MODIFIED DESIGN OF EXISTING URBAN RESIDENTIAL WATER DISTRIBUTION NETWORK BASED ON NETWORK APPRAISAL AND RELIABILITY ANALYSIS

A Dissertation submitted in partial fulfillment of the requirement for the Award of degree of

MASTER OF TECHNOLOGY
IN
HYDRAULICS AND WATER ENGINEERING

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CANDIDATE'S DECLARATION

I do hereby certify that the work presented is the report entitled "**MODIFIED DESIGN OF EXISTING URBAN RESIDENTIAL WATER DISTRIBUTION NETWORK BASED ON NETWORK APPRAISAL AND RELIABILITY ANALYSIS**" in the partial fulfillment of the requirements for the award of the degree of "Master of Technology" in Hydraulics & Flood engineering submitted in the Department of Civil Engineering, Delhi Technological University, is an authentic record of my own work carried out from January 2015 to July 2015 under the supervision of Dr. Rakesh Mehrotra (Associate Professor), Department of Civil Engineering. I have not submitted the matter embodied in the report for the award of any other degree or diploma.

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ABSTRACT

Water distribution networks deliver water from its sources to the customers of the network. In Indian cities including metropolitan cities, the water distribution is a major problem. The cities are battling with old water distribution system and the demand has grown beyond the level the system can satisfy. Moreover unequal Head at major nodal points in a water distribution network due to various reasons is adding to the problem. This leaves consumers unsatisfied. The continuous increase in failure of distribution system has reduced the system reliability to a new low. The consumers remain uncertain for the availability of water and reduces their confidence towards existing water distribution network. This leads consumers to harness water through individual bores and hence exploiting water table.

EPANET, hydraulic simulation software is used to analyze Head and Demand at nodes in water distribution networks. Simulation results from EPANET helps us to design a water distribution network which provides an adequate volume of water with adequate head to consumers as per their demands under any condition over a period of time.

Delhi Jal Board is one of the prime water distribution agency in Delhi. A water distribution network for the Pradhan Enclave in area Burari is being examined for hydraulic parameter sufficiency and its reliability under various water demands.

ORGANISATION OF REPORT

The study described in this report provides an approach for assessing the performance and reliability of a selected water distribution network for urban residential area. The report consists of six chapters. Chapter 2 provides a synopsis of a detailed review of the relevant literature. Chapter 3 gives the brief introduction of the study area. Chapter 4 provides the

methodology of the processing of data using EPANET hydraulic simulation software. The reliability of the network under study and associated hydraulic parameters are also depicted. Chapter 5 details the modification of the network. Chapter 6 is about the results and discussion of the hydraulic analysis of the urban residential water distribution network.

KEYWORDS: EPANET, Water Demand, Population Increase, Pumps, Head, Reliability

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LIST ABBREVIATIONS

ACO	Ant Colony Optimization
CPHEEO	Central Public Health and Environmental Engineering Organization
DJB	Delhi Jab Board
EPANET	Environmental Protection Agency's Network
GIS	Geographic Information System
GPCD	Gallons per Capita per Day
HS	Harmony Search
LPCD	Liters per Capita per Day
LPS	Liters per second
MMAS	Max-Min Ant System
MTTF	Mean Time to Failure
MTTR	Mean Time to Repair
PHD	Peak Hourly Demand
SCE	Shuffle Complex Evolution
UGR	Underground Reservoir
WADISCO	Water Distribution Simulation and Optimization
WADSOP	Water Distribution System Optimization Program
WDN	Water Distribution Network
WHO	World Health Organization

Chapter 1

INTRODUCTION

1.1 GENERAL

Water distribution networks are the integral part of the cities to accomplish the water supply needs of people at all times. Water is a precious natural resource and one of the most essential requirements of all living beings. The perception of water availability and its quality is commonly discussed, studied and evaluated by the hydraulic researchers over the past decade. It is believed to be modest owing to various hydraulic, environmental controllable and uncontrollable factors. Many issues are thus apprehended pertaining to the capability, capacity and reliability of these water distribution networks. There are many areas, large and small, with high population growth rate which are not able to access water both in terms of quantity (average demand per capita) and hydraulic head (water Head at the consumer's end). The optimal design, operation and maintenance of WDN is of utmost importance in present times to prevent overexploitation of water resources.

In the Delhi state, Delhi Jal Board (DJB), constituted under Delhi Jal Board Act 1998, is responsible for production and distribution of drinking water as well as for collection, treatment, and disposal of domestic sewage in Delhi. Water supply distribution network has been developed to cover both planned and unplanned areas.

The water supply in most cities of developing countries like India is generally intermittent and erratic, leading to dissatisfaction among the consumers. A reliable supply of water to the residents constitutes an essential component of civic infrastructure of a city. At the launch of the international drinking water supply and sanitation decade (November 1980), Dr H.T.Mahler, Director – General of the World Health Organisation, stated that:

“… the number of water taps per 1000 population is a much better indicator of a country’s health status than the number of hospital beds” (Development Forum, 1987). As the population of Delhi is increasing, and is projected to reach 190 Lakhs by 2017, it is important that there be an adequate water system to support the increasing demand of water.

The DJB is working to increase the water supply and expand the distribution networks by laying new water pipelines, construction of new water treatment plants (Dwarka and Okhla) and construction of UGRs. The water treatment and supply capacity for Delhi has increased steadily from 1956. It was raised from 66 MGD in 1956 to 437 MGD in 1990 and 855 MGD in 2012. Looking to the future, this increasing trend needs to continue in order to support growth and development of Delhi. Providing a water supply for a community involves tapping the most suitable source of water, ensuring that is safe for domestic consumption and then supplying it in adequate quantities. Upgrading the existing water distribution network is the next step towards supporting the imminent growth in demand for water.

The World Health Organization (WHO Study Group, 1987) defines safe water as “… water that does not contain harmful chemical substances or micro-organisms in concentrations that cause illness in any form”. It defines adequate water supply as “… one that provides safe water in quantities sufficient for drinking, and for culinary, domestic, and other household purposes so as to make possible the personal hygiene of member of the household. A sufficient quantity should be available on a reliable, year-round basis near to, or within the household where the water is to be used”. According to the 2011 census, 81% of the 3.41 lakhs of households in Delhi were provided water through piped water supply system. The demand for water per capita is estimated to be 60GPCD. This includes both domestic requirement and non-domestic requirement. The total water demand, for the projected population of 190 Lakhs would be 1140 MGD. In the duration of two years, 2010-2012, water production by DJB was 845 MGD. The sources of water for DJB include the river West Yamuna Canal, Bhakra storage, underground water resources, as well as the Upper Ganga Canal.

Given the adequate supply of water from the numerous water sources, Delhi has a water supply network that is about 11350 kilometers. However, large part of the network is approximately 40 – 50 years old and is prone to leakages at several nodes. According to the estimates of DJB, the total unaccounted distribution losses are nearly 40 percent of the total water supplied. This is quite significant as majority of the water being supplied is unable to reach the consumer. Action is being taken by DJB to solve this problem. They have setup Leak Detection and Investigation (LDI) cell that has replaced 1200 km of old, damaged pipeline with new and sturdy pipelines. This effort needs to continue to ensure minimal loss of scarce water resource. Therefore, the board is currently working towards bringing down the losses to 20 percent of the total water supplied. The board is also investing resources in recycling of backwash water. This resulted in an additional 45 MGD of water from four water treatment plants, without the need of extra water sources. The combined effort will result in more water reaching the consumers. Given that there is adequate supply of water, however, the consumers still don’t get water at adequate heads. Hence, it is essential to improve the water distribution network and make it reliable in order to reduce wastage of water and support the growth of the Delhi.

Reliability of the water distribution networks is of paramount importance considering the growing number of urban consumers and growing demand for piped water to meet their everyday need. The water distributing agencies are, therefore continuously working towards upgrading the existing networks and optimizing the design to a great extent to achieve the highest efficiency at the minimum installation and overall maintenance cost of the water distribution network. A system failure occurs when all the consumer demands are not met.

In developing countries like India, the cities are growing at a fast pace with increasing population. The existing WDNs have become old and are not able to meet their requirement. So, it is essential to investigate, establish and upgrade existing network which satisfies the following conditions (McGhee, 1991): 1) Maintain water quality standard in distribution pipes; 2) establish economic design and layout; 3) deliver adequate quantity of water; 4) maintain required hydraulic pressure; 5) Assure reliability of supply during any period.

A case study of one typical water distribution network is selected for study.

1.2 OBJECTIVES

The present study of urban residential water distribution network under the control of Delhi Jal Board involves the appraisal, analysis for its capacity and reliability estimation to fulfill the water demands at various nodes by applying the EPANET hydraulic simulation software. The main objectives are:

- To check the inadequacies of design of water supply schemes
- To test the strength of network in terms of head and quantity of water the network delivers by varying demand

- To study the implication of growing population on the network
- To determine reliability of WDN for head at nodes and quantity of water reaching the consumer
- To suggest possible modifications in the network to improve its reliability

1.3 SCOPE

- To extend the study to other networks in the country
- To incorporate latest technology and IT sector to make WDN advanced and responsive, making WDN smart
- To develop methods for more accurate estimation of demands at nodes
- To develop more advanced techno-economical solution for a reliable water distribution network

1.4 METHODOLOGY

1. Study location : WDN Pradhan enclave Burari Delhi.
2. Data collection on WDN of selected area includes network layout, age of network, supply duration, basic water demand at the nodes, number of pumps, parameters of installed pumps, pipe type, population etc.
3. Water distribution network reconstructed in EPANET.
4. Obtained pump curve by feeding pump parameters in EPANET.
5. Reliability calculation procedure is established for the study with main focus on head and quantity of water satisfaction for the consumer. Population is also assessed based on geometric incremental method for four decades.

6. Basic demand of 0.219 LPS at each node is given and simulation is run. Then demand is increased till there is negative pressure at any node in the network. Reliability is calculated for each incremental demand.
7. The network is studied for the two cases, 2 pumps/3 pumps running.
8. Based on the result obtained from the study of the network a modification of the network with 24X7 is suggested to increase the reliability of water reaching consumers.

1.4 ASSUMPTIONS

The project assumes that the various fittings in the network function properly, and there is adequate water supply to meet the domestic needs of the occupants in the residential area. In general, a daily per capita water consumption of at least 172 litres and 8 persons per plot is taken as design figure to calculate the domestic water needs. A plot is defined as a piece of land which homes 8 persons and water consumption as 172 LPCD. According to CPHEEO for one storied building, the minimum pressure at ferrule point should be 7m, for 2 storied building it may be 12m and 3 storied building it should be 17m or as stipulated by local bylaws. A residual head of 20 m at least is taken at the farthest node of WDN.

The study also assumes that the topology of the entire area under study as flat with normal conditions which may affect the performance of the network. There is no seismic activity which may deteriorate the performance of fittings in the piped network. There is also no corrosion, water leakages or pipe blocks which may have detrimental affect on the performance of the water distribution network.

CHAPTER 2

LITERATURE REVIEW

The economy and human development are powered by receiving water through WDN. The main components of water supply systems are 1) treatment works; 2) supply network; 3) distribution network. Advancements in the development of effective software tools viz. EPANET, HAMMER, waterCAD and waterGEMS facilitated for the calculations and in-depth analysis of hydraulic parameters of any water distribution network.

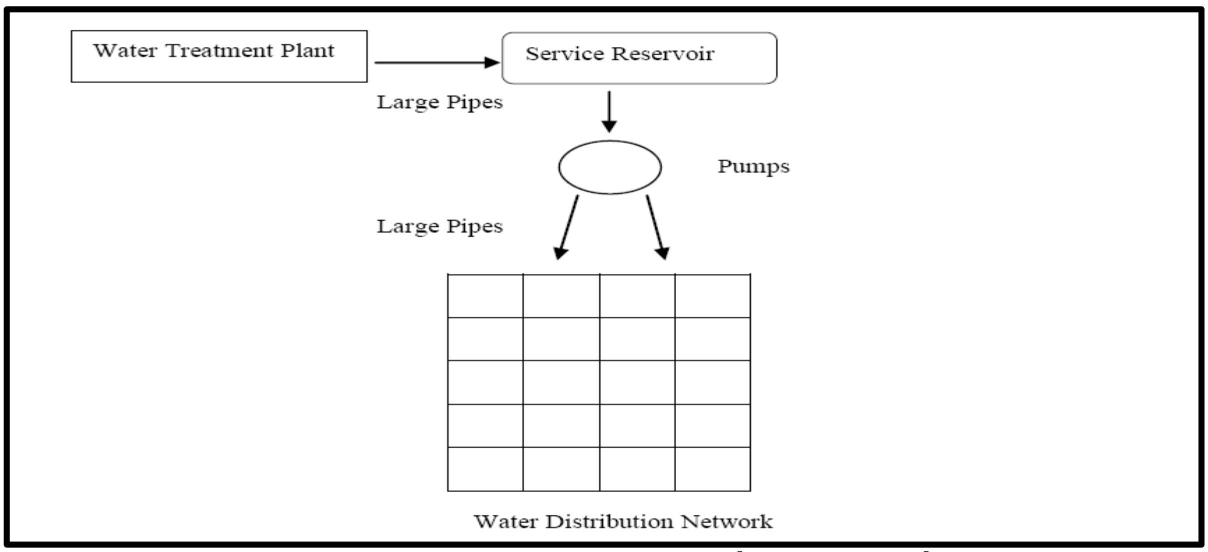


FIGURE 2.1 Water Distribution Network

This chapter reviews the applications of various methods, tools and techniques for the design and performance evaluation models in WDNs. Section 2.1 presents several models, techniques and heuristics used for the design of WDNs. A literature survey on operational strategies of urban WDNs is given in section 2.2. The literature on topics such as failure analysis and prevention, reliability-based design in WDNs are presented in section 2.3. A review of literature on the maintenance, rehabilitation and strengthening WDNs is included in section 2.4.

2.1 DESIGN OF WATER DISTRIBUTION NETWORKS

A WDN must be so designed that it can supply and fulfill the desired water demand of the consumers at sufficient defined pressure. The design involves specifying the sizes of different elements of the distribution network and checking the adequacy of this network (Mays 2000). Significant effort has been placed in developing approaches to solve for designs of WDNs.

There exists large literature on the network design and reporting the application of classical methods (including linear programming, dynamic programming and nonlinear programming). These methods have been used, sometimes at the cost of considerable simplifications of the models. One of the earliest approaches, the linear programming gradient method was proposed by Alperovits and Shamir (1977). Other authors followed this innovative course and introduced alternative derivations from the linear programming-based gradient expressions (Kessler and Shamir 1989, Fujiwara and Khang 1990). These approaches lead to solutions in which pipes have one or two fixed diameter segments. For practical implementation, this type of solution is unrealistic.

The state-of-the-art principles and methods of pipe network are presented by Walski (1985). Su et al (1987) presented a basic framework for a model that can be used to determine the least-cost design of a WDN, subject to conservation of energy and reliability constraints. The limitation of this model is that the resulting pipe diameters may not be commercially available pipe sizes and should be rounded off to the appropriate sizes. These approximated diameters might affect the feasibility of the resulting solution.

A redundancy-constrained minimum-cost design of water distribution networks is presented by Park and Liebman (1993). Redundancy is quantified using the expected shortage due to failure of individual pipes as a measure of reliability that permits incorporation of some considerations of frequency, duration and severity of damage.

Liong and Atiquzzaman (1994) proposed a powerful algorithm, Shuffled Complex Evolution (SCE) linked with EPANET, the network simulation model to solve WDN design problems. Taher and Labadie (1996) developed a prototype decision support system WADSOP (Water Distribution System Optimization Program) to guide water distribution system design and analysis in response to changing water demands, timing, and use patterns, and accommodation of new developments. WADSOP integrates a geographic information system (GIS) for spatial database management and analysis with optimization theory to provide a computer-aided decision support tool for water engineers. Xu and Goulter (1999) proposed a fuzzy linear program optimization approach for the optimal design of WDNs.

Wu and Simpson (2001) applied a Genetic Algorithm to design and rehabilitation of a WDN. Two benchmark problems of water pipeline design and a real WDN are presented to demonstrate the application of the proposed technique.

A Fuzzy linear programming model is formulated by Bhave and Gupta (2004) for minimum cost design of water distribution networks. Future water demands being difficult to predict with any uncertainty are considered as fuzzy demands and modeled by trapezoidal possibility distribution function. The proposed linear programming model avoided iterative procedure and also provided a cheaper solution.

Vairavamoorthy and Ali (2005) proposed a methodology for the design of water distribution systems based on Genetic Algorithms. The objective is to minimize the capital cost, subject to ensuring adequate pressures at all nodes during peak demands. The method involves the use of a pipe index vector to control the genetic algorithm search. The method has been tested on several networks, including networks used for benchmark testing least cost design algorithms, and has been shown to be efficient and robust.

A least-cost design of WDNs under demand uncertainty is developed by Babayan et al (2005). A new approach to quantifying the influence of demand uncertainty is proposed. The original model is reformulated as a deterministic one, and it is coupled with an efficient genetic algorithm solver to find robust and economic solutions.

Geem (2006) presented a cost minimization model using Harmony Search (HS) algorithm for the design of WDNs. The model is applied to five WDNs and the results showed that the Harmony Search-based model is suitable for water network design. Zecchin et al (2006) proposed an advanced Ant Colony Optimization (ACO) algorithm known as Max-Min Ant System (MMAS) for the WDN.

Branch and Bound integer linear programming technique is used by Samani and Mottaghi (2006), for the design of municipal WDNs. The constraints include pipe sizes, reservoir levels, pipe flow velocities and nodal pressures. This procedure helped to design a WDN that satisfies all required constraints with a minimum total cost. It is observed that Genetic Algorithm is more efficient when dealing with a medium-sized network, but other methods outperformed it when dealing with a real complex one.

2.1.1 Network Analysis

2.1.1 a) Flow dependent analysis

The method is widely used for the static and dynamic analysis of the network. The water demands are assumed to be satisfied at all nodes. The analysis provides heads at all demand nodes. The head values are thus compared with the standard values. This is further evaluated if the network performance is satisfactory. The fluctuations in the nodal water demands over a period of time are also sometimes considered for the analysis of the network.

2.1.1 b) Head dependent analysis

Nodal heads are also at times used for the analysis of WDNs. The analysis provides nodal flows as unrestricted values. However, the outflows are to be positive and more than the required values. The network performance is thus evaluated with such predefined conditions for any WDN.

The network performance as highlighted is evaluated for the predefined and set parameters. However, in reality these parameters change and considered as fuzzy parameters. Under such a condition, the analysis is termed as Fuzzy-Value analysis.

2.1.2 Intermittent Vs Continuous WDNs

Intermittent water supply systems are more prevalent in developing countries like India. The existing WDNs are generally extended. The design approach for the intermittent systems receives little or no attention. WDNs are usually designed and built for continuous water supply to consumers for 22 hours of operation, allowing 2 hours spare for maintenance to meet their daily water demand. Yet, the networks are mostly utilized for intermittent water supply to different consumers at different times. The operation of intermittent systems is mostly based on the experience, analysis of the water availability and demand of consumers.

