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Analysis of Water Distribution Network In Rural Area

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Abstract: This paper concerns for the design of rural water distribution systems in developing countries. Most of population of India is staying in rural area. At end of nineteen century community is not getting water at their resident in the village. But water is basic need of human being and it is directly effect on human health. Indian government is decided to provide safe, regular and adequate water to the community at their resident. This paper is helpful to water supply engineers are facing the problem of designing new distribution network in haphazard developed rural area. For designing of best economical water distribution system LOOP version software is used with a case study. Design procedure satisfied all constraints with a minimum total cost. The constraints include residual nodal pressure, velocity of flow in pipe, pipe material, reservoir level, peak factor and available commercial pipe diameters. In investigation, it is found that water distribution network cost occupied almost 70% of the total cost of water supply system. Extensive research has been done to minimize cost through optimization in design of water distribution network. In addition to the simulation tool, optimization techniques to identify the least cost design of distribution systems, while achieving the most equitable distribution of water have been developed.

Keywords- Water Distribution Network Design, Cost Analysis, Rural Area.

1. INTRODUCTION

1.1 General

Water distribution system is a hydraulic infrastructure consisting of elements such as pipes, tanks reservoirs pumps and valves etc. It is crucial to provide drinking or potable water to the end users; hence, effective water supply is of paramount importance in designing a new water distribution network or in expanding the existing one. Computation of flows and pressures in a complex network has been of great challenge and interest for those involved with designs, construction and maintenance of public water distribution systems. Analysis and design of pipe networks create a relatively complex problem, particularly if the network consists of range of pipes as frequently occurs in water distribution systems of large metropolitan areas. In the absence of significant fluid acceleration, the behavior of a network can be determined by a sequence of steady state conditions, which form a small but vital component for assessing the adequacy of a network. Such an analysis is needed each time changing pattern of consumption or delivery are significant or, added features such as supplying of water, addition of booster pumps, pressure regulating valves or storage tanks, change the system. Many methods have been used in the past to compute flows in network of pipes such methods range from graphical methods to the use of physical analogies and finally to the use of mathematical models. These methods of network analysis have been developed and implemented on the computer over the last fifty years. Of all the available methods, the first and probably the most widely used method of analysis is the Hardy Cross Technique. This method makes corrections to initial assumed value by using a first order expansion of the energy equation in terms of selection factor for the flow rate in each loop. In certain cases, it has been found that the Hardy Cross method converges very slowly or not at all. This leads to suggest special measures to improve convergence and a constrained model for the minimum cost design of water distribution networks. This methodology attempted to account for the uncertainties in required demands, required pressure heads, and pipe roughness coefficients. It was formulated an optimization problem as a non-linear programming model which is solved using a generalized reduced gradient method. It shows that uncertainties in future demands, pressure head requirements, and pipe roughness can have significant effects on the optimal design and cost. Further the reliability of water distribution system can be computed by treating the demand, pressure head, and pipe roughness as random variables. It can also be assumed that water demand and pipe roughness coefficient follow a probability distribution, and then used a random number generator to generate the values of random variables for each node and pipe. It leads to hydraulic simulation and computed the pressure heads at the demand nodes, provided the demands are satisfied. Finally, nodal and system hydraulic reliabilities can be computed using EPANET.

1.2 Need of water supply

Human life, as with all animal and plant life on the planet, is dependent upon water. Not only do we need water to grow our food, generate our power and run our industries, but we need it as a basic part of our daily lives - our bodies need to ingest water every day to continue functioning. "Basic needs of about 70litres per person per day". It includes the need for water to maintain a basic standard of personal and domestic hygiene sufficient to maintain health. The effects of inadequate water supply causes disease, time and energy expended in daily collection, high unit costs, etc. provision of basic daily water needs is yet to be regarded by many countries as a human right.

1.3 EPANET

EPANET is a computer program that performs extended period simulation of hydraulic and water quality behavior 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. EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. EPANET can help assess alternative management strategies for improving water quality throughout a system. These can include:

(i) Altering source utilization within multiple source systems,

(ii) Altering pumping and tank filling/emptying schedules,

(iii) Use of satellite treatment, such as re-chlorination at storage tanks,

(iv) Targeted pipe cleaning and replacement.

1.4 Description of area

The area for which we are analyzing the water distribution system is Village Deulgoan Gada. The population of the area is 1248 (source IPH, personal comm..). The distribution system designed here is tree system or dead end system. By the use of EPANET that is by filling the data into it about number of nodes, demand, elevation, tanks and pipes we design the respective distribution system. The number of nodes designed here are 36 and 2 overhead water tanks and these all are shown in figure1 below

