply and sanitation systems has been devolved to 86 of the 127 urban local bodies (ULBs) in the state. Also some of the ULBs such as Nabadiganta Industrial Township Authority (NDITA) which comprises of mainly industrial and commercial consumers in the IT industry have entered into arrangements with private players for setting up and maintaining the water supply and waste evacuation systems. KMA covers an area of 1851 sq. km. and caters to a population of 14.7 million out of a total 22.5 million urban populations across WB according to 2001 census (KMDA, 2010). Water in the KMA area is sourced from two sources – river Hooghly, the only surface water source and rest is procured from ground water. Treated water from the river is supplied through pipelines to a limited area of KMA, the majority of areas depending on ground water from deep tube-wells connected to a network of reservoirs and pipelines (City Development Plan, Kolkata).

A snapshot of the water supply scenario in the state is presented in the Table 1. There is an increasing deficit of water requirement in the state and it is worthwhile to examine the efficiencies of various water delivery mechanisms in the state. Another interesting point to note is only 56% of the population is being supplied surface water, the rest depending on fast depleting groundwater sources. Also in terms of access, only 53% of the population has access to water within their premises. Thus depending on groundwater sources has an additional impact on health issues in the state. The data for expected water requirement is shown below in Table 1.

Table 1: Water Requirement and Supply in West Bengal **{***Source: Compiled from data of State Irrigation Department (Rudra)}*

Year	Water Requirement (Million hectare meter, Mham)	Deficit (in %)
2000	10.85	38
2011	13.02	48
2025	16.60	59

1.2 Research GAP

This study will investigate the hydraulic design of Water Distribution system in terms of flow and pressure parameters from the Overhead Tank to the consumer points using EPANET, WATERGEMS V8i software. The EPANET, WATERGEMS V8i software programme analyses the pressure at eachnode, track the flow of water in each pipe. After simulation of the water distribution network, results were presented in various forms. A procedure for the destructuralization of a waterdistribution system is presented.

1.3 Study Area

South Dumdum is a city and a municipality in North 24 Pargnas district in the Indian state of West Bengal. South Dumdum is located at 22.61°N,88.40°E. It is under Barackpur Sub Division. This Municipality is surrounded by Kolkata Municipal Corporation, Dum Dum Municipality, North Dum Dum Municipality, Bidhannagar Municipal Corporation and Baranagar Municipality. South Dumdum Municipality is strategically located immediately adjacent to Kolkata, the capital of West Bengal separated by the railway track of the Eastern Railway on the south west and the Keshtopur Canal on the south. Here the analyses have been carried out for the network of distribution line at Ward-30 under Zone-6 in South Dumdum Municipality. Location Map showing the location as shown in Figure 4.

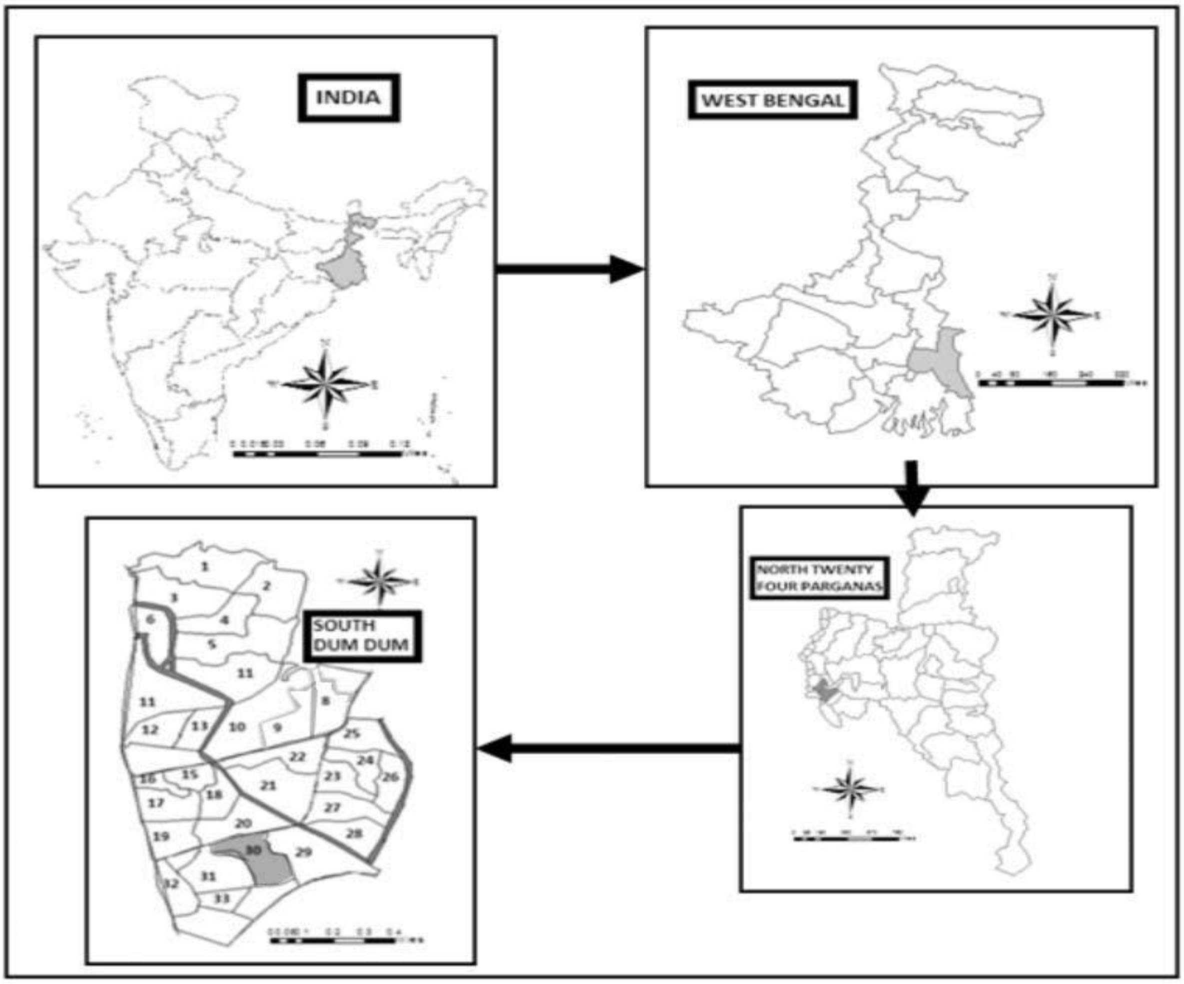

Figure 4: Study Area Location Map

1.4 Objective of the Study

The main objective of this present research study is to develop & design hydraulic model for the South Dumdum Municipality water supply distribution network with the help of EPANET and WaterGEMS software.

1.5 Brief Methodology of the Study

The overall research methodology is mainly divided into five components: Data collection from the present situation by topographical survey and consumer survey, Total Demand Estimation, Model creation by using software WaterGEMS and EPANET, Hydraulic simulation and optimization and Result & Discussion. The layout map for the whole pipeline network in relation to Ward No.-30 at South Dumdum Municipality in the North 24 Parganas District of the State of West Bengal has been studied for this study. The required data for the design have been taken from the available topographic survey drawing of the study area.

The model has been prepared by assigning the proper input parameters to the different element such as the pipes are given the input values length, trial diameter (based on experience and engineering judgement) and Hazen-Williams Constant/Roughness co- efficient. The junctions have been attributed with the input parameters like base demand and elevation. The model is simulated and checked against the relevant CPHEEO clauses and modified if found any error/warning.

1.6 Possible Outcome of the Study

The following are the research outputs:

- To integrate and develop optimized water distribution network in the study area.
- To design a Water Zone based hydraulic model.
- To maintain optimum/stipulated flow parameters in the entire system.

1.7 Outline of the Thesis

The thesis consists of eight chapters.Chapter 2 contains literature review. Chapter 3 illustrates the study area and the researchmethodology. Chapter 4 reveals the governing equations and the relevance of numerical model for the design and Analysis of the Distribution Network. Chapter 5 contains methodology of the study. Chapter 6 presents result and discussion of the hydraulic modeling of the water distribution network by using EPANET & WaterGEMS software. Finally, the summery of the research study describes inconclusions and recommendations with the future scope of the study which are provided inchapter 7. References are given at end of the dissertation.

CHAPTER 2

2.1 Literature Review

Engelhard *et al.* (2000) discussed about rehabilitation strategies for water distribution networks: a literature view with a UK perspective. The multi-objective optimization approaches which have been developed recently have the potential to be developed into the required whole-life costing model based on the appropriate economic model and performance criteria.

Ulanicki *et al.* (2000) discussed about open and closed loop pressure control for leakage reduction of pipes. This paper formulates and investigates methods for planning and implementation of on-line control strategies of predictive and feedback control for areas with many pressures reducing valves and many target points. The considered methods explicitly take into account a leakage model. The results are applied to an area with three pressure reducing valves and two target points.

Das *et al.* (2006) carried out a pipe network analysis for a water treatment plant. As a case study, the total schematic drawing of the rising main for South 24 Parganas water supply scheme under Public Health Engineering Directorate, Government of West Bengal was considered. The network comprises of a clear water reservoir from which two pumps are supplying water to 24 numbers of zonal overhead reservoirs and two ground level reservoirs (for feeding 27 more overhead reservoirs) through the rising main. It was found that in this network, the time requirement taken for filling different overhead reservoir fully are not similar at all. Some of these fill up early and some other take a long duration. This may cause differential water supply and may be even insufficient supply to the some of the local consumers. That is why, fill up times of overhead reservoirs are to be maintained almost same by adjusting the flow control valves. The objective of this study was to simulate the present condition through a computer model which would act as a base for any modifications in the network if required and then suggest modifications to achieve the flow in such a way to fill up all the overhead reservoirs and ground level reservoirs almost simultaneously. The head losses at different valves and pipes were optimized without disturbing the required flow. In the first part, the required flows to each overhead reservoir were balanced in such a way that all of them get fully fill up after almost in a same interval. In the second part, a lot of modifications were done in the valves for optimizing the head losses without disturbing the first phase. After modelling the network, it was found that the time requirements of 24 overhead reservoirs are almost same. Then optimum head losses in pipes and valves were also achieved during the operation. Also it was suggested that the actual administration of chlorine could be done in such a way that the concentration goes through the total network for effective disinfection of water at the consumer point.

Das *et al.* (2008) carried out a study in two phases. In the first phase, a model of pipe network based on EPANET in such a way that the modelled flow data for various zonal overhead reservoirs takes almost equal fill up times by adjusting the flow control valves (FCV) and changing some other parameters such as flow parameter of flow control valves, loss coefficient of throttle control valves (TCV), percentage of opening of pressure reducing valves (PRV) etc. In second phase, a lot of modifications have been done by adjusting different parameters of TCV, PRV and FCV for optimizing the head losses in pipes and valves in the modelled system so that the desired water inflow to the zonal overhead reservoirs is achieved. It has been found that in this network, the time requirement taken for filling different overhead reservoir fully are not similar at all. Some of these fill up early and few other take a long duration. This will cause differential water supply and the result may be indicated insufficient supply to the local consumers. The results concluded that 60 % and more reduction of losses were occurred in the pipelines and valves (average head loss of 90 valves out of 95 is less than 24 m). The validation of the model also revealed that almost 92 % success (22 overhead reservoirs out of 24 reservoirs and two booster stations, are filled up completely after 5.5 hr and 6 hr respectively for maintaining the same fill up times of all the overhead reservoirs.

Izquierdo *et al.* (2008) discussed about sensitivity analysis to assess the relative importance of pipes in water distribution networks. On the contrary, their designs are closely related to their hydraulic performance. Recognizing the sundry and relative importance of the different pipes in a water distribution network may help in assessing their impact on the hydraulic performance of the network. This information, in turn, will be helpful in the different aspects of a water distribution network make-up, namely design, planning, control and management.

Liana and Vlad (2008) made computer simulation by using EPANET 2.0 software and info works WS and obtained the results with two software programs on the Hanoi water distribution network which was a proper evaluation of the network under normal and abnormal conditions.

Koppel and Vassiljev (2009) discussed about calibration of a model of an operational water distribution system containing pipes of different age. The aim of the paper is to demonstrate that the Levenberg–Marquardt algorithm can give successful results when operational water distribution systems are calibrated with the proper selection of parameter increment for the calculation of partial derivatives.

Saha (2010) worked on the alignment of pipelines have two major considerations: i) head loss and ii) hammer degradation. The loss of water in pipelines can cause pressure transiency and can create stress on the system. Pipeline alignment will also cause land use change and displacement of local population. Resistance may arise later. Another important parameter was considered is the cost. High head loss or large change in land use and compensation to the displaced population can increase the total cost of alignment.

Arunkumar and Nethaji (2011) examined a study on water demand analysis of Public Water Supply in Municipalities using EPANET 2.0 software with the aim of providing effective planning, development and operation of water supply and distribution networks which is one of the most essential components of urban infrastructure. Using EPANET 2.0 water model software, the demand for the Underserved and Un- served area is calculated. A framework for taking management decisions such as an extension of the supply network and location of new facilities was given.

Chandramouli and Malleswararao (2011) discussed about reliability based optimal design of water distribution network for municipal water supply using EPANET2.0 software.

Chatterjee (2011) analysed the pressure transient of a long pipeline through simulation using HAMMER software during power failure of Mejia Thermal Power Station, Damodar Valley Corporation, Durgapur considering the case of water hammer with controlling devices and different types of valves. The study has been based on several runs of the software module under different conditions of failure of hydraulic system. The study is conducted for the transient analysis in case of 15 km long 1470 mm diameter rising main. The results revealed that the transient can be minimized either by repositioning air chamber and different types of valves along the pipeline. Attempt was also made to find out alternative device like surge tank. The results also revealed that position of protection device has influence on the magnitude of transient head.

Mukherjee *et al.* (2012) studied about a pipe network design and analysis for a Water Treatment Plant. As a case study, the total schematic drawing of the distribution network of Dhapa Water Treatment Plant under KMC which serves the population of Eastern Kolkata has been considered. The network comprises of clear water reservoir from which three pumps were supplied by a common header to three semi underground reservoirs (head works) situated at Anadapur, Mukundapur and Patuli. From these three head works the treated water is conveyed to sum total of 17 elevated storage reservoirs which would serve 14 KMC wards. Network was modeled and the optimum head losses in pipes and valves, pressures and hydraulic head in junctions were calculated by EPANET and HAMMER software. HAMMER is further used in the four pump houses separately to check maximum transient head which is developed in case of failure of the supply pumps. A stress analysis was carried out in the pipes where the maximum hammer head was observed and for safety analysis of the pipes the ultimate stress observed from the hammer analysis of the pipe is compared with the standard tensile stress of the pipe material used by considering the safety factor. In each pump house the above analysis was done and it was also checked when the flow capacity of the pumps was increased by 25%, 50%, 75% and 100% of the original rated flow whether the pipes are safe or not. Moreover, a study was also done on pipe network system for pseudo loop system. In support to which for example it was considered an example and tried to check whether the difference in elevation between source and receiving end is equal to the total head lost in the pipes between them. Due to non-availability of practical field data in run condition few data help was taken from HAMMER and 98.5% accuracy was found out.

Mukherjee *et al.* (2012) carried out a comparative study about the hydraulic analysis outputs of pipeline network between EPANET and HAMMER software. The main objective was to model a pipeline of water distribution network and accordingly putting the inputs as required in both the software. The outputs mainly at junctions (i.e. hydraulic grade line and pressure), pipes (i.e. flow, unit head loss) and velocity were considered for representing the above study. The comparative study has been carried out with the help of statistical regression analysis by finding out correlation coefficient and probable error coefficient. A relation was found out between the outputs of EPANET and HAMMER by the properties of linear regression, so that in unavailability of any one of the software, the results of the other software could be found out. Any network analysis software gives the same result for a fixed input, when statistical analysis of results is considered a significant difference though difference is too low but still an important one where precision becomes first criteria of designing a pipeline network. For a particular distribution network, it is seen that during hydraulic analysis the output obtained from the two softwares are moreover the same but a very slight difference could be found among them while undergoing statistical analysis by the process shown in this paper. Graphical representation was also illustrated between the outputs of pipes and junctions of the above network and discussions were made on the probable amount of correlation between the outputs of the two software.

Das *et al.* (2013) investigated that with the increase demand of water in an urban area it becomes necessary to increase the capacity of the water pipeline networks by keeping the pipes and valves elements unchanged as it is quite hazardous work to change those in an urban area. This paper presents a study on characteristics of mainly hammer head at increased flow demand in pipeline networks using HAMMER software. For modelling the pipeline distribution networks, the parameters of the pipes, junctions and other elements were inserted in the above software according to the layout of distribution network of Dhapa water treatment plant, KMC. The flow capacity of the pipeline networks was increased in the order of 25 %, 50 %, 75 % and 100 % more than the existing flow capacity of the above networks and the transient analyses were done accordingly. Increasing trends of hammer head, pressure and circumferential stress with respect to increased flow demand were observed for all the zones which satisfy with vide validation with basic equation for water hammer theoretically.

Mukherjee *et al.* (2015a) found out if measuring steps can be taken in the water distribution systems then the flow received at many outlets can be very high. From comparison between measuring steps and initial running condition at various outlets highest incremental percentage was recorded to be around 600 %. By comparing the above with initial flow condition it can be said that this hike was recorded that those places where the flow was near to zero during the peak time.

Mukherjee *et al.* (2015b) worked on transient analysis of pipeline network for drinking purpose in Assam, India is to study the transient behaviour of the proposed raw water pipeline for drinking purpose which is linked between intake well at Brahmaputra river to Jalukbari water treatment plant (WTP).

Das *et al.* (2015) studied the basic techniques of landscape irrigation system design. Intended as a very basic text for irrigation design, these proceeds with the application of software like GPS Garmin Map Source, AutoCAD, WaterGEMS and HAMMER this software are equally important in water treatment pipeline designs.

Mukherjee *et al.* (2016) worked on hydraulics of pipeline systems and checked the behaviour of such steam pipeline which are mainly affected by thermal stresses where realized on the real life operation of a captive power plant at Chhattisgarh, India.

Ganguly *et al.* (2016) dealt with a study that is basically a test through software simulation of the usage of flow control valves (FCVs) to control the flow in order to meet the demand in a drinking water pipeline network system. A case study in this research has been undertaken to affirm the usage of the valves. By seeing the results of the pressures and hydraulic grades of the total pipe network system along with few geographical undulations as depicted in the Table of elevations, it can be said that the usage of air valves are very much necessary to the proper demand in this case.

Halder *et al.* (2016) studied a closed pipe network of one of the command area zone under the water supply scheme from River Rupnarayan to calculate various parameters such as hydraulic grade, pressure, flow, velocity and unit headloss in the whole piped network. Before that study, the River Rupnarayan was chosen by PHED to supply potable water to the residents of a rural area near Panskura block of Purba Medinipur District in West Bengal. During the analysis in this study, EPANET software has been used. The whole network of service area under one of the zone of the said water supply scheme, here Brindabanchak Gram Panchayat consists about 364 pipes and 337 junctions. There were junctions where the flow has become very less inspite of the fact that the pressure is there is due to the non-utilisation of valves. The results were also compared with a popular pipeline designing software LOOP. The obtained flow outcomes of the pipe network system using EPANET and LOOP software's had a similarity of 96%. It suggests that by using any one of the software, these results could be adjusted.

Mukherjee *et al.* (2017b) conducted a study on clear water pipeline distribution network of a Water Treatment Plant in Assam, India. Water was supplied in three different routes in 42 numbers of elevated storage reservoirs through three routes. The treated water was conveyed by using different number of pipelines of different lengths, diameters. Sixteen numbers of flow control valves were used in the places where the outflow computed is excess from the demand. This deviation was adjusted by increasing the capacity of the pumps with proper usage of air valves with valve operations directly helping to achieve the demand and thus reducing energy consumption cost.

Pulai *et al.* (2017) worked on operation of Anandapur Head work at Anandapur in ward no - 108 of Borough-XII under K.M.C. dedicated to serve water at three wards that is 106,107 and 108 under Borough -XII specially serves Ward no-108 that is the study area of which population about sixty thousand including floating persons as per census 2011. The operation of the above Head works is generally scheduled in three shifts that is morning, noon and evening. Complaints are there regarding low pressure of flow of water at various areas. So, to solve the problem it is required to understand the real life operation through the software and change the required elements from the pipeline network. EPANET software has been used in this study to analysis head, base demand, pressure, elevation at different junctions of the said model and from this to look what changes occur in different sections of pipes in terms of flow, velocity, head losses etc and to relate the outcomes with the help of EPANET software to analysis the different said parameters at different junctions of pipe network.

Sarkar *et al.* (2017) dealt with a study on the pipe line network drawing of Panskura-II Zone-1 Boosting Station, Zone-1, under Panskura Municipal Corporation (KMC). Panskura-II Zone-1 pumping station is dedicated to serve mainly six mouza the JL no. 289, 290, 291, 292, 293 and 302 consisting of huge population. The operation of the above station will start in full flow in 2017. To examine the zone-1 water distribution network we have to draw the network and calculate its output like pressure, head loss, velocity so that no problem will arise when it works in a full flow. WaterGEMS software developed by Bentley Systems.

Mondal *et al.* (2017) worked on the clear water pipeline system of treated water to be supplied from the clear water reservoir, having capacity of 107 MGD, located at Jalukbari Water Treatment Plant (WTP) at Assam in India. The potable water is to be conveyed from Jalukbari WTP to Kamakhya Hill Top Reservoir by using pumps through a pipeline of 1400 mm diameter and the distance to be covered is about 5.585 km along with the elevation difference of about 127 m between Jalukbari WTP and Kamakhya reservoir. For the detailed study and analysis at steady state condition, the complete WTP pipeline network including pipeline rising main, pump house, reservoirs and valves have been considered in the total pipeline system. The steady state simulation has been undertaken by the help of WaterGEMS software. From the results obtained through software simulation, the hydraulic grade or corresponding pressures at different junctions in the whole pipeline system have been found out.

Das *et al*. (2018) worked on scheme of potable water supply to the residents of rural area of Brindabanchak Gram Panchayat at Panskura-II block of East Medinipur district in West Bengal from river Rupnarayan, under the Public Health Engineering Directorate under Govt. of West Bengal. During the analysis, EPANET software was used. The entire network system consists about 364 numbers of pipes and 337 numbers of junctions. The results were also compared with the outcomes from a popular pipeline designing software LOOP. The obtained flow using EPANET and LOOP software had a similarity of around 96%. It suggests that by using either EPANET software or LOOP software the results could be adjusted for pipeline design purpose in future.

Das *et al.* (2019) worked on the pipeline balancing network of rising main alignment of ground level reservoir (GLR II) to overhead reservoirs for surface water based water supply scheme at Bhangar II BLOCK in South 24 Parganas under West Bengal by the useded of EPANET and WaterGEMS software. After getting output, both the software results have been compared with various parameters to show the merits and limitations of both the systems and to understand how the system can be optimized in an economical manner.

Goswami *et al.* (2020) worked on scheme of hydraulic modeling of a rural drinking water distribution network in bankura using watergems. The design of 24 X 7 rural water distribution systems in Chausal area consisting of 29 Kms of Pipeline and serving to a design population of 8381 within the Gangajalghati Block in the Bankura district. The procedure is based on the use of techniques like (a) allocating all the network nodes among an assigned number of District Metering Areas (DMAs) and (b) determining which pipes should be closed to delimit such districts and which pipes should be left open and fitted with flow meters in order to maximize the resilience of the entire system after the creation of districts. It simulates the water distribution network and calibrates the different parameters for implementing in the rural area.