Borjesson and Bobeda (1964), Sridharan and Dutta (1987) and Thorley and Wood (1987), showed that the concepts and methods used are identical with those used for the developed countries. The only differences are the criterion adopted, such as per capita consumption allocation, minimum pipe sizes, residual heads etc. (Haukland, 1991). It is obvious from the literature studied that there is an implicit assumption that the design procedure used in developed countries is appropriate for the developing countries.

Maintenance costs are higher in intermittent supply systems. The pipes are alternately exposed to air and water and thus corrode faster. They need to be repaired and changed more often to control leakage and avoid bursts. Operation costs are also higher as compared to continuous systems since they only need to balance consumption when needed in the continuous systems. (Mcintosh 2003)

A number of service reservoirs, ground level and overhead storage, are provided along the supply network at distribution stations. Direct pumping into the distribution pipes is also done. Usually 80-90% of total storage is provided at the distribution station as underground/ground level storage and the rest as elevated service reservoirs.

The problem arising from intermittent supply systems, which is generally ignored is associated with high levels of contamination. This occurs in networks where there are prolonged periods of interruption of supply due to negligible or zero pressure in the system.

However, the disadvantages are that the systems are not used as designed and causes inconvenience to consumers since water is not available all the time and become contaminated. The wear and tear of component increases and more man power is required. The pumping capacities are higher and storage capacities are to be extended.

In India, the WDNs are though generally designed for the continuous water supply but used as intermittent water supply system and mostly works out to be uneconomical. The existing water distribution networks are seldom abandoned and are rather used with expanded and rehabilitated network as per the water demands. Suribabu et al (2009) illustrated the advantages of converting the existing intermittent supply system to continuous supply for fulfilling the increased demand of water.

2.1.3 Types of WDNs

Water distribution networks could be serial, branched or looped. However, in practice, a suitable combination of such networks is used due to many physical and operational constraints and limitations of water source. Bhave, Gupta and Agrawal (2007) demonstrated the method for strengthening the networks and improve their reliability adopting two-stage iterative method. In practice, the networks may not be provided with the much needed pipeline connections to achieve better reliability but the overall economics and physical conditions at times restrain for dealing with such solutions. A level-one redundant network is considered sufficient enough considering the failure level and pattern of pipes and fittings as well as the overall economics of the water system. The economics for expansion or strengthening of the network is thus always premeditated and is calculated.

The joint problem of layout and component design of WDNs is addressed by Rowel and Barnes (1982). A two-level hierarchically integrated system of models is developed for the layout of both single and multiple source water distribution systems. Bhave and Lam (1983) proposed a Dynamic Programming approach to obtain a less costly distribution layout. An integrated model for the least cost layout and design of WDNs is developed by Goulter and Morgan (1985). The model consists of two linked linear programming formulations. One linear program determines the least cost layout of a WDN given an initial pressure distribution. The other program determines the least cost component design given an initial pipe layout.

A model for the layout of WDNs under single loadings is presented by Awumah et al (1989). The zero-one integer programming model is used to select the pipes that should form the network, while satisfying redundancy and hydraulic requirements. A network component optimization step, using well established design models is then applied to this solution to refine the pipe sizes and pressure heads, thus giving a layout and component optimal solution.

Genetic Algorithm approach is presented by Afshar (2007) for the simultaneous layout and component size of WDNs. The method starts with a predefined maximum layout which includes all possible and useful connections. An iterative design-float procedure is then used to move from the current to a cheaper layout satisfying a predetermined reliability set by the user.

Tanyimboh and Setiadi (2008) presented a multi-criteria maximum-entropy approach to the joint layout, pipe size and reliability of WDNs. The capital cost of the system is taken as the principal criterion, and so the trade-offs between cost, entropy, reliability and redundancy are examined sequentially in a large population of optimal solutions.

2.1.4 Pipe Theory

Several parameters of WDN are involved in the analysis of the network. The pipe length, diameter, roughness coefficient, minor appurtenances, hydraulic gradient levels at source and demand nodes and water supply pattern at source and demand side are generally key factors which are taken into account for the study and analysis of WDNs. Many other relationships such as pipe Head loss are also used for the analysis of the WDN.

When water flows in pipes, energy is lost due to friction and is termed as friction head loss. Head loss due to minor appurtenances is termed as minor head loss. Friction head loss is due to viscosity of the fluid and turbulence of flow. Frictional head loss is larger than other losses hence also called major head loss in pipe networks.

To determine head loss in WDN following empirical relations are used:

i) Darcy Weisbach Formula

$$h_f = \frac{fLV^2}{D2g}$$

Where,

L = length of pipe ‘m’

V = velocity of flow ‘m/s’

D = diameter of pipe ‘m’

f = coefficient of pipe

ii) Hazen Williams Formula

$$h_f = \frac{10.68\ L\ Q^{1.852}}{C_{HW}^{1.852} D^{4.87}}$$

L = length of pipe ‘m’

Q = flow ‘m3/s’

D = diameter of pipe ‘m’

C_{HW} = Hazen William coefficient

iii) Manning Formula

$$h_f = \frac{10.29\ N^2\ L}{D^{16/3}} Q^2$$

L = length of pipe ‘m’

Q = flow ‘m3/s’

D = diameter of pipe ‘m’

N = Manning constant

EPANET has capability to solve WDN with all the three formulas. For analysis in this report Hazen William formula is used.

Hazen William formula	**C value**
Cast Iron	130
Concrete	100-140
Ductile Iron	140
Polyvinyl chloride	150

Ductile Iron pipes are used in the network, hence C = 140 is taken for the pipes.

2.1.5 METHODS FOR THE ANALYSIS OF WDNs

No method is available that can directly solve non-linear equations and therefore, an iterative procedure is necessary. Four methods are commonly used in practice for iterative solutions of equations and thereby, for the analysis of water distribution networks. They are :-

- Hardy Cross Method
- Newton Raphson Method
- Linear Theory Method
- Gradient Method

Hardy Cross Method

Hardy Cross Method was first suggested in 1936. It is a systematic, iterative procedure for network analysis. The approach is based on loop flow correction equations. It is also known as method of balancing heads. Cornish applied the principle to nodal head correction equation that is ΔH equations. His approach is known as method of balancing flows. Both the approaches, method of balancing heads and method of balancing flows, included under one name, the Hardy Cross Method.

Hardy Cross Method attempts to solve the non-linear equations involved in network analysis by making assumptions. The effect of neglecting higher power correction terms is tolerable and a small number of iterations are required for a single loop, even when the initial guess is poor. However, the effect of neglecting adjacent loops and considering only one correction equation at a time is considerable and the number of iterations required for conversions increases as the size of the network increases. Even the partial consideration of the effect of adjacent loops in the Hardy

Cross Method does little to improve the situation. Therefore, instead of considering only one correction equation at a time and solving it, if the effect of all adjacent loops is considered and if all the correction equations are solved simultaneously, conversions can be achieved in a smaller number of iterations.

Newton Raphson Method

Newton Raphson Method expands the non-linear terms in Taylor Series neglecting the residues after two terms and thereby considering only the linear term. The Newton Rapson Method linearizes the non-linear equations through partial differentiation. This method is general and works even when the non-linear equations are transcendental, containing exponential, trigonometric or logarithmic terms. In pipe network analysis, the terms are algebraic, uniform and simple, variables are raised to the same non-unity exponents.

The Newton Raphson Method approximates the value of the root using tangent line. The figure illustrates how after successive iterations the approximation value of x approaches the actual value, r. This method is limited in some cases where the series does not converge.

The series has two cases whereby it does not converge.

i) The approximation, x_2 is further from the actual x_1 value, which is closer to r. This usually happens when the gradient of the function is close to zero.

ii) Sometimes the approximation falls outside the domain of the function. This is illustrated as x_3 falls outside of the domain.

Therefore, given that there are cases when the Newton Method might not work. The most effective rule is to stop when successive approximations for xn and xn+1 are consistent to eight decimal places.

Linear Theory Method

There are some equations involved in pipe network analysis that are non-linear. These equations are linearized since no direct method is available for their solution. An iterative procedure is continued until a satisfactory accuracy of solution is reached. The pipe discharge and nodal head equations are solved for different situations such as, the networks with known pipe resistances, unknown pipe resistances, networks with pumps and valves. Hence, the linear theory method linearizes the non-linear equations by merging a part of the non-linear term in the resistance of the pipes.

Gradient Method

The Gradient Method directly obtains the improve Q and H values instead of computing corrections to them. In an iterative procedure, that is continued till no further improvement is observed. This method also does not need balancing of node flow continuity equations at each node to begin the process. The pipe discharges and nodal heads are taken as the basic unknown in formulating the Q-H equations for different situations as described above. The advantage of Gradient Method over other methods is that the number of iterations required for convergence is minimum, whereas Newton-Raphson method has maximum iterations.

2.2 OPERATION OF WATER DISTRIBUTION NETWORK

2.2.1 Leak Detection and Monitoring

Leakage in water supply networks can represent a large percentage of the total water supplied, depending on the age and deterioration of the system. As a result of water losses and increasing population, urban areas may experience shortages of water. Coulbeck and Orr (1993) presented a reliability perspective of the required systems and activities for control of WDNs with an objective of cost control, quality control and leakage control. The ways in which computers are being used for control purposes are described.

Reis et al (1997) have addressed the problem of appropriate location of control valves in a WDN and their settings via Genetic Algorithm to obtain maximum leakage reduction for given nodal demands and reservoir levels.

A new method for detecting the magnitude of leaks in small residential service zones of a drinking WDN is proposed by Buchberger and Nadimpalli (2004). Several examples, based on observed and simulated pipe flows are presented to demonstrate the application of the leak detection method.

A model to support decision systems regarding quantification, location and opening adjustment of control valves in a network system, with the main objective to minimize pressure and consequently leakage levels is developed by Araujo et al (2006). EPANET model (Rossman 2000) is used for hydraulic network analysis and Genetic algorithm optimization method for pressure control and leakage reduction. A case study is presented is used to show the efficiency of the system by pressure control through valves management.

2.2.2 Pumping Mains Operations

The problem of daily controlling a water distribution network, including pumping devices, and storage capacities, in order to supply the consumers at the lowest cost is addressed by Joalland and Cohen (1980). Discrete Dynamic Programming approach is used to solve the problem. Walski (1982) made an economic comparison of the cost between the lining of main and the associated savings in the energy costs of pumping.

Chen (1988) considered a network without tanks and determined the optimal allocation of supply between the pump sources. Dynamic programming approach was used to select the actual pumps given the continuous outflows. Kim and Mays (1994) described how to minimize the pumping costs by including the rehabilitation action for each main of the hydraulic model as a decision variable. This system cost, subject to a hydraulic constraint formulation is minimized using a nonlinear optimization package. Klempous et al (1997) presented a multilevel two algorithms for finding optimal control in a static water distribution system based on the idea of aggregation technique. The first is a simulation algorithm of the pipeline network and the other is an algorithm for finding an optimal control at the pumping station.

Sakarya and Mays (2000) used a mathematical programming approach for determining the operation of WDN pumps with water quality considerations. The methodology is based upon describing the operation as a discrete time optimal scheduling problem that can be used to determine the operation schedules of the pumps in distribution systems.

Cembrano et al (2000) proposed an optimal supervisory control system which can be used as an efficient means of scheduling water transfer operations to achieve management goals, such as cost minimization and quality improvement. The case study presented showed that significant savings may be achieved by using the optimal control procedures to compute the strategies for pumping and water transfer operation.

An optimal pump scheduling problem in water supply systems is addressed by McCormick and Powell (2003). Medium term maximum demand policies are assumed to be represented in daily scheduling by constraints on power use or by penalty costs. The problem is formulated as a Dynamic Program in which variations in daily demand for water are modeled as a Markov process.

The objective is to find the optimal operating strategy to provide an acceptable level of service to the customer within system constraints, while minimizing the operational cost. Farmani et al (2005) investigated the application of multi-objective evolutionary algorithms to the identification of the payoff characteristics between total cost and reliability of a WDN. The pipe rehabilitation decisions, tank sizing, and pump operation schedules are the decision variables considered.

Broad et al (2010) have proposed the application of meta-models, which can act as a surrogate or substitute for simulation models, for optimal operation of WDNs. The study considered average daily pumping costs and chlorine costs and demonstrated the effectiveness of the approach by applying it to an actual distribution network.

2.3 RELIABILITY OF WATER DISTRIBUTION NETWORK

2.3.1 Failure Analysis and Prevention

There are many reasons which are directly or indirectly responsible for the failure of a WDN. The causes include deterioration of pipes structurally or at times internally due to corrosion effects resulting in reduced hydraulic capacity, degradation of water quality. The pipes eventually break causing failures of WDNs, water-main break and leaks. The number of breaks also vary as per the diameter and time for cast iron and asbestos-concrete mains. There is a linear relationship between pipe breaks and age. Shamir and Howard (1979) applied regression analysis to obtain a relationship for the breakage rate of a pipe as a function of time. Walski and Pellicca (1982) developed an exponential pipe failure prediction model based on investigation of the correlation between pipe age and break frequency. Clark et al (1982) suggested a model that combines two equations, one to predict the time of first break and the second to predict the number of subsequent breaks, which are assumed to grow exponentially over time in an attempt to account for the relative impacts of various external agents.

A practical way of assessing the impact of various pipe failure conditions on water distribution networks is described by Jowitt and Xu (1993). The method assesses the vulnerability of the network to the loss of any particular pipe element, and provides a quantitative estimate of the impact on each nodal demand, and the post-failure utilization of nodal sources and pipe elements. The results of the method can be combined with pipe failure probabilities to provide measures of network reliability. Misiunas et al (2005) have proposed a new continuous monitoring approach for detecting and locating breaks in water pipelines.

Water supply pumps failures and then repair times also cause much anxiety and affect water supply continuity. The deterioration processes as well as pipe structural failure models are therefore very complex and difficult to model. Although significant work has been done in modeling the physical process of pipe deterioration and failure (Doleac et al 1980, Ahammed and Melchers 1994, Rajani et al 1996), the complex processes, lack of pertinent data and highly variable environmental conditions posed severe challenges to these research efforts and a comprehensive model is required. Damelin et al (1972) considered water supply pump inter-failure times and repair times as random variables, and assumed them to be exponentially distributed and lognormally distributed respectively. They studied pumps with different capacities and presented statistical data on mean time to failure (MTTF) and mean time to repair (MTTR). The failure data were based on inter-arrival times of working hours, not including times when the pumps were inoperative due to scheduled outages for maintenance. Shamir and Howard (1981) used these data for computing mean annual number of failures, presuming that pump operates 8400 h per year with some 20 to 44 h per month for preventive maintenance and other scheduled outages.

2.3.2 Reliability Analysis

Reliability analysis of a water distribution system is concerned with measuring its ability to meet consumer's demands in terms of quantity and quality, under normal and emergency conditions. The analytical methods developed for reliability analysis of WDNs considered two states: working condition and shutdown condition. Kettler and

Goulter (1985) introduced reliability constraint – the probability of breakage should not exceed a specified acceptable level- in a least-cost network design model.

Node isolation-simultaneous failure of all links connected to a node or multiple link failure, topology of the network are some of the probabilities which are analysed for improving system reliability and thus a reliable WDN.

Goulter and Coals (1986) developed two quantitative approaches to the incorporation of reliability measures in the least-cost design model with constraint on probability of load isolation of WDNs. In both approaches Linear Programming technique is used to obtain an optimal layout design.

A least-cost methodology is presented by Ormsbee and Kessler (1990) for use in upgrading existing single-source WDNs in order to sustain single component failure. The methodology is developed by casting the network-reliability problem in terms of an explicit level of system redundancy.

The concept of hydraulic reliability has been widely used in determining system reliability (Hobbs and Beim 1988, Duan and Mays 1990, Fujiwara and Silva 1990, Fujiwara and Tung 1991, Fujiwara and Ganesharajah 1993, Gupta and Bhave 1994, 1996, Prasad and Park 2004). Taking into consideration the hydraulic requirements, the basic concept of system reliability commonly perceived is that the water should be provided from sources to each demand point at the desired time, at the desired pressure and at the desired flow rate.

Fujiwara and Silva (1990) proposed a heuristic method to obtain a least-cost WDN design with a given reliability. The method first determines an optimal design without the consideration of reliability. The reliability of the network design is then assessed. An iterative feedback procedure is then employed, which improved the reliability with a small increase in cost.

Quimpo and Wu (1997) suggested a capacity-weighted reliability surface as a tool to assess the condition of deteriorating water supply infrastructure. The method considered the reliability to meet demands at nodes to depend on the reliability and hydraulic capacity of all the network elements leading from the sources. The present project demonstrates an application of EPANET simulation software for reliability analysis of water distribution systems, taking into account the quality of the water supplied, as well as hydraulic reliability considerations. The correlations between nodal demands are shown to significantly increase distribution network costs designed to meet a specific reliability target.

Jacobs and Goulter (1988) made evaluation of various graph theory approaches for their applicability to reliability analysis of water distribution networks. Bao and Mays (1990) proposed a methodology to estimate the nodal and hydraulic reliabilities of water distribution system that account for uncertainties. The method is based on a Monte Carlo simulation which can be used for the assessment of existing systems, the design of new systems or the expansion of existing systems.

Goulter (1995) defined the reliability of water distribution networks as “The ability of a water distribution system to meet the demands that are placed on it where such demands are specified in terms of the flows to be supplied (total volume and flow rate) and the range of pressures at which those flows must be provided.” Many research outcomes, methods, theories and applications are dedicated to estimate and ascertain the hydraulic parameters and overall reliability of water distribution networks (Torii and

Lopez 2012; Islam, Sadiq and others 2014; Atkinson, Farmani and others 2014; Suribabu, Neelakantan and others 2009).

The application of probability theory by modeling the hydraulic parameters of any water distribution network under various scenarios of pipe failures as random variables is adapted to assess its reliability (Torii and Lopez, 2012). The first-order reliability method approach is applied to assess the reliability and analysis of the network. It helps in establishing the locations where redundancy of the pipes is absolutely essential or redundant.

Atkinston et al (2014) assessed the mechanical and hydraulic reliability with varying the performance of various reliability indicators such as resilience index, entropy, minimum surplus Head and performance. The performance of the water distribution network is analyzed for the trading-off of these parameters with the cost and reliability to obtain an optimized solution of the network design and operation such that the consumers are ensured of proper Head and quantity of water.

2.4 MAINTENANCE OF WATER DISTRIBUTION NETWORKS

2.4.1 Rehabilitation

Since the performance of the water distribution system depends on the performance of every single pipe, the decision on pipe rehabilitation or renewal should consider the individual pipe in the context of the network performance. The literature provides a variety of models that can assist the decision maker in scheduling rehabilitation of a water distribution system.

2.4.2 Strengthening and Expansion

When an existing WDN is deficient, several pipes together would need strengthening. The literature related to strengthening of an existing network to enhance its capability; and its expansion to cover additional localities is presented below.

Bhave (1985) used a criterion to increase the capability of the network by optimally selecting links to be strengthened by parallel piping and expansion of the network to cover additional localities.

Boulos and Wood (1990) proposed a method for determining explicitly different parameters for upgrading and enhancing WDNs. The method can directly determine a variety of design parameters such as pipe diameter, pipe length, pump power, pump head, storage level and valve characteristics; and operating parameters such as pump speed, control valve setting and specified flow or pressure requirements.

WADISCO (Water Distribution Simulation and Optimization) computer software is developed by Walski et al (1990) which can be used for optimal design or strengthening and expansion of a network. The algorithm enumerates all possible pipe size combinations within the user specified size ranges and tests them for feasibility against pressure requirements. The program can handle any number of links and nodes; booster pumps and check and pressure reducing valves.

There are many tools for water distribution network planning and management. An optimization model is developed for the management and operation of a large-scale, multi-reservoir water distribution system with preemptive priorities. When the water supply is insufficient to meet the planned demand, appropriate rationing factors are applied to reduce water supply. The model and its user-friendly interface form a decision support system, which can be used to configure a water distribution system to facilitate capacity expansion and reliability studies.

Optimized cost solution of upgrading the existing piped network using genetic algorithm, Naik and Gupta (2009) has suggested. The economics of pipe network optimization is resolved for 15 and 30 years water requirements. The decision of limit of investment would certainly vary for place to place and region to region predominantly due to demographic factors but certainly a helping tool at the design stage. Islam, Sadiq and others (2014) introduced uncertainty in hydraulic parameters such as water volume, pressure and quality and estimated overall reliability of the network by simulating a steady state condition using EPANET hydraulic simulation software. The paper discusses overall reliability of the network by quantifying the uncertainties including the belief factor which is also introduced to indicate the confidence of consumers if the utility would be able to ensure adequate water to their requirements.

CHAPTER 3

STUDY AREA

3.1 LOCATION

An urban residential WDN of Delhi Jal board is taken for study. The network is situated in Burari area which is near Yamuna Biodiversity Park and flood plains of river Yamuna, Northern part of Delhi as shown in the fig 3.1.1 and fig 3.1.2. The residential area is named as Pradhan enclave. The size of this area is 24 hectares. For suppling water to this area, water is first received in an underground reservoir (UGR) from Wazirabad water treatment plant which is approximately 7.0 kms away from the residential area. The UGR is connected to Wazirabad water treatment plant by a pipeline of 700mm diameter. The topography of the overall area is flat with slope in between 1% to 2%. The area Pradhan Enclave is situated at longitude 77˚12'22.9"E and latitude 28˚46'08.4" N. Fig 3.1.3 shows satellite view of Pradhan Enclave.

The climate of the area is humid subtropical. The area experiences high humidity during the rainy season with heavy rainfall. There is high variation between summer and winter temperatures. The average maximum and minimum temperature and total precipitation of the residential area is shown the fig. 3.1.4.

FIGURE 3.1.1 LOCATION OF BURARI AREA IN NORTHERN DELHI

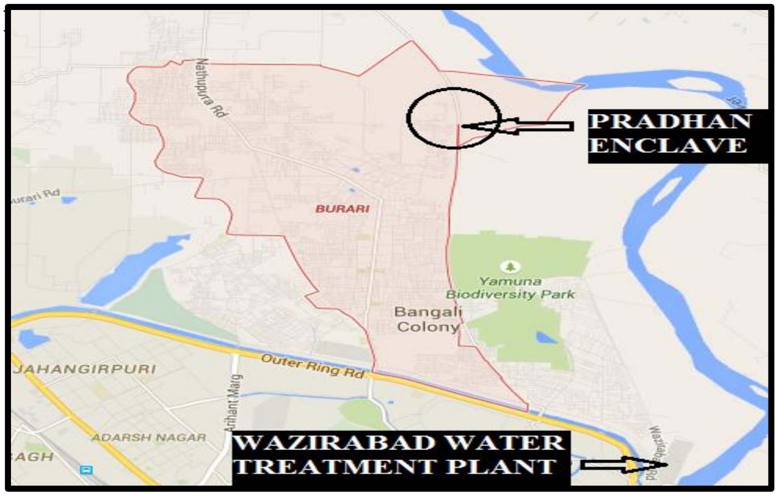


FIGURE 3.1.2 LOCATION OF PRADHAN ENCLAVE IN BURARI AREA

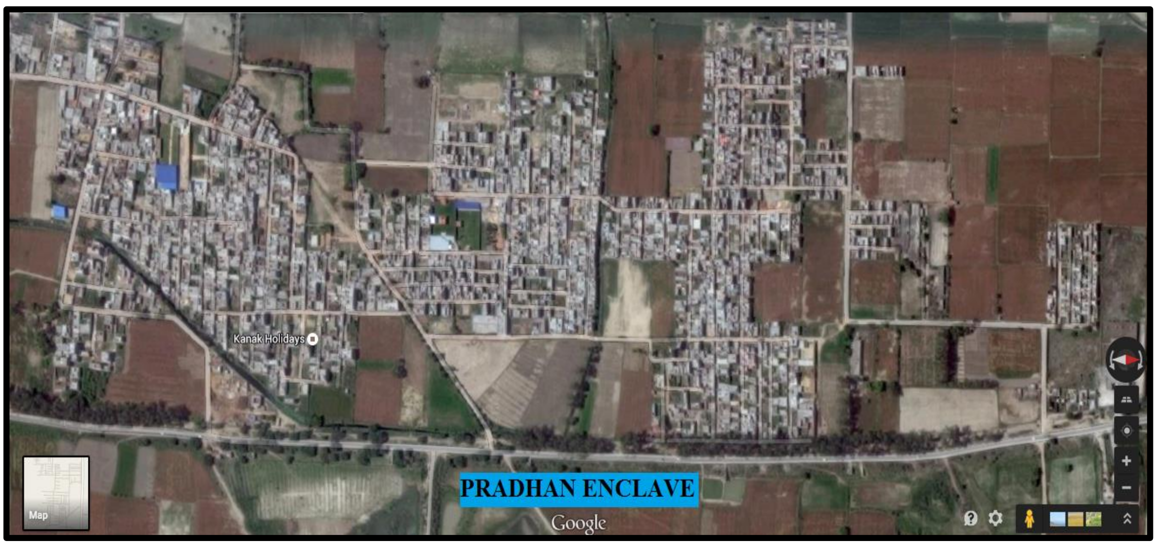
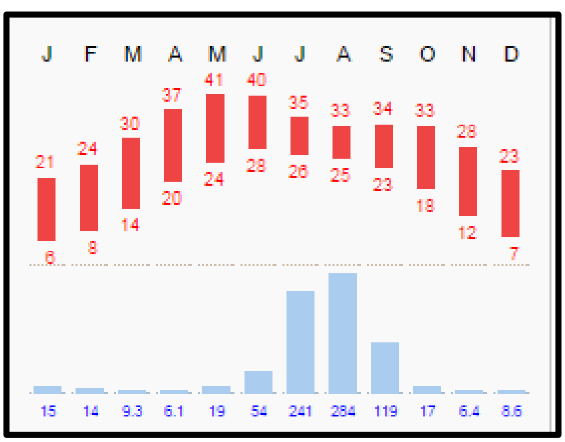


FIGURE 3.1.3 REMOTE SENSING IMAGE: SATELLITE VIEW OF PRADHAN ENCLAVE



AVERAGE MAX. & MIN. TEMPERATURES IN ˚C

PRECIPITATION IN MM

FIGURE 3.1.4 CLIMATE CHART OF BURARI (DELHI)

3.2 SALIENT DATA

The WDN under study is a dead end system. The salient data of the network are:-

ITEM	VALUE
AREA	24 Hectares
NO. OF PUMPS	3
DISCHARGE AND HEAD OF PUMP	2 MGD and 28 m
HORSE POWER OF PUMP	100-150 HP
NO. OF OVER HEAD TANK	Nil
NO. OF UNDER GROUND RESERVOIR (UGR)	1
SIZE/VOLUME OF UGR	10 Million Liters
NO. OF CURRENT CONSUMERS	15480 approx.
NO. OF PIPES	233
MATERIAL OF PIPE	Ductile Iron (DI)
DIAMETER OF THE PIPELINE	100, 200 and 250 mm
AGE OF THE WATER SUPPLY NETWORK	Less than 5 years
DESIGN PERIOD OF THE NETWORK	40 Years
NO. OF NODES	215
NO. OF PLOTS	1935
MAXIMUM HEIGHT ALLOWED TO BE CONSTRUCTED	15 Meters

CHAPTER 4

BURARI WATER DISTRIBUTION NETWORK – A CASE STUDY

4.1 GENERAL

A water distribution system consists of network of pipes, reservoirs, pumps, valves and other hydraulic elements. To analyze the inter-relationship among various components, a water distribution system is transformed into a network representation called water distribution network (WDN) Yang et al... 1996. Its purpose is to supply adequate quantity of water to customers within adequate pressure levels under varying demand conditions.

The improvement and augmentation of water supply for Delhi is one of the sub-mission for urban infrastructure of cities focused under Jawaharlal Nehru National Renewal Mission (JNNURM), city development plan, department of urban development, Govt. of Delhi; 2006 since inadequacy of water supply has been identified as one of the weak area.

4.2 EPANET - WATER DISTRIBUTION MODELING TOOL

The present study employs EPANET hydraulic simulation software, which is a free software developed by USEPA, for analyzing the water distribution network. EPANET assists for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. EPANET provides a fully equipped and extended period of hydraulic analysis that can handle large systems. It also supports the simulation of temporarily varying water demand, constant or variable speed pumps, and the minor head losses for bends and fittings. The modeling provides information such as flows in pipes, pressures at junctions, propagation of a contaminant, chlorine concentration, water age, and even alternative scenario analysis. This helps to compute pumping energy and cost and then model various types of valves, including shutoffs, check pressure regulating and flow control.

EPANET uses the "Gradient Method" to solve the network hydraulics. The results obtained from EPANET are used to check the performance of the network based on its reliability. For analysis in this report Hazen William formula is used. Ductile Iron pipes are used in the network, hence C = 140 is taken for the pipes.

4.3 RELIABILITY

In this study, network reliability is calculated for water head at nodes and quantity of water reaching to our consumers. If it is satisfied that reliability of quantity of water with adequate head is reaching consumers, then their belief in the system will increase. To calculate reliability in the network under study two ways have been adopted:

1. **Head reliability of the network**:

The head of the network is divided into five slabs as shown in fig 4.3.1

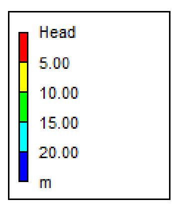


FIG 4.3.1 – HEAD SCALE

To calculate head reliability of the system, nodes in each slab of head are counted for the result obtained from EPANET. Reliability of head in the network is calculated as:

$$\mathbf{R_H = \frac{node\ X\ 0 + node\ X\ 5 + node\ X\ 10 + node\ X\ 15 + node\ X\ 20}{215X20}} \quad \text{…….. 4.3.1}$$

The formula is used when the demand in the network is assumed constant during the hour of supply. If the demand is varying during the hour of supply then reliability is calculated for demand of each period separately and then average of the reliability of WDN is calculated.

2. Reliability of water demand:

According to BIS 1172:2007 135 LPCD to 255 LPCD is the water demand for one person in a day. According to CPHEEO 227LPD was the requirement of water in India during 2011. The economic survey of Delhi 2012 has presented the current water demand at 172 LPCD and 180 LPCD by 2021. For network analysis 172 LPCD is taken i.e. water that should be made available to a consumer having metered supply. Hence, Reliability of quantity of water is calculated as:

$$\mathbf{R_Q} = \frac{\textbf{total amount of water that is made available to one person}}{\textbf{total amount of water should be available to one person}}$$

4.4 POPULATION

The present population of an area can be determined by conducting an official enumeration "census". These official surveys are carried out at intervals of 10 years. This data help planners of WDN to calculate future population of the area under study. There are a number of ways to forecast population. Geometric increase method is adopted in the present study. Accordingly,

$$\boldsymbol{P_n = P_0\left(1+\frac{r}{100}\right)^n} \quad \textbf{.....4.4.1}$$

Where,

P_0 = Initial population

P_n = Future population

r = assumed rate of growth

n = number of decade

The rate of growth is calculated based on the old population data of the country. The census of year 2011 is referred (Table 4.4.1). It gives us the population increase in India for the previous six decades.

S.NO.	CENSUS YEAR	POPULATION	% CHANGE
1	1951	361,088,000	
2	1961	439,235,000	21.6
3	1971	548,160,000	24.8
4	1981	638,329,000	24.7
5	1991	846,387,888	23.9
6	2001	1,028,737,436	21.5
7	2011	1,210,726,932	17.7

TABLE 4.4.1 – CENSUS OF INDIA

Constant ‘r’ is calculated by two methods:

1. Empirical relation

$$r = \sqrt[t]{\frac{P_2}{P_1}} - 1$$

Where $P_1 = intitial\ known\ population$

$P_2 = final\ known\ population$

t = number of decades between $P_1\ and\ P_2$

From the table 4.5.1:

$P_1 = 1{,}028{,}737{,}436$

$P_2 = 1{,}210{,}726{,}932$

t = 1

$$r = \sqrt[1]{\frac{1{,}210{,}726{,}932}{1{,}028{,}737{,}436}} - 1 = 1.1769 - 1 = 0.1769$$

r = 17.69%

2. Average of rate of increase:

$$r = \frac{r_1 + r_2 + r_3 + \cdots r_t}{t}$$

$$r = \frac{21.6 + 24.8 + 24.7 + 23.9 + 21.5 + 17.7}{6} = 22.37$$

'r' value calculated in empirical method matches the data given in table 4.4.1. So 'r' value is taken as 17.69% and population for next 40 years is estimated. Population is than divided evenly on 1935 plots of Pradhan Enclave as shown in Table 4.4.2

Decade	Population	% Increase	New Population	People Per Plot
1	15480	17.69	18218	9
2	18218	17.69	25234	13
3	25234	17.69	41135	21
4	41135	17.69	78916	41

TABLE 4.4.2 – Projected Growth of population with 'r' constant

In census data, it is observed that there is a constant decline in the 'r' value hence a calculation more suitable to actual census data is shown in table 4.4.3. The constant for rate of growth is therefore reduced by 3 percent every decade and results are shown in table 4.4.3.

Decade	Population	% Increase	New Population	People Per Plot
1	15480	17.69	18218	9
2	18218	15	24094	12
3	24094	12	33850	17
4	33850	9	47782	25

TABLE 4.4.3 – Projected Growth of population with ‘r’ changing

4.5 PUMPS

The water in the WDN is pumped from a pump house. Pump is a device that raises the hydraulic head of water. Pump house contains 3 pumps of 2 MGD (87.63 LPS) capacity. The design head of the pump is 28 meters. Pump house has three pumps, 2 are operated at a time and third is a spare.

In EPANET, pumps are described with a pump characteristic curve or pump curve. Pump curve describes the additional head imparted to a fluid as a function of its discharge through the pump. In EPANET, pumps act as links of negligible length with specified upstream and downstream junction nodes.

For EPANET to design a single point pump curve, a combination of head and flow that represents a pump’s desired operating point is provided. EPANET fills the rest of the curve by assuming:

1. A shut off head at zero flow equal to 133% of the design head
2. A maximum flow at zero head equal to twice the design flow

FIG 4.5.1 gives the pump curve produced by EPANET for 2 MGD (87.63 LPS) flow and 28 meter head, which is the operating point of the pump.

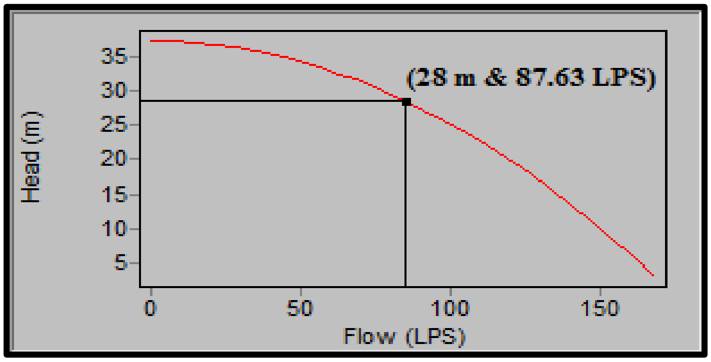


FIGURE 4.5.1 – Pump curve

4.6 DESIGN OF NETWORK IN EPANET

The simulations are performed for current population of approximate 15480 people, living on 1935 plots. Each plot having 8 people if divided equally. There are 215 nodes, each node supplying to 9 plots in the study area to identify the critical nodes and thus zone for drop in Head values for various demand variations.

The water network collected from DJB was originally designed in waterGEM. It is reconstructed in EPANET software. The layout PLAN of Pradhan Enclave WDN, Burari is first fed as INPUT PLAN (FIG 4.6.1) as under:

VIEW ➡ BACKDROP ➡ LOAD

Nodes at the junction of the roads are identified. These nodes are connected with pipes, taking diameter and length of the pipelines from waterGEM data. Roughness coefficient 'C' is taken 140. A WDN "hydraulic model" is thus formed for EPANET. Then the background map is removed so that the network with pipe diameters is clearly visible FIG (4.6.2). The network has further been divided into four zones for the purpose of its appraisal as shown in (FIG 4.6.3).