CHAPTER 3

3.1 Study Area

Here the analyses have been carried out for the network of distribution line at Ward-30 under Zone-6 in South Dumdum Municipality. Location map showing the study area as shown in Figure 5.

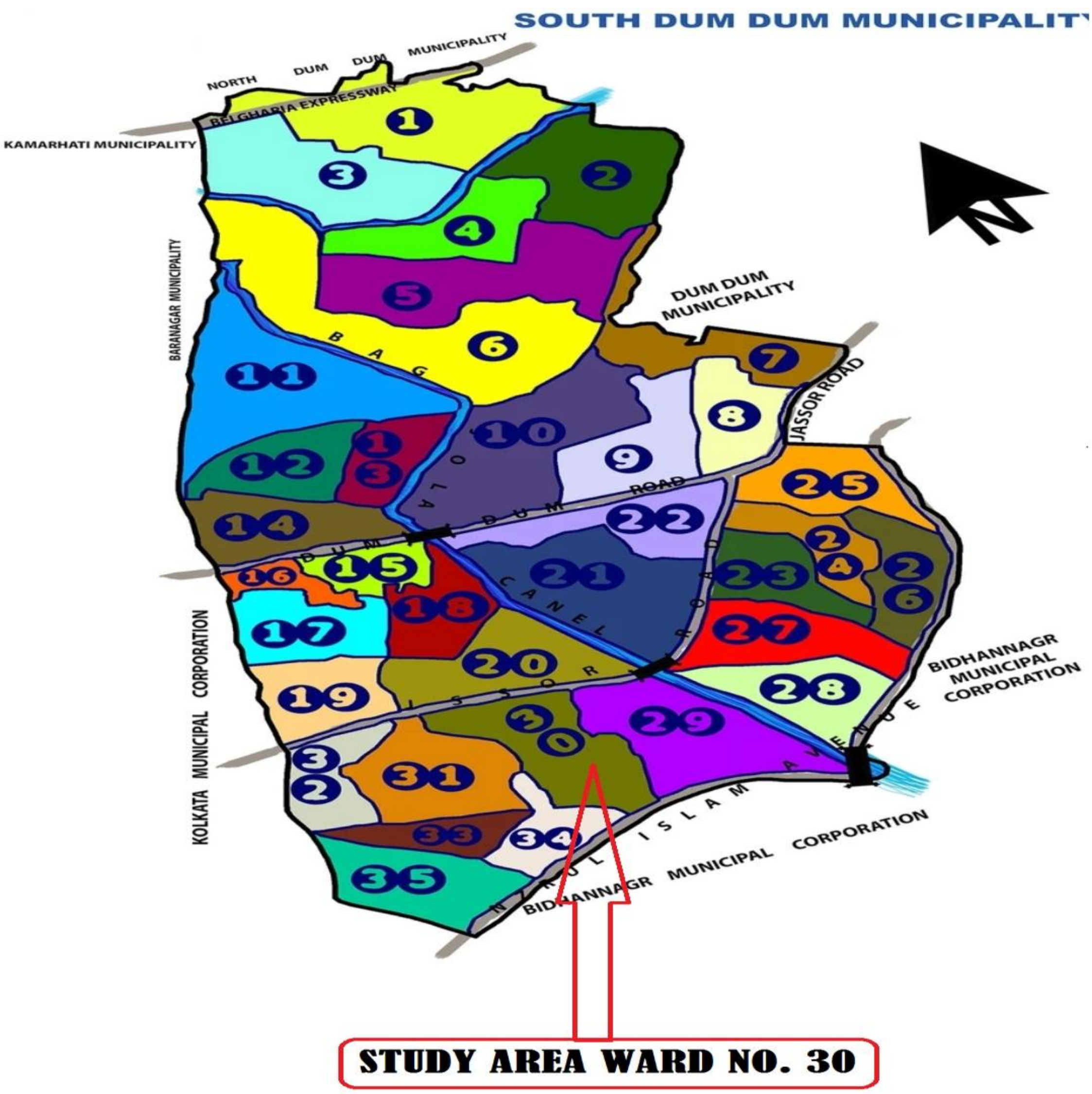

Figure 5: Study Area Map Ward No.-30

A satellite image of ward no. 30 in South Dumdum Municipality showing the road pattern and ESR in Figure 6.

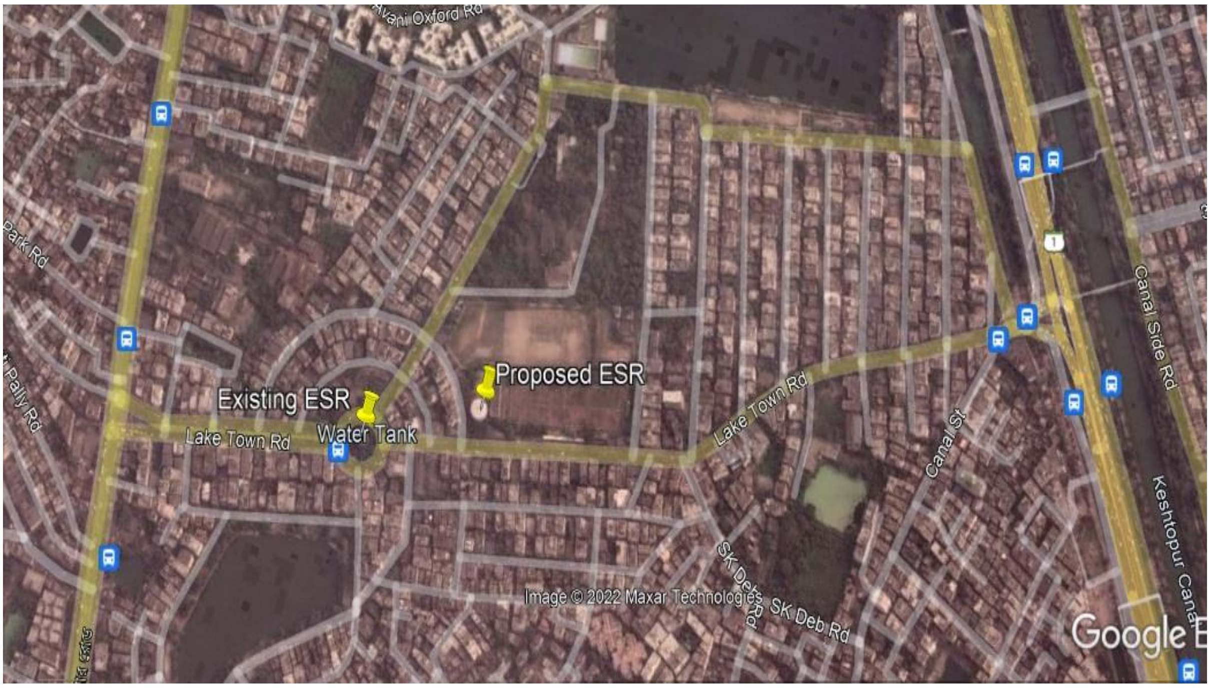

Figure 6: Satellite image of the study area

The ESR picture of ward no. 30 in South Dumdum Municipality as shown in Figure 7.

Figure 7: ESR at Ward No.- 30 in South Dumdum Municipality

This study will investigate the hydraulic design of Water Distribution system in Ward-30 under Zone-6 in South Dumdum Municipality area in terms of flow and pressure parameters as per the CPHEEO norms from the Overhead Tank to the consumer points using EPANET & WaterGEMS software.

3.2 Research Area Overview

3.2.1 Background

South Dumdum is a city and a municipality in North 24 Pargnas district in the Indian state of West Bengal. South Dum Dum consists of localities like Nagerbazar, Bangur Avenue, Dum Dum Park, Lake Town, Paikpara. South Dumdum is located at 22.61°N ,88.40°E. It is under Barackpur Sub Division. This Municipality is surrounded by Kolkata Municipal Corporation, Dum Dum Municipality, North Dum Dum Municipality, Bidhannagar Municipal Corporation and Baranagar Municipality. Area of South Dum Dum Municipality is 17.39 Sq.km with 35 nos municipal ward.

Presently South Dum Dum Municipality is having 15 MGD treated surface water from KMDA & KMC and extracting ground water through 81Nos of Deep Tube well. South Dum Dum Municipality is having total water storage capacity of 5.56 MGD. Out of which Clear Water Under Ground Reservoir is of capacity 5 MGD and there are 12 nos of ESRs of total 0.56 MGD capacity as per slip prepared by South Dum Dum Municipal authority.

Presently clear water being supplied to consumers mostly through Elevated Storage Reservoirs and partly by direct pumping. Total length of distribution line already laid in the city is about 300km as per Slip prepared by South Dum Dum Municipal Authority. There is total road length of 450 Km. Out of which 300 km of road having clear water distribution line under various water supply project as implemented under GOI projects. It is required to be mentioning here the South Dum Dum Municipality itself with his own fund has implemented some clear water distribution network as per information received from South Dum Dum Municipal Authority.

In the previous project under JnNURM Intake structure for 30 MGD WTP has been constructed at Mallickbari, WTP of capacity 30 MGD has been constructed at Kamarhati Baranagar water works, from the CWR of the WIP clear water has been supplied to three numbers of Municipality namely Dumdum, South Dumdum, North Dumdum.

The pipe line network has been developed generally over the years in an unplanned and piece meal manner depending on spot requirement without any proper planning and design before implementation of JnNURM project. The present distribution network is not covering entire municipality. In some cases, insufficient supply of water has been identified as major problems where distribution network had not been provided. Following issues are also identified during citizen feedback: Wastage of water within household and from street stand post.

As the demand of water for industrial purpose, floating population and number of residential high rise buildings have been increased abruptly in the above mentioned municipalities so due to rapid urbanization the scenario of land use pattern has been totally changed and consequently, the present production capacity of the above mentioned 30 MGD WTP is now insufficient to meet the demand of water for the intermediate phase (2035). So there is a need for construction a new WTP.

KMDA in consultation with Municipal Authority mainly concentrated on the Demand of Ward No. 8, 9, 12, 13, 15, 16, 17, 18, 19, 20, 21, 22, 23,24, 25, 26, 27, 28, 29, 30, 31, 32,33, 34, 35 out of its 35 Nos of wards. In this scheme South Dum Dum Municipal area has been divided in 9 water Zones. The projected population figure of the above mentioned wards for the town is 255181 in the year 2020 (Base Year), 310440 in year 2035 (Intermediate Year) and 371240 in year 2050 (Ultimate Year) and will be catered from the WTP which will be constructed in this phase of 15 MGD capacity. Forecasted demand for this scheme for the year 2020 is 54.40 MLD, for the year 2035 is 65.97 MLD and for the year 2050 is 78.69 MLD. It is required to be mention here that wards no. 18,19,20,29,30,31,32,33,34,35 of South Dumdum Municipality are being supplied clear water from Palta Water Treatment Plant, KMC.

In this scheme water needs of municipality at least up to the year 2035. This will have a good impact on the public health scenario in terms of reducing the incidence of various water borne diseases. Pipe lines will be capable to meet the demand of the year 2050 population of the municipality in usual growth pattern. Thus, the scheme shall cater to the water demand for the entire municipality comprising of 35 wards with population. Details of the scheme have been given in Table 2.

Table 2: Salient Features at a Glance

<table>
<tr><th>Description</th><th>Details</th></tr>
<tr><td>Name of the Scheme</td><td>Augmentation of Water Supply Scheme of South Dumdum Municipality</td></tr>
<tr><td>Location</td><td>South Dumdum Municipality, North 24 Parganas District, West Bengal.</td></tr>
<tr><td>Zone Wise Ward Covered</td><td><table>
<tr><td>Zone Number</td><td>Ward Number</td></tr>
<tr><td>Zone- 1</td><td>Ward- 21,22,9</td></tr>
<tr><td>Zone- 2</td><td>Ward- 15,16,18</td></tr>
<tr><td>Zone- 3</td><td>Ward- 19,17,20(50%)</td></tr>
<tr><td>Zone- 4</td><td>Ward- 31,32,33(50%)</td></tr>
<tr><td>Zone- 5</td><td>Ward- 35,34,33(50%)</td></tr>
<tr><td>Zone- 6</td><td>Ward- 29,30,20(50%)</td></tr>
<tr><td>Zone- 7</td><td>Ward- 25,24,23,8</td></tr>
<tr><td>Zone- 8</td><td>Ward- 27,28,26</td></tr>
<tr><td>Zone- 9</td><td>Ward- 12,13</td></tr>
</table></td></tr>
<tr><td>Source of Raw Water</td><td>River Hooghly</td></tr>
<tr><td>Intake Structure</td><td>Construction of Intake Structure with Jetty Mounted Pump House for 15 MGD Capacities WTP at Mallickbari.</td></tr>
</table>

Description	Details
Raw water Pumping main	Pipe Material: DI(Ductile Iron) Pipe /Mild Steel Pipe; Length of Pipe:15100 m; Pipe diameter: 1000 mm, thickness 12.5 mm.
Water Treatment Plant	Capacity: 15 MGD at Bangur Avenue Lake Town.
Clear Water Reservoir (CWR)	Capacity 1.5 MGD
Clear Water Pumping main	Pipe Material: DI-K9 Length of Pipe: 7248 m, Diameter: 300 mm to 900 mm
Elevated Storage Reservoirs (ESR)	5 Nos; i) Capacity 2034 Cum at Ward No.31, ii) Capacity 1984 Cum at Ward No.35, iii) Capacity 1270 Cum at Ward No.18, iv) Capacity 1941 Cum at Ward No.19, v) Capacity 1941 Cum at Ward No.30,
Clear Water Distribution Pipe	Pipe Material: DI-K7 ;Total Length: 40 km ; Diameter: 100 mm to 450 mm.

Schematic diagrams of various stages of the project have been shown below the Figure 8.

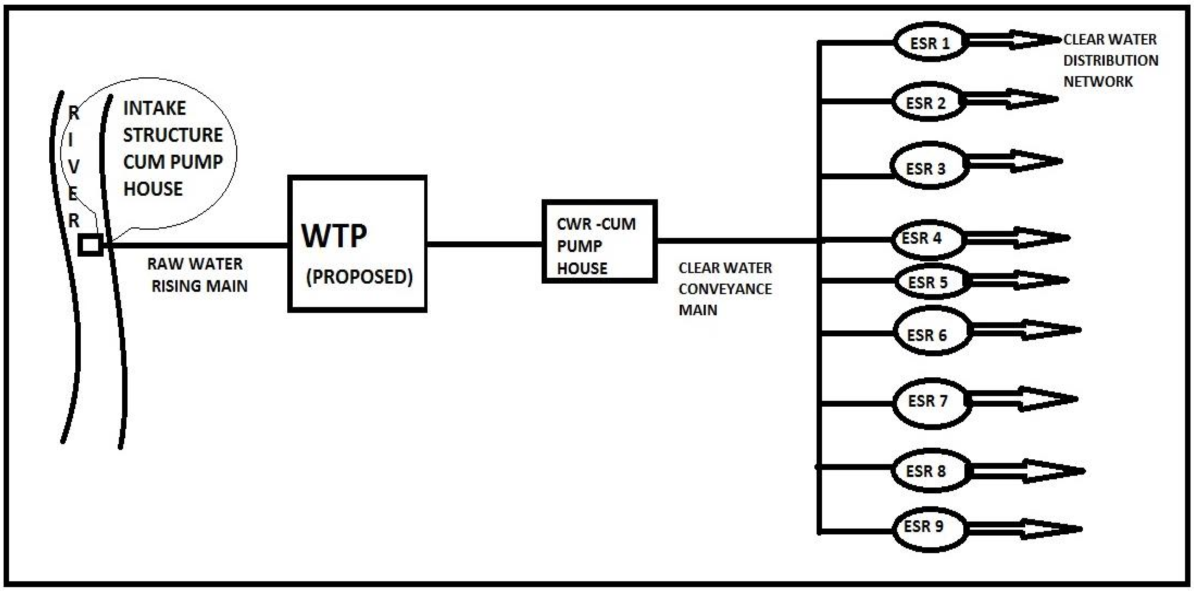

Figure 8: Schematic diagram of the project

A plan showing the proposed command area of the 9 water zones in South Dumdum Municipality as shown below in Figure 9.

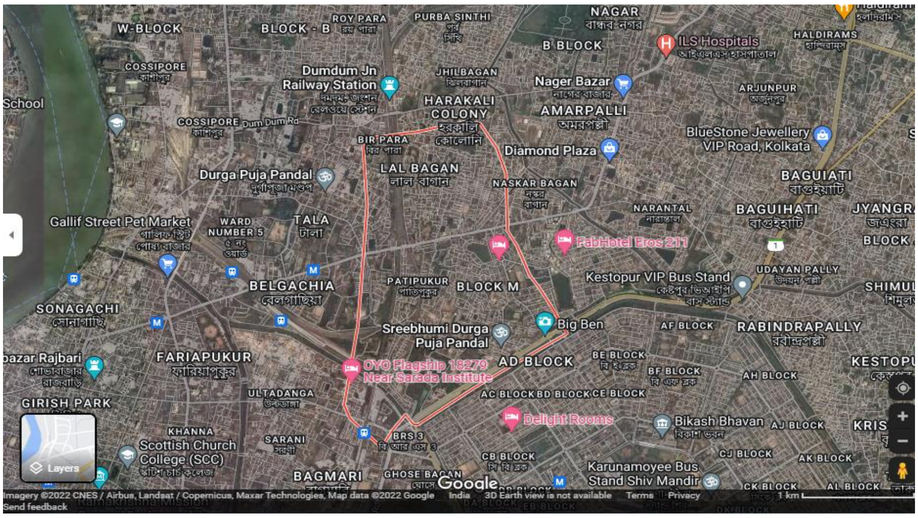

Figure 9: Surface Water Supply Augmentation Scheme areas

3.3 Surface Water Supply Augmentation Scheme

Surface Water Supply Augmentation Scheme for South Dumdum Municipality, the raw water is sourced from river Hooghly, at Mallickbari, near Kamarhati Baranagar. An Intake structure of capacity 15 MGD constructed at Mallickbari. The 15.1 km raw water transmission main (1000 mm diameter MS/DI pipe) is laid for conveyance of raw water from the Ganges to the location of Water Treatment Plant (WTP) of Ccapacity 15 MGD at Bangur Avenue, Laketown. A Clear Water Reservoir of capacity 1.5 MGD has been installed at WTP. The clear water pumping main DI-K9 pipe of length 7248 m is laying from WTP to the 05 numbers ESR located in the different water zone in South Dumdum Municipality. The Clear Water Distribution Pipe (DI-K7) of length of 40 Km has been laying for deliver clear water in different water zones in South Dumdum Municipality. A plan showing the proposed WTP and ESR in South Dumdum Municipality as shown below in Figure 10.

Figure 10: Intake Jetty, WTP and ESR Location at South Dumdum Municipality

Location co-ordinate and picture of Water Treatment Plant (WTP) of Ccapacity 15 MGD at Bangur Avenue, Laketown in South Dumdum Municipality as shown in Figure 11.

Figure 11: WTP (15 MGD Capacity) at Laketown, South Dumdum Municipality

3.4 Source sustainability and intake location

The river Hooghly, which is a perennial river, is the source of raw water. The Intake structure of capacity 15 MGD is constructed at Mallickbari near Kamarhati Baranagar for delivering raw water from Hooghly River to Water Treatment Plant at Bangur Avenue, Laketown as shown in Figure 12.

Figure 12: Intake Structure Cum Pump House located at Mallickbari on Hooghly River

CHAPTER 4

4.1 Pipe Network Analysis Theory

One of the main problems faced by the engineers in design of large water distributing system is to calculate the flow through pipelines in the network and the corresponding pressure heads at intake and outlet points. This enables one to select the system elements, like pipe sizes, pump specifications etc.

The engineer is often engaged to design the original distributing network or to recommend an improvement or rehabilitation to the existing network system. In this case, the network in the system already designed and we want to analyse them at different operating conditions (i.e. at different demands) in order to determine the capability of the networks to deliver the required pressures and flows. The general procedures of analysing any pipe network are as follows:

a) The layout of the pipe network should be determined.

b) The characteristic of all network components should be determined from the source to the consumers.

c) The two basic hydraulic equations are applied:

4.1.1 Flow Continuity Equation

The sum of inflows entering the junction equals to the sum of outflows leaving the junctions. When the junction J is considered as shown in Fig. 4.5 where five pipes are interconnected and there is an outflow demand. The directions of flows Q1, Q2 and Q5 are entering the junction J1, while the directions of flows Q3, Q4 and Q6 are leaving the junction J1. By common sense, the total flow entering the junction should be equal to the total flow leaving the junction because it neither creates nor stores any flow inside, and thus, Figure 13 represents it.

$$Q1+Q2+Q5=Q3+Q4+Q6 \quad \ldots(4.1)$$

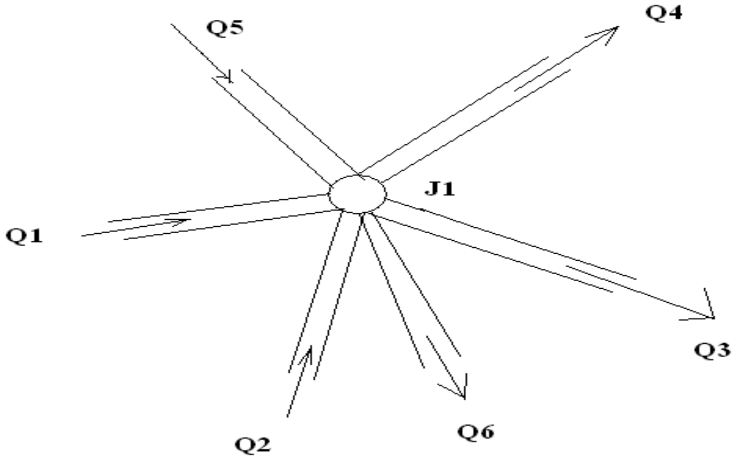

Figure 13: Flow across a junction

4.1.2 Energy equation

The difference in hydraulic head between any two junctions within any pipe system equals to the net head losses between junctions. The notations are described in Figure

$$(P_a + Z_a) - (P_b + Z_b) = h_l - h_p \qquad ...(4.2)$$

The piezometric head ($\frac{p}{\gamma} + z$) of the water, the pressure head adder to water by a pump (h_l), the kinetic head of water $\frac{v^2}{2g}$ and the total head losses (h_p) lost from water by friction and fitting minor losses when water moves inside pipes from the point of higher head (hydraulic pressure) to the point of lower head with mean velocity (V) and flow rate (Q). To relate all above pressure head together, let's take the pipe in Figure.14.

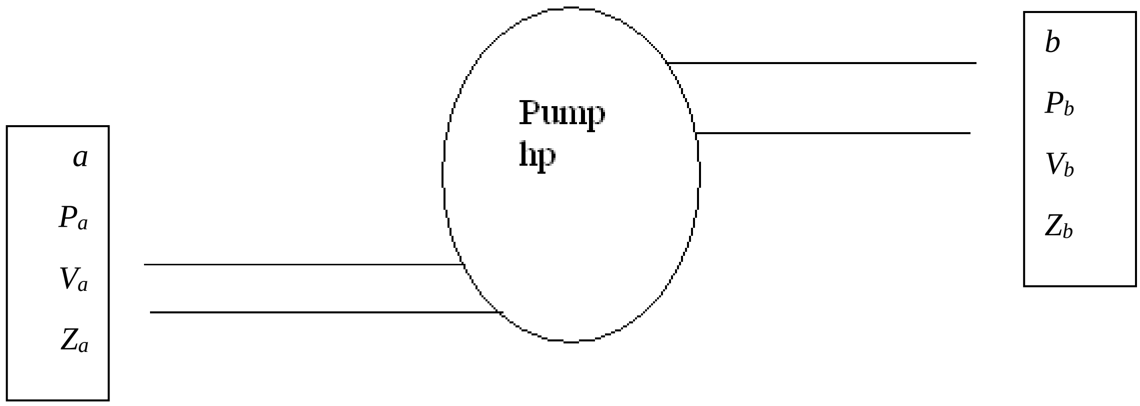

Figure 14: Flow through a pump

Assume that the flow direction in the pipe is from junction (a) to (b) and its value is Q, the pressure and elevation head and the mean velocity at (a) are P_a , Z_a and V_a respectively, the total pressure, elevation head and mean velocity at (b) are P_b, Z_b and V_b respectively, the total friction and fitting head losses from (a) to (b) are h_l, the pump's head is h_p, all heads in are in m. for this system:

$$P_a + Z_a + \frac{v_a^2}{2g} + h_p = P_b + Z_b + \frac{v_b^2}{2g} + h_l \qquad ... (4.3)$$

The combination of the flow continuity equation and energy equation results in a system of non–linear equations that are solved by trial and error/hand computations or by using computer software that solve such equations.

4.2 Hydraulic Head Loss:

The hydraulic head loss by water flowing in a pipe due to friction with the pipe walls can be computed using one of three different formulas:

- Hazen-Williams formula
- Darcy-Weisbach formula
- Chezy-Manning formula

The Hazen-Williams formula is the most commonly used headloss formula in the US. It cannot be used for liquids other than water and was originally developed for turbulent flow only. The Darcy-Weisbach formula is the most theoretically correct. It applies over all flow regimes and to all liquids. The Chezy-Manning formula is more commonly used for open channel flow. Each formula uses the following equation to compute headloss between the start and end node of the pipe:

$$h_L = Aq^B \qquad \ldots(4.4)$$

where h_L = head loss (Length), q = flow rate (Volume/Time), A = resistance coefficient, and B = flow exponent. Table 4.1 lists expressions for the resistance coefficient and values for the flow exponent for each of the formulas. Each formula uses a different pipe roughness coefficient that must be determined empirically. Table 4.2 lists general ranges of these coefficients for different types of new pipe materials. Be aware that a pipe's roughness coefficient can change considerably with age.

With the Darcy-Weisbach formula uses different methods to compute the friction factor f depending on the flow regime:

- The Hagen–Poiseuille formula is used for laminar flow (Re < 2,000).
- The Swamee and Jain approximation to the Colebrook-White equation is used for fully turbulent flow (Re > 4,000).
- A cubic interpolation from the Moody Diagram is used for transitional flow (2,000 < Re < 4,000). Table 3 has shown the pipe head loss formulas for full flow.

Table 3: Pipe Headloss Formulas for Full Flow (for head loss in m and flow rate in m^3/s)

Formula	*Resistance Coefficient (A)*	*Flow Exponent (B)*
Hazen-Williams	$469.855C^{-1.852}\ d^{-4.871}\ L^{1.852}$	1.852
Darcy-Weisbach	$2.9197\ f(\boldsymbol{v}, d, q)\ d^{-5}\ L^{2}$	2
Chezy-Manning	$799.1\ n^2\ d^{-5.33}\ L^{2}$	2

Notes: C = Hazen-Williams roughness coefficient

ν = Darcy-Weisbach roughness coefficient (m)

f = friction factor (dependent on e, d, and q)

n = Manning roughness coefficient

d = pipe diameter (m)

L = pipe length (m)

q = flow rate (m^3/s)

Roughness coefficients for new pipe are mentioned in Table 4.

Table 4: Roughness Coefficients for New Pipe

Material	*Hazen-Williams C (unit less)*	*Darcy-Weisbach ν ($m \times 10^{-3}$)*	*Manning's n (unit less)*
Cast Iron	130 – 140	0.259	0.012 - 0.015
Concrete or Concrete Lined	120 – 140	0.3-3.0	0.012 - 0.017
Galvanized Iron	120	0.152	0.015 - 0.017
Plastic	140 – 150	0.002	0.011 - 0.015
Steel	140 – 150	0.046	0.015 - 0.017
Vitrified Clay	110		0.013 - 0.015

Pipes can be set open or closed at preset times or when specific conditions exist, such as when tank levels fall below or above certain set points, or when nodal pressures fall below or above certain values.

4.2.1 Minor Losses

Minor head losses (also called local losses) are caused by the added turbulence that occurs at bends and fittings. The importance of including such losses depends on the layout of the network and the degree of accuracy required. They can be accounted for by assigning the pipe a minor loss coefficient. The minor headloss becomes the product of this coefficient and the velocity head of the pipe, i.e.

$$H_L = K \frac{v^2}{2g} \qquad ...(4.5)$$

where K = minor loss coefficient, v = flow velocity (Length/Time), and g = acceleration of gravity (Length/Time^2). Table 5 provides minor loss coefficients for several types of fittings.

Table 5: Minor Loss Coefficients for Selected Fittings

FITTING	*LOSS COEFFICIENT*
Globe valve, fully open	10
Angle valve, fully open	5
Swing check valve, fully open	2.5
Gate valve, fully open	0.2
Short-radius elbow	0.9
Medium-radius elbow	0.8
Medium-radius elbow	0.6
45 degree elbow	0.4
Closed return bend	2.2
Standard tee - flow through run	0.6
Standard tee - flow through branch	1.8
Square entrance	0.5
Exit	1

4.3 Water distribution system

The history of water distribution is very ancient in development. Indeed, urban WDSs date back to the Bronze Age (circa 3200–1100 B.C.), with 'several astonishing examples' from the mid-third millennium B.C. (Mays *et al*., 2012). These include, for example, a system of hundreds of wells supplying water to domestic demands, and private and public baths (Mays *et al*., 2012). Crouch (1993), who documented water management in ancient Greece, revealed that the very first piped water supplies including pressure pipes had been known as early as the second millennium B.C. It is documented that ancient Minoan and Greek civilizations had urban water reticulation, sewerage and drainage systems, with wells, cisterns, tanks, reservoirs, dams, channel sand water pipes made of terracotta (clay) and lead (Crouch, 1993; Angelakis *et al.*, 2005; Mays *et al*., 2012)

More-over, the ancient Greeks constructed 'long-distance water supply lines with tunnels and bridges' referred to as aqueducts, which are dated back to the eighth to sixth century B.C. (Crouch 1993). Greek technologies were subsequently inherited by the Romans (circa 100 B.C. to 500 A.D.), who developed them further and implemented them at an enlarged scale (Mays *et al*., 2007; Angelakis *et al*., 2012). In particular, Roman aqueducts, which carried water from a source to the Roman cities, could extend over more than 100 km in length (Viollet, 2000; Haut and Vivier, 2012). They could incorporate an inverted siphon, which was pressurized pipeline carrying water across a valley (Haut and Vivier, 2012). The Romans also used wooden pipes as an alternative to the terracotta pipes, prevalent in Northern Europe (Hodge, 2002). The durability of the Roman constructions is remarkable, with some of them having operated up to modern times (Mays *et al*., 2012). Furthermore, it is recognized that the Romans had an advanced knowledge of water supply engineering (Hodge, 2002; Haut and Vivier, 2012).After the fall of the Roman Empire at circa fifth century A.D., it is unclear if the Roman knowledge about water management survived the collapse of these civilizations (Crouch, 1993) and the following Middle Age period referred to as Dark Ages (5th–15th centuryes A.D.).Even though it is agreed that Roman achievements 'were not totally forgotten' (Angelakis *et al*., 2012), it is admitted that there was a decline in the quality of water management practices during those several centuries (Burian and Edwards, 2002; Angelakis *et al*., 2012). This decline with very poor sanitary conditions including polluted water in sources and waste in the streets is reported, especially in Europe (Gray, 1940). Water supply was provided to a central delivery point, from where it was brought to the homes by people themselves or servants, or else water carriers who made a business of selling and delivering water. It was not until after the Renaissance (14th–17th centuries) when water management practices began to evolve once more (Walski, 2006). Possibly the first major pipe line was a 25 km line from Marly-On-Seine to the Palace of Versailles in France, which was completed in 1664 (Walski, 2006). By the mid-1700s, London had more than 50 km of water mains constructed of wood, cast-iron and lead pipes (Sanks, 2005; Walski, 2006). In the United States, the first piped water supply was in Boston in 1652, when water was brought from springs and wells to near what is now the restored Quincy Market area (Mays, 2000).

4.4 Devices for raising water and water pumps

It is believed that ancient devices for raising water originate from the Assyrians, Babylonians and other ancient nations. The early devices used to raise water from wells are a pulley bucket, windlass and their various modifications to transfer motion. While they may seem basic to mention here, they were of a great significance. For example, the theft of a 'fatal bucket' from a public well in Bologna, by Modena soldiers, is believed to have resulted in a war between those two nations (the incident occurred at the beginning of the 11th century A.D.) In the first century B.C., Vitruvius compiled the existing knowledge of hydraulics (Rouse & Ince 1963) and described the principal hydraulics mechanisms to raise water invented since antiquity to date.

The tympanum and the noria (Egyptian 'wheel of fortune') both have the form of a wheel partially submerged into water, in which water is elevated by gutters or vessels, respectively, using its rotary motion. Figure 15 represents the same.

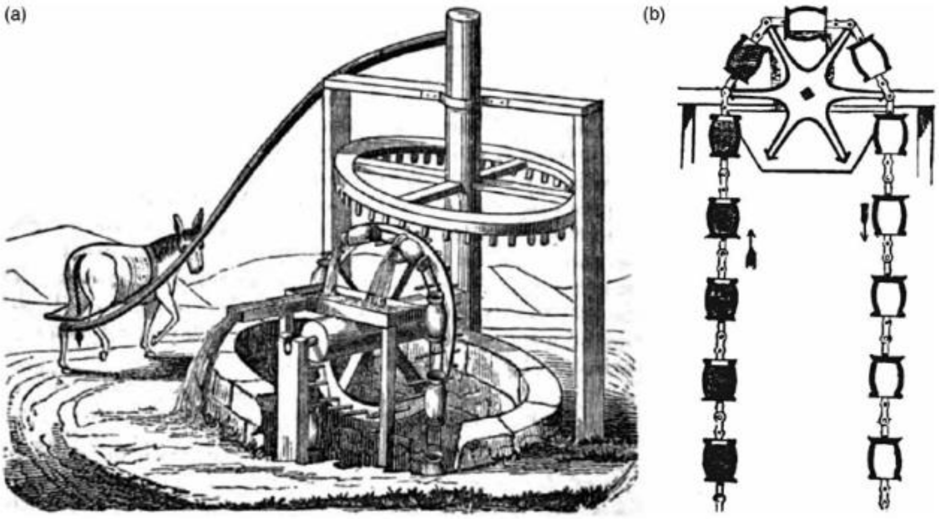

Figure 15: A chain of pots in Spain

4.5 Network analysis

Network analysis, which is invaluable for the water professional involved with design, operation, maintenance and optimization of Water Distribution System (WDS), consists of two distinct components, namely, analysis of (i) hydraulic and (ii) water quality behaviour of flow through a WDS. This section focuses on hydraulic analysis only. Hydraulic analysis calculates flows, head losses and pressures in a specified pipe network by simultaneously solving a set of equations (further in the text referred to as 'network equations'). These network equations arise from the conservation of mass of flow and energy as (i) the sum of flows toward any junction is zero, (ii) the sum of head losses in a closed loop is zero and (iii) the head loss in a pipe is directly proportional to the power of the flow. Due to the size of WDSs and the associated large number of non-linear network equations, hydraulic analysis is a complex task. Historically, hydraulic analysis methods range from graphical methods, through the use of physical analogies, to mathematical models.

Prior digital computers

Hydraulic analysis of WDSs involves tedious calculations applying a combination of simplifications, engineering experience and practice, and conservatism (Walski *et al.*, 2006). The first method reported was the graphical method introduced by Spiess (1887) and (Aldrich, 1937), followed by the more popular graphical method of Freeman (1892). The former method presented solutions for basic branched and looped systems, whereas the latter method investigated simple and more complex WDSs with fire demands. Free-man's graphical method was later expanded by Aldrich (1937) using the Hazen–Williams formula. Other well-known methods include the electric network analyzer method (Camp and Hazen, 1934) based on the analogy between the laws governing hydraulic flow and electric current in net-works (Ramalingam *et al.*, 2004) and the Hardy–Cross method (Cross 1936), which was the first method to solve hydraulic analysis mathematically. The graphical and electric analyzer methods have not been widely used due to time and equipment requirements, respectively (Aldrich 1937), the Hardy–Cross method became popular with numerous sub-sequent publications describing its application to various systems (Ramalingam *et al.*, 2004)

After digital computers

Several iterative methods have been applied to hydraulic analysis of a WDS. The first method adapted to the digital computer was the Hardy–Cross method (Cross, 1936) in 1957, with application to the WDS of the city of Palo Alto, California (Ormsbee, 2006). Because this method could take a long time to converge to a solution or could fail to converge at all, other methods were proposed (Ormsbee, 2006). These methods included the Newton–Raphson method (simultaneous node method) (Martin and Peters, 1963), simultaneous loop method (Epp and Fowler, 1970), the linear theory approach (simultaneous pipe method) (Wood and Charles, 1972; Tavallaee 1974) and the gradient method (simultaneous network method) (Todini and Pilati, 1988). The Newton–Raphson method may converge more quickly for small networks, but very slowly for large networks compared to the linear theory approach (Mays, 1989). The simultaneous loop method is the improved Newton–Raphson method with the benefit of significantly improved convergence characteristics of the original algorithm (Ormsbee, 2006). The linear theory approach has the capacity to analyse all network components and is more flexible regarding the representation of pumps (Mays 1989). The gradient method was adopted in the development of the hydraulic simulation package EPANET (Rossman, 1993). The next significant step in hydraulic analysis of WDSs was development of hydraulic simulation packages, accessible for wide use by water professionals. The first such package titled KYPIPE, which uses the simultaneous loop method to solve the network equations, was developed by the University of Kentucky in 1980 (Wood, 1980).

Following the advent of window based packages introduced by Microsoft and Apple Macintosh software, developers started developing software like FOTRAN, BASIC, COBOL, C++, MS Excel etc. This software has logic circuits and capacities to carry out complex calculations in short period (Adeniran, 2007). Engineers and scientists soon found that most of the manual iterative works they carried out with Hardy Cross Method can easily be performed using these platforms. Recently, researchers focus on stochastic optimization methods that deal with a set of points simultaneously in its search for the global optimum. Savic and Walters (1997) combined gradient algorithm with EPANET network solver. Many modeling programs are now available for commercial and educational use. Recently, several computer programs running on personal computers, such as EPANET, UNWB-LOOP, WADISO, InfoWorks Water and WaterGEMS have been created and made available.

4.6 About the software EPANET

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated.

4.6.1 Physical component of network

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. Figure 16 illustrates how these objects can be connected to one another to form a network.

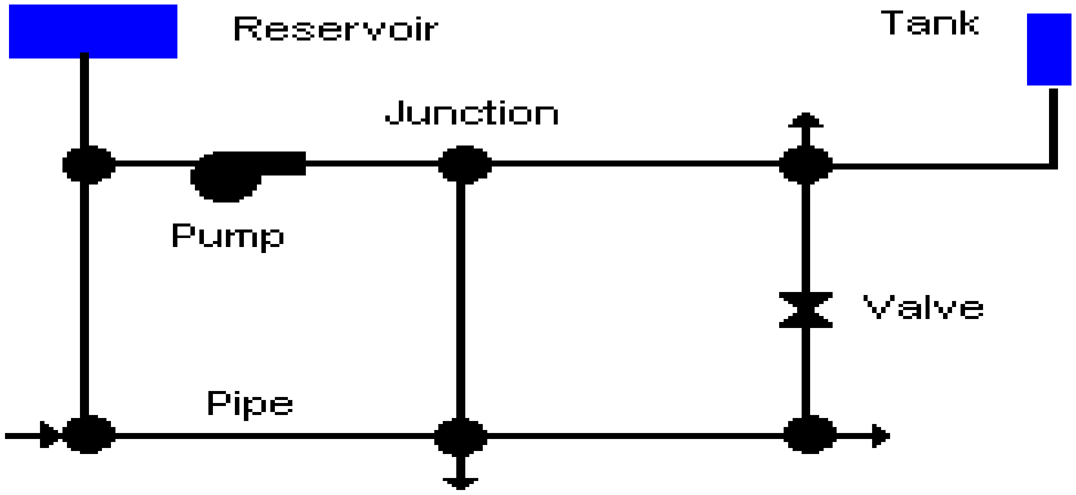

Figure 16: Physical component of network

4.6.2 Junctions

Junctions are points in the network where links join together and where water enters or leaves the network. The basic input data required for junctions are:

i. Elevation above some reference (usually mean sea level)
ii. Water demand (rate of withdrawal from the network)
iii. Initial water quality.

The output results computed for junctions at all time periods of a simulation are:

i. Hydraulic head (internal energy per unit weight of fluid)
ii. Pressure
iii. Water quality

Junctions can also:

- Have their demand vary with time
- Have multiple categories of demands assigned to them
- Have negative demands indicating that water is entering the network
- Be water quality sources where constituents enter the network
- Contain emitters (or sprinklers) which make the outflow rate depend on the pressure.

4.6.3 Reservoirs

Reservoirs are nodes that represent an infinite external source or sink of water to the network. They are used to model such things as lakes, rivers, groundwater aquifers, and tie-ins to other systems. Reservoirs can also serve as water quality source points. The primary input properties for a reservoir are its hydraulic head (equal to the water surface elevation if the reservoir is not under pressure) and its initial quality for water quality analysis. Because a reservoir is a boundary point to a network, its head and water quality cannot be affected by what happens within the network. Therefore, it has no computed output properties. However, its head can be made to vary with time by assigning a time pattern to it (see Time Patterns below).

4.6.4 Tanks

Tanks are nodes with storage capacity, where the volume of stored water can vary with time during a simulation. The primary input properties for tanks are:

- Bottom elevation (where water level is zero)
- Diameter (or shape if non-cylindrical)
- Initial, minimum and maximum water levels
- Initial water quality

The principal outputs computed over time are:

- Hydraulic head (water surface elevation)
- Water quality

Tanks are required to operate within their minimum and maximum levels. EPANET stops outflow if a tank is at its minimum level and stops inflow if it is at its maximum level. Tanks can also serve as water quality source points.

4.6.5 Emitters

Emitters are devices associated with junctions that model the flow through a nozzle or orifice that discharges to the atmosphere. The flow rate through the emitter varies as a function of the pressure available at the node:

$$q = C\, p^{\gamma} \qquad \ldots(4.6)$$

where q = flow rate, p = pressure, C = discharge coefficient, and γ= pressure exponent. Emitters are used to model flow through sprinkler systems and irrigation networks. They can also be used to simulate leakage in a pipe connected to the junction (if a discharge coefficient and pressure exponent for the leaking crack or joint can be estimated) or compute a fire flow at the junction (the flow available at some minimum residual pressure EPANET treats emitters as a property of a junction and not as a separate network component.

4.6.6 Pipes

Pipes are links that convey water from one point in the network to another. EPANET assumes that all pipes are full at all times. Flow direction is from the end at higher hydraulic head (internal energy per weight of water) to that at lower head. The principal hydraulic input parameters for pipes are:

- Start and end nodes
- Diameter
- Length
- Roughness coefficient (for determining headloss)
- Status (open, closed, or contains a check valve).

The status parameter allows pipes to implicitly contain shutoff (gate) valves and check (non-return) valves (which allow flow in only one direction).

The water quality inputs for pipes consist of:

- Bulk reaction coefficient
- Wall reaction coefficient.

Computed outputs for pipes include:

- Flow rate
- Velocity
- Head loss
- Darcy-Weisbach friction factor
- Average reaction rate (over the pipe length)
- Average water quality (over the pipe length).

4.6.7 Pumps

Pumps are links that impart energy to a fluid thereby raising its hydraulic head. The principal input parameters for a pump are its start and end nodes and its pump curve (the combination of heads and flows that the pump can produce). In lieu of a pump curve, the pump could be represented as a constant energy device, one that supplies a constant amount of energy (horsepower or kilowatts) to the fluid for all combinations of flow and head. The principal output parameters are flow and head gain. Flow through a pump is unidirectional and EPANET will not allow a pump to operate outside the range of its pump curve. Variable speed pumps can also be considered by specifying that their speed setting be changed under these same types of conditions. By definition, the original pump curve supplied to the program has a relative speed setting of 1. If the pump speed doubles, then the relative setting would be 2; if run at half speed, the relative setting is 0.5 and so on. Changing the pump speed shifts the position and shape of the pump curve (see the section on Pump Curves below). As with pipes, pumps can be turned on and off at present times or when certain conditions exist in the network. A pump's operation can also be described by assigning it a time pattern of relative speed settings. EPANET can also compute the energy consumption and cost of a pump. Each pump can be assigned an efficiency curve and schedule of energy prices. If these are not supplied, then a set of global energy options will be used. Flow through a pump is unidirectional. If system conditions require more head than the pump can produce, EPANET shuts the pump off. If more than the maximum flow is required, EPANET extrapolates the pump curve to the required flow, even if this produces a negative head. In both cases a warning message will be issued.

4.6.8 Valves

Valves are links that limit the pressure or flow at a specific point in the network. Their principal input parameters include:

- Start and end nodes
- Diameter
- Setting
- Status.

The computed outputs for a valve are flow rate and headloss. The different types of valves included in EPANET are:

- Pressure Reducing Valve (PRV)
- Pressure Sustaining Valve (PSV)
- Pressure Breaker Valve (PBV)
- Flow Control Valve (FCV)
- Throttle Control Valve (TCV)
- General Purpose Valve (GPV).

PRVs limit the pressure at a point in the pipe network. EPANET computes in which of three different states a PRV can be in:

- Partially opened (i.e., active) to achieve its pressure setting on its downstream side when the upstream pressure is above the setting
- Fully open if the upstream pressure is below the setting
- Closed if the pressure on the downstream side exceeds that on the upstream side (i.e., reverse flow is not allowed). PSVs maintain a set pressure at a specific point in the pipe network. EPANET computes in which of three different states a PSV can be in:
- Partially opened (i.e., active) to maintain its pressure setting on its upstream side when the downstream pressure is below this value
- Fully open if the downstream pressure is above the setting
- Closed if the pressure on the downstream side exceeds that on the upstream side (i.e., reverse flow is not allowed).

PBVs force a specified pressure loss to occur across the valve. Flow through the valve can be in either direction. PBV's are not true physical devices but can be used to model situations where a particular pressure drop is known to exist.

FCVs limit the flow to a specified amount. The program produces a warning message if this flow cannot be maintained without having to add additional head at the valve (i.e., the flow cannot be maintained even with the valve fully open).

TCVs simulate a partially closed valve by adjusting the minor head loss coefficient of the valve. A relationship between the degree to which a valve is closed and the resulting head loss coefficient is usually available from the valve manufacturer.

GPVs are used to represent a link where the user supplies a special flow - head loss relationship instead of following one of the standard hydraulic formulas. They can be used to model turbines, well draw-down or reduced-flow backflow prevention valves.

Shutoff (gate) valves and check (non-return) valves, which completely open or close pipes, are not considered as separate valve links but are instead included as a property of the pipe in which they are placed.

Each type of valve has a different type of setting parameter that describes its operating point (pressure for PRVs, PSVs, and PBVs; flow for FCVs; loss coefficient for TCVs, and head loss curve for GPVs).