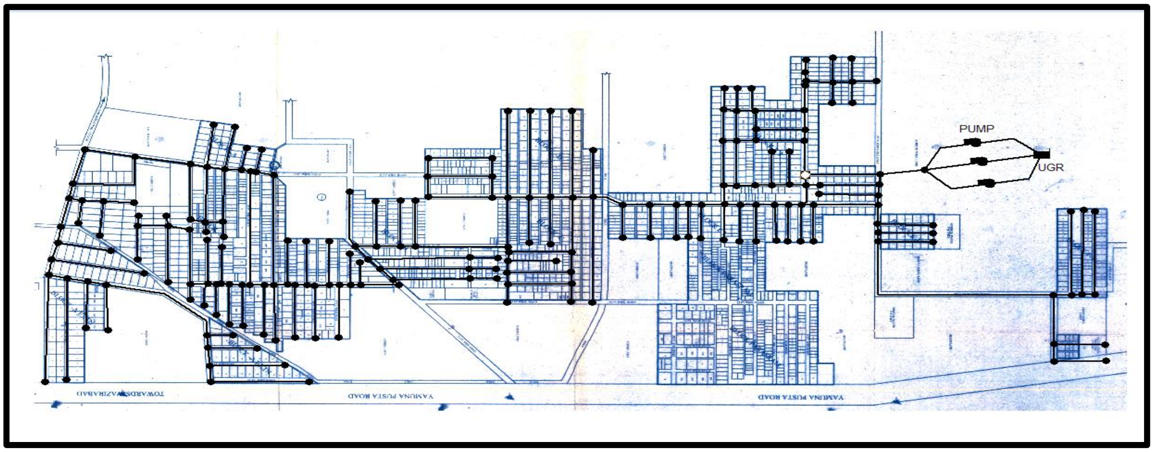


FIGURE 4.6.1 – Layout plan of Pradhan enclave, Burari

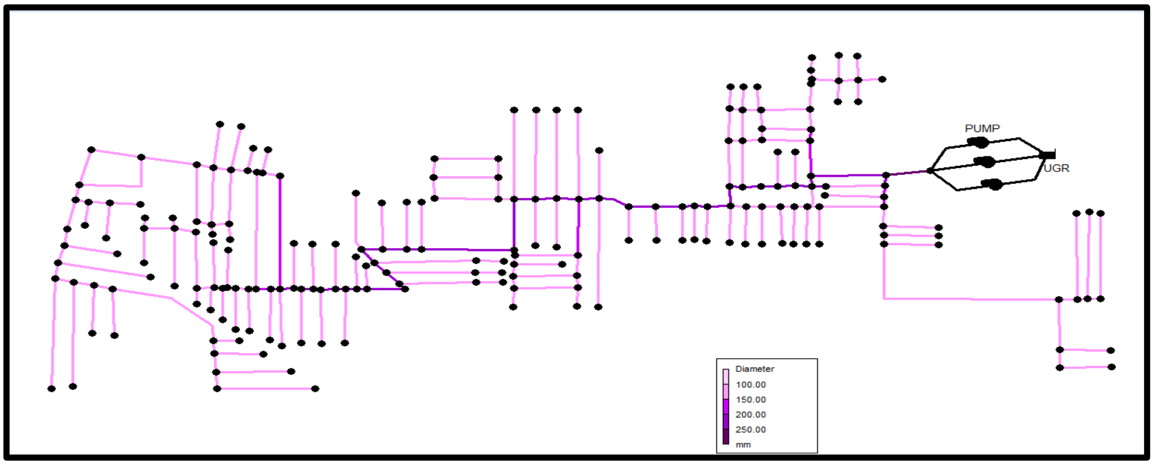


FIGURE 4.6.2 - WDN of Pradhan Enclave, Burari

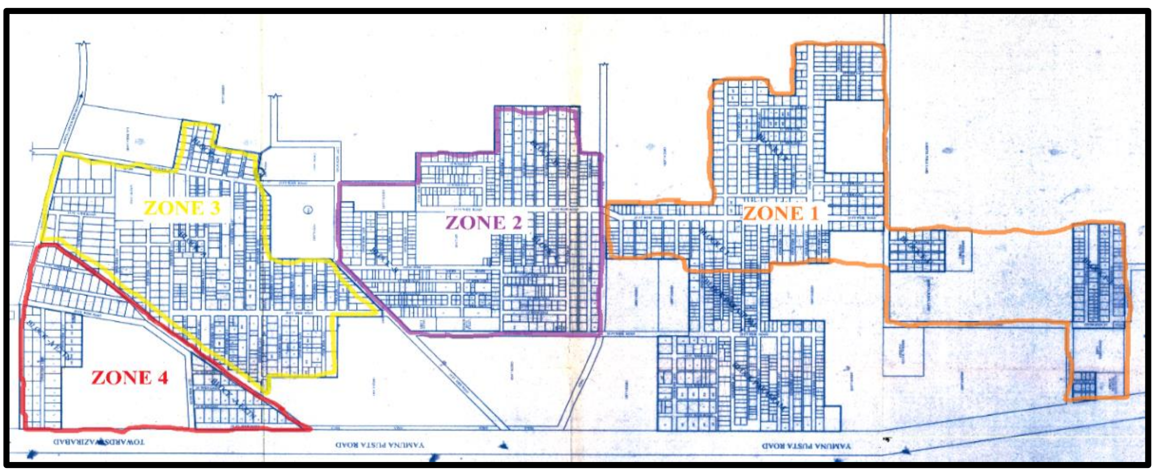


FIGURE 4.6.3 – Zones defined in WDN Pradhan Enclave, Burari

4.7 NETWORK APPRAISAL

The given network is now tested for its strength in terms of head and quantity of water for the current operating conditions at Burari which is intermittent water supply from UGR.

4.7.1 PUMPING FROM UGR WITH INTERMITTENT WATER SUPPLY

This is the case used at Pradhan enclave by DJB currently. The community is supplied water two times a day for 3 hours each (Morning 6:30 to 9:30 and evening 5:30 to 8:30). Pump (3 X 2MGD at 28m) are installed but 2 pumps are currently utilized to pump water through the pipe network. The calculations show that:-

Demand at each node as given by DJB	$\rightarrow$	0.219 L/S
Water delivered at 1 node in 1 hour	$\rightarrow$	0.219 X 3600 = 788.4 L
Water delivered at 215 node in 1 hour	$\rightarrow$	788.4 X 215 = 169506 L
Total water in one day at 215 node	$\rightarrow$	169506 X 6 = 1017036 L
Quantity of water for one person	$\rightarrow$	1017036/15480 = 65.7 L

EPANET simulation is run for 0.219 LPS demand at each node as shown in FIG 4.7.1.1.

Thus, reliability is calculated (refer to 4.3):

1. R_H

   All 215 nodes are above 20m

   $R_H = \frac{(215X20)}{(215X20)} X\ 100 = 100\%$

2. R_Q

   Total available volume of water to a person in a day = 65.7 L

   Total required volume of water for a person in a day = 172 L

   $R_Q = \frac{(65.7)}{(172)} = 38.2\%$

The above calculations indicate that although head at all nodes is fulfilled yet the quantity of water reaching the consumers to fulfill their requirement is low. This would lead the consumers dissatisfied and search for other options to fulfill their water demands.

Keeping the population constant, the water demand is now increased. The volume of water reaching one plot is increased in steps by 50 L and the demand generated at a node is fed in the WDN. The variation of head at nodes by varying demand is observed by EPANET and reliability of head of the network is calculated. Also reliability of quantity of water delivered to a person is calculated. Table 4.7.1 indicates how the network responds when the water demand at Nodes is increased with two pumps operating in parallel.

Table 4.7.1.1 shows that by increasing available volume of water at a plot there is considerable reduction in Head. Initially till 87.5L per person i.e. 700 L of water at a plot, the Head reliability is 100% but as the water demand is increased further there is reduction in Head reliability of the system and hence needed to be addressed.

As the quantity of water demand is increased to 143.75 L per person, nodes of zone 4 have head less than 5m and at 162.5 L per person there is negative pressure in zone 4 as shown in figure 4.7.1.13.

Thus we can derive from table 4.7.1.1 that in current scenario water consumers in zone 1 will always have water with adequate supply and consumers in zone 4 would be dissatisfied since the head of water in zone 4 would fall below 20 meters.

Further, for each demand value at the nodes, the EPANET simulation is run and the results are shown in FIG 4.7.1.1 to 4.7.1.13. We observe that as the water demand at nodes increase, the water head at the nodes in zone 4 starts falling the effect is shown by increasing red nodes. Finally at water demand 0.542, there is negative head in zone 4.

Water per plot (L)	Water per person (L)	Total water for network (L)	Demand at one node (LPS)	Nodes with head below				Nodes with head above 20 (M)	R_H	R_Q	ZONE Affected
				5m	10m	15m	20m				
525.6	65.7	1017036	0.219	0	0	0	0	215	100.00	38.20	
550	68.75	1064250	0.229	0	0	0	0	215	100.00	39.97	
600	75	1161000	0.250	0	0	0	0	215	100.00	43.60	
650	81.25	1257750	0.271	0	0	0	0	215	100.00	47.24	
700	87.5	1354500	0.292	0	0	0	0	215	100.00	50.87	
750	93.75	1451250	0.313	0	0	0	8	207	99.07	54.51	4
800	100	1548000	0.333	0	0	0	30	185	96.51	58.14	4
850	106.25	1644750	0.354	0	0	0	39	176	95.47	61.77	4
900	112.5	1741500	0.375	0	0	6	37	172	94.30	65.41	4
950	118.75	1838250	0.396	0	0	29	38	148	88.84	69.04	3,4
1000	125	1935000	0.417	0	0	32	60	123	85.58	72.67	3,4
1050	131.25	2031750	0.438	0	16	25	63	111	81.28	76.31	3,4
1100	137.5	2128500	0.458	0	30	14	66	105	78.60	79.94	3,4
1150	143.75	2225250	0.479	8	24	38	63	82	71.74	83.58	2,3,4
1200	150	2322000	0.500	28	13	51	44	79	65.47	87.21	2,3,4
1250	156.25	2418750	0.521	31	13	58	38	75	63.14	90.84	2,3,4
1300	162.5	2515500	0.542	**NEGATIVE PRESSURE**							

TABLE 4.7.1.1 – Effect on nodal heads due to varied water demand

To understand much better EPANET simulation for each demand is indicated next. This shows how network reacts under increasing demand and areas affected.

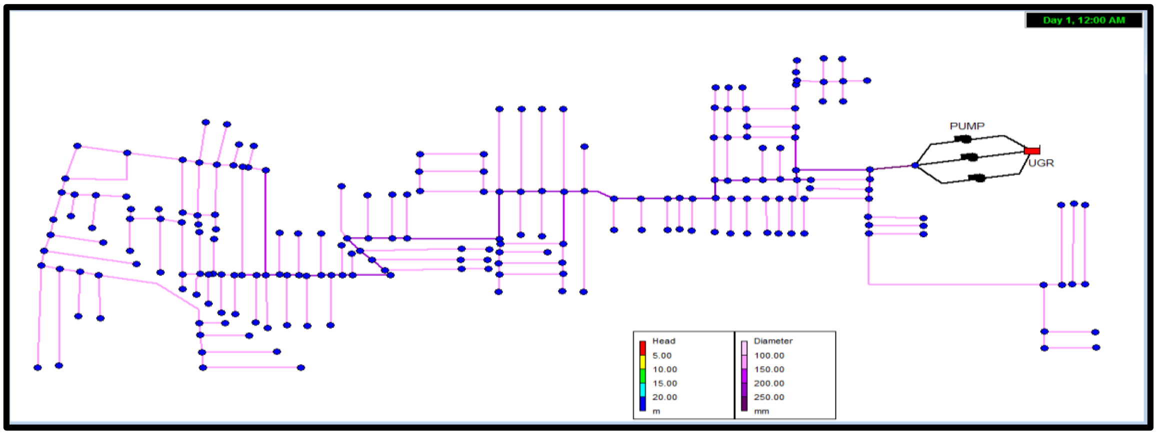


Figure 4.7.1.1 EPANET simulated WDN for water demand 0.219 to 0.292 (LPS)

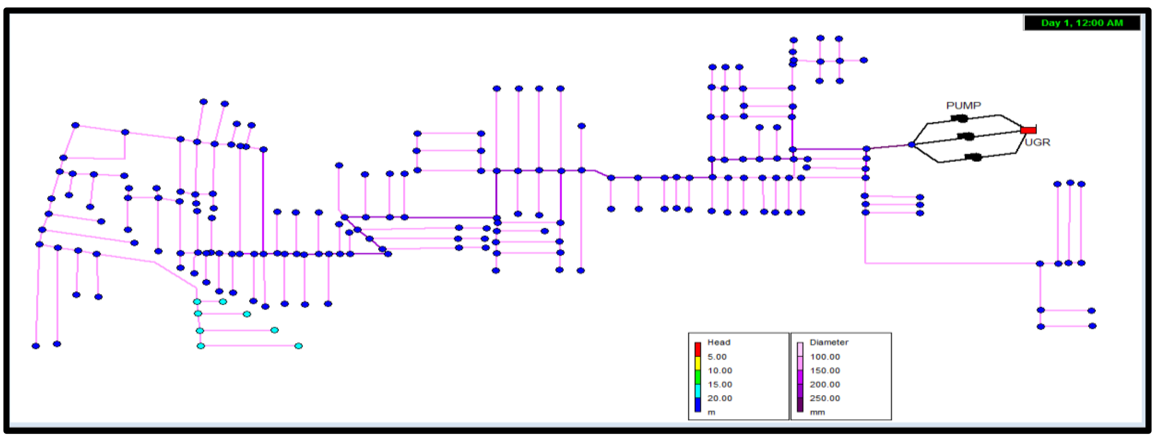


Figure 4. 7.1.2 EPANET simulated WDN for water demand 0.313 (LPS)

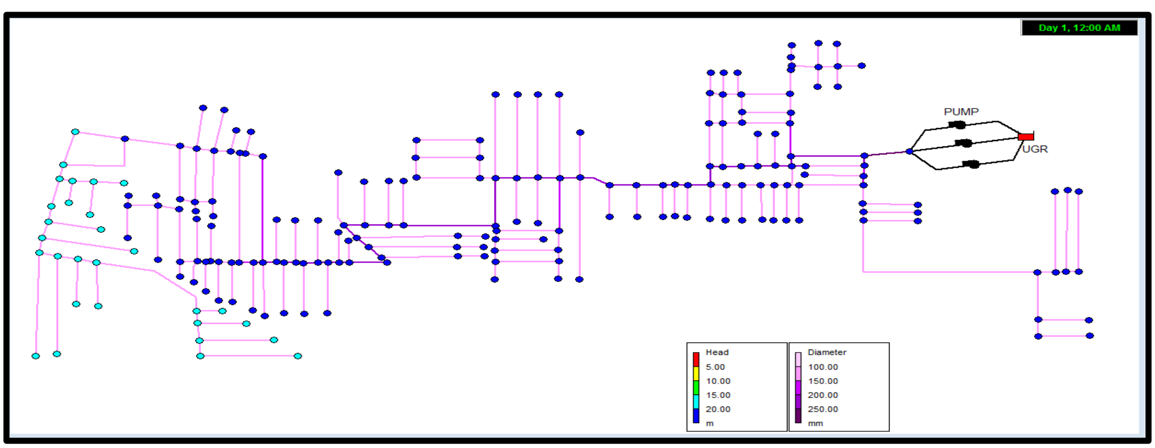


Figure 4.7.1.3 EPANET simulated WDN for water demand 0.333 (LPS)

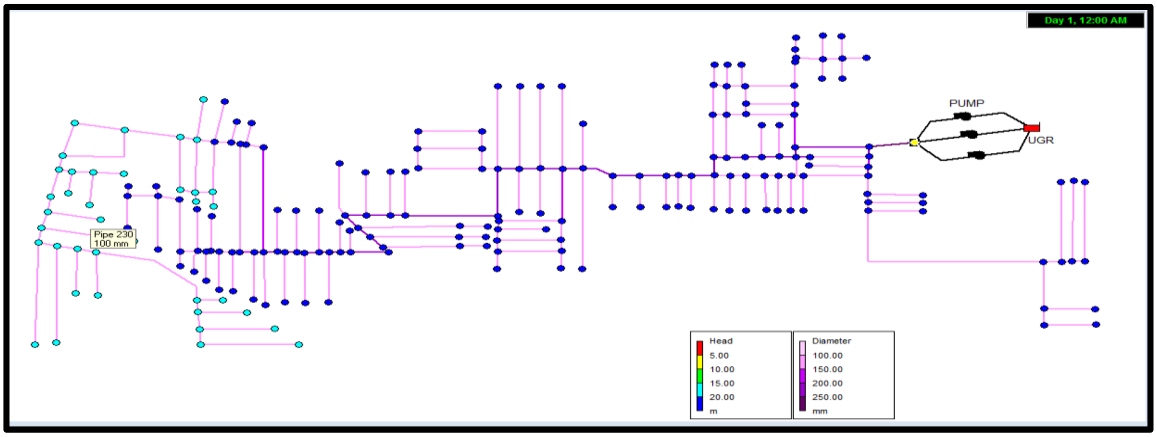


Figure 4.7.1.4 EPANET simulated WDN for water demand 0.354 (LPS)

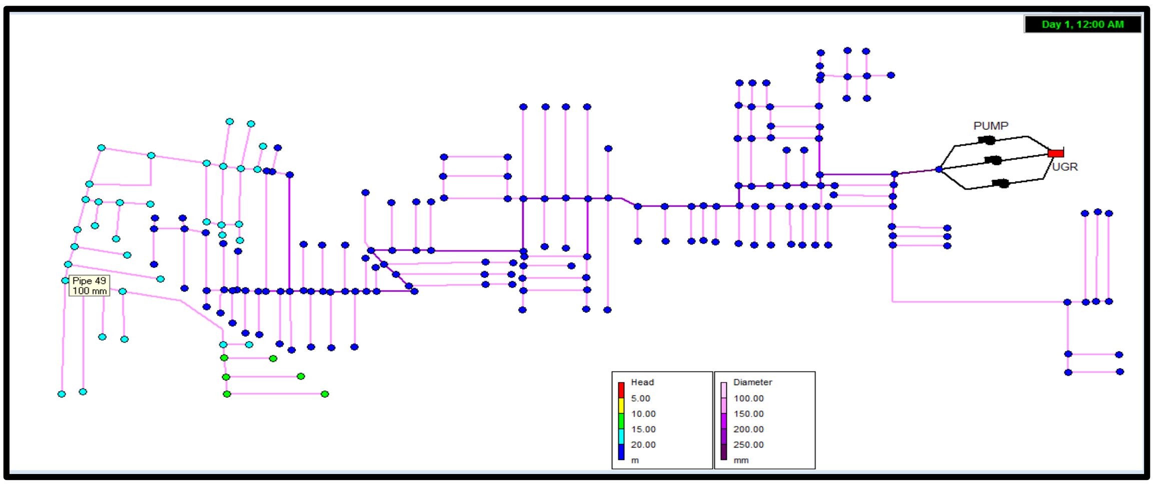


Figure 4.7.1.5 EPANET simulated WDN for water demand 0.375 (LPS)

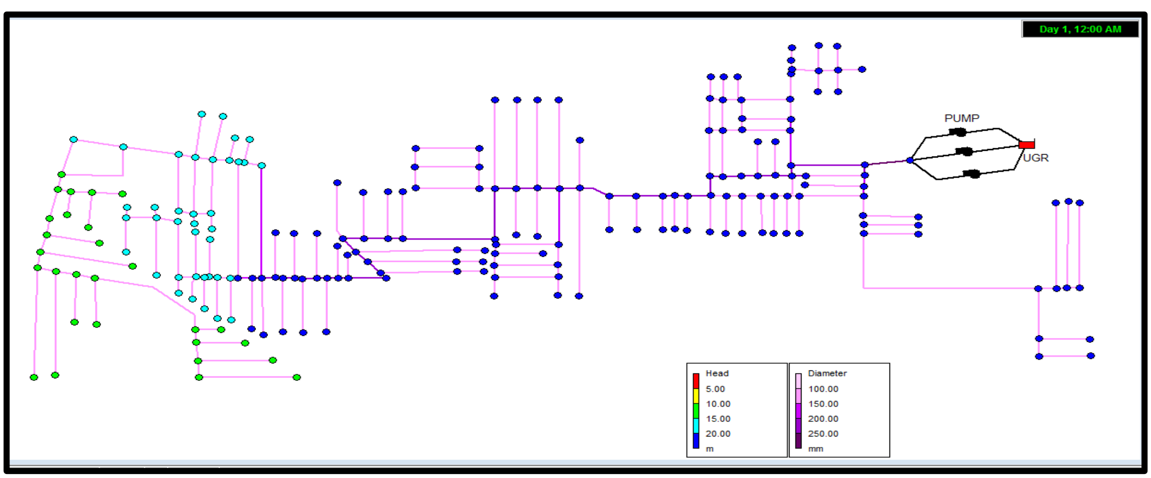


Figure 4.7.1.6 EPANET simulated WDN for water demand 0.396 (LPS)