Valves can have their control status overridden by specifying they be either completely open or completely closed. A valve's status and its setting can be changed during the simulation by using control statements. Because of the ways in which valves are modelled the following rules apply when adding valves to a network.

- A PRV, PSV or FCV cannot be directly connected to a reservoir or tank (use a length of pipe to separate the two)
- PRVs cannot share the same downstream node or be linked in series
- Two PSVs cannot share the same upstream node or be linked in series.
- A PSV cannot be connected to the downstream node of a PRV.

4.6.9 Non-Physical Components

In addition to physical components, EPANET employs three types of informational objects – curves, patterns, and controls - that describe the behaviour and operational aspects of a distribution system.

4.6.10 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 the relationship between the head and flow rate that a pump can deliver at its nominal speed setting. Head is the head gain imparted to the water by the pump and is plotted on the vertical (Y) axis of the curve in feet (meters).

Flow rate is plotted on the horizontal (X) axis in flow units. A valid pump curve must have decreasing head with increasing flow.

EPANET will use a different shape of pump curve depending on the number of points supplied (see Figure 17)

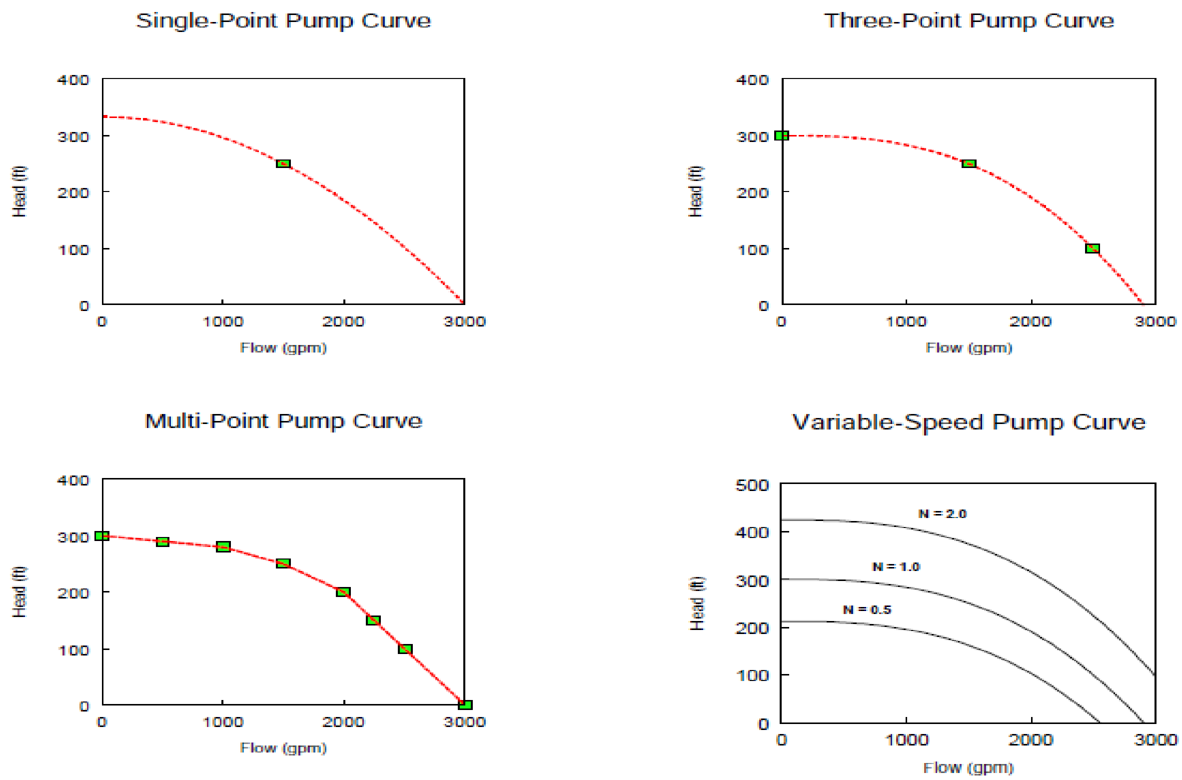

Figure 17: Various types of pump curves

Single-Point Curve - A single-point pump curve is defined by a single head-flow combination that represents a pump's desired operating point. EPANET adds two more points to the curve by assuming a shutoff head at zero flow equal to 133% of the design head and a maximum flow at zero head equal to twice the design flow. It then treats the curve as a three-point curve.

Three-Point Curve - A three-point pump curve is defined by three operating points: a Low Flow point (flow and head at low or zero flow condition), a Design Flow point flow and head at desired operating point), and a Maximum Flow point (flow and head at maximum flow). EPANET tries to fit a continuous function of the form

$$H_g = A - Bq^c \qquad \dots \quad (4.7)$$

through the three points to define the entire pump curve. In this function, H_g = head gain, q = flow rate, and A, B, and C are constants.

Multi-Point Curve – A multi-point pump curve is defined by providing either a pair of head-flow points or four or more such points. EPANET creates a complete curve by connecting the points with straight-line segments.

An Efficiency Curve determines pump efficiency (Y in percent) as a function of pump flow rate (X in flow units). An example efficiency curve is shown in Figure 18. Efficiency should represent wire-to-water efficiency that takes into account mechanical losses in the pump itself as well as electrical losses in the pumps motor. The curve is used only for energy calculations. If not supplied for a specific pump, then fixed global pump efficiency will be used.

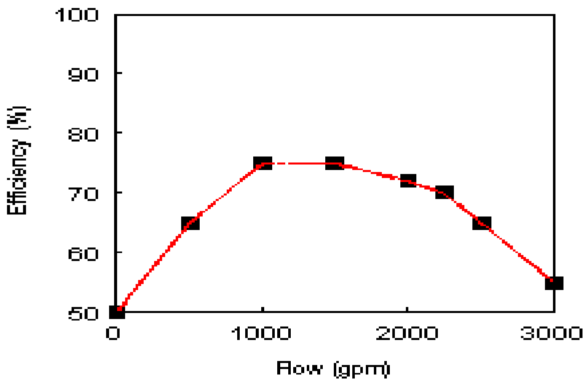

Figure 18: Pump Efficiency Curve

Volume Curve

A Volume Curve determines how storage tank volume (Y in cubic feet or cubic meters) varies as a function of water level (X in feet or meters). It is used when it is necessary to accurately represent tanks whose cross-sectional area varies with height. The lower and upper water levels supplied for the curve must contain the lower and upper levels between which the tank operates. An example of a tank volume curve is given below in Figure 19.

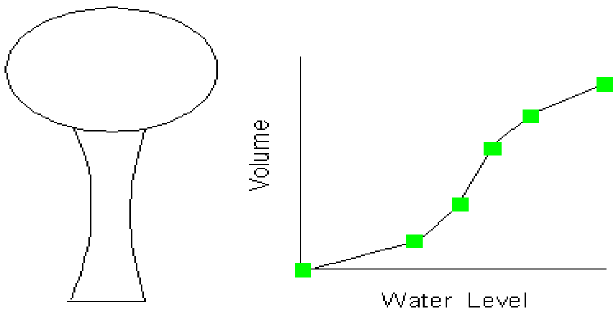

Figure 19: Tank Volume Curve

Head loss Curve

A Head Loss Curve is used to describe the head loss (Y in feet or meters) through a General Purpose Valve (GPV) as a function of flow rate (X in flow units). It provides the capability to model devices and situations with unique head loss-flow relationships, such as reduced flow - backflow prevention valves, turbines, and well draw-down behaviour

4.6.11 Hydraulic Simulation Model

EPANET's hydraulic simulation model computes junction heads and link flows for a fixed set of reservoir levels, tank levels, and water demands over a succession of points in time. From one-time step to the next reservoir levels and junction demands are updated according to their prescribed time patterns while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving simultaneously the conservation of flow equation for each junction and the headloss relationship across each link in the network. This process, known as "hydraulically balancing" the network, requires using an iterative technique to solve the nonlinear equations involved. EPANET employs the "Gradient Algorithm" for this purpose.

The hydraulic time step used for extended period simulation (EPS) can be set by the user. A typical value is 1 hour. Shorter time steps than normal will occur automatically whenever one of the following events occurs:

- The next output reporting time period occurs
- The next time pattern period occurs
- A tank becomes empty or full
- A simple control or rule-based control is activated.

4.7 Overview of WaterGEMS software

4.7.1 WaterGEMS

WaterGEMS stands for Water Geographic Engineering Modelling System that clarifies the complex science of hydraulic analysis and puts the power to perform this critical work in our hands.

a) Develop cost-effective surge control strategies.
b) Prevent costly infrastructure damage.
c) Trim construction, operation and maintenance budgets.
d) Model any surge protection device.
e) Minimize wear and tear on pipes.
f) Ensure the longevity of our water system.
g) Prepare for power failures and minimize service interruptions.

WaterGEMS is the industry's hydraulic modelling solution according to a recent industry study.

a) Preventing leakages.
b) Prevent catastrophic failures.
c) Design surge control measures.
d) Reduce operation and maintenance costs.
e) Eliminate costly over design.
f) Minimize service interruptions.

4.7.2 Creating or Importing a Steady-State Model:

We can create an initial steady-state model of our system within WaterGEMS directly, using the advanced WaterGEMS Modeller interface, or import one from an existing steady-state model created using other software.

4.7.3 Creating a Model:

WaterGEMS is an extremely efficient tool for laying out a water-transmission pipeline or even an entire distribution network. It is easy to prepare a schematic model and let WaterGEMS take care of the link-node connectivity and element labels, which are assigned automatically. For a schematic model only pipe lengths must be entered manually to complete the layout. We may need to input additional data for some hydraulic elements prior to a run.

Modelling capabilities

Modelling capabilities include:

a) Steady-State/Extended Period Simulation
b) Global Demand and Roughness Adjustments
c) Check Data/Validate
d) Calculate Network
e) Flow Emitters
f) Parallel VSPs
g) Calculation Options
h) Patterns
i) Controls
j) Active Topology

4.7.4 Steady-State/Extended Period Simulation

WaterGEMS Software gives the choice between performing a steady-state analysis of the system and performing an extended-period simulation over any time period.

Steady-State Simulation

Steady-state analyses determine the operating behaviour of the system at a specific point in time or under steady-state conditions (flow rates and hydraulic grades remain constant over time). This type of analysis can be useful for determining pressures and flow rates under minimum, average, peak, or short term effects on the system due to fire flows.

For this type of analysis, the network equations are determined and solved with tanks being treated as fixed grade boundaries. The results that are obtained from this type of analysis are instantaneous values and may or may not be representative of the values of the system a few hours, or even a few minutes, later in time.

Extended Period Simulation (EPS)

When the variation of the system attributes over time is important, an extended period simulation is appropriate. This type of analysis allows we to model tanks filling and draining, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies formulated by the WaterGEMS. While a steady-state model may tell whether the system has the capability to meet a certain average demand, an extended period simulation indicates whether the system has the ability to provide acceptable levels of service over a period of minutes, hours, or days. Extended period simulations (EPSs) can also be used for energy consumption and cost studies, as well as water quality modelling. Data requirements for extended period simulations are greater than for steady-state runs. In addition to the information required by a steady-state model, we also need to determine water usage Patterns, more detailed tank information, and operational rules for pumps and valves.

4.7.5 Numerical model calibration and validation

As part of its expert witness and break-investigation service, GENIVAR has calibrated and validated WaterGEMS Edition's numerical simulations for different fluids and systems for clients in the civil (water and wastewater), mining (slurry), and hydropower sectors. Comparisons between computer models and validation data can be grouped into the following three categories:

a) Cases for which closed-form analytical solutions exist given certain assumptions. If the model can directly reproduce the solution, is considered valid for this case.

b) Laboratory experiments with flow and pressure data records. The model is calibrated using one set of data and, without changing parameter values, it is used to match a different set of results. If successful, it is considered valid for these cases.

c) Field tests on actual systems with flow and pressure data records. These comparisons require threshold and span calibration of all sensor groups, multiple simultaneous datum and time base checks and careful test planning and interpretation. Sound calibrations match multiple sensor records and reproduce both peak timing and secondary signals—all measured every second or fraction of a second.

It is extremely difficult to develop a theoretical model that accurately simulates every physical phenomenon that can occur in a hydraulic system. Therefore, every hydraulic model involves some approximations and simplifications of the real problem. For designers trying to specify safe surge-control systems, conservative results are sufficient.

The differences between computer model results and actual system measurements are caused by several factors, including the following difficulties:

a) Precise determination of the pressure-wave speed for the piping system is difficult, if not impossible. This is especially true for buried pipelines, whose wave speeds are influenced by bedding conditions and the compaction of the surrounding soil.

b) Precise modelling of dynamic system elements (such as valves, pumps, and are influenced by protection devices) is difficult because they are subject to deterioration with age and adjustments made during maintenance activities. Measurement equipment may also be inaccurate.

c) Unsteady friction coefficients and losses depend on fluid velocities and accelerations. These are difficult to predict and calibrate even in laboratory conditions.

4.7.6 Steady state friction method

In WaterGEMS an Initial Conditions (steady state) calculation, this computes the heads and flows for every pipe in the system. Prior to beginning the transient calculations, WaterGEMS automatically determines the friction factor based on this information:

If a pipe has zero flow at the initial steady-state, WaterGEMS use the Friction Coefficient specified in the Pipe Physical properties.

If a pipe has a nonzero flow at the initial steady-state, WaterGEMS automatically calculates a Darcy-Weisbach friction factor, f, based on the heads at each end of thepipe, the pipe length and diameter, and the flow in the pipe. It uses this calculated value in the transient simulation.

WaterGEMS always uses the Darcy-Weisbach friction method in performing the hydraulic transient calculations, regardless of which method is specified in the Steady State/EPS Solver Calculation Options. If required, WaterGEMS will automatically convert the friction factors to the appropriate format.

4.7.7 Active topology

The Active Topology functionality allows we to make elements inactive (and to change them back to active again), so as to either be excluded (when inactive) or included (when active) from the network and its calculations. This lets we create before and after scenarios and alternatives for proposed construction projects and to test the redundancy, if any, in existing networks.

The following conditions apply to all inactive elements:

• Reports
• Inclusion in profiles
• Inactive elements
• Current scenario
• They are not evaluated in any network calculations or hydraulic equations. They are not included when generating project inventory reports, element details reports, or element results report.

Inactive Topology option is turned on. By default, tabular reports do not include Inactive elements are differentiated visually from Active ones in the main drawing pane, in the Aerial View window, and in either of the plan view types. Inactive elements are still available for inclusion in selection sets.

Any changes made to the Active Topology through the drawing pane or the Property grids are applied to the Active Topology Alternative associated with the current scenario.

4.7.8 Assumptions

For running the analysis in the software few assumptions were made:

(a) The fluid under consideration is supposed to be homogeneous.
(b) Elasticity of the fluid and pipeline material follows a linear pattern.
(c) The flow is one dimensional and the fluid is incompressible.
(d) The software uses average velocity.
(e) Due to unavailability of valves data at the distribution end the system is modelled without them.
(f) The pipe elevations at many areas were unknown so these were interpolated. Excluding software few more assumptions that were taken:
(g) All pipes were taken as new pipes having same roughness coefficient as 140 and maximum amount of pipe material as ductile iron.
(h) Average temperature was assumed to be 20^0 C.

(i) Pump input data are being rated data without having the pump curves.
(j) Water quality simulation was beyond the scope of this present study

4.8 Losses in water distribution systems

The most basic way to determine losses is to calculate the difference between the system input and output. These losses can be divided into "apparent losses" and "real losses". Apparent losses are caused by unauthorized consumption by illegal connections and metering inaccuracies. Real losses are caused by leakage and overflows. The term unaccounted for water (UFW) describes the combination of real and apparent losses.

4.9 Leakage

Leakage from the water distribution pipeline network can be defined as that water which, having been obtained from the source, treated and put into supply, leaks and escapes other than by a deliberate action. In India, much of transported water is lost through leakage. This Figure can be even higher for older pipes. The loss of such large volumes of water is environmentally and economically damaging. The AWWA manual "Water Audits and Leak Detection" (AWWA 1999) lists six main sources of leakage that may occur in any section of the system:

(a) Material defects induced by poor design or insufficient planning at the concept stage.
(b) Pipe breaks caused by poor workmanship in the construction phase – laying and support of pipes.
(c) Operational errors – overpressure, water hammer, valve operation, etc.
(d) Corrosion due to soil and/or water chemistry effects and groundwater effects (for example, seawater).
(e) Leakage from any of the installed fittings (valves, saddles, bends, tees, hydrants etc.).
(f) Accidental or deliberate damage of hydrants and line air valves (including unauthorized tapings).

For leak detection Ultrasonic Leak Detector machine can be used.

CHAPTER 5

5.1 Methodology

In this chapter, methodologies to achieve various activities of the study objectives are discussed. The various activities carried out in the study is shown through the flow chart. The detailed description of methodology is as below:

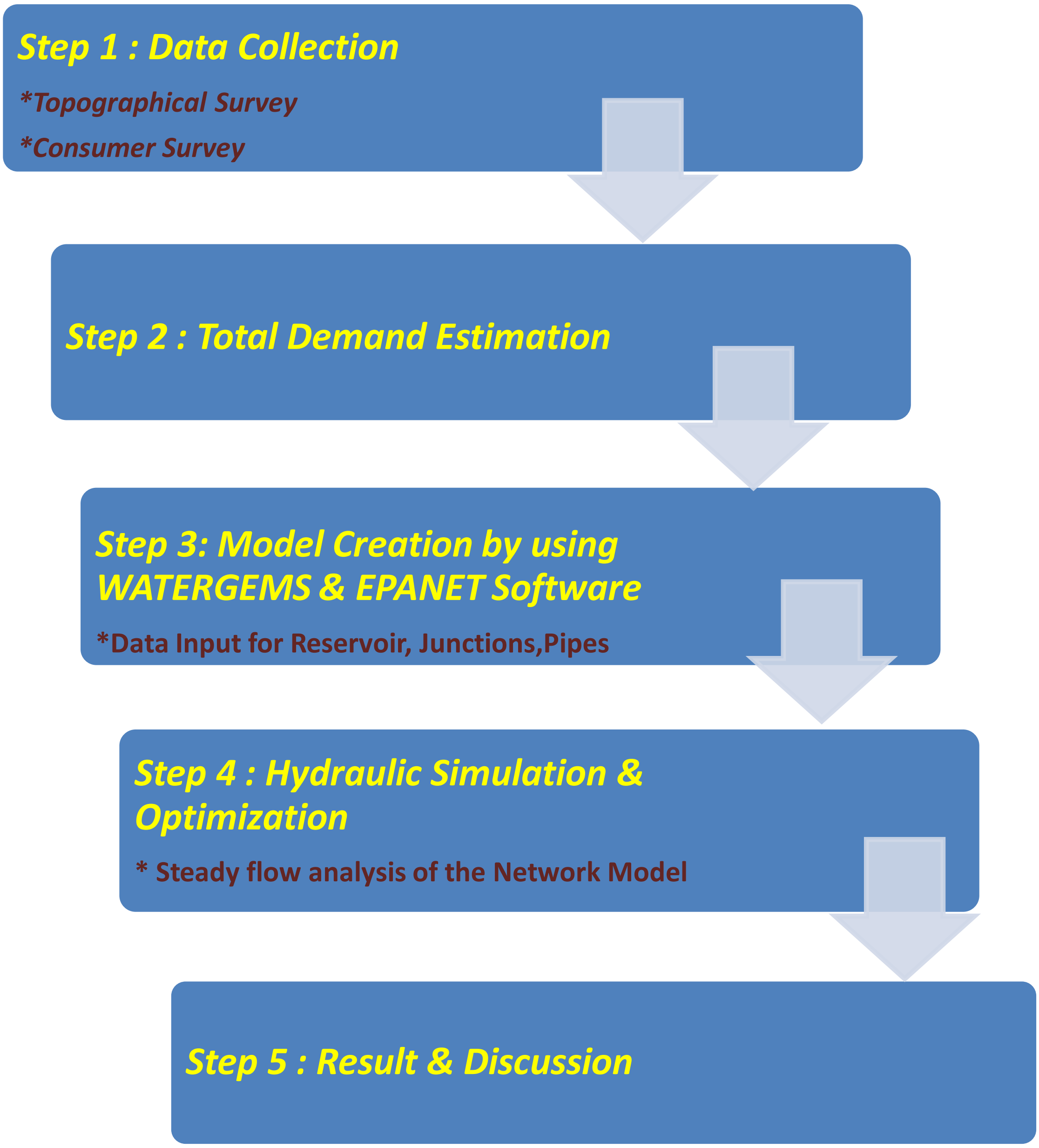

Figure 20: Methodology of Study

5.1.1 Data Collection

The necessary investigation required for design of water supply distribution network shall relate to carrying out / validating the property survey data for all habitations within the project area including Topographic survey along roads, pathways and or proposed alignment of the pipes.

Topographical Survey:

The available Topographic survey data for the Water Supply Distribution network (in AutoCAD format), showing the length, pipe diameter and alignment of the pipeline have been used for the study.

The survey data however is updated and validated with the concerned authority to providefor pipeline networks in all roads / pathways to provide accessibility to all the properties and provide for service connections.

Topographical survey has been carried along existing road network / pathways as maybe required for detail design of the water supply distribution network beyond the surveyed length. The survey shall update all objects/ roads / steps / drains/ gully pits/ culverts etc.,on both sides of the road (black bituminous top or concrete or brick) edges including road portion. Type and width of road and offset distance of important structures within or adjacent to road, from a fixed point shall be shown in the map.

The entire topographic survey has been carried out by DGPS and/or Total Station having facility of transferring data to computer. All traverse stations shall be recorded with X, Y andZ coordinates and shall be checked to eliminate any error. The BMs have been connected forX and Y coordinates by Total Stations with closing at intervals

Consumer Survey:

The finalized consumer data have been considered for the study. The survey has been done to identify all properties and other consumer points in each water supply zone. It is to arriveat the present population to calculate and project it for the design Year (2050). The consumer survey taken note of the following parameters within the command area.

- details with regards to property number, name, address, village and habitation, contact numbers,
- number of resident members (male, female, children),
- Categories of properties (domestic, institutional, commercial, slums, residential, mixed usage etc).

The data collected from household survey has been geo-coded to the base map. This database has been used for assessment of water demand of each property at the junction of distribution network pipe and thus the system has been designed and modeled accordingly.

5.1.2 Estimation of Design Demand

Design Years and Design Population

The distribution network has been designed for a period of 30 years. The base year for the project is 2020, the intermediate design year is 2035 and the ultimate design year is 2050.

In this research we consider the area of Ward no. 30 under Zone No. 6 in South Dumdum Municipality. The South Dumdum Municipality populations were 232811 and 392444 in the year 1991 and 2001 respectively as per population census data. The design population is calculated in the below Table 6.

Table 6: Population Projections (Source: Water works Department of South Dumdum Municipality, Population Census Data)

Area	Population in 2011	% of total population	Projected Population 2020	Projected Population 2035	Design Population 2050	Remarks
South Dumdum Municipality	403316	100%	480043	607922	735801	By Arithmetical Increase Method
Water Zone No.-6	39153	9.71%	46613	59030	71447	By Arithmetical Increase Method
Ward No.-30	13626	3.38%	16226	20548	24871	By Arithmetical Increase Method

Design water demand

This study intends to provide potable water in all wards of the South Dum Dum Municipality with adequate pressure head at consumer end as per CPHEEO guidelines as per present and future status of urbanization takes place. Base year 2020, Intermediate year 2035 and the ultimate year 2050 has been considered.

The computation of demand has been based on the above population as well as the following assumptions:

i. For estimation of domestic water demand of 150 lpcd as per CPHEEO manual has been considered for municipal towns with 24x7 supply system. This is inclusive of the demand for floating population.

ii. The unaccounted for water (UFW) has been considered at 15% of municipal domestic demand.

iii. The water demand for firefighting for communities larger than 50,000 has been estimated using the standard formula, $100/\sqrt{P}$ Kilo Liter per day where P is population in 1000.

iv. Industrial Water Supply Demand @20% of the Domestic and Non Domestic Water Supply Demand.

v. Institutional Water Supply Demand @20% of the Domestic and Non Domestic Water Supply Demand.

vi. Water Demand for Public Uses @ 5% of the Domestic and Non Domestic Water Supply Demand.

vii. Peak factor considers 3.0 for Population less than 50000 as per CPHEEO manual.

Table 7: Calculation of Design Water Demand

Description	Base Year 2020	Intermediate Year 2035	Design Year 2050	Unit	Remarks
Population	16226	20548	24871	Nos.	From calculation of Design Population
Floating Population@ 6%	974	1233	1493	Nos.	
Total Population	17200	21781	26364	Nos.	
Domestic and Non-Domestic Water Supply Demand @ 150 LPCD	2580000	3267150	3954600	Lit/Day	CPHEEO Manual
Industrial Water Supply Demand 20%	516000	653430	790920	Lit/Day	Assumed
Institutional Water Supply Demend@20%	516000	653430	790920	Lit/Day	Assumed
Fire Fighting water Demand @ 100*(P)^0.5 KLD, where P is population in thousand	414729	466702	513459	Lit/Day	CPHEEO Manual (Zone-6 population greater than 50,000)
Water Demand for Public Uses @ 5%	129000	163357.5	197730	Lit/Day	Assumed
UFW @ 15%	387000	490072.5	593190	Lit/Day	CPHEEO Manual
Total Estimated Demand(Lit/Day)	4542729	5694142	6840819	Lit/Day	
Hours of Supply	24	24	24	Hours	Continuous Water Supply
Total Estimated Demand (Lit/Sec.)	52.58	65.90	79.18	Lit/Sec	
Total Estimated Demand (MLD)	4.54	5.69	6.84	MLD	

Table 8: Service Reservoir Details

Study Area	Estimated Demand	ESR Capacity	Dia of ESR	Stagging Height	ESR Height
Ward No. 30	6.84 MLD	1941 CUM	20.3 M	19 M	6.0 M

5.1.3 Hydraulic Model Creation by using EPANET

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.

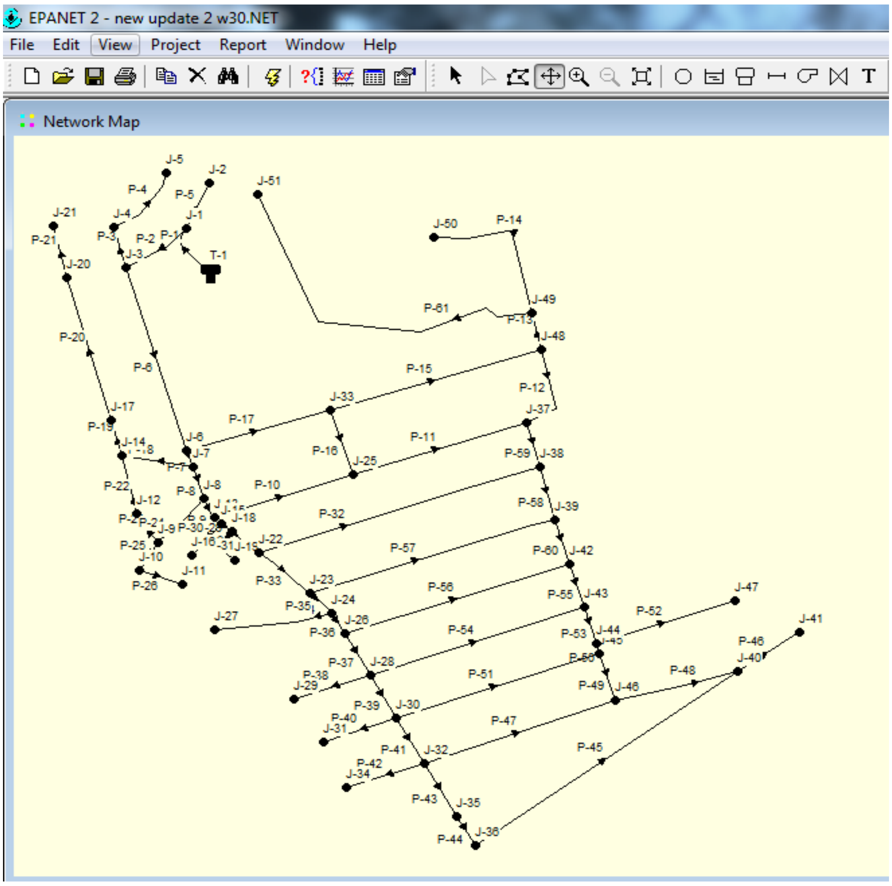

Figure 21: Water Distribution Network Drawing by using EPANET

5.1.4 Hydraulic Model Creation by using WaterGEMS

WaterGEMS is an extremely efficient tool for laying out a water-transmission pipeline or even an entire distribution network. It is easy to prepare a schematic model and let WaterGEMS takes care of the link-node connectivity and element labels, which are assigned automatically. For a schematic model only pipe lengths must be entered manually to complete the layout. We may need to input additional data for some hydraulic elements prior to a run.

Data input in WaterGEMS software:

In constructing a distribution network, we need not to be concerned with assigning labels to pipes and nodes, because Bentley WaterGEMS V8i will assign labels automatically. When creating a schematic drawing, pipe lengths are entered manually. In a scaled and geo-referenced drawing, pipe lengths are automatically calculated from the position of the pipes' bends and start and stop nodes on the drawing pane. There are several elements to be considered while creating a model such as Pipes, Junctions, Hydrants, Tanks, Reservoirs, Pumps, valves etc.

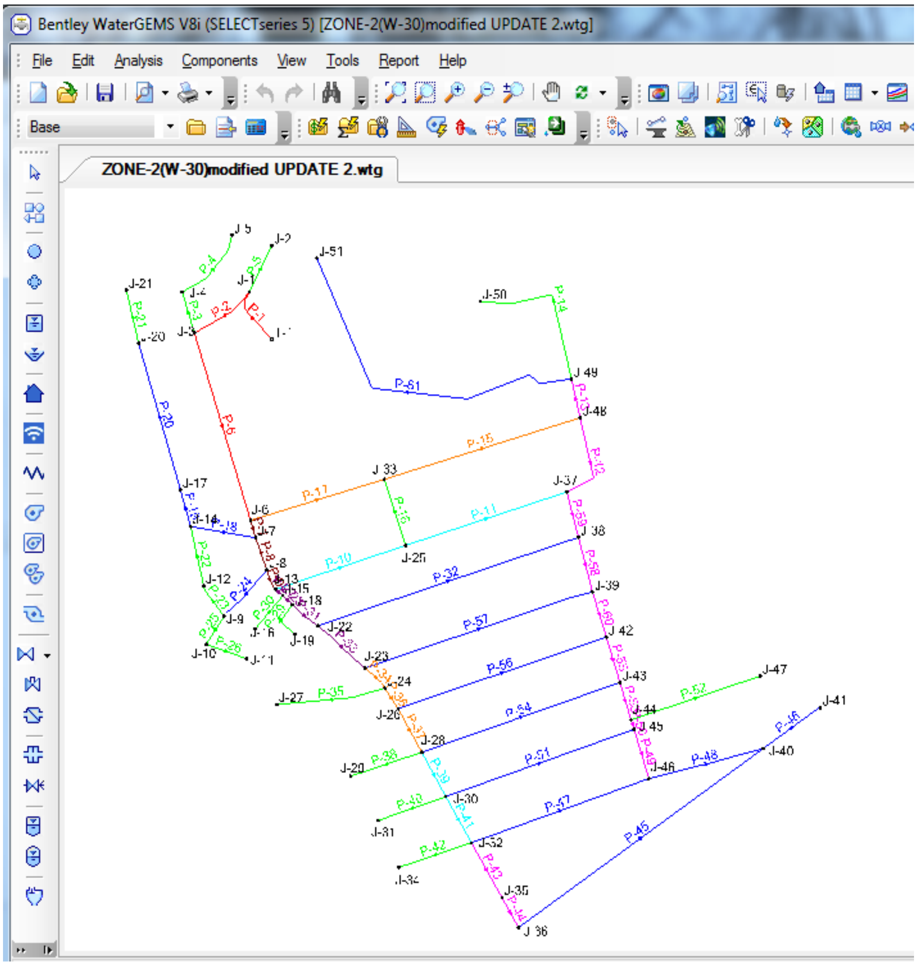

Figure 22: Water Distribution Network Drawing by using WaterGEMS

The input parameters of some elements are shown below:

Tanks: Tanks are nodes with storage capacity, where the volume of stored water canvary with time during a simulation. The primary input properties for tanks are:

- ✓ Bottom elevation (where water level is zero)
- ✓ Diameter (or shape if non-cylindrical)
- ✓ Initial, minimum and maximum water levels.

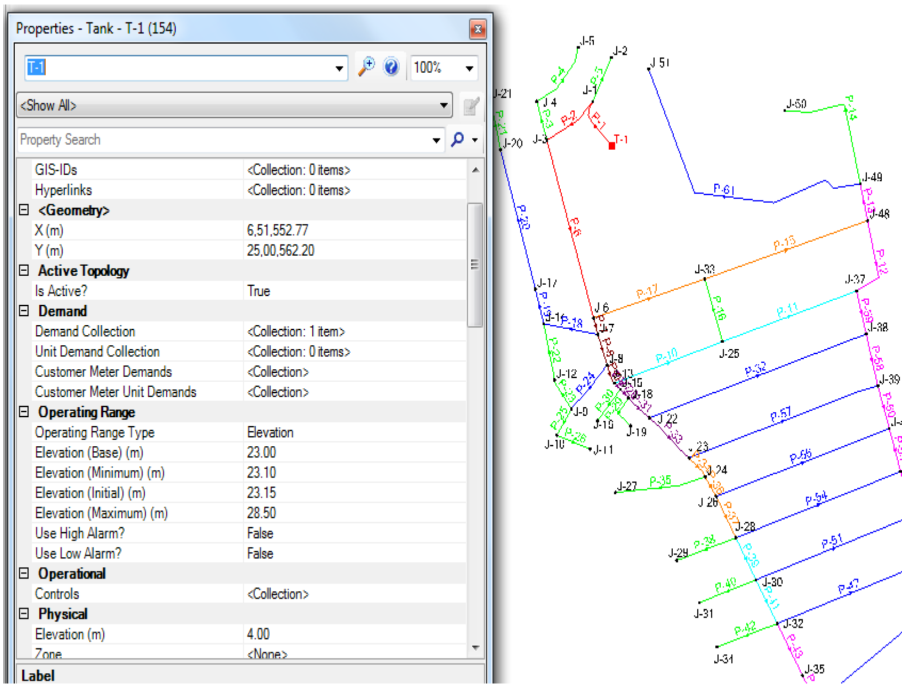

Figure 23: Tank Input Fields

Pipes: Pipes are links that convey water from one point in the network to another. WaterGEMS assumes that all pipes are full at all times. Flow direction is from the endat higher hydraulic head (internal energy per weight of water) to that at lower head. The principal hydraulic input parameters for pipes are: Start and end nodes, Diameter, Length, Roughness coefficient (for determining headloss), Status (open, closed, or contains a check valve). The status parameter allows pipes to implicitly contain shutoff (gate) valves and check (non-return) valves (which allow flow in only one direction).

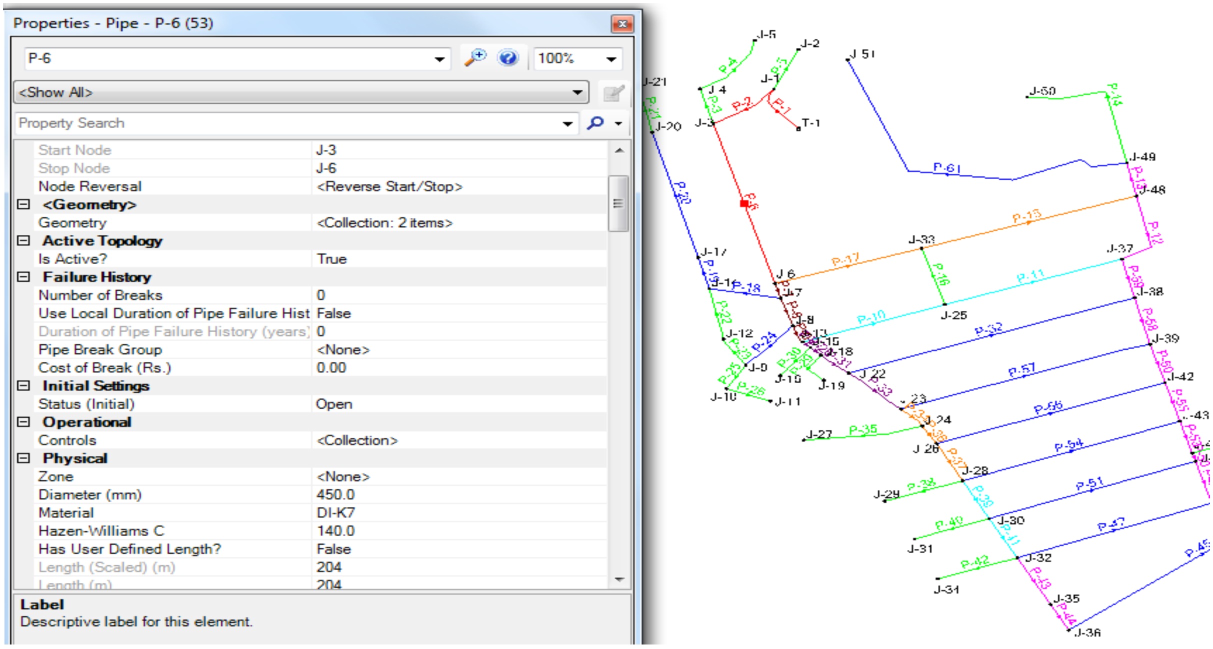

Figure 24: Pipe Input Fields

Junctions: Junctions are points in the network where links join together and where water enters or leaves the network. The basic input data required for junctions are: Elevation above some reference (usually RL), Water demand (rate of withdrawal from the network). Junctions can also:

Have their demand vary with time.

Have multiple categories of demands assigned to them.

Have negative demands indicating that water is entering the network.

Be water quality sources where constituents enter the network.

Contain emitters (or sprinklers) which make the outflow rate depend on the pressure.

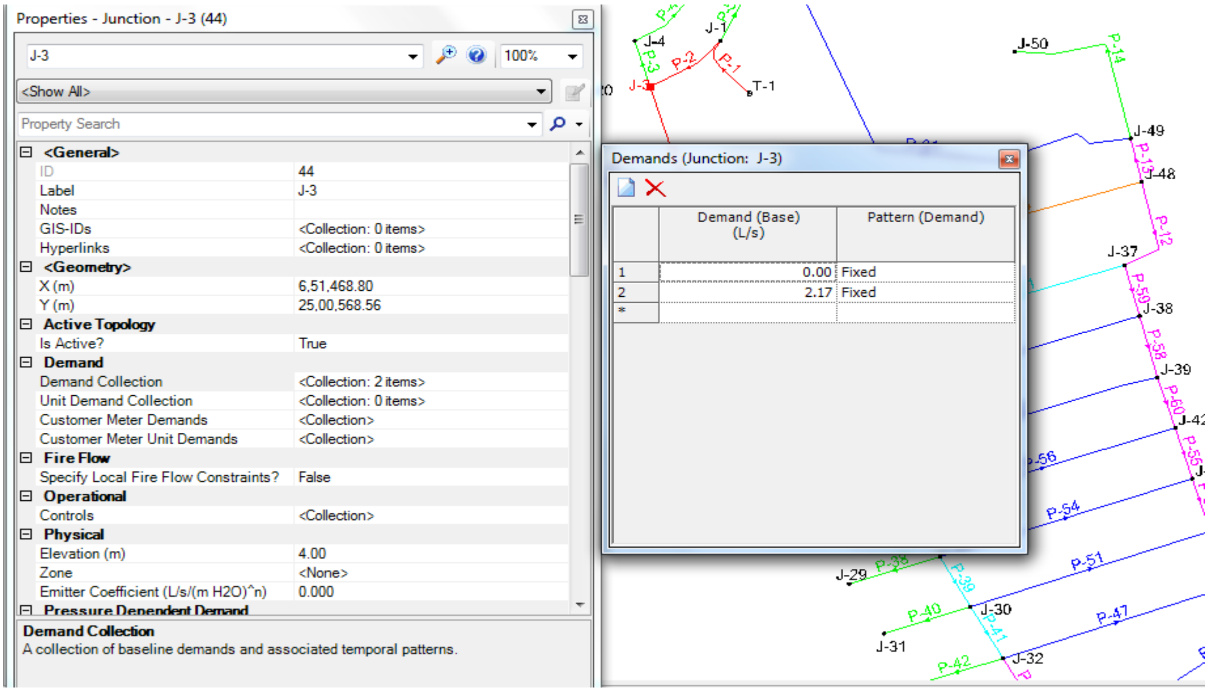

Figure 25: Junction Input Fields

5.1.5 Hydraulic Simulation & Optimization

When the base map is ready and all the model elements are annoyed with correct set of input data the Bentley WaterGEMS V8i provides modeling capabilities the model has been computed to optimize distribution system aspect of Hydraulic Analysis & performed a steady-state analysis for a snapshot view of the system, or perform an extended-period simulation to see how the system behaves over time using any common friction equation like Hazen-Williams, Darcy-Weisbach, or Manning’s methods.

It is also possible to take advantage of scenario management to see how your system reactsto different demand and physical conditions, including fire and emergency usage. Control pressure and flow completely by using flexible valve configurations. The engineering judgment has been applied to control pipe connections to optimize the different parameters like pressure, velocity and unit head loss.

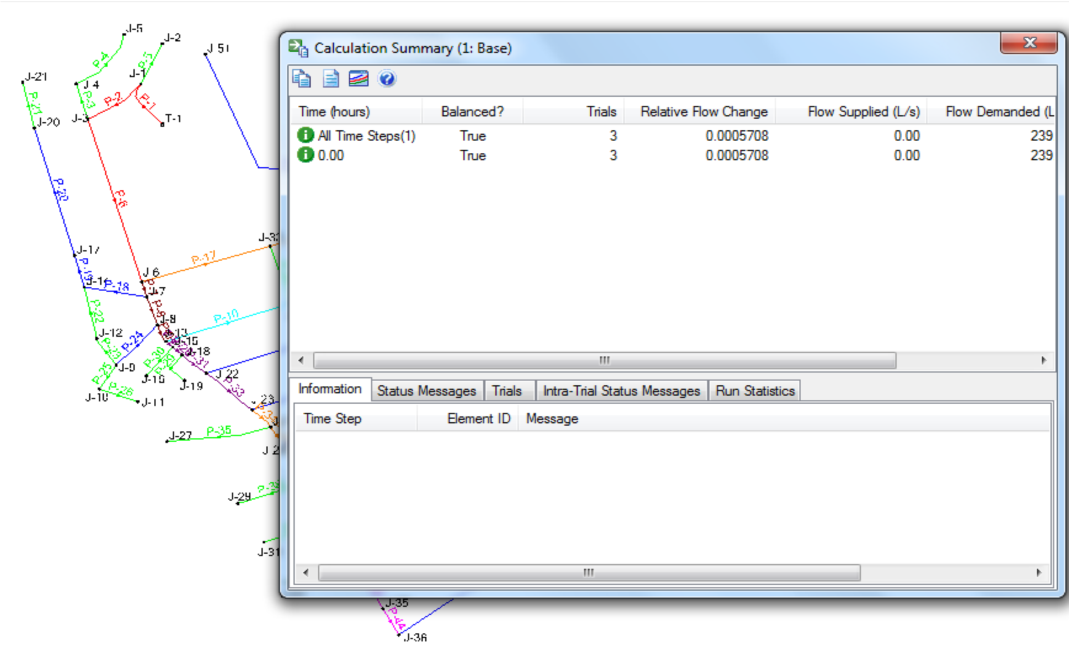

Figure 26: Simulation Window

Detailed discussions about the output of the results are analyzed in the subsequent chapters.

CHAPTER 6

6.1 Results Analysis and Discussions

The chapter below report and discuss the results obtained from the study. After collecting and computing data of the distribution network of the study area, the parameters like pressure, flow, unit head loss and velocity have been computed using WaterGEMS and EPANET.