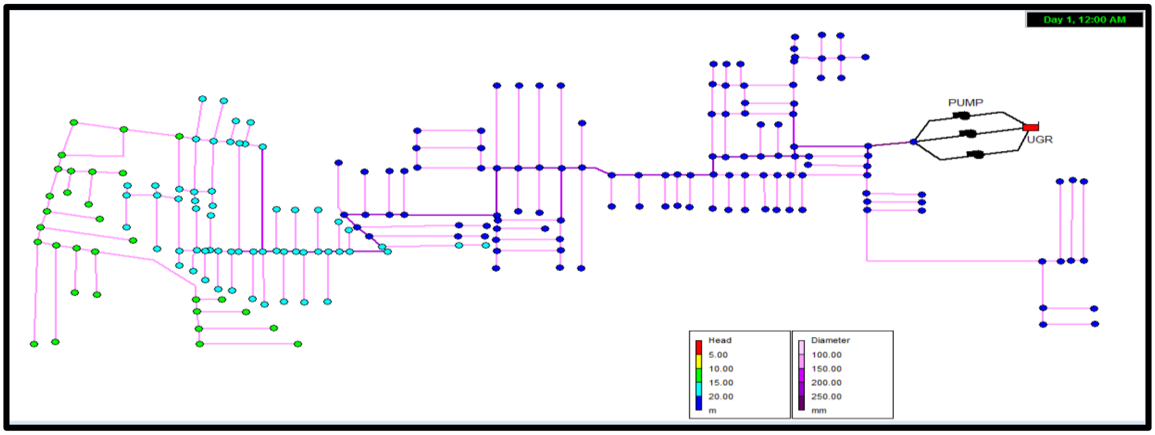


Figure 4.7.1.7 EPANET simulated WDN for water demand 0.417 (LPS)

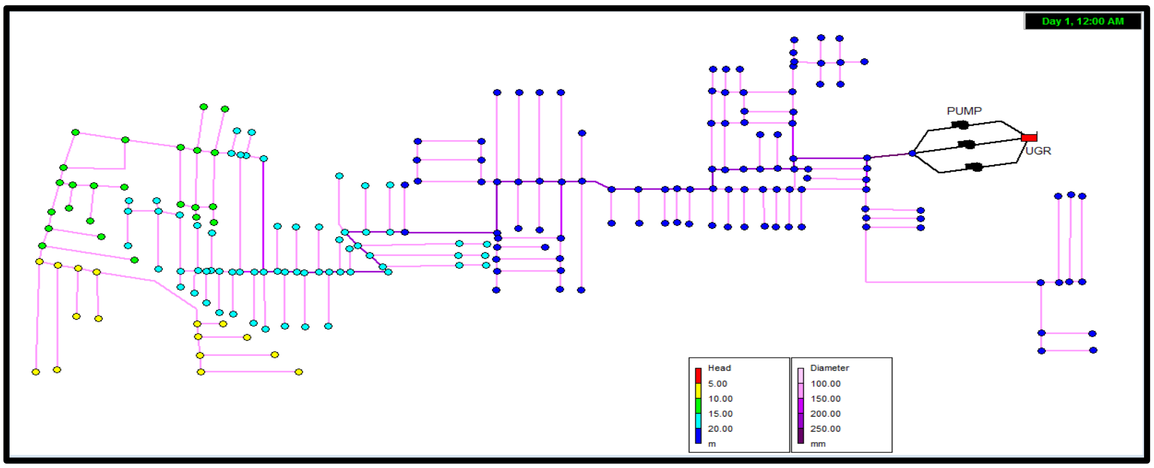


Figure 4.7.1.8 EPANET simulated WDN for water demand 0.438 (LPS)

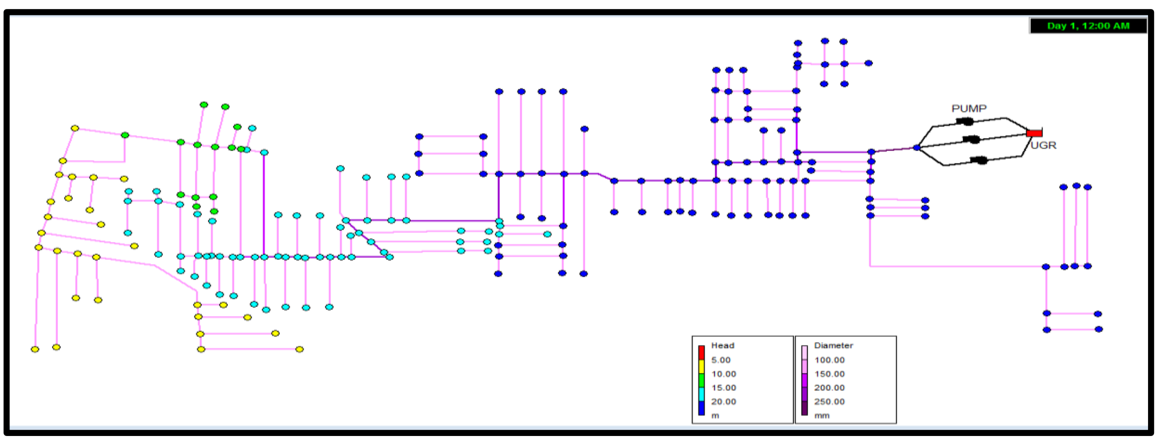


Figure 4.7.1.9 EPANET simulated WDN for water demand 0.458 (LPS)

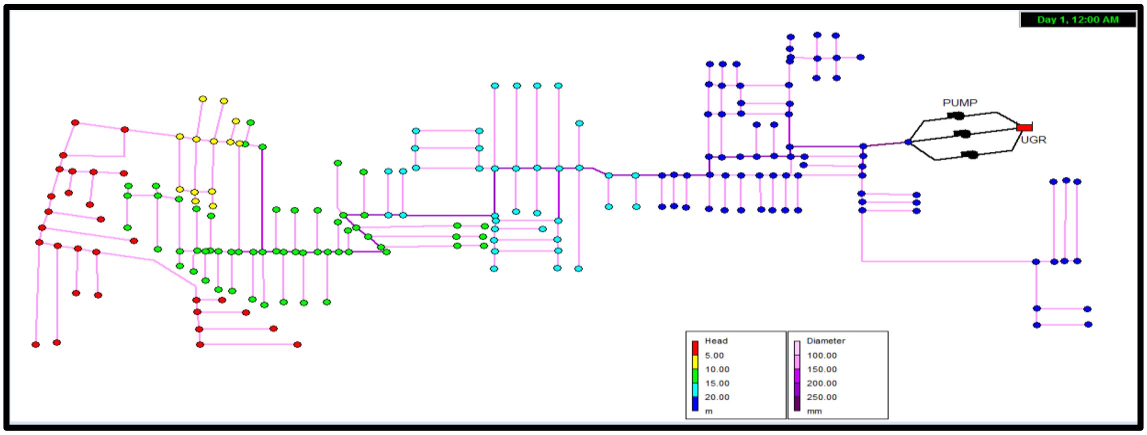


Figure 4.7.1.10 EPANET simulated WDN for water demand 0.479 (LPS)

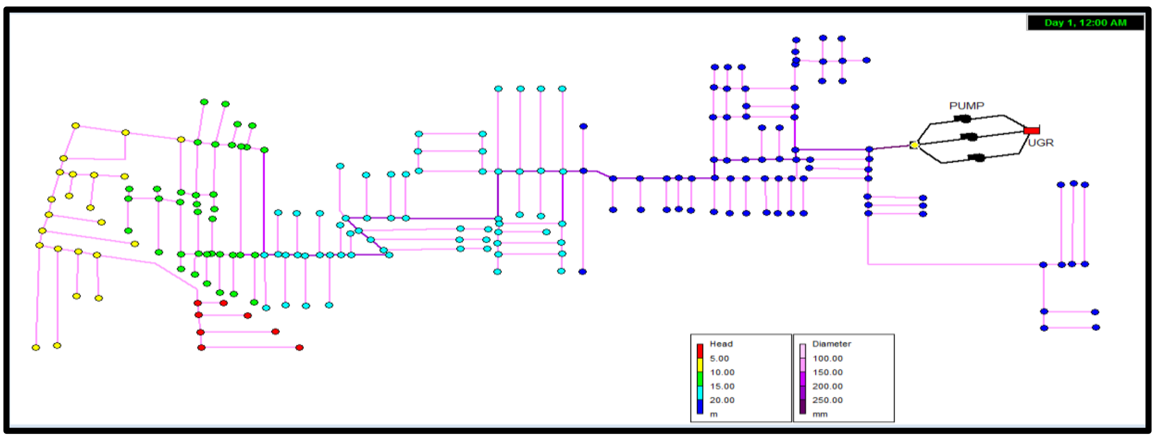


Figure 4.7.1.11 EPANET simulated WDN for water demand 0.500 (LPS)

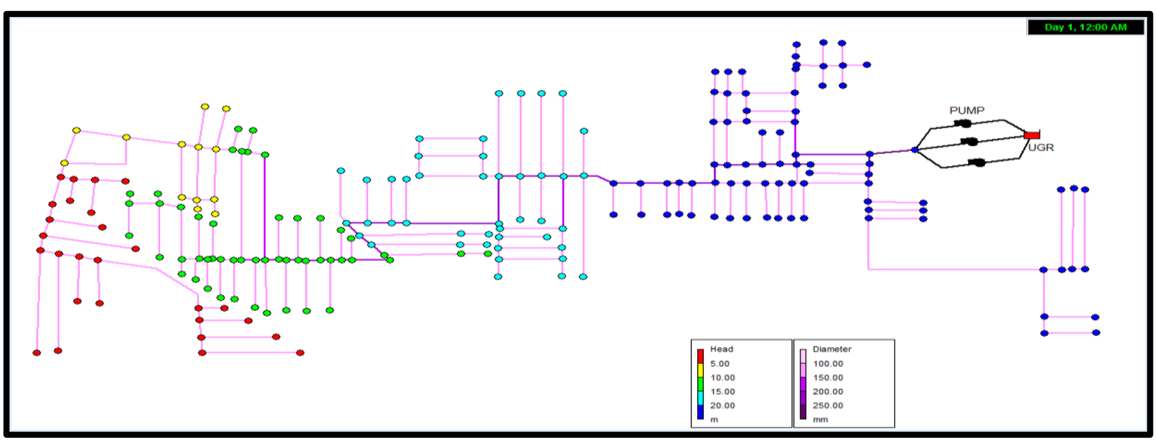


Figure 4.7.1.12 EPANET simulated WDN for water demand 0.521 (LPS)

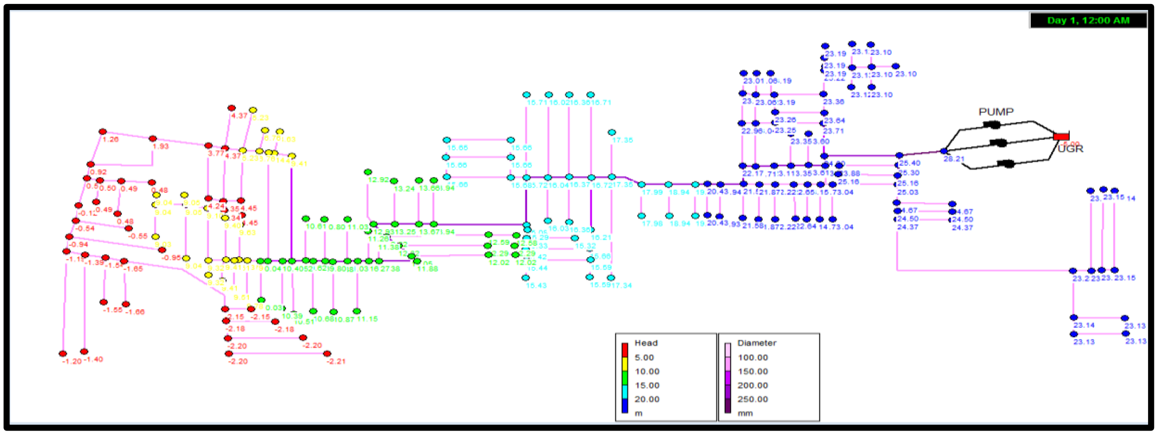


Figure 4.7.1.13 EPANET simulated WDN for water demand 0.542 (LPS)

Now R_H and R_Q are plotted as calculated in table 4.7.1.1. The figure 4.7.1.14 shows variation of reliability when demand is increased. It shows that as the consumer starts to get adequate amount of water, head at nodes in the network starts to fall i.e. as R_Q increases R_H decreases.

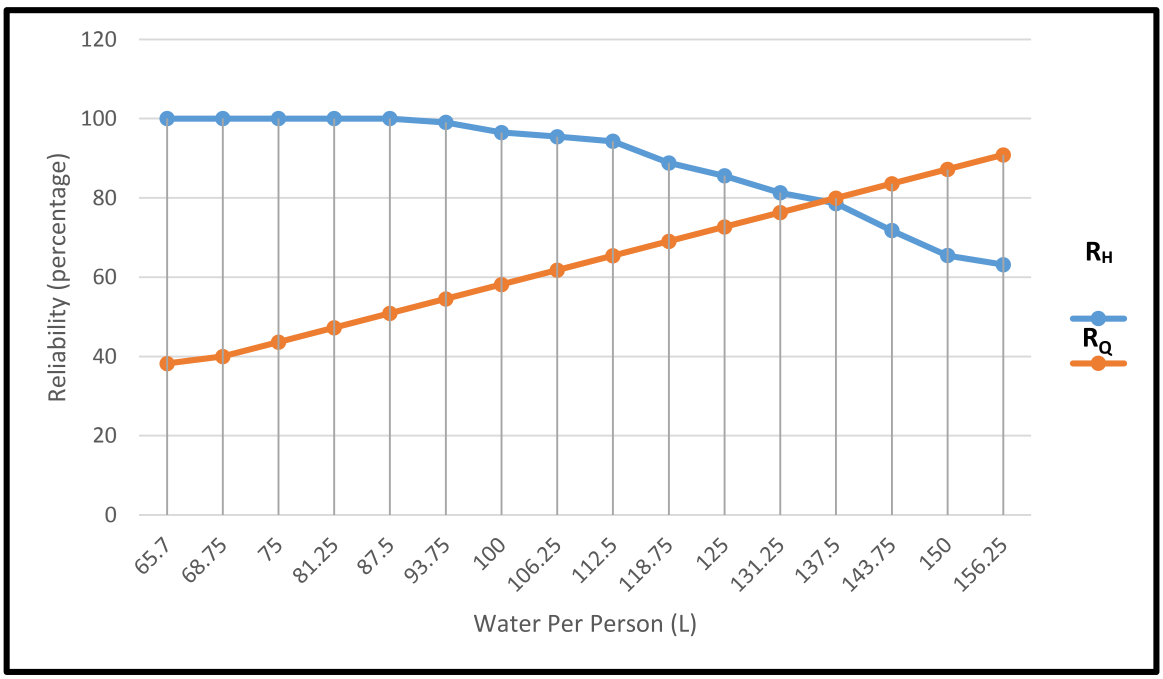


Figure 4.7.1.14 – Variation of reliability with increasing water demand

Figure 4.7.1.15 shows the relationship between the head of water at the first node and farthest node in the WDN. The difference in both head is increasing as the water consumption per person per day is increased.

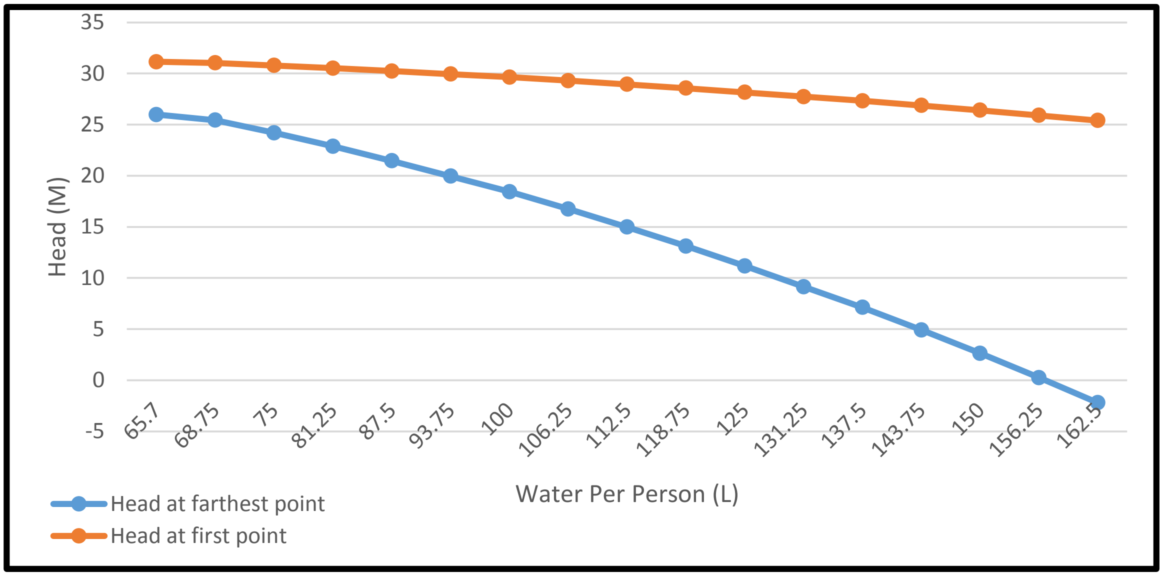


Figure – 4.7.1.15 – Variation of head with demand increase

Figure 4.7.1.16 shows the flow through one pump when two pumps are operational in parallel. Also the head created by the pump is shown. The figure shows that the pumps capacity is sufficient since the head starts falling in WDN while there is enough discharge capacity of pump i.e. (87.63 58.27=29.36 LPS) is left.

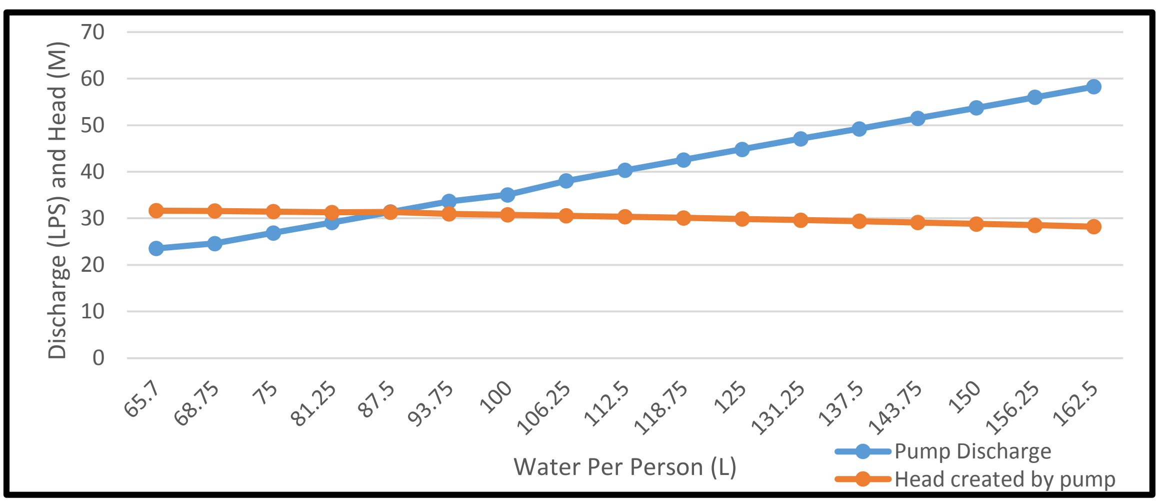


Figure 4.7.1.16 – Pump output

The network strength is again tested with all three pumps running in parallel to simply understand and show the results based on EPANET simulation software, how the head would vary if in future demand increases further and all the three pumps are run during 6 hours of supply to the network. The exercise done next is only to observe if it is possible to increase head and quantity of water reaching to consumers. Table 4.7.1.2 again shows the reliability of head and quantity of water with increasing demand with 3 pumps operational.

Water per plot (L)	Water per person (L)	Total water for network (L)	Demand at one node (LPS)	Nodes with head below				Nodes with head above (M)	R_H	R_Q	ZONE Affected
				5m	10m	15m	20m				
525.6	65.7	1017036	0.219	0	0	0	0	215	100.00	36.50	
550	68.75	1064250	0.229	0	0	0	0	215	100.00	38.19	
600	75	1161000	0.250	0	0	0	0	215	100.00	41.67	
650	81.25	1257750	0.271	0	0	0	0	215	100.00	45.14	
700	87.5	1354500	0.292	0	0	0	0	215	100.00	48.61	
750	93.75	1451250	0.313	0	0	0	0	215	100.00	52.08	
800	100	1548000	0.333	0	0	0	20	195	97.67	55.56	4
850	106.25	1644750	0.354	0	0	0	31	184	96.40	59.03	4
900	112.5	1741500	0.375	0	0	0	41	174	95.23	62.50	4
950	118.75	1838250	0.396	0	0	16	27	172	93.14	65.97	4
1000	125	1935000	0.417	0	0	29	36	150	89.07	69.44	3,4
1050	131.25	2031750	0.438	0	0	33	55	127	85.93	72.92	3,4
1100	137.5	2128500	0.458	0	20	21	59	115	81.28	76.39	3,4
1150	143.75	2225250	0.479	0	30	14	62	109	79.07	79.86	3,4
1200	150	2322000	0.500	8	24	33	57	93	73.60	83.33	2,3,4
1250	156.25	2418750	0.521	28	13	45	47	82	66.51	86.81	2,3,4
1300	162.5	2515500	0.542	31	12	55	38	79	64.19	90.28	2,3,4
1350	168.75	2612250	0.563	NEGATIVE PRESSURE							

TABLE 4.7.1.2 – Effect on reliability due to varied water demand

For EPANET simulation result for each demand in table 4.7.1.2 refer to APPENDIX 1.

Figure 4.7.1.17 shows the comparison of reliability of head R_H and reliability of quantity of water R_Q with 2 pumps and 3 pumps running.

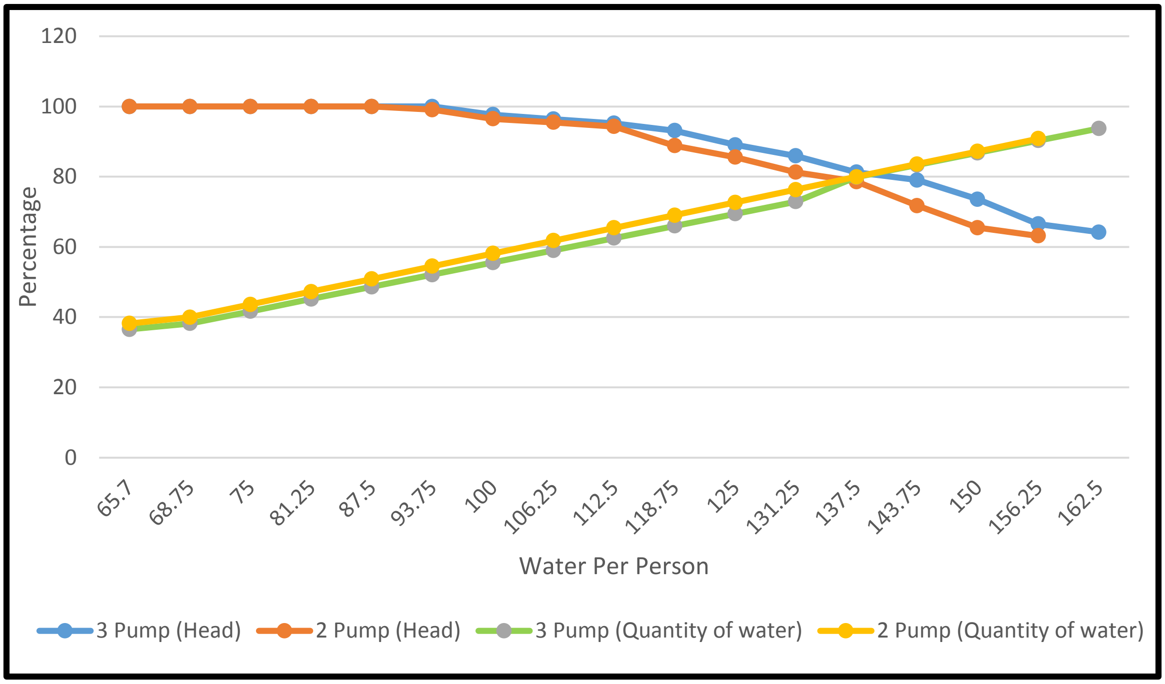


Figure 4.7.1.17 R_H, R_Q Variation for 2 and 3 pumps working

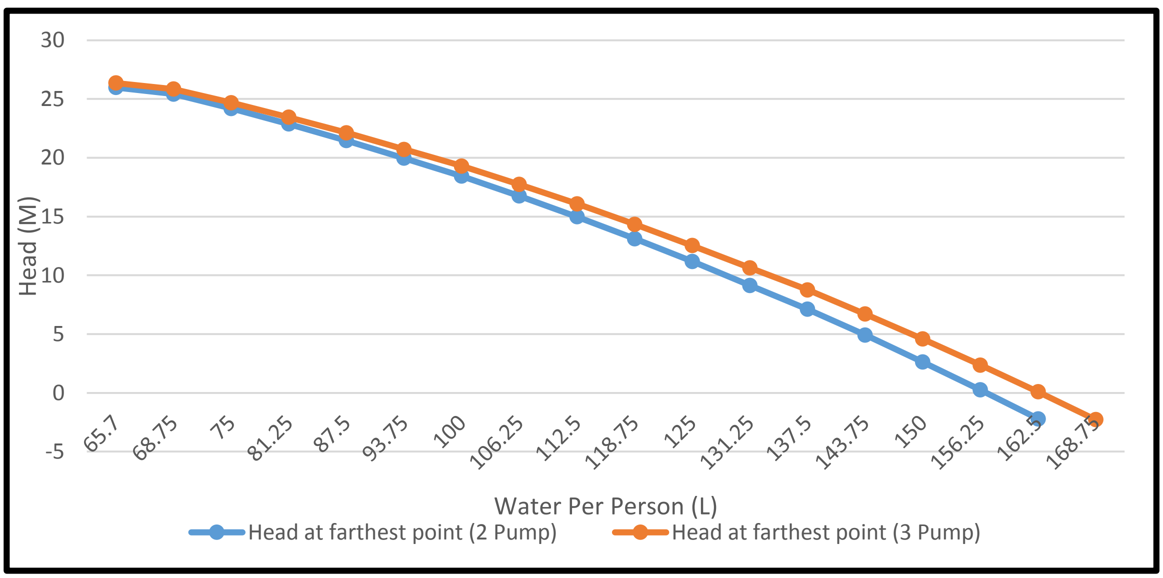


Figure 4.7.1.18 head variation at farthest node in WDN for 2 and 3 pumps working

This indicates that even with 3 pumps operating the head at the farthest point on network does not change much indicating that adding pump will not solve the problem.

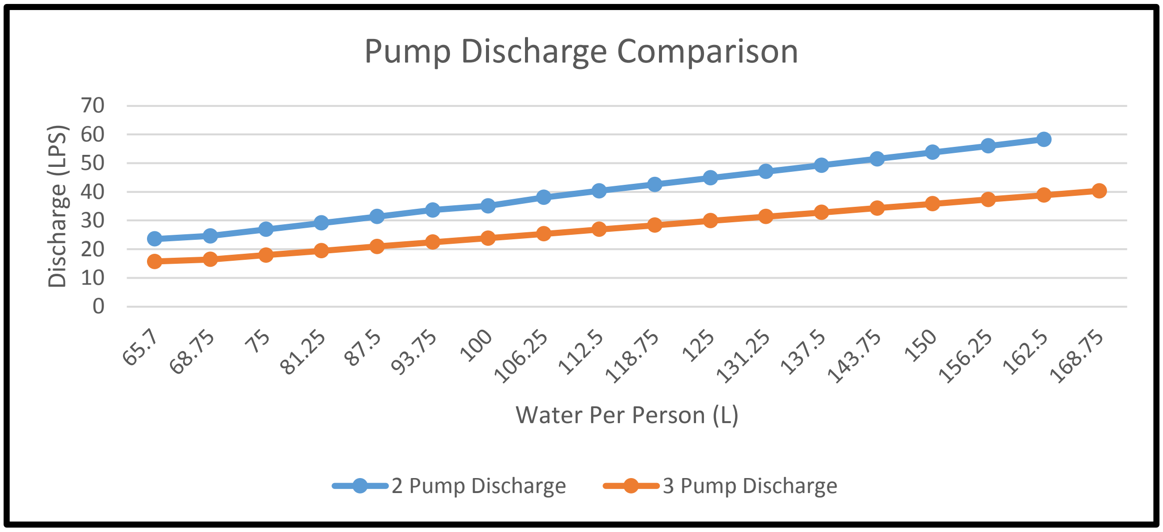


Figure 4.7.1.19 – Discharge per pump for 2 and 3 pumps working

Figure 4.7.1.19 shows that only half of pump capacity is used at 168.75 LPS but the head in Zone four has fallen below 0 m i.e. negative pressure are obtained. This shows the installed capacity of pump is more than sufficient but the network is not supporting the pump.

4.7.2 DEMAND AS POPULATION INCREASE

In section 4.7.1 demand was increased assuming population of the area is not changing. Now, the population in increased and new water demands are calculated. The population increment over four decade from present is studied taking the life of WDN as 40 years.

Referring from section 4.5 population growth is calculated. Table 4.7.2.1 indicates population growth in next four decades.

Decade	New Population	People Per Plot
1	18218	9
2	24094	12
3	33850	17
4	47782	25

TABLE 4.7.2.1: POPULATION GROWTH

As the population grows, the water demand at each node increases. In the first decade population growth is from 8 people per plot to 9 people per plot.

Daily consumption as	→	180 LPCD
Total quantity of water per plot	→	9X180 = 1620 L per day
Water supplied at one node	→	1620 X 9 = 14580 L
Water demand at node for 6 hours intermittent supply	→	14580/ (6 X 3600) = 0.68 LPS

Table 4.7.2.2 further show the calculation for next 3 decades.

Decade	People per plot	supply LPCD	Total water per plot	Total water at one node	Node Demand 6 hrs (Intermittent supply)
1	9	180	1620	14580	0.68
2	12	180	2160	19440	0.90
3	17	180	3060	27540	1.28
4	23	180	4140	37260	1.73

TABLE 4.7.2.2 Effect on Population growth on nodal water demand

From table 4.7.2.2 the node demand after first decade i.e. 0.68 LPS is much more the network can satisfy even when 3 pumps are running as shown in table 4.7.1.2 for 6 hours of water supply per day. Therefore, even if the WDN is sufficient to some extent today it would not fulfill the water requirement of future population. Figure 4.7.2.1 shows EPANET simulation of 0.68 LPS demand when all the three pumps are running. The figure 4.7.2.1 shows that Zone 1 and 2 consumer are satisfied with their need but zone 3 and 4 have no water at all. This shows the limitations of network. Similarly, for decade 2 and 3 conditions, the figure 4.7.2.2 and 4.7.2.3 show that the network is not able to supply water to the consumers for 6 hours of water supply per day.

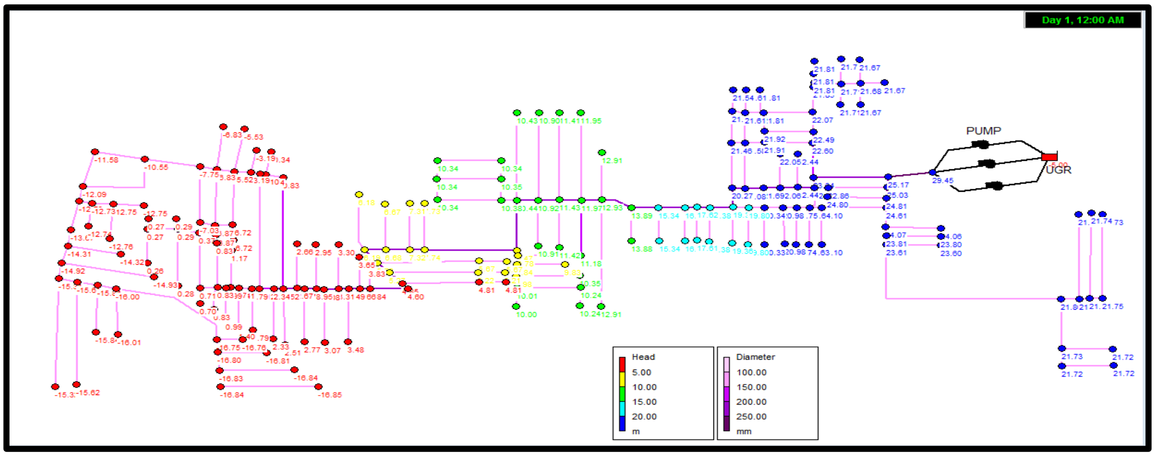


Figure 4.7.2.1 EPANET simulated WDN for water demand 0.68 (LPS)

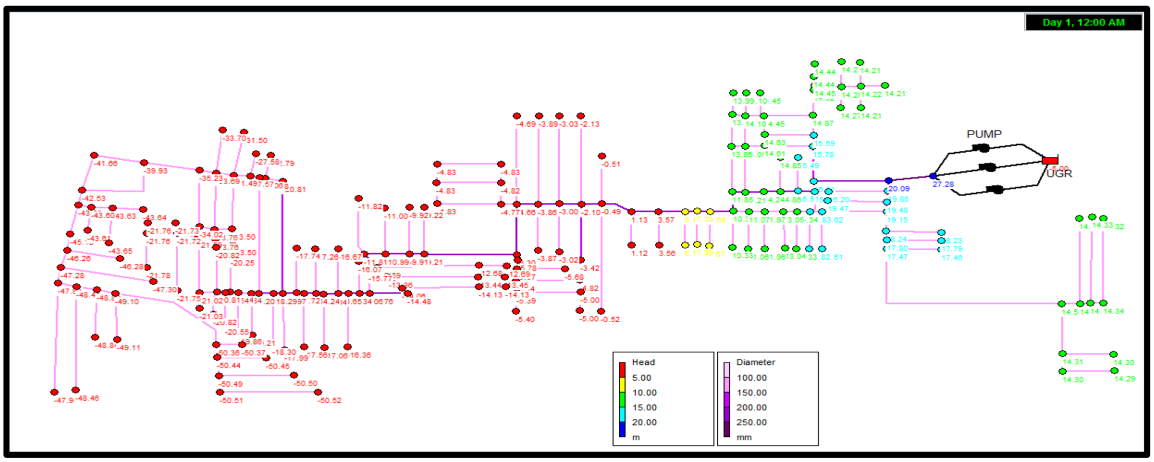


Figure 4.7.2.2 EPANET simulated WDN for water demand 0.9 (LPS)

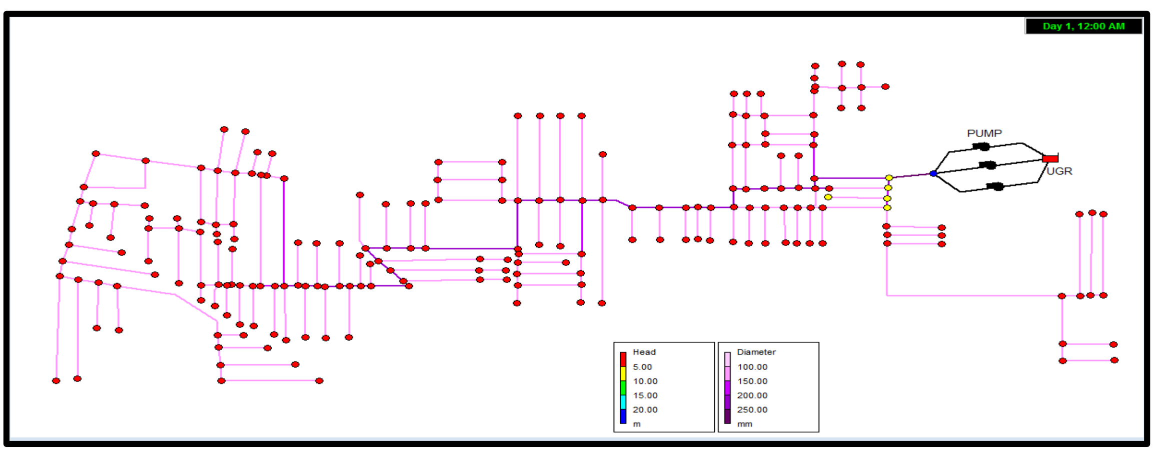


Figure 4.7.2.3 EPANET simulated WDN for water demand 1.28 (LPS)

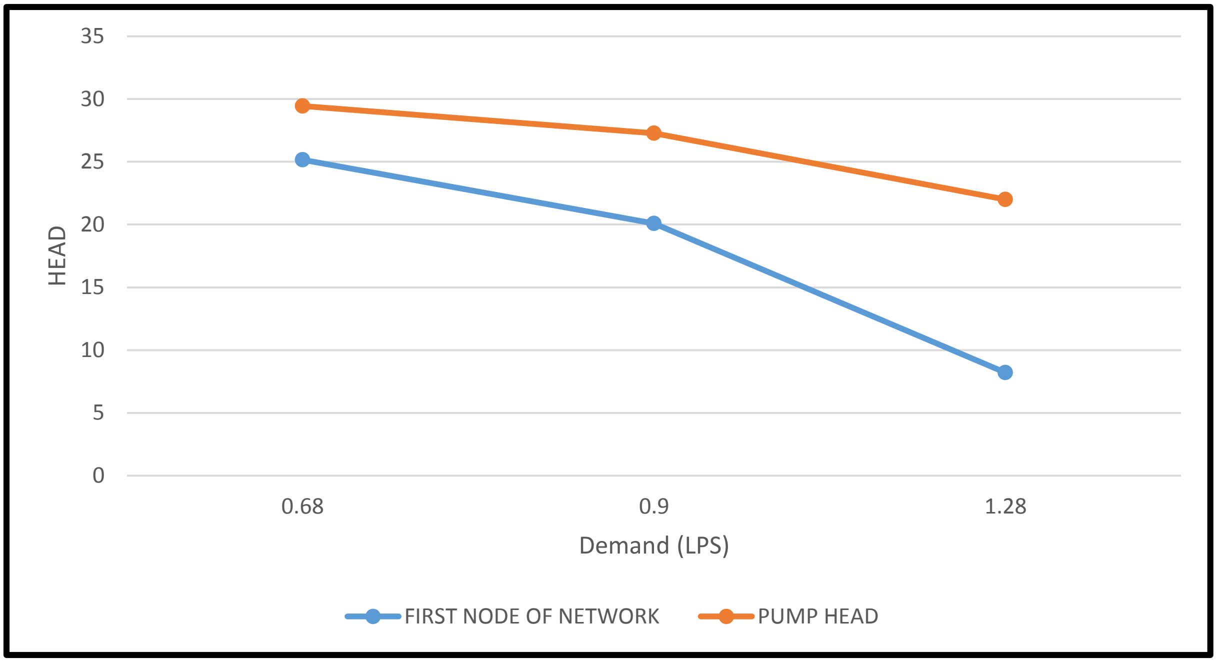


Figure 4.7.2.4 – Effect on head at first node due to increase of water demand

The figure 4.7.2.4 shows the head variation between first node and last pump end when all three pumps are operating. It depicts that the pipe diameter between pump and first node is not sufficient. But as changing of pipe in the network is very costly and not a viable option it is not recommended. Instead it would be prudent to increase the present hours of water supply.