6.1.1 Output data of the Distribution Network by using WaterGEMS

Table 9: Output data of Tank in the Distribution Network using WaterGEMS Software

ID	Label	Elevation (Base) (m)	Elevation (Minimum) (m)	Elevation (Initial) (m)	Elevation (Maximum) (m)	Flow (Out net) (L/s)	Hydraulic Grade (m)
154	T-1	23	23.1	23.15	28.5	239.34	23.15

Table 10: Output data of different Junction in the Distribution Network using WaterGEMS Software

ID	Label	Elevation (m)	Demand (L/s)	Hydraulic Grade (m)	Pressure (m H_2O)
42	J-1	4	5.09	22.907	18.87
50	J-2	4	1.09	22.892	18.85
44	J-3	4	6.51	22.631	18.59
46	J-4	4	2.58	22.478	18.44
48	J-5	4.001	1.68	22.427	18.39
52	J-6	4.015	7.53	21.933	17.88
54	J-7	4.002	2.53	21.868	17.83
56	J-8	4.025	2.52	21.767	17.71
84	J-9	4.057	2.83	21.668	17.58
87	J-10	4.1	1.62	21.622	17.49
89	J-11	4.13	0.92	21.613	17.45
82	J-12	4.15	2.06	21.62	17.43
58	J-13	4.003	3.63	21.711	17.67
74	J-14	4.12	3.51	21.617	17.46
91	J-15	4.004	1.41	21.679	17.64
97	J-16	4.016	0.92	21.67	17.62
76	J-17	4.06	4.01	21.534	17.44
93	J-18	4.089	1.85	21.638	17.51
95	J-19	4.039	0.86	21.63	17.56
78	J-20	4.057	4.35	21.413	17.32
80	J-21	4.19	1.14	21.396	17.17

ID	Label	Elevation (m)	Demand (L/s)	Hydraulic Grade (m)	Pressure (m H_2O)
99	J-22	4.005	8.04	21.539	17.5
103	J-23	4.15	7.16	21.394	17.21
105	J-24	4	3.51	21.294	17.26
60	J-25	4	8.11	21.548	17.51
109	J-26	4.014	6.34	21.22	17.17
107	J-27	4.019	2.39	21.156	17.1
111	J-28	4.075	8.3	21.114	17
113	J-29	4.09	1.63	21.068	16.94
115	J-30	4.081	8.07	20.968	16.85
117	J-31	4.098	1.56	20.927	16.8
119	J-32	4.068	8.2	20.898	16.8
70	J-33	4.035	8.95	21.686	17.62
121	J-34	3.99	1.66	20.848	16.82
123	J-35	4.017	2.05	20.83	16.78
125	J-36	4.034	7.24	20.802	16.73
62	J-37	4.12	6.62	21.413	17.26
101	J-38	4.001	8.14	21.274	17.24
147	J-39	4.05	7.37	21.134	17.05
127	J-40	4.056	10.58	20.631	16.54
129	J-41	4.091	1.51	20.626	16.5
144	J-42	4.083	6.76	21.026	16.91
141	J-43	4.003	6.37	20.929	16.89
136	J-44	3.995	3.99	20.853	16.82
134	J-45	3.985	5.62	20.844	16.83
131	J-46	4.019	7.73	20.807	16.75
139	J-47	4.005	2.96	20.597	16.56
64	J-48	4.17	7.24	21.464	17.26
66	J-49	4.17	11.72	21.358	17.15
68	J-50	4.079	3.38	20.984	16.87
152	J-51	4	7.5	20.854	16.82

Table 11: Output data of different pipe in the Distribution Network using WaterGEMS Software

ID	Label	Length (m)	Start Node	Stop Node	Dia mm	Material	Hazen Williams C	Flow (L/s)	Velocity (m/s)	Head loss Gradient m/km	Loss in Pipes
43	P-1	62	T-1	J-1	450	DI-K7	140	239.34	1.5	3.91	0.2424
45	P-2	74	J-1	J-3	450	DI-K7	140	233.17	1.47	3.73	0.276
47	P-3	45	J-3	J-4	100	DI-K7	140	4.26	0.54	3.42	0.1539

ID	Label	Length (m)	Start Node	Stop Node	Dia mm	Material	Hazen Willi-ams C	Flow (L/s)	Velocity (m/s)	Head loss Gra-dient m/km	Loss in Pipes
49	P-4	83	J-4	J-5	100	DI-K7	140	1.68	0.21	0.61	0.0506
51	P-5	54	J-1	J-2	100	DI-K7	140	1.09	0.14	0.27	0.0146
53	P-6	204	J-3	J-6	450	DI-K7	140	222.4	1.4	3.42	0.6977
55	P-7	19	J-6	J-7	400	DI-K7	140	163.3	1.3	3.42	0.065
57	P-8	35	J-7	J-8	400	DI-K7	140	148.17	1.18	2.86	0.1001
59	P-9	23	J-8	J-13	400	DI-K7	140	137.8	1.1	2.5	0.0575
61	P-10	147	J-13	J-25	250	DI-K7	140	25.76	0.52	1.1	0.1617
63	P-11	183	J-25	J-37	250	DI-K7	140	20.75	0.42	0.74	0.1354
65	P-12	97	J-48	J-37	200	DI-K7	140	9.68	0.31	0.53	0.0514
67	P-13	41	J-48	J-49	200	DI-K7	140	22.6	0.72	2.57	0.1054
69	P-14	168	J-49	J-50	100	DI-K7	140	3.38	0.43	2.23	0.3746
71	P-15	221	J-33	J-48	300	DI-K7	140	39.52	0.56	1	0.221
72	P-16	73	J-33	J-25	100	DI-K7	140	3.11	0.4	1.91	0.1394
73	P-17	151	J-6	J-33	300	DI-K7	140	51.57	0.73	1.64	0.2476
75	P-18	71	J-7	J-14	150	DI-K7	140	12.59	0.71	3.53	0.2506
77	P-19	40	J-14	J-17	150	DI-K7	140	9.5	0.54	2.1	0.084
79	P-20	159	J-17	J-20	150	DI-K7	140	5.49	0.31	0.76	0.1208
81	P-21	57	J-20	J-21	100	DI-K7	140	1.14	0.15	0.3	0.0171
83	P-22	63	J-12	J-14	100	DI-K7	140	0.42	0.05	0.05	0.0032
85	P-23	39	J-9	J-12	100	DI-K7	140	2.48	0.32	1.25	0.0488
86	P-24	67	J-8	J-9	150	DI-K7	140	7.86	0.44	1.47	0.0985
88	P-25	35	J-9	J-10	100	DI-K7	140	2.55	0.32	1.32	0.0462
90	P-26	46	J-10	J-11	100	DI-K7	140	0.92	0.12	0.2	0.0092
92	P-27	10	J-13	J-15	350	DI-K7	140	108.41	1.13	3.07	0.0307
94	P-28	14	J-15	J-18	350	DI-K7	140	106.08	1.1	2.95	0.0413
96	P-29	43	J-18	J-19	100	DI-K7	140	0.86	0.11	0.18	0.0077
98	P-30	46	J-15	J-16	100	DI-K7	140	0.92	0.12	0.2	0.0092
100	P-31	35	J-18	J-22	350	DI-K7	140	103.37	1.07	2.81	0.0984
102	P-32	296	J-22	J-38	150	DI-K7	140	6	0.34	0.9	0.2664
104	P-33	68	J-22	J-23	350	DI-K7	140	89.33	0.93	2.15	0.1462
106	P-34	30	J-23	J-24	300	DI-K7	140	75.77	1.07	3.35	0.1005
108	P-35	118	J-24	J-27	100	DI-K7	140	2.39	0.3	1.17	0.1381
110	P-36	26	J-24	J-26	300	DI-K7	140	69.88	0.99	2.88	0.0749
112	P-37	52	J-26	J-28	300	DI-K7	140	57.82	0.82	2.03	0.1056
114	P-38	81	J-28	J-29	100	DI-K7	140	1.63	0.21	0.58	0.047
116	P-39	53	J-28	J-30	250	DI-K7	140	42.17	0.86	2.75	0.1458
118	P-40	77	J-30	J-31	100	DI-K7	140	1.56	0.2	0.53	0.0408
120	P-41	55	J-30	J-32	250	DI-K7	140	27.8	0.57	1.27	0.0699
122	P-42	83	J-32	J-34	100	DI-K7	140	1.66	0.21	0.6	0.0498

ID	Label	Length (m)	Start Node	Stop Node	Dia mm	Material	Hazen Willi-ams C	Flow (L/s)	Velocity (m/s)	Head loss Gra-dient m/km	Loss in Pipes
124	P-43	66	J-32	J-35	200	DI-K7	140	13.81	0.44	1.03	0.068
126	P-44	36	J-35	J-36	200	DI-K7	140	11.75	0.37	0.77	0.0277
128	P-45	323	J-36	J-40	150	DI-K7	140	4.52	0.26	0.53	0.1712
130	P-46	75	J-40	J-41	150	DI-K7	140	1.51	0.09	0.07	0.0053
132	P-47	203	J-32	J-46	150	DI-K7	140	4.13	0.23	0.45	0.0914
133	P-48	127	J-46	J-40	150	DI-K7	140	7.58	0.43	1.38	0.1753
135	P-49	54	J-45	J-46	200	DI-K7	140	11.18	0.36	0.7	0.0378
137	P-50	10	J-44	J-45	200	DI-K7	140	12.06	0.38	0.8	0.008
138	P-51	215	J-30	J-45	150	DI-K7	140	4.74	0.27	0.58	0.1247
140	P-52	147	J-44	J-47	100	DI-K7	140	2.96	0.38	1.74	0.2558
142	P-53	41	J-43	J-44	200	DI-K7	140	19.01	0.61	1.87	0.0767
143	P-54	226	J-28	J-43	150	DI-K7	140	5.73	0.32	0.82	0.1853
145	P-55	49	J-42	J-43	200	DI-K7	140	19.65	0.63	1.98	0.097
146	P-56	237	J-26	J-42	150	DI-K7	140	5.71	0.32	0.82	0.1943
148	P-57	258	J-23	J-39	150	DI-K7	140	6.4	0.36	1.01	0.2606
149	P-58	59	J-38	J-39	200	DI-K7	140	21.67	0.69	2.38	0.1404
150	P-59	49	J-37	J-38	200	DI-K7	140	23.81	0.76	2.83	0.1387
151	P-60	49	J-39	J-42	200	DI-K7	140	20.7	0.66	2.18	0.1068
153	P-61	372	J-49	J-51	150	DI-K7	140	7.5	0.42	1.35	0.5022
TOTAL		5940									7.8119

6.1.2 Output data of the Distribution Network by using EPANET

Table 12: Output data of different Junction in the Distribution Network using EPANET Software

Node ID	Elevation (m)	Base Demand (LPS)	Demand	Head (m)	Pressure(m)
J-1	4	1.695164	5.09	22.91	18.87
J-3	4	2.168808	6.51	22.63	18.59
J-4	4	0.859872	2.58	22.48	18.44
J-5	4.001	0.559888	1.68	22.43	18.39
J-2	4	0.362408	1.09	22.89	18.85
J-6	4.015	2.510545	7.53	21.93	17.88
J-7	4.002	0.844805	2.53	21.87	17.83
J-8	4.025	0.839484	2.52	21.77	17.71
J-13	4.003	1.209947	3.63	21.71	17.67
J-25	4	2.704967	8.11	21.55	17.51
J-37	4.12	2.206821	6.62	21.41	17.26

Node ID	Elevation (m)	Base Demand (LPS)	Demand	Head (m)	Pressure(m)
J-48	4.17	2.413867	7.24	21.46	17.26
J-49	4.17	3.905467	11.72	21.36	17.15
J-50	4.079	1.12732	3.38	20.98	16.87
J-33	4.035	2.983098	8.95	21.69	17.62
J-14	4.12	1.170377	3.51	21.62	17.46
J-17	4.06	1.335901	4.01	21.53	17.44
J-20	4.057	1.449739	4.35	21.41	17.32
J-21	4.19	0.380247	1.14	21.4	17.17
J-12	4.15	0.686241	2.06	21.62	17.43
J-9	4.057	0.944465	2.83	21.67	17.58
J-10	4.1	0.541271	1.62	21.62	17.49
J-11	4.13	0.307322	0.92	21.61	17.45
J-15	4.004	0.469156	1.41	21.68	17.64
J-18	4.089	0.616146	1.85	21.64	17.51
J-19	4.039	0.286194	0.86	21.63	17.56
J-16	4.016	0.306516	0.92	21.67	17.62
J-22	4.005	2.679113	8.04	21.54	17.5
J-38	4.001	2.71323	8.14	21.27	17.24
J-23	4.15	2.386204	7.16	21.39	17.21
J-24	4	1.169085	3.51	21.29	17.26
J-27	4.019	0.795373	2.39	21.16	17.1
J-26	4.014	2.11481	6.34	21.22	17.17
J-28	4.075	2.765701	8.3	21.11	17
J-29	4.09	0.542612	1.63	21.07	16.94
J-30	4.081	2.691129	8.07	20.97	16.85
J-31	4.098	0.519258	1.56	20.93	16.8
J-32	4.068	2.732095	8.2	20.9	16.8
J-34	3.99	0.554671	1.66	20.85	16.82
J-35	4.017	0.68408	2.05	20.83	16.78
J-36	4.034	2.41186	7.24	20.8	16.73
J-40	4.056	3.528305	10.58	20.63	16.54
J-41	4.091	0.504502	1.51	20.63	16.5
J-46	4.019	2.577527	7.73	20.81	16.75
J-45	3.985	1.873822	5.62	20.84	16.83
J-44	3.995	1.329531	3.99	20.85	16.82
J-47	4.005	0.986406	2.96	20.6	16.56
J-43	4.003	2.122603	6.37	20.93	16.89
J-42	4.083	2.254232	6.76	21.03	16.91
J-39	4.05	2.457788	7.37	21.13	17.05
J-51	4	2.500026	7.5	20.85	16.82
Tank T-1	23	#N/A	-239.34	23.15	0.15

Table 13: Output data of different pipe in the Distribution Network using EPANET Software

Link ID	Length (m)	Dia mm	Rough-ness	Friction Factor	Flow (L/s)	Velocity (m/s)	Unit Head loss (m/Km)	Loss in Pipes	Status
P-1	62	450	140	0.015	239.34	1.5	3.91	0.2431	Open
P-2	74	450	140	0.015	233.17	1.47	3.73	0.2761	Open
P-3	45	100	140	0.023	4.26	0.54	3.42	0.1527	Open
P-4	83	100	140	0.026	1.68	0.21	0.61	0.0508	Open
P-5	54	100	140	0.028	1.09	0.14	0.27	0.0146	Open
P-6	204	450	140	0.015	222.4	1.4	3.42	0.6982	Open
P-7	19	400	140	0.016	163.3	1.3	3.42	0.0651	Open
P-8	35	400	140	0.016	148.17	1.18	2.86	0.1015	Open
P-9	23	400	140	0.016	137.8	1.1	2.5	0.0563	Open
P-10	147	250	140	0.02	25.76	0.52	1.1	0.1622	Open
P-11	183	250	140	0.02	20.75	0.42	0.74	0.1352	Open
P-12	97	200	140	0.022	9.68	0.31	0.53	0.0514	Open
P-13	41	200	140	0.019	22.6	0.72	2.57	0.1064	Open
P-14	168	100	140	0.024	3.38	0.43	2.23	0.3742	Open
P-15	221	300	140	0.019	39.52	0.56	1	0.221	Open
P-16	73	100	140	0.024	3.11	0.4	1.91	0.1386	Open
P-17	151	300	140	0.018	51.57	0.73	1.64	0.2468	Open
P-18	71	150	140	0.02	12.59	0.71	3.53	0.2514	Open
P-19	40	150	140	0.021	9.5	0.54	2.1	0.0833	Open
P-20	159	150	140	0.023	5.49	0.31	0.76	0.121	Open
P-21	57	100	140	0.028	1.14	0.15	0.3	0.017	Open
P-22	63	100	140	0.032	0.42	0.05	0.05	0.0032	Open
P-23	39	100	140	0.025	2.48	0.32	1.25	0.0485	Open
P-24	67	150	140	0.022	7.86	0.44	1.47	0.0984	Open
P-25	35	100	140	0.025	2.55	0.32	1.32	0.046	Open
P-26	46	100	140	0.029	0.92	0.12	0.2	0.0091	Open
P-27	10	350	140	0.017	108.41	1.13	3.07	0.0311	Open
P-28	14	350	140	0.017	106.08	1.1	2.95	0.0415	Open
P-29	43	100	140	0.029	0.86	0.11	0.18	0.0077	Open
P-30	46	100	140	0.029	0.92	0.12	0.2	0.0091	Open
P-31	35	350	140	0.017	103.37	1.07	2.81	0.0985	Open
P-32	296	150	140	0.023	6	0.34	0.9	0.2665	Open
P-33	68	350	140	0.017	89.33	0.93	2.15	0.1454	Open
P-34	30	300	140	0.017	75.77	1.07	3.35	0.1	Open
P-35	118	100	140	0.025	2.39	0.3	1.17	0.1385	Open

Link ID	Length (m)	Dia mm	Rough-ness	Friction Factor	Flow (L/s)	Velocity (m/s)	Unit Head loss (m/Km)	Loss in Pipes	Status
P-36	26	300	140	0.017	69.88	0.99	2.88	0.0742	Open
P-37	52	300	140	0.018	57.82	0.82	2.03	0.1058	Open
P-38	81	100	140	0.026	1.63	0.21	0.58	0.0468	Open
P-39	53	250	140	0.018	42.17	0.86	2.75	0.1456	Open
P-40	77	100	140	0.026	1.56	0.2	0.53	0.041	Open
P-41	55	250	140	0.019	27.8	0.57	1.27	0.0704	Open
P-42	83	100	140	0.026	1.66	0.21	0.6	0.0495	Open
P-43	66	200	140	0.021	13.81	0.44	1.03	0.0678	Open
P-44	36	200	140	0.021	11.75	0.37	0.77	0.0277	Open
P-45	323	150	140	0.024	4.52	0.26	0.53	0.1712	Open
P-46	75	150	140	0.028	1.51	0.09	0.07	0.0053	Open
P-47	203	150	140	0.024	4.13	0.23	0.45	0.0913	Open
P-48	127	150	140	0.022	7.58	0.43	1.38	0.1754	Open
P-49	54	200	140	0.022	11.18	0.36	0.7	0.0376	Open
P-50	10	200	140	0.021	12.06	0.38	0.8	0.0083	Open
P-51	215	150	140	0.024	4.74	0.27	0.58	0.1247	Open
P-52	147	100	140	0.024	2.96	0.38	1.74	0.2555	Open
P-53	41	200	140	0.02	19.01	0.61	1.87	0.0762	Open
P-54	226	150	140	0.023	5.73	0.32	0.82	0.1852	Open
P-55	49	200	140	0.02	19.65	0.63	1.98	0.0977	Open
P-56	237	150	140	0.023	5.71	0.32	0.82	0.1943	Open
P-57	258	150	140	0.023	6.4	0.36	1.01	0.2602	Open
P-58	59	200	140	0.02	21.67	0.69	2.38	0.1401	Open
P-59	49	200	140	0.019	23.81	0.76	2.83	0.1384	Open
P-60	49	200	140	0.02	20.7	0.66	2.18	0.1074	Open
P-61	372	150	140	0.022	7.5	0.42	1.35	0.5024	Open
TOTAL	5940							7.81	

6.1.3 Different Parameter Analysis in the Distribution Network using WaterGEMS and EPANET

Table 14: Different Parameter Analysis in the Distribution Network

Distribution Network Analysis using WaterGEMS Software					
Parameter	**Max/Min**	**Location**	**Value**	**Diameter (mm)**	**Material**
Flow (L/s)	Maximum	Pipe P-1	239.34	450	DI-K7
	Minimum	Pipe P-22	0.42	100	DI-K7

Distribution Network Analysis using WaterGEMS Software					
Velocity(m/s)	Maximum	Pipe P-1	1.5	450	DI-K7
	Minimum	Pipe P-22	0.05	100	DI-K7
Head Loss(m/Km)	Maximum	Pipe P-1	3.91	450	DI-K7
	Minimum	Pipe P-22	0.05	100	DI-K7
Pressure Head(m)	Maximum	Junction J-1	18.87		
	Minimum	Junction J-41	16.5		
Distribution Network Analysis using EPANET Software					
Parameter	**Max/Min**	**Location**	**Value**	**Diameter (mm)**	**Material**
Flow (L/s)	Maximum	Pipe P-1	239.34	450	DI-K7
	Minimum	Pipe P-22	0.42	100	DI-K7
Velocity(m/s)	Maximum	Pipe P-1	1.5	450	DI-K7
	Minimum	Pipe P-22	0.05	100	DI-K7
Head Loss(m/Km)	Maximum	Pipe P-1	3.91	450	DI-K7
	Minimum	Pipe P-22	0.05	100	DI-K7
Pressure Head(m)	Maximum	Junction J-1	18.87		
	Minimum	Junction J-41	16.5		

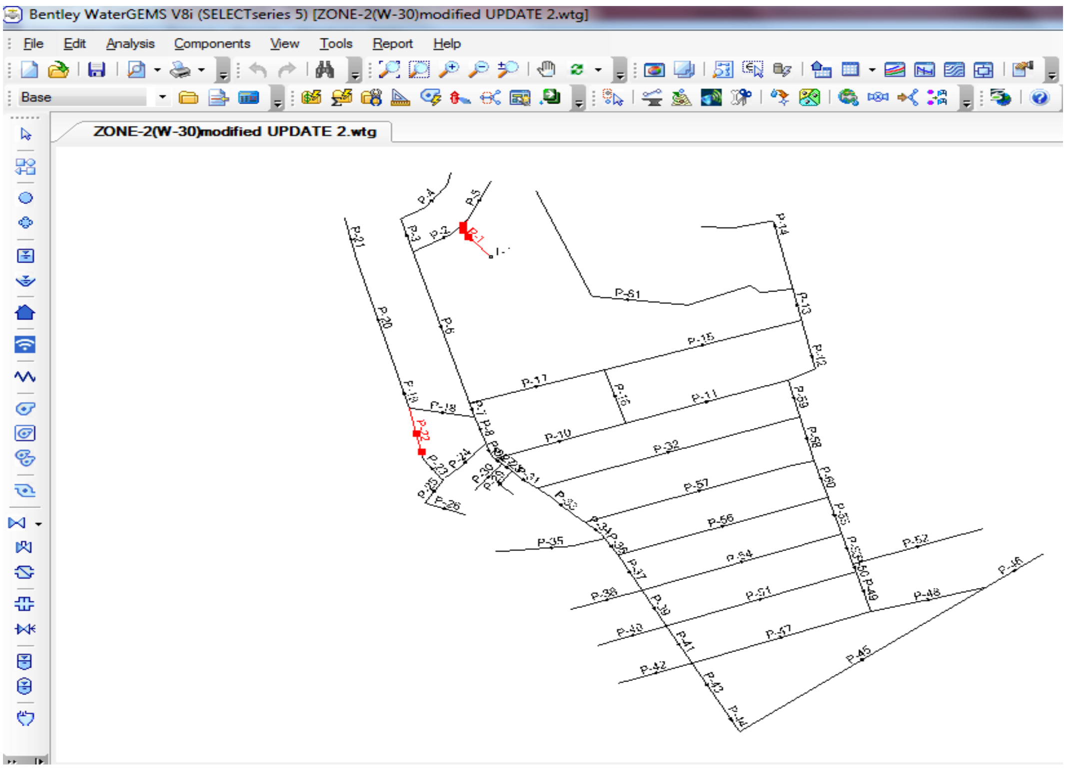

Figure 27: Pipe showing Maximum and Minimum Flow, Velocity, Head Loss in the Distribution Network

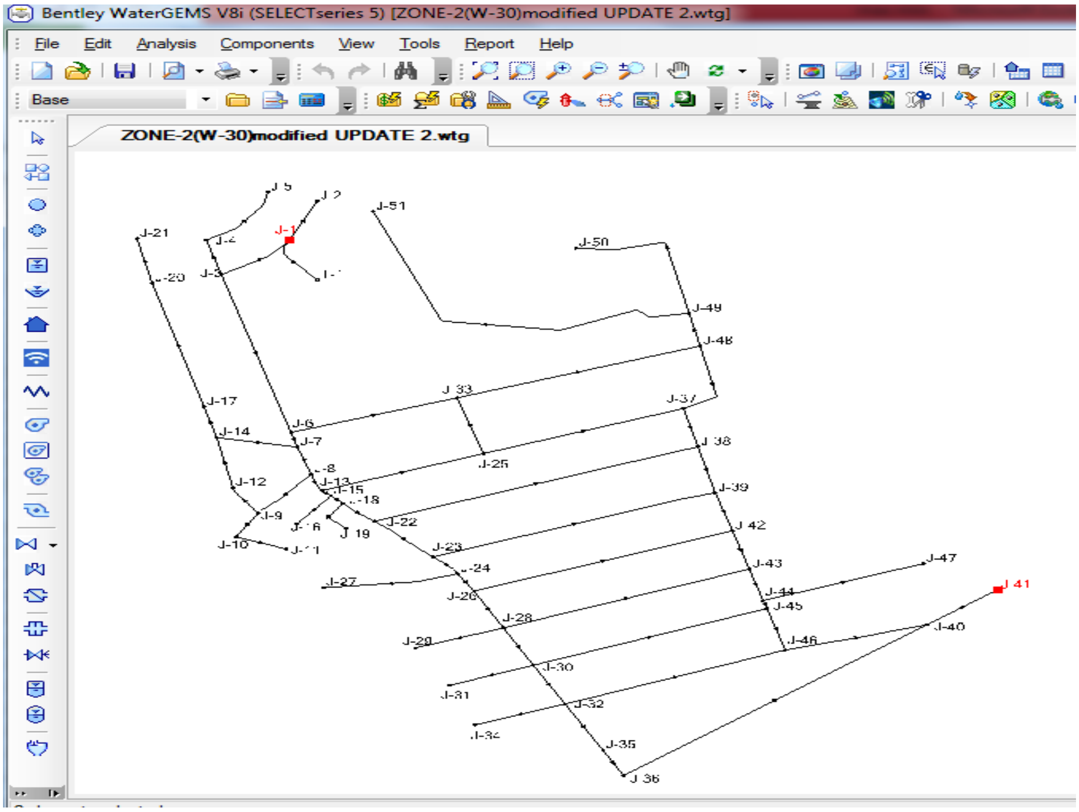

Figure 28: Junction showing Maximum and Minimum Pressure Head in the Distribution Network