CHAPTER 5

MODIFICATIONS BASED ON REVIEW OF EXISTING SYSTEM

The strength to which the present network of Pradhan enclave could supply water to the consumer has so far been verified and analyzed. From the analysis done in section 4.7.1 and 4.7.2, it is evident that the modification to network is necessary in order to use the existing network to fulfill the current demand with higher reliability and to satisfy the demand of future growing population.

Figure 5.1 shows how water demand vary if we increase the duration of water supply for 6 hours per day to 24 hours in the network. It shows that current 6 hours of supply will not be sufficient to supply 180 LPCD water to each individual in a day, since the table 4.7.1.2 indicates that any demand above 0.5 LPS will leave consumers in the area without water.

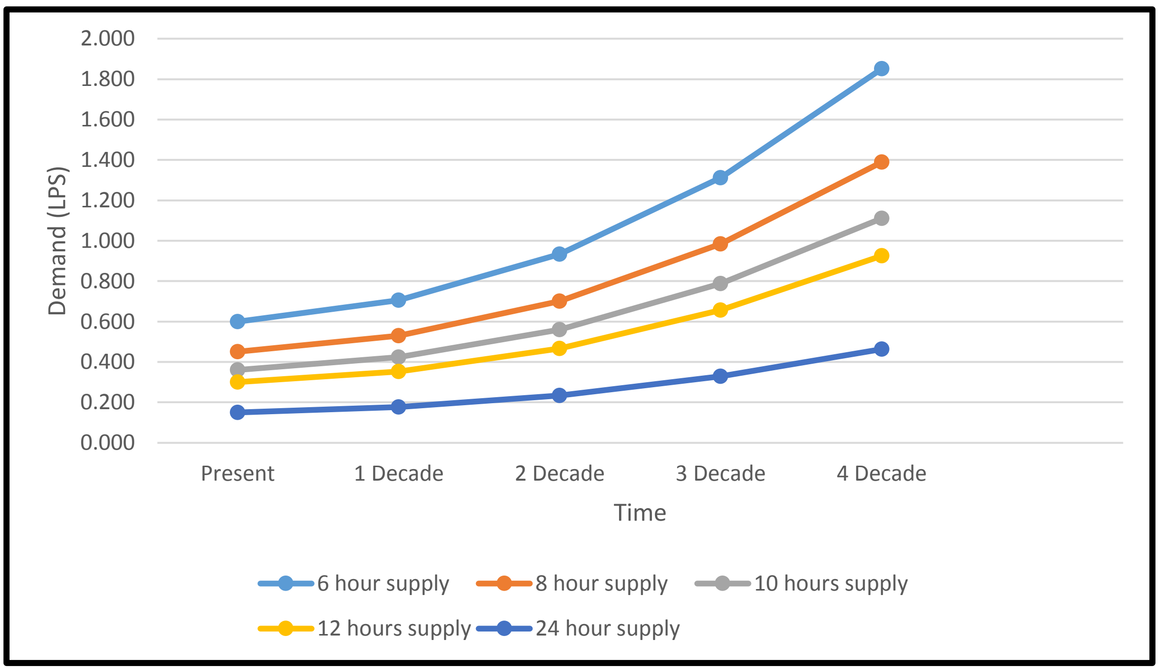


Figure 5.1- Demand based on duration of supply

From the interpretation of the figure 5.1 and table 4.7.1 it is suggested that if the duration of water supply to the network is increased to 24 hours supply than only the demand at nodes will decrease to the levels that the WDN could satisfy today as well as in future.

Hence, for continuous supply to the network a time pattern for water demand is assumed for a day. Figure 5.2 shows the demand in 24 hours supply for present population.

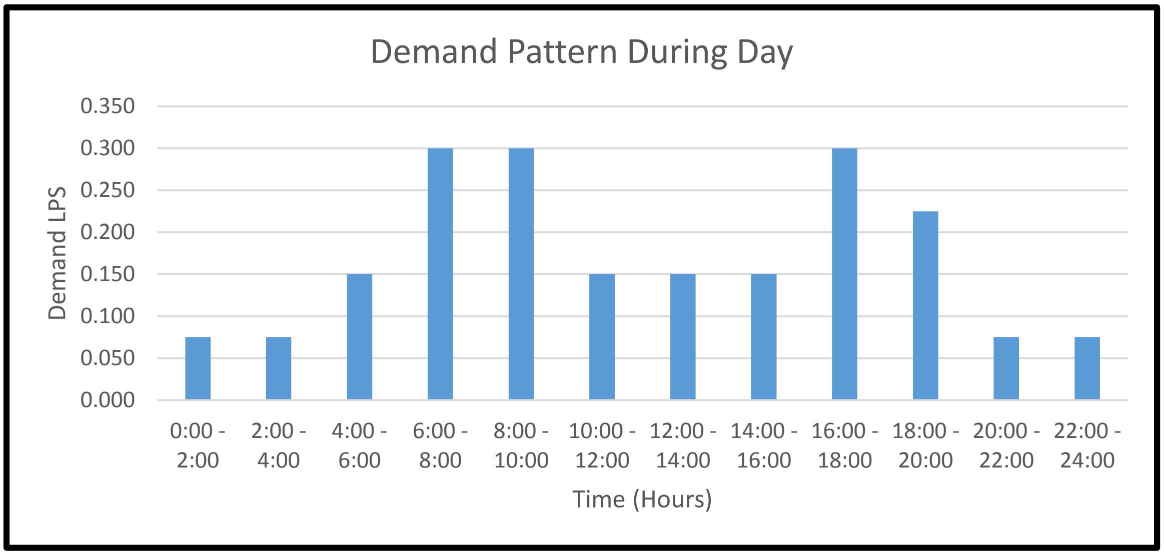


Figure 5.2 – Water Demand Pattern during day

Time	Population	Supply (LPCD)	Total Water Required	Demand at each node for 24 hour supply (LPS)
Present	15480	180	2786400	0.150
1 Decade	18218	180	3279240	0.177
2 Decade	24094	180	4336920	0.233
3 Decade	33850	180	6093000	0.328
4 Decade	47782	180	8600760	0.463

TABLE 5.1 – Demand variation for continuous water supply

EPANET simulation is run with demand value 0.15 LPS as given in table 5.1 and water demand as per figure 5.2 is fed into the EPANET. The simulation is run. The network is able to supply water to all the nodes with head above 20 meters for complete duration of supply. Hence, reliability R_H is 100% for current demands of the population. Also R_Q is 100% as the consumers are supplied water throughout the day. If such a system is used then the belief of the consumer in the network will increase many folds and consumers will take special care to make sure water is not wasted. Also as the society under study is metered therefore water wastage will be less if supply is 24 hours. This would reduce the need for people to look for other sources of water such as tankers and bore wells which create large wastage of water and detrimental to environment.

The water demand at each node for continuous supply is less than 0.542 LPS as depicted in table 5.1 and hence there shall not be any negative pressure in any of the four zones in the network.

The effect of population on nodal reliability R_H is shown in FIG 5.3. The average reliability of the network is above 90% up to 3 decades and remains above 70% in 4^{th} decade as per the water demand pattern during day (FIG 5.2).

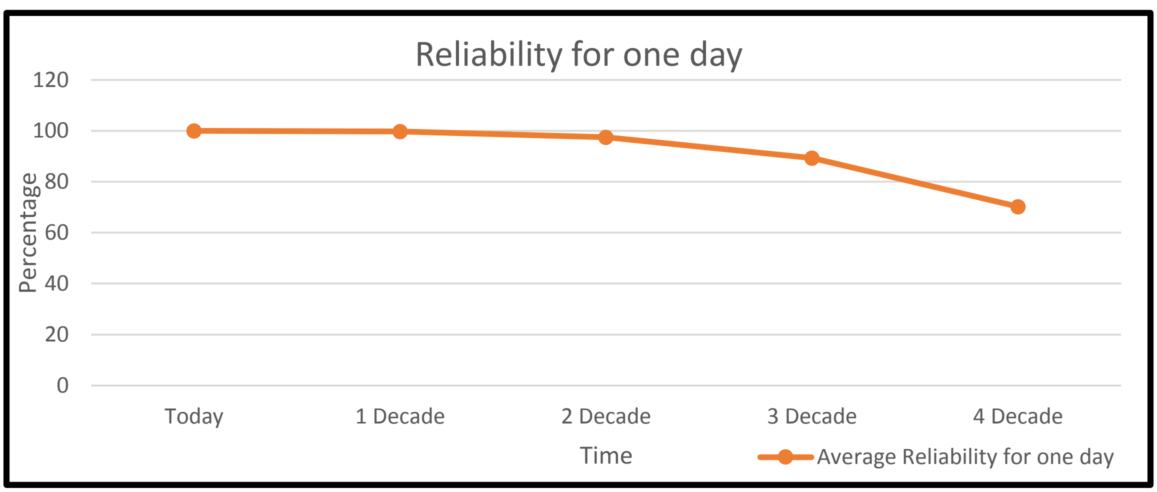


Figure 5.3 – Effect of population on nodal reliability R_H

CHAPTER 6

CONCLUSION

The water distribution network of Pradhan Enclave, Burari has been studied using EPANET simulation software for the following:

(i) Capacity of WDN to supply water, its capability to satisfy the consumers for their future water needs for four decades.
(ii) Effect of population increase as well as its reliability.

The literature review on the subject emphasized that the water distribution networks in the cities have been studied extensively by the researchers during the past two decades using the specialized simulation software and has helped in developing more reliable and efficient water distribution networks of present times. Based on the study and discussion in this report, following is concluded:-

1. The water demand at one node taken as 0.219 LPS by DJB appears to be inadequate considering the expected population increase even as per town planning in Delhi.

2. The existing water distribution network is being used as intermittent water distribution network. It is able to supply water demand up to 156.25 LPCD with existing population.

3. The existing WDN can barely satisfy the water demand of 0.68 LPS in the network for intermittent water supply after one decade from present as per the rate of increase of population.

4. The head reliability R_H of the existing water distribution network is more than 90% for the water demand up to 112.5 LPCD for current population for 6 hours of water supply.

5. The existing water supply network is recommended for continuous water supply system so as to cater the existing water demand up to 180 LPCD and shall thus fulfill the water needs as per the population growth up to three decades.

6. The reliability of the existing WDN when used as continuous water supply system shall be above 90% up to three decades and shall remain above 70% in fourth decade as per the water demand at nodes.

7. In case the network is extended in the near future, it will not be able to supply water with sufficient head in zone 3 and 4.

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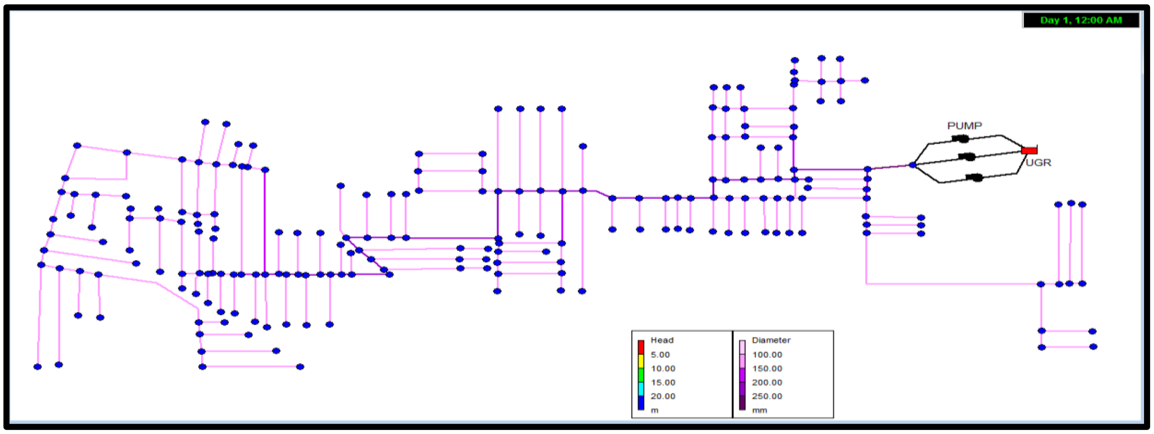
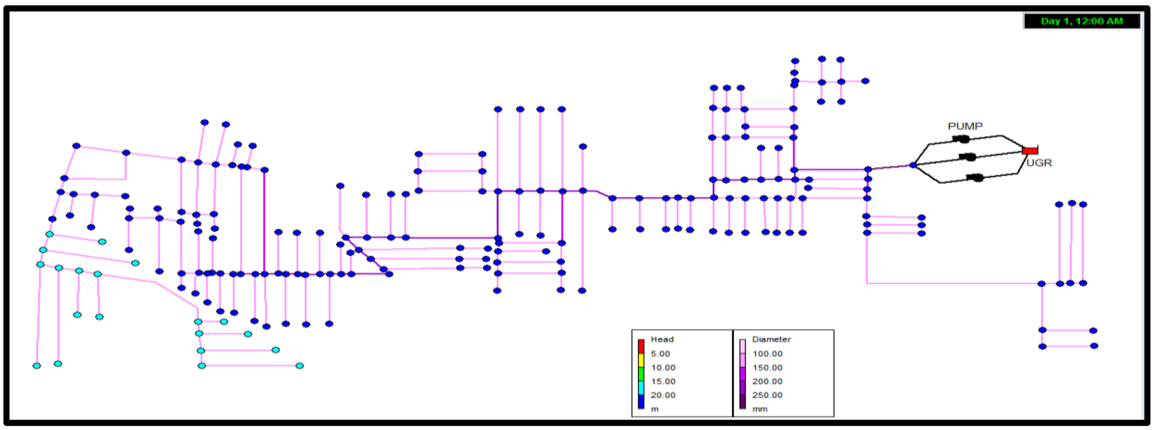
APPENDIX 1

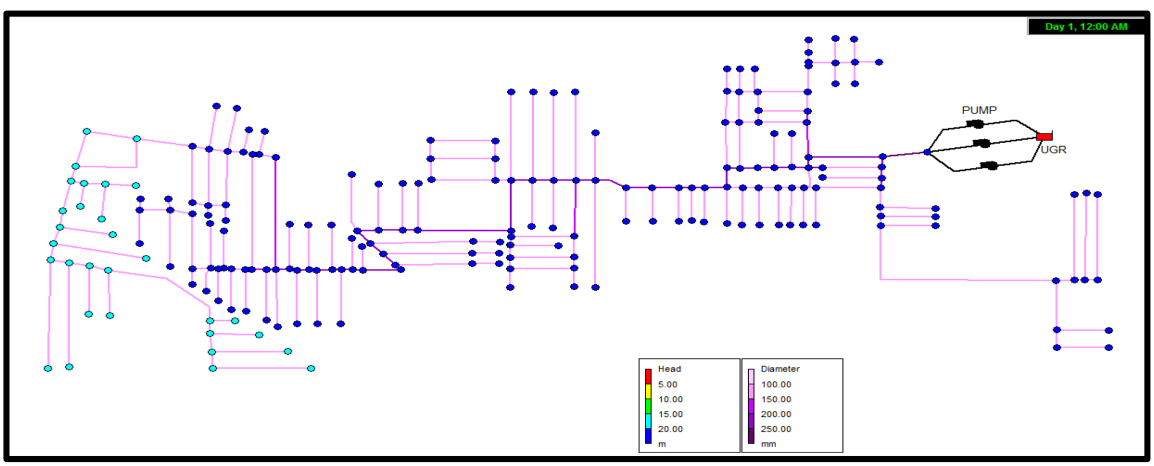
EPANET simulation result for Table 4.7.1.2. when three pumps are running.



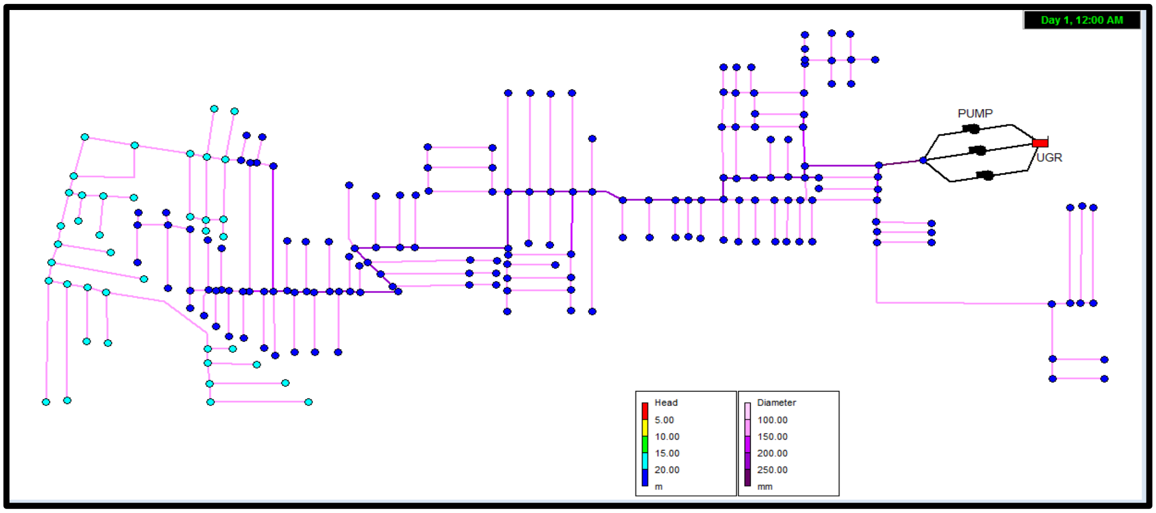
EPANET simulated WDN for water demand 0.219 to 0.313 (LPS)



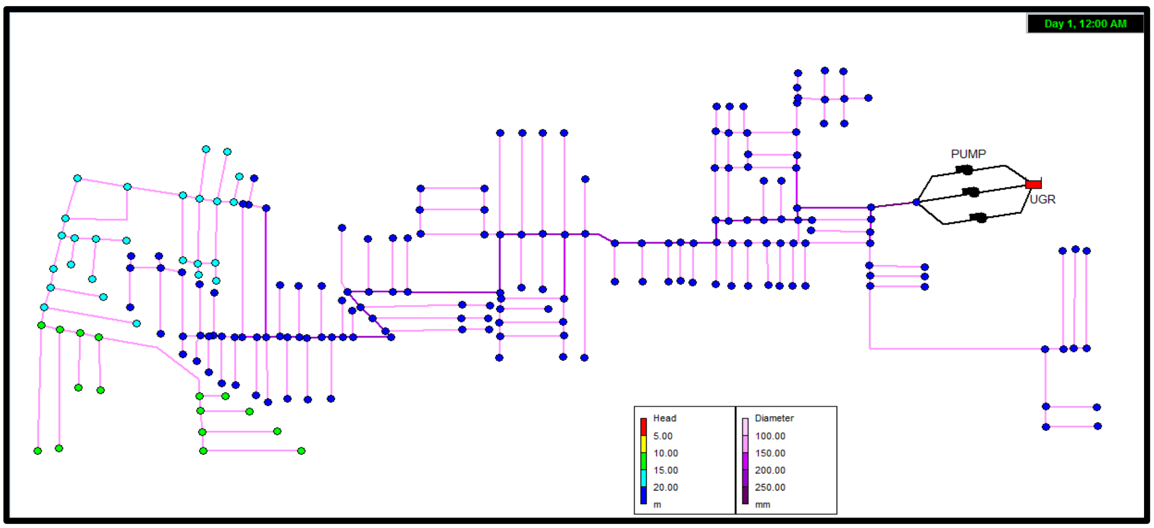
EPANET simulated WDN for water demand 0.333 (LPS)



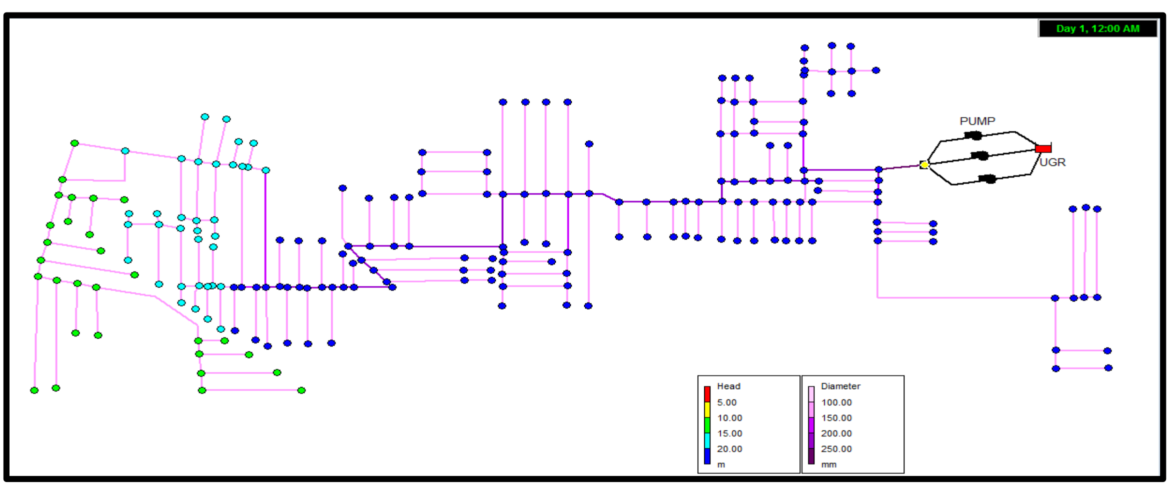
EPANET simulated WDN for water demand 0.354 (LPS)



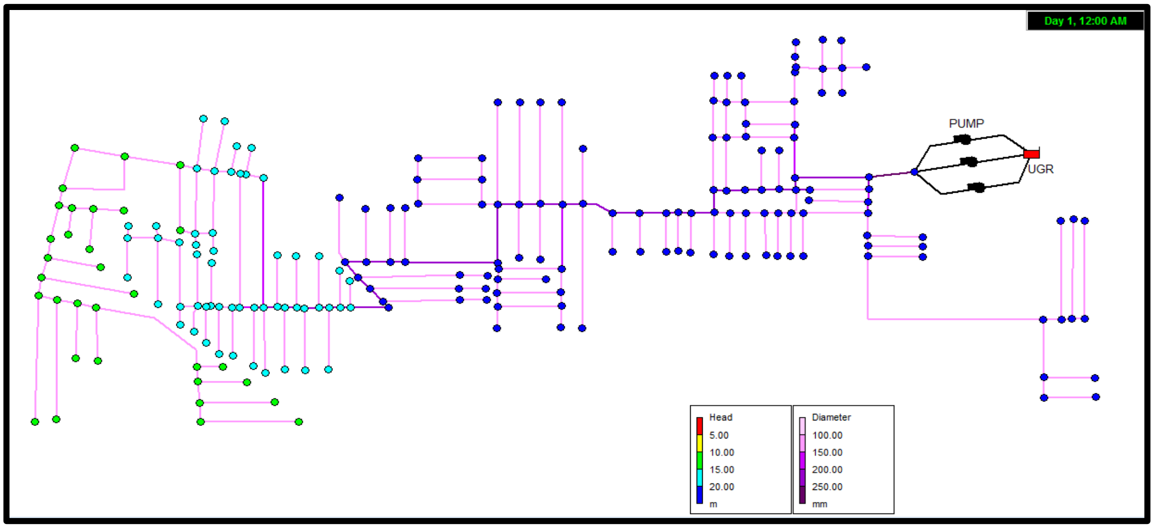
EPANET simulated WDN for water demand 0.375 (LPS)



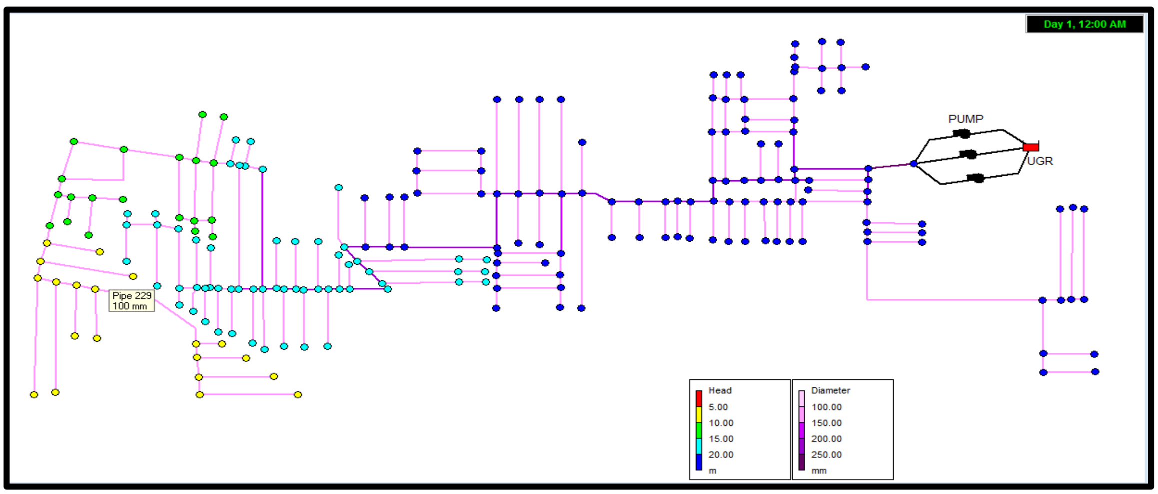
EPANET simulated WDN for water demand 0.396(LPS)



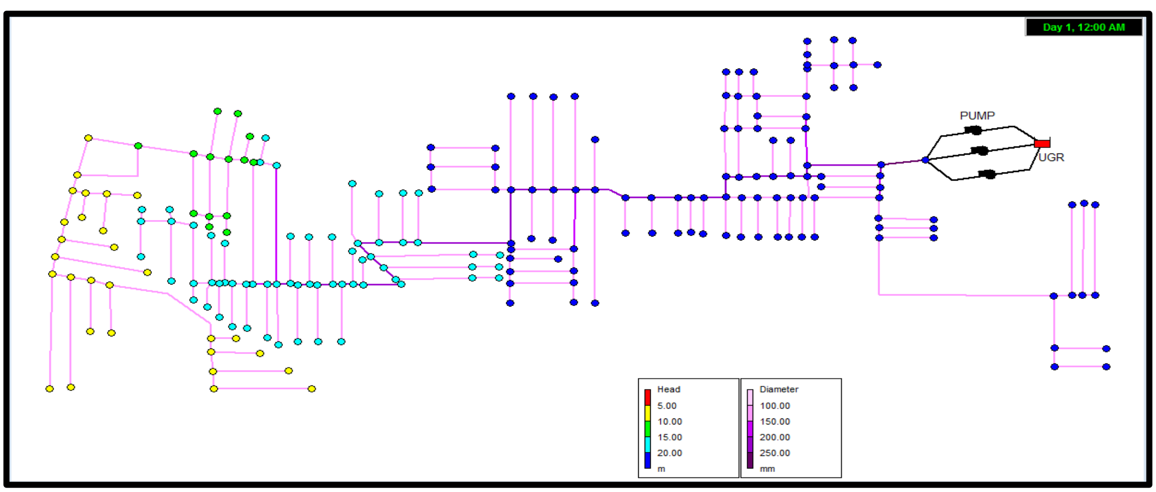
EPANET simulated WDN for water demand 0.417(LPS)



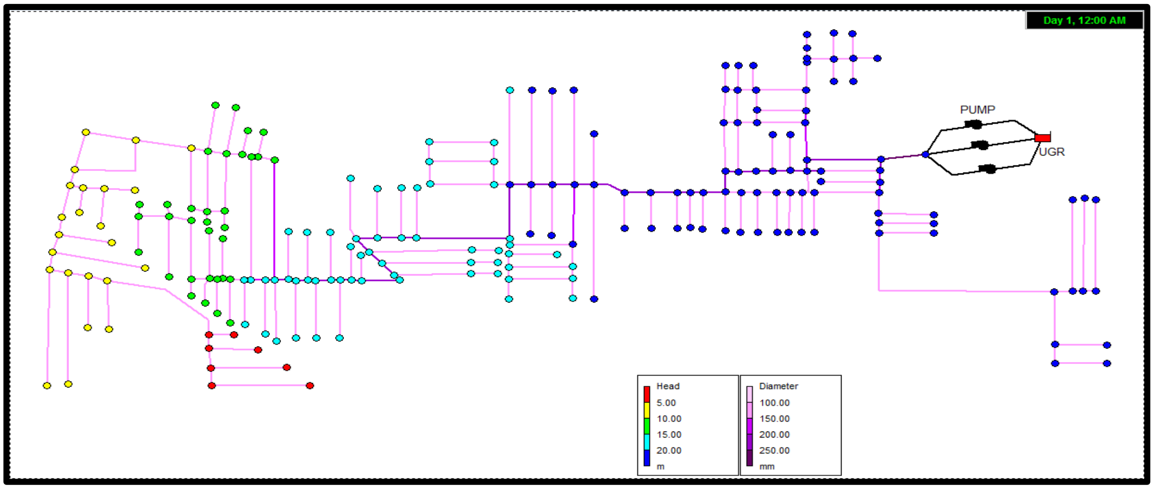
EPANET simulated WDN for water demand 0.438 (LPS)



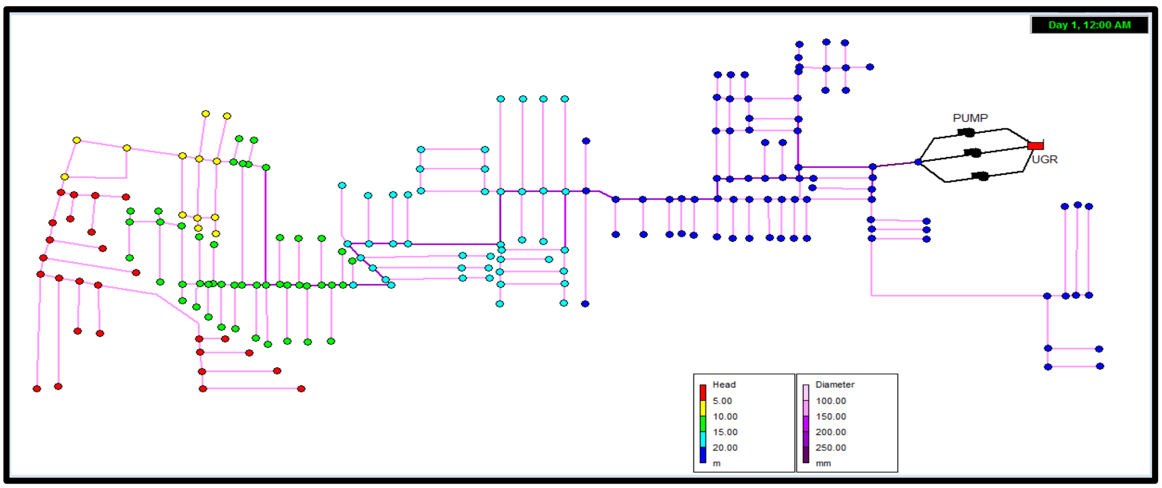
EPANET simulated WDN for water demand 0.458 (LPS)



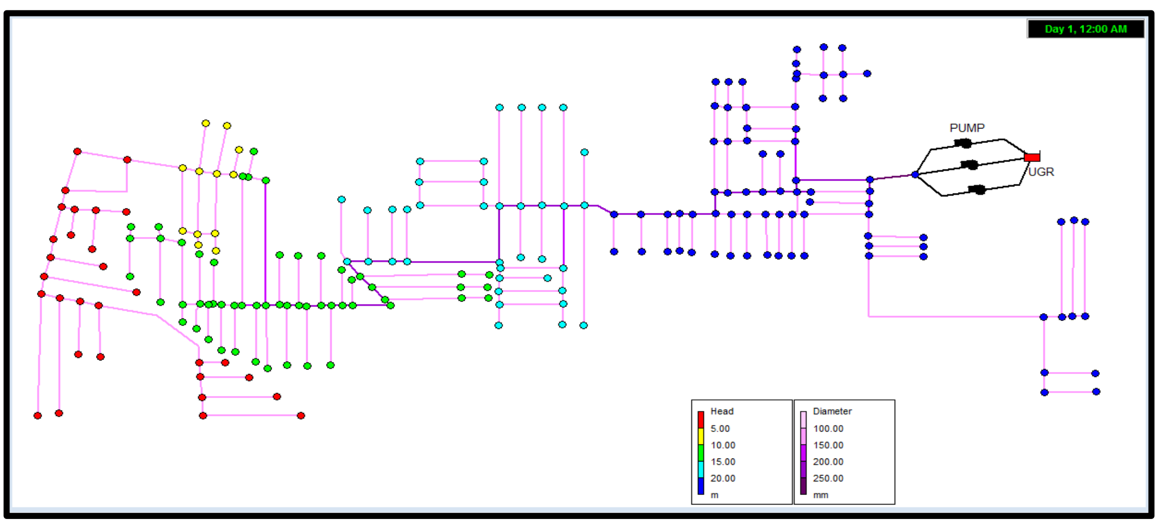
EPANET simulated WDN for water demand 0.479 (LPS)



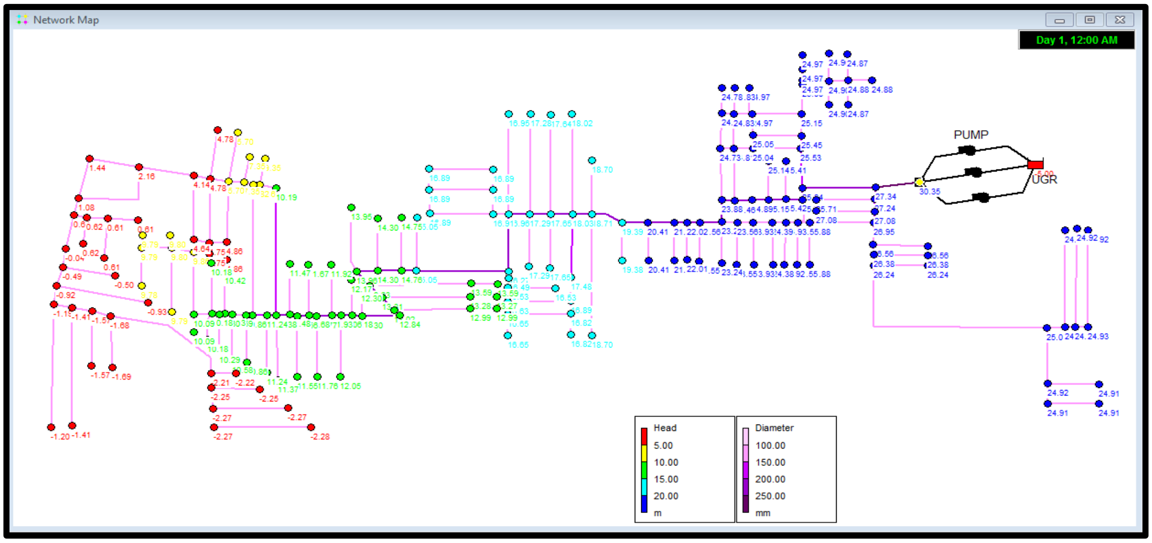
EPANET simulated WDN for water demand 0.500 (LPS)



EPANET simulated WDN for water demand 0.521 (LPS)



EPANET simulated WDN for water demand 0.542 (LPS)



EPANET simulated WDN for water demand 0.563 (LPS)