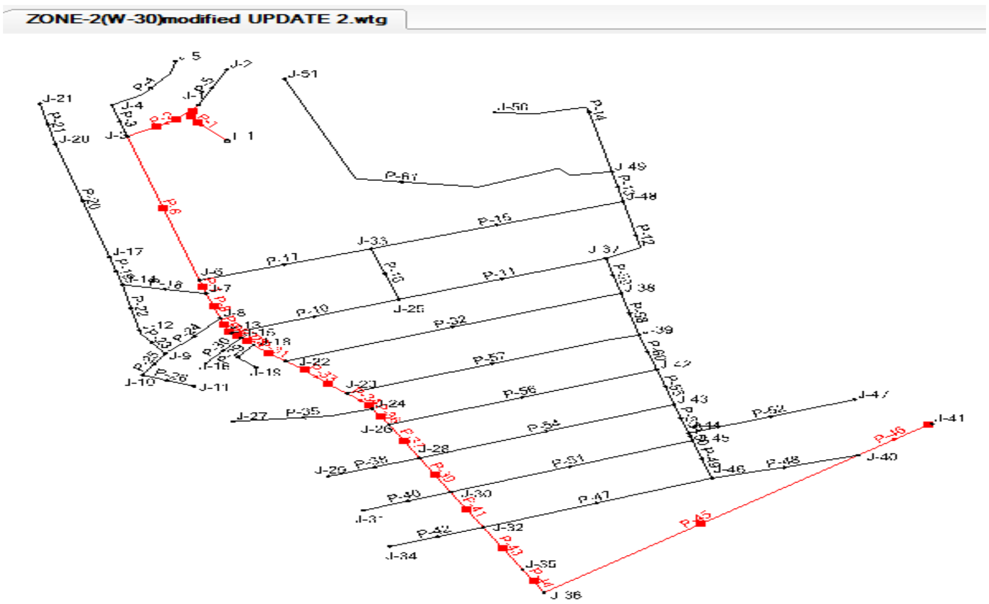

Figure 29: Showing Critical path in the Distribution Network

6.1.4 Different Graphs of various Parameters in the Distribution Network using WaterGEMS and EPANET

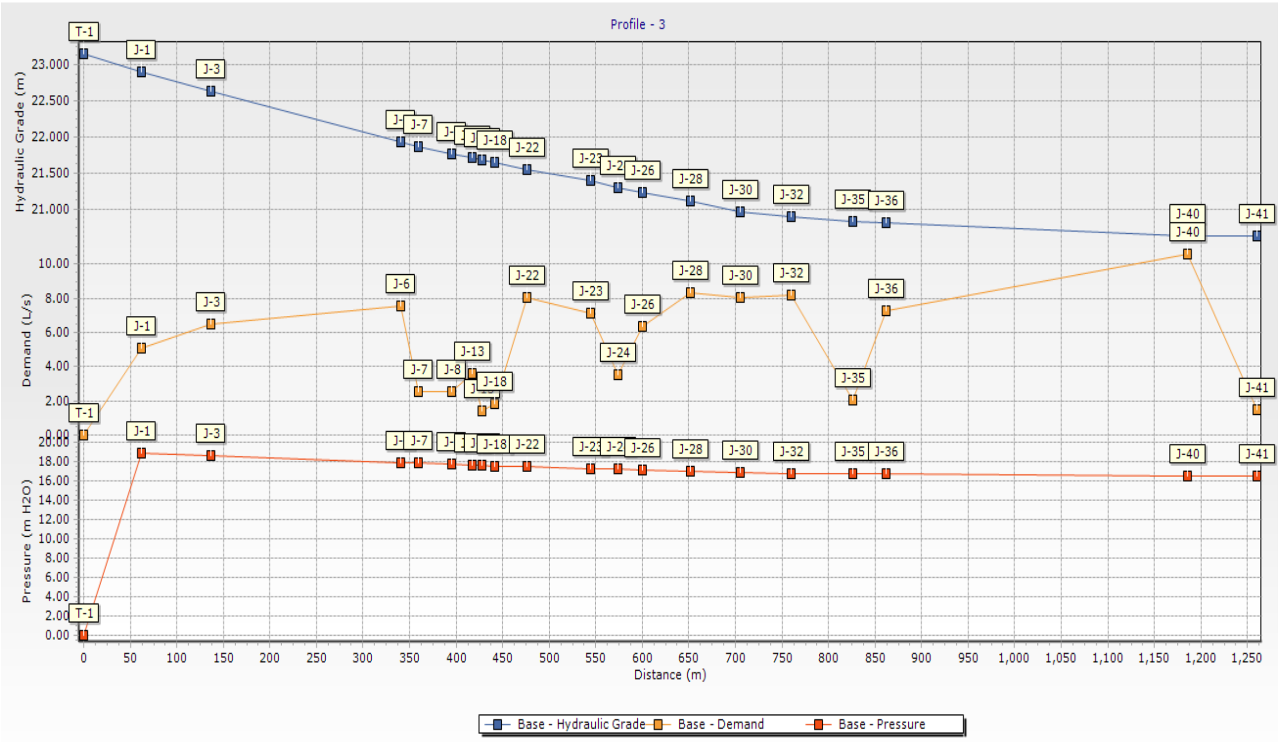

Figure 30: Hydraulic Grade, Pressure Head, Demand variation along the Critical path

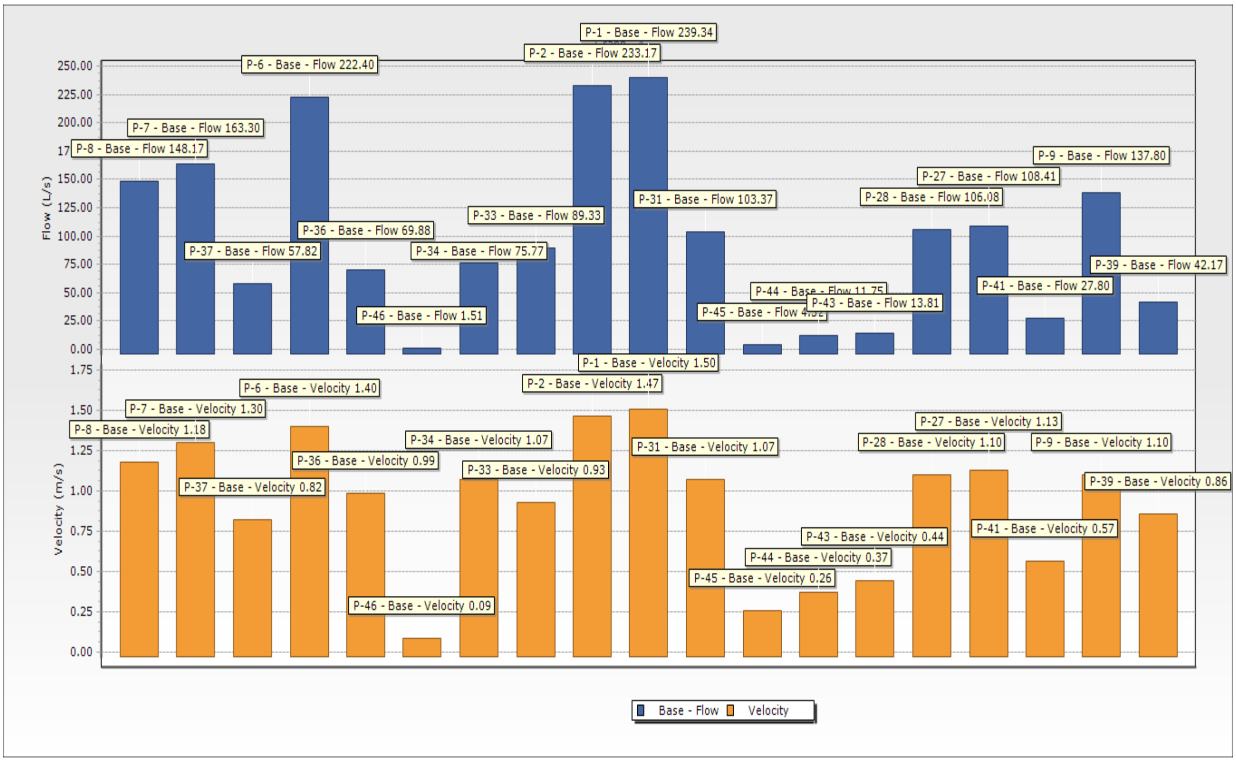

Figure 31: Flow and Velocity variation in the pipe along the Critical path

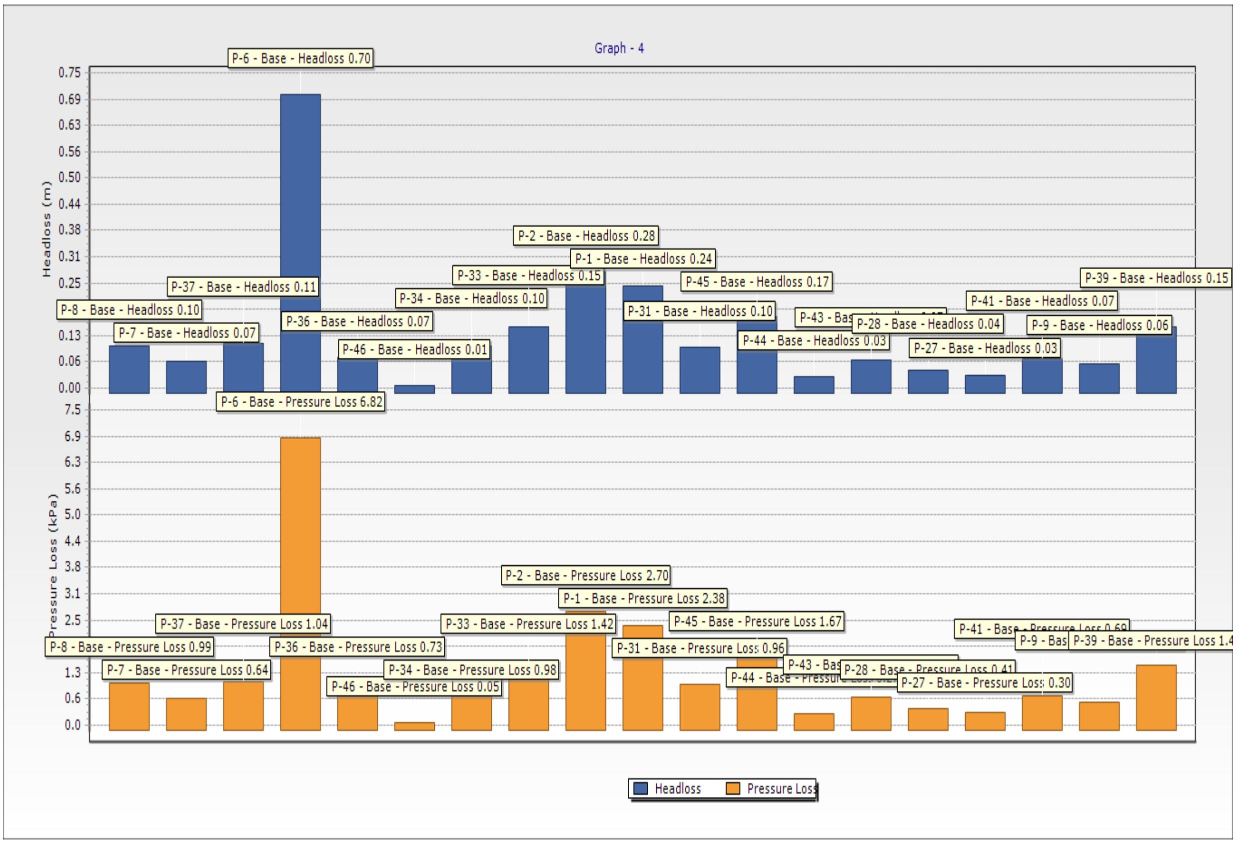

Figure 32: Headloss and Pressure loss variation in the pipe along the Critical path

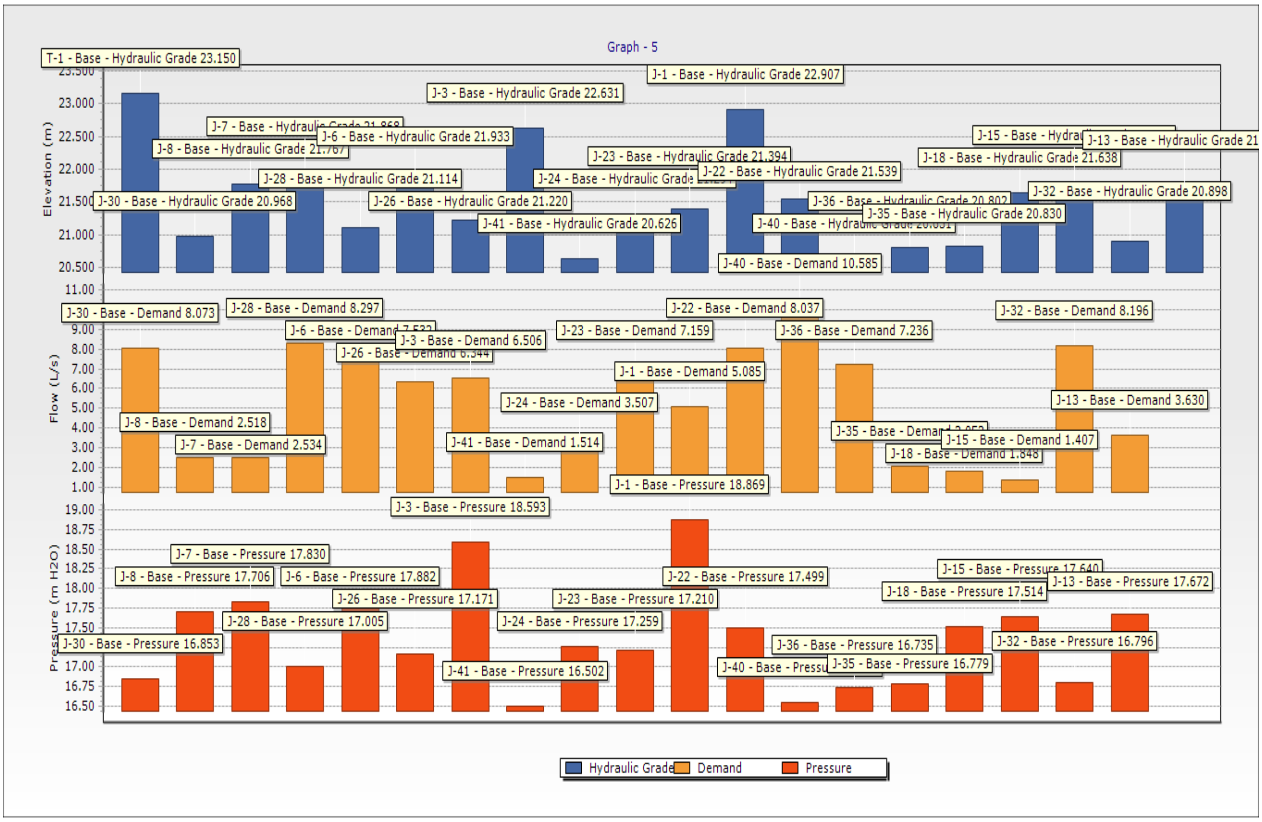

Figure 33: Demand, Hydraulic Grade, Headloss variation in the Junction along the Critical Path

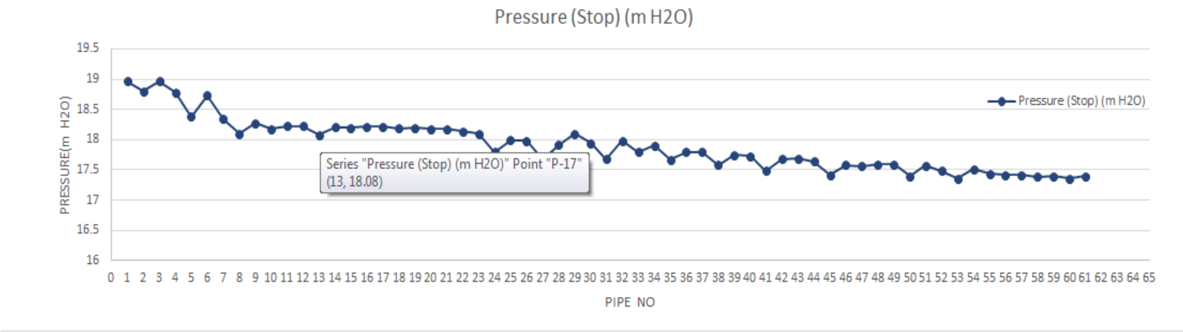

Figure 34: Headloss variation along Pipe

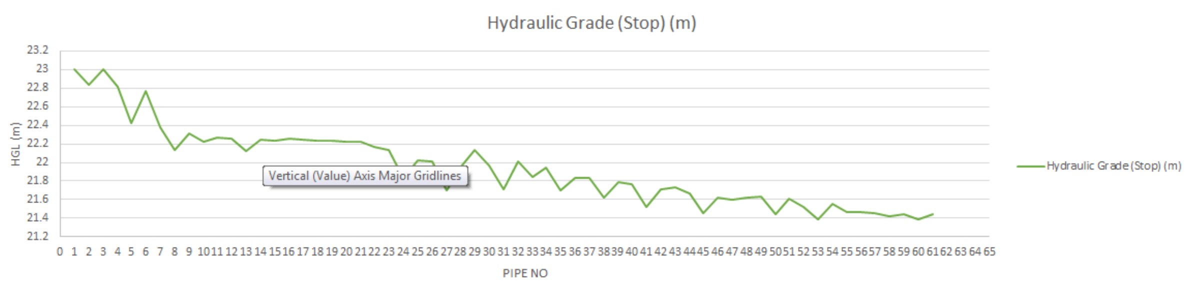

Figure 35: Variation of Hydraulic Grade along Pipe

The variations of the residual pressure are along the alignment to the furthest node J- 41. The ground level is undulated so that the Hydraulic Gradient Line has varied from18.87 m to 16.5 m which is shown in the figure. As per CPHEEO manual minimum residual pressure required at ferrule point should be 7 m for one storeyed building, 12 m for two storeyed building and 17 m for three storeyed building for direct supply. In this distribution network minimum residual pressure is 16.5 m. In this study area upto two storyed building water supplied by direct supply and for multi-storeyed building pumping is required. So, it fulfills the minimum pressure guideline of CPHEEO.

The velocity and the headloss gradient have also varied along the alignment to the furthest node. The velocity has varied from 1.5 m/sec to 0.05m/sec. The maximum head loss gradient in the entire network is 3.91 m/Km. The headloss gradient is less than 4m/Km stipulated by the CPHEEO manual.

After analysing different parameter likes flow, velocity, head loss, pressure head in this distribution network it can be easily seen that there is very less difference in both the system.

The detailed output results of WaterGEMS and EPANET for Junctions and Pipes are given as Appendix 1and Appendix 2 respectively.

6.1.5 Comparison of losses

Losses can be computed in following manner in both the system.

$$\text{Percentage of losses} = \frac{\sum\left\{ \mathit{Unit}\ \text{headloss} \times \text{length of the pipe} \right\}}{\sum \text{length of the pipe}} \times 100$$

Percentage of losses in EPANET system = (7.81 m/5940 m) *100

= 0.1314 %

Percentage of losses in WaterGEMS system = (7.8119 m/5940 m) *100

= 0.1315 %

After analysing the losses, it can be easily seen that there is very less difference in both the system. The length of the pipe is enhanced in WaterGEMS due to proper definition of length of pipe of valves and pumps whereas; in EPANET such data is not taken into account.

CHAPTER 7

7.1 Conclusion

In this study, the empirical analysis of the water distribution network in the Ward No.-30 area ofthe South Dumdum Municipality has been carried out using EPANET, WaterGEMS computer-based simulation software for water distribution network. Prelude to the analysis, a review of literature was carried out. The current conditions of water supply and distribution in thestudy have been studied to explore the real need to have a surface water based 24 X 7 watersupplies for the consumers within the study area. Relevant data required for the analysis were collected. In this distribution network minimum residual pressure is 16.5 m. The maximum pressure head of 18.87 m observed at junction J-1 and minimum pressure head of 16.5 m observed at junction J-41. The maximum velocity of 1.5 m/s observed at pipe P-1 and minimum velocity of 0.05 m/s observed at pipe P-22. The maximum head loss of 3.91 m observed at pipe P-1 and minimum head loss of 0.05 m observed at pipe P-22. The results of all the analysis were supported by charts, screen-shots & pictures inadherence to the CPHEEO manual.

With the use of this software one can indeed analyses the network at the desk and can foresee the error, if any, in the design and consequently the changes required to be done in such designs for its successful execution at site. The result reveals that the software used for thedesign has the capability to handle various pipe network problems without changing in model of or mathematical formulation.

Being a comparatively smaller network, results from both the software hardly varies. So either of the software’s may be used for these sort of smaller network.

7.2 Future Scope of Study

It is extremely difficult to develop a theoretical model that accurately simulates every physical phenomenon that can occur in a hydraulic system. Therefore, every hydraulic model involves some approximations and simplifications of the real problem. The field data can be calibrated for the accuracy assessment of the model. The differences between computer model results and actual system measurements are caused by several factors which require detailed emphasis and rigorous study. The case study presented in this report may be magnified for the research and further works whichare not limited to;

- Different scenario analysis to study whether the network is capable to cater the peak demand if all the consumers withdraw water at the same time.
- Extended period simulation with varying demand.
- Unsteady friction coefficients and losses depend on fluid velocities and accelerations.
- These are difficult to predict and calibrate even in laboratory conditions.

However, the efficacy of the both the software’s may be better understood when these will be applied to comparatively bigger network and simulated for extended time.

References

- Engelhardt, M.O., Skipworth, P.J., Savic, D.A., Saul, A.J. and Walters, G.A. (2000). Rehabilitation strategies for water distribution networks: a literature review with a UK perspective. Urban Water, **2**(2), pp. 153-170.

- Ulanicki, B., Bounds, P.L.M., Rance, J.P. and Reynolds, L. (2000). Open and closed loop pressure control for leakage reduction. Urban Water, **2**(2), pp. 105-114.

- Das, S. (2006). Study of pipeline network system and temporal decay of chlorine for the water treatment plant at Dakshin Raipur, West Bengal. M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Das, S. Roy, P.K. and Mazumdar, A. (2008). Analysis of pipe flow and head loss of a modeled network based on EPANET in a water treatment plant at Raipur, West Bengal. Journal of the Institution of Public Health Engineers, India, 2008-09(3), pp. 21-28.

- Izquierdo, J., Montalvo, I., Perez, R. and Herrera, M. (2008). Sensitivity analysis to assess the relative importance of pipes in water distribution networks. Mathematical and Computer Modelling, 48(1-2), pp. 268–278.

- Liana, V. and Vlad, P. (2008). INFOWORKS WS and EPANET V2- Modelling the water distribution networks, UPB Scientific Bulletin, Series D,70(4), pp. 91-102.

- Koppel, T. and Vassiljev, A. (2009). Calibration of a model of an operational water distribution system containing pipes of different age. Advances in Engineering Software, 40(8), pp.659–664.

- Saha, S. (2010). Analysis of pipeline alignment of a Thermal power plant based on pipeline identification with help of Geoinformatics tools (PIGMI) and HAMMER. M.E. Thesis. School of Water Resources Engineering, Jadavpur University: Kolkata, India.

- Arunkumar, M. and Nethaji, M.V.E. (2011). Water Demand Analysis of Municipal Water Supply using EPANET Software. International Journal of Applied Bioengineering, 5(1), pp. 9-19.

- Chandramouli, S. and Malleswararao, P. (2011). Reliability based optimal design of a water distribution network for Municipal water supply. International Journal of Engineering and Technology, 3(1), pp. 9-19.

- Chatterjee, B. (2011). Pipeline transient analysis of Mejia Thermal Power Station, DVC by HAMMER Software. M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Mukherjee, B. (2012). Pipeline design and analysis of distribution network of Dhapa water treatment plant, KMC by EPANET and HAMMER software. M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Mukherjee, B., Das, S. and Mazumdar, A. (2012)- "Comparison of Pipeline Hydraulic Analysis between EPANET and HAMMER software", International Journal of Advances in Science and Technology, 4(6), pp. 52-63.

- Das S., Mukherjee B. and Mazumdar A. (2013). Analysis of Hammer Head at Increased Flow Demand in Pipe Networks: A Case Study. International Review of Mechanical Engineering, 7(4): 757-766.

- Nag, T., Garg, A. (2013). Strategies to Improve Urban Water Delivery in West Bengal, India: An Analysis of Water Institutions and Benchmarking of Water Delivery Organizations, W.P. No. 2013-04-02, Indian Institute of Management Ahmedabad, India.

- Sarkar, P.K. (2014). Extended Period Simulations of Pipeline Distribution Network of Garfa Boosting Station by Hammer and WaterGEMS Softwares. M.E. Thesis School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Mandal, S. (2015). Pipeline analysis of ash slurry disposal system of (2×520) MW Thermal Power Station by HAMMER software. M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Das M., (2015). Design of Landscape Irrigation System by AutoCAD Software and Simulations of Pipeline Distribution Network of a proposed site by Hammer and WaterGEMS Softwares. *M.E. Thesis*. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Mala-Jetmarova, H., Barton, A. and Bagirov, A. (2015). A history of water distribution systems and their optimisation, Faculty of Science and Technology, Federation University Australia, Mt Helen Campus, University Drive, Ballarat, Victoria 3350, Australia.

- Mukherjee, B., Das, S., Mazumdar, A. and Sarkar, P. (2015a). Management in Real Time Operation of Drinking Water Pipeline Distribution Network: A case study. 47th IWWA Annual Convention, pp. 298-303.

- Mukherjee, B., Das, S. and Mazumdar, A. (2015b). Transient Analysis of a Pipeline Network for Drinking Purpose in Assam, India. Journal of the Institution of Public Health Engineers, India, 2014-15 (4), pp. 26-32.

- Ganguly, S. (2016). Hydraulic Analysis for a Composite Piped Water Supply Network using EPANET Software: A Case Study. M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Halder, K. (2016). Comparison between EPANET and LOOP Software's using a Gravity Flow Water Supply Network. M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Mondal A. (2017). Analysis of a Potable Water Supply Pipeline at Guwahati using WaterGEMS Software, M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Pulai, S. (2017). Model Study of a Water Distribution Pipeline System under Anandapur Head Work in Kolkata through EPANET Software, M.E. Thesis. School of Water Resources Engineering, Jadavpur University, Kolkata: India.

- Mukherjee, B., Das, S. and Mazumdar, A. (2017a). Stress analysis of smart support system for a steam pipeline of Sagardighi Thermal Power Plant, West Bengal, India. International Journal of Innovative and Emerging Research in Engineering, **4**(1), pp. 137-146.

- Mukherjee, B., Das, S. and Mazumdar, A. (2017b). Simulation of Composite Piped Water Supply by EPANET and LOOP Softwares: A Case Study. Journal of the Institution of Public Health Engineers, India, XXXXV (1), pp. 44-54.

- Das, S., Mukherjee, B. and Mazumdar, A. (2018). Comparison of outcomes through EPANET and LOOP softwares using a gravity flow water supply network at East Medinipur in West Bengal. Journal of the Indian Chemical Society, 95(March), pp. 313-324.

- Das, P. (2019). Simulation of clearwater pipeline network of bhangar ii boosting station in west bengal using epanet and watergems softwareS. M.E. Thesis School of Water Resources Engineering, Jadavpur University, Kolkata, India.

- Goswami, U. (2020). hydraulic modeling of a rural drinking water distribution network in bankura using WaterGEMS Softwares. M.E. Thesis School of Water Resources Engineering, Jadavpur University, Kolkata, India.

- The United Nations World Water Development Report 2020, WATER AND CLIMATECHANGE, pp 10-30

- The dimensions of water security - Latin American Water Funds Partnership, https://www.iberdrola.com/

- Strategic Plan – 2011- 2022, Department of Drinking Water and Sanitation – Rural Drinking Water, "Ensuring Drinking Water Security in Rural India", Department of Drinking Water and Sanitation Ministry of Rural Development Government of India, pp 11-41

- Manual on Water Supply and Treatment, 1999, CPHEEO, Chapter-2, pp 4 -13 and Chapter-10, pp 359-361.

- Vision Plan 2020, Directorate of Public Health Engineering, Govt. of West Bengal, pp05-13, Bentley WaterGEMS V8i User's Guide

Appendix – 1

1. Junction Details in WaterGEMS Software:

ID	Label	Elevation (m)	Demand (L/s)	Hydraulic Grade (m)	Pressure (m H2O)
42	J-1	4	5.09	22.907	18.87
50	J-2	4	1.09	22.892	18.85
44	J-3	4	6.51	22.631	18.59
46	J-4	4	2.58	22.478	18.44
48	J-5	4.001	1.68	22.427	18.39
52	J-6	4.015	7.53	21.933	17.88
54	J-7	4.002	2.53	21.868	17.83
56	J-8	4.025	2.52	21.767	17.71
84	J-9	4.057	2.83	21.668	17.58
87	J-10	4.1	1.62	21.622	17.49
89	J-11	4.13	0.92	21.613	17.45
82	J-12	4.15	2.06	21.62	17.43
58	J-13	4.003	3.63	21.711	17.67
74	J-14	4.12	3.51	21.617	17.46
91	J-15	4.004	1.41	21.679	17.64
97	J-16	4.016	0.92	21.67	17.62
76	J-17	4.06	4.01	21.534	17.44
93	J-18	4.089	1.85	21.638	17.51
95	J-19	4.039	0.86	21.63	17.56
78	J-20	4.057	4.35	21.413	17.32
80	J-21	4.19	1.14	21.396	17.17
99	J-22	4.005	8.04	21.539	17.5
103	J-23	4.15	7.16	21.394	17.21
105	J-24	4	3.51	21.294	17.26
60	J-25	4	8.11	21.548	17.51
109	J-26	4.014	6.34	21.22	17.17
107	J-27	4.019	2.39	21.156	17.1
111	J-28	4.075	8.3	21.114	17
113	J-29	4.09	1.63	21.068	16.94
115	J-30	4.081	8.07	20.968	16.85
117	J-31	4.098	1.56	20.927	16.8
119	J-32	4.068	8.2	20.898	16.8
70	J-33	4.035	8.95	21.686	17.62
121	J-34	3.99	1.66	20.848	16.82
123	J-35	4.017	2.05	20.83	16.78
125	J-36	4.034	7.24	20.802	16.73
62	J-37	4.12	6.62	21.413	17.26

ID	Label	Elevation (m)	Demand (L/s)	Hydraulic Grade (m)	Pressure (m H2O)
101	J-38	4.001	8.14	21.274	17.24
147	J-39	4.05	7.37	21.134	17.05
127	J-40	4.056	10.58	20.631	16.54
129	J-41	4.091	1.51	20.626	16.5
144	J-42	4.083	6.76	21.026	16.91
141	J-43	4.003	6.37	20.929	16.89
136	J-44	3.995	3.99	20.853	16.82
134	J-45	3.985	5.62	20.844	16.83
131	J-46	4.019	7.73	20.807	16.75
139	J-47	4.005	2.96	20.597	16.56
64	J-48	4.17	7.24	21.464	17.26
66	J-49	4.17	11.72	21.358	17.15
68	J-50	4.079	3.38	20.984	16.87
152	J-51	4	7.5	20.854	16.82

2. **Pipe Details in WaterGEMS Software:**

ID	Label	Length (Scaled) (m)	Start Node	Stop Node	Dia (mm)	Mate-rial	Hazen Williams C	Flow (L/s)	Velocity (m/s)	Head loss Grad-ient (m/km)
43	P-1	62	T-1	J-1	450	DI-K7	140	239.34	1.5	3.913
45	P-2	74	J-1	J-3	450	DI-K7	140	233.17	1.47	3.728
47	P-3	45	J-3	J-4	100	DI-K7	140	4.26	0.54	3.419
49	P-4	83	J-4	J-5	100	DI-K7	140	1.68	0.21	0.61
51	P-5	54	J-1	J-2	100	DI-K7	140	1.09	0.14	0.273
53	P-6	204	J-3	J-6	450	DI-K7	140	222.4	1.4	3.416
55	P-7	19	J-6	J-7	400	DI-K7	140	163.3	1.3	3.421
57	P-8	35	J-7	J-8	400	DI-K7	140	148.17	1.18	2.858
59	P-9	23	J-8	J-13	400	DI-K7	140	137.8	1.1	2.498
61	P-10	147	J-13	J-25	250	DI-K7	140	25.76	0.52	1.105
63	P-11	183	J-25	J-37	250	DI-K7	140	20.75	0.42	0.74
65	P-12	97	J-48	J-37	200	DI-K7	140	9.68	0.31	0.534
67	P-13	41	J-48	J-49	200	DI-K7	140	22.6	0.72	2.569
69	P-14	168	J-49	J-50	100	DI-K7	140	3.38	0.43	2.231
71	P-15	221	J-33	J-48	300	DI-K7	140	39.52	0.56	1.004
72	P-16	73	J-33	J-25	100	DI-K7	140	3.11	0.4	1.907
73	P-17	151	J-6	J-33	300	DI-K7	140	51.57	0.73	1.643
75	P-18	71	J-7	J-14	150	DI-K7	140	12.59	0.71	3.532

ID	Label	Length (Scaled) (m)	Start Node	Stop Node	Dia (mm)	Mate-rial	Hazen Williams C	Flow (L/s)	Velocity (m/s)	Head loss Grad-ient (m/km)
77	P-19	40	J-14	J-17	150	DI-K7	140	9.5	0.54	2.095
79	P-20	159	J-17	J-20	150	DI-K7	140	5.49	0.31	0.759
81	P-21	57	J-20	J-21	100	DI-K7	140	1.14	0.15	0.298
83	P-22	63	J-12	J-14	100	DI-K7	140	0.42	0.05	0.046
85	P-23	39	J-9	J-12	100	DI-K7	140	2.48	0.32	1.252
86	P-24	67	J-8	J-9	150	DI-K7	140	7.86	0.44	1.474
88	P-25	35	J-9	J-10	100	DI-K7	140	2.55	0.32	1.318
90	P-26	46	J-10	J-11	100	DI-K7	140	0.92	0.12	0.201
92	P-27	10	J-13	J-15	350	DI-K7	140	108.41	1.13	3.07
94	P-28	14	J-15	J-18	350	DI-K7	140	106.08	1.1	2.949
96	P-29	43	J-18	J-19	100	DI-K7	140	0.86	0.11	0.176
98	P-30	46	J-15	J-16	100	DI-K7	140	0.92	0.12	0.2
100	P-31	35	J-18	J-22	350	DI-K7	140	103.37	1.07	2.811
102	P-32	296	J-22	J-38	150	DI-K7	140	6	0.34	0.896
104	P-33	68	J-22	J-23	350	DI-K7	140	89.33	0.93	2.145
106	P-34	30	J-23	J-24	300	DI-K7	140	75.77	1.07	3.351
108	P-35	118	J-24	J-27	100	DI-K7	140	2.39	0.3	1.169
110	P-36	26	J-24	J-26	300	DI-K7	140	69.88	0.99	2.884
112	P-37	52	J-26	J-28	300	DI-K7	140	57.82	0.82	2.031
114	P-38	81	J-28	J-29	100	DI-K7	140	1.63	0.21	0.576
116	P-39	53	J-28	J-30	250	DI-K7	140	42.17	0.86	2.751
118	P-40	77	J-30	J-31	100	DI-K7	140	1.56	0.2	0.531
120	P-41	55	J-30	J-32	250	DI-K7	140	27.8	0.57	1.272
122	P-42	83	J-32	J-34	100	DI-K7	140	1.66	0.21	0.6
124	P-43	66	J-32	J-35	200	DI-K7	140	13.81	0.44	1.032
126	P-44	36	J-35	J-36	200	DI-K7	140	11.75	0.37	0.766
128	P-45	323	J-36	J-40	150	DI-K7	140	4.52	0.26	0.529
130	P-46	75	J-40	J-41	150	DI-K7	140	1.51	0.09	0.07
132	P-47	203	J-32	J-46	150	DI-K7	140	4.13	0.23	0.449
133	P-48	127	J-46	J-40	150	DI-K7	140	7.58	0.43	1.38
135	P-49	54	J-45	J-46	200	DI-K7	140	11.18	0.36	0.698
137	P-50	10	J-44	J-45	200	DI-K7	140	12.06	0.38	0.803
138	P-51	215	J-30	J-45	150	DI-K7	140	4.74	0.27	0.577
140	P-52	147	J-44	J-47	100	DI-K7	140	2.96	0.38	1.742
142	P-53	41	J-43	J-44	200	DI-K7	140	19.01	0.61	1.866
143	P-54	226	J-28	J-43	150	DI-K7	140	5.73	0.32	0.821
145	P-55	49	J-42	J-43	200	DI-K7	140	19.65	0.63	1.983
146	P-56	237	J-26	J-42	150	DI-K7	140	5.71	0.32	0.816
148	P-57	258	J-23	J-39	150	DI-K7	140	6.4	0.36	1.01

ID	Label	Length (Scaled) (m)	Start Node	Stop Node	Dia (mm)	Mate-rial	Hazen Williams C	Flow (L/s)	Velocity (m/s)	Head loss Grad-ient (m/km)
149	P-58	59	J-38	J-39	200	DI-K7	140	21.67	0.69	2.378
150	P-59	49	J-37	J-38	200	DI-K7	140	23.81	0.76	2.83
151	P-60	49	J-39	J-42	200	DI-K7	140	20.7	0.66	2.185
153	P-61	372	J-49	J-51	150	DI-K7	140	7.5	0.42	1.353

3. Tank Details in WaterGems Software:

ID	Label	Elevation (Base) (m)	Elevation (Minimum) (m)	Elevation (Initial) (m)	Elevation (Maximum) (m)	Flow (Out net) (L/s)	Hydraulic Grade (m)
154	T-1	23	23.1	23.15	28.5	239.34	23.15

Appendix – 2

1. Junction Details in EPANET software

Node ID	Elevation (m)	Base Demand (LPS)	Demand	Head (m)	Pressure(m)
J-1	4	1.695164	5.09	22.91	18.87
J-3	4	2.168808	6.51	22.63	18.59
J-4	4	0.859872	2.58	22.48	18.44
J-5	4.001	0.559888	1.68	22.43	18.39
J-2	4	0.362408	1.09	22.89	18.85
J-6	4.015	2.510545	7.53	21.93	17.88
J-7	4.002	0.844805	2.53	21.87	17.83
J-8	4.025	0.839484	2.52	21.77	17.71
J-13	4.003	1.209947	3.63	21.71	17.67
J-25	4	2.704967	8.11	21.55	17.51
J-37	4.12	2.206821	6.62	21.41	17.26
J-48	4.17	2.413867	7.24	21.46	17.26
J-49	4.17	3.905467	11.72	21.36	17.15
J-50	4.079	1.12732	3.38	20.98	16.87
J-33	4.035	2.983098	8.95	21.69	17.62
J-14	4.12	1.170377	3.51	21.62	17.46
J-17	4.06	1.335901	4.01	21.53	17.44
J-20	4.057	1.449739	4.35	21.41	17.32
J-21	4.19	0.380247	1.14	21.4	17.17
J-12	4.15	0.686241	2.06	21.62	17.43
J-9	4.057	0.944465	2.83	21.67	17.58
J-10	4.1	0.541271	1.62	21.62	17.49
J-11	4.13	0.307322	0.92	21.61	17.45
J-15	4.004	0.469156	1.41	21.68	17.64
J-18	4.089	0.616146	1.85	21.64	17.51
J-19	4.039	0.286194	0.86	21.63	17.56
J-16	4.016	0.306516	0.92	21.67	17.62
J-22	4.005	2.679113	8.04	21.54	17.5
J-38	4.001	2.71323	8.14	21.27	17.24
J-23	4.15	2.386204	7.16	21.39	17.21
J-24	4	1.169085	3.51	21.29	17.26
J-27	4.019	0.795373	2.39	21.16	17.1
J-26	4.014	2.11481	6.34	21.22	17.17
J-28	4.075	2.765701	8.3	21.11	17
J-29	4.09	0.542612	1.63	21.07	16.94
J-30	4.081	2.691129	8.07	20.97	16.85
J-31	4.098	0.519258	1.56	20.93	16.8

Node ID	Elevation (m)	Base Demand (LPS)	Demand	Head (m)	Pressure(m)
J-32	4.068	2.732095	8.2	20.9	16.8
J-34	3.99	0.554671	1.66	20.85	16.82
J-35	4.017	0.68408	2.05	20.83	16.78
J-36	4.034	2.41186	7.24	20.8	16.73
J-40	4.056	3.528305	10.58	20.63	16.54
J-41	4.091	0.504502	1.51	20.63	16.5
J-46	4.019	2.577527	7.73	20.81	16.75
J-45	3.985	1.873822	5.62	20.84	16.83
J-44	3.995	1.329531	3.99	20.85	16.82
J-47	4.005	0.986406	2.96	20.6	16.56
J-43	4.003	2.122603	6.37	20.93	16.89
J-42	4.083	2.254232	6.76	21.03	16.91
J-39	4.05	2.457788	7.37	21.13	17.05
J-51	4	2.500026	7.5	20.85	16.82
Tank T-1	23	#N/A	-239.34	23.15	0.15

2. Pipe Details in EPANET software

Link ID	Length (m)	Dia mm	Rough-ness	Friction Factor	Flow (LPS)	Velocity (m/s)	Unit Head loss (m/Km)	Loss in Pipes	Status
P-1	62	450	140	0.015	239.34	1.5	3.91	0.2431	Open
P-2	74	450	140	0.015	233.17	1.47	3.73	0.2761	Open
P-3	45	100	140	0.023	4.26	0.54	3.42	0.1527	Open
P-4	83	100	140	0.026	1.68	0.21	0.61	0.0508	Open
P-5	54	100	140	0.028	1.09	0.14	0.27	0.0146	Open
P-6	204	450	140	0.015	222.4	1.4	3.42	0.6982	Open
P-7	19	400	140	0.016	163.3	1.3	3.42	0.0651	Open
P-8	35	400	140	0.016	148.17	1.18	2.86	0.1015	Open
P-9	23	400	140	0.016	137.8	1.1	2.5	0.0563	Open
P-10	147	250	140	0.02	25.76	0.52	1.1	0.1622	Open
P-11	183	250	140	0.02	20.75	0.42	0.74	0.1352	Open
P-12	97	200	140	0.022	9.68	0.31	0.53	0.0514	Open
P-13	41	200	140	0.019	22.6	0.72	2.57	0.1064	Open
P-14	168	100	140	0.024	3.38	0.43	2.23	0.3742	Open
P-15	221	300	140	0.019	39.52	0.56	1	0.221	Open
P-16	73	100	140	0.024	3.11	0.4	1.91	0.1386	Open
P-17	151	300	140	0.018	51.57	0.73	1.64	0.2468	Open
P-18	71	150	140	0.02	12.59	0.71	3.53	0.2514	Open

Link ID	Length (m)	Dia mm	Rough-ness	Friction Factor	Flow (LPS)	Velocity (m/s)	Unit Head loss (m/Km)	Loss in Pipes	Status
P-19	40	150	140	0.021	9.5	0.54	2.1	0.0833	Open
P-20	159	150	140	0.023	5.49	0.31	0.76	0.121	Open
P-21	57	100	140	0.028	1.14	0.15	0.3	0.017	Open
P-22	63	100	140	0.032	0.42	0.05	0.05	0.0032	Open
P-23	39	100	140	0.025	2.48	0.32	1.25	0.0485	Open
P-24	67	150	140	0.022	7.86	0.44	1.47	0.0984	Open
P-25	35	100	140	0.025	2.55	0.32	1.32	0.046	Open
P-26	46	100	140	0.029	0.92	0.12	0.2	0.0091	Open
P-27	10	350	140	0.017	108.41	1.13	3.07	0.0311	Open
P-28	14	350	140	0.017	106.08	1.1	2.95	0.0415	Open
P-29	43	100	140	0.029	0.86	0.11	0.18	0.0077	Open
P-30	46	100	140	0.029	0.92	0.12	0.2	0.0091	Open
P-31	35	350	140	0.017	103.37	1.07	2.81	0.0985	Open
P-32	296	150	140	0.023	6	0.34	0.9	0.2665	Open
P-33	68	350	140	0.017	89.33	0.93	2.15	0.1454	Open
P-34	30	300	140	0.017	75.77	1.07	3.35	0.1	Open
P-35	118	100	140	0.025	2.39	0.3	1.17	0.1385	Open
P-36	26	300	140	0.017	69.88	0.99	2.88	0.0742	Open
P-37	52	300	140	0.018	57.82	0.82	2.03	0.1058	Open
P-38	81	100	140	0.026	1.63	0.21	0.58	0.0468	Open
P-39	53	250	140	0.018	42.17	0.86	2.75	0.1456	Open
P-40	77	100	140	0.026	1.56	0.2	0.53	0.041	Open
P-41	55	250	140	0.019	27.8	0.57	1.27	0.0704	Open
P-42	83	100	140	0.026	1.66	0.21	0.6	0.0495	Open
P-43	66	200	140	0.021	13.81	0.44	1.03	0.0678	Open
P-44	36	200	140	0.021	11.75	0.37	0.77	0.0277	Open
P-45	323	150	140	0.024	4.52	0.26	0.53	0.1712	Open
P-46	75	150	140	0.028	1.51	0.09	0.07	0.0053	Open
P-47	203	150	140	0.024	4.13	0.23	0.45	0.0913	Open
P-48	127	150	140	0.022	7.58	0.43	1.38	0.1754	Open
P-49	54	200	140	0.022	11.18	0.36	0.7	0.0376	Open
P-50	10	200	140	0.021	12.06	0.38	0.8	0.0083	Open
P-51	215	150	140	0.024	4.74	0.27	0.58	0.1247	Open
P-52	147	100	140	0.024	2.96	0.38	1.74	0.2555	Open
P-53	41	200	140	0.02	19.01	0.61	1.87	0.0762	Open
P-54	226	150	140	0.023	5.73	0.32	0.82	0.1852	Open
P-55	49	200	140	0.02	19.65	0.63	1.98	0.0977	Open
P-56	237	150	140	0.023	5.71	0.32	0.82	0.1943	Open

Link ID	Length (m)	Dia mm	Rough-ness	Friction Factor	Flow (LPS)	Velocity (m/s)	Unit Head loss (m/Km)	Loss in Pipes	Status
P-57	258	150	140	0.023	6.4	0.36	1.01	0.2602	Open
P-58	59	200	140	0.02	21.67	0.69	2.38	0.1401	Open
P-59	49	200	140	0.019	23.81	0.76	2.83	0.1384	Open
P-60	49	200	140	0.02	20.7	0.66	2.18	0.1074	Open
P-61	372	150	140	0.022	7.5	0.42	1.35	0.5024	Open
TOTAL	5940							7.81	