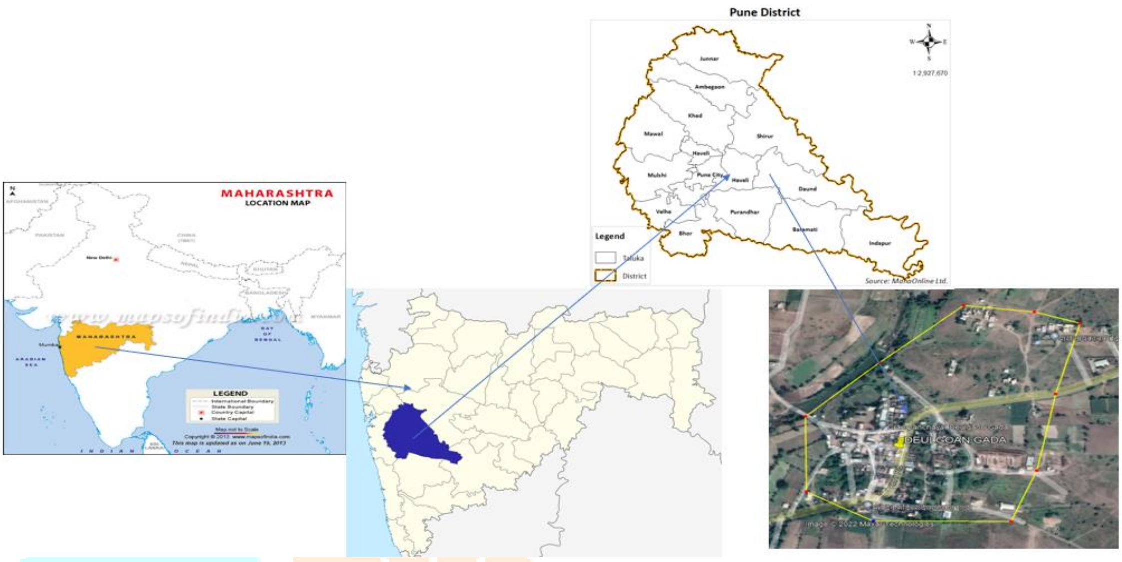

Fig. 1: Location of Deulgoan Gada

The geographical extents of the study area varying from 20° 0' 59.9976" N to 76° 1' 59.9988" E". As, the optimized design of distribution network for supplying of drinking water is not done earlier for the present study area using EPANET tool, hence, this area is considered for the present analysis in this paper. In addition to the above, basic topographical data such as base map, layout for distribution network to be designed; the population of the study area was collected from the concern authorities at the mandal level.

1.5 Equations used for design and analysis of network

Hydraulic equations Hydraulic equations commonly used for design and analyses of water transmission network are as follows:

1. Darcy-Weisbach equation
2. Hazen-William's equation.

In this paper,

The design and analysis were worked out by using the most popular Hazen Williams equation. This equation is conventionally acceptable equation for design of water conveyance system as it is simple to use. Hazen-William's equation with hydraulic mean depth, slope and velocity is given by equation 1.

$$V = 0.852 CHR^{0.63} S^{0.54}$$

Where, CH = Hazen-William's coefficient of pipe;

S = slope of hydraulic gradient line (m/m); and

R = Hydraulic mean depth, m.

The value of CH of pipe for design purposes (Manual, 1999).

Substituting,

$$V = 4Q/(\pi d^2),$$

R = D/4 and S = hf /L in Eq. (1)

And after some algebraic manipulations, one can obtain Equations.

$$hf = 10.68LQ\ 1.852\ CH^{\ 1.852}D^{4.87}$$

$$hf = KQ^{\ 1.852}$$

Where, K = Resistance coefficient of a pipe and given by

$$K = 10.68L\ CH^{\ 1.852}D^{4.87}$$

The Hazen Williams formula expressed in the forms of above equations can be used to compute the loss of head in a pipe flowing under pressure. Table 1 gives the value of Hazen-William's coefficient for common pipe materials (Manual, 1999).

Table 1: Values of Hazen-Williams Coefficient for Pipes

Sr. No.	Pipe	Value of C_H
1	Cast iron, Ductile iron	100
2	Mild steel	100
3	RCC up to 1200 mm diameter	140
4	RCC above 1200 mm diameter	145
5	PVC, GRP, and other Plastic	145

1.6 Methods of balancing heads

I. Assume the clockwise flows as positive and counterclockwise flows as negative;

II. Assign positive sign to the head losses for flows towards the junction and negative sign to those away from the junction.

The procedure, for the looped network analysis using hardy cross method of balancing head as follows:

1. Number all the nodes and pipe links. Also the number of loops. Adopt a sign convention that a pipe discharge is positive if it flows from a lower node number to a higher node number, otherwise negative. Apply nodal continuity equation at all nodes to obtain initial pipe discharges
2. Obtain discharge in other pipes repeat procedure until all the pipe flows are known. Compute corresponding K
3. Assume the loop pipe flow sign convention to apply loop discharges corrections
4. Take the value of CH from table 1;
5. Calculate ΔQ for the existing pipe flows using equation and apply pipe corrections algebraically
6. Apply the similar procedure in all the loops of a pipe network. Repeat step 5 until the discharge corrections in all the loops are relatively very small i.e. Within allowable limits of ±2% or the sum of the head losses in a closed loops relatively very small i.e. Within allowable limit of ±0.150m. When the corrections are less than allowable limits, the assumed flows are correct, and iterations are terminated.

1.7 Advantages of EPANET

Following are some basic advantages of EPANET for using in network and distribution analysis.

Flow rates in the network is obtained by using linear method.

Head losses due to friction are computed using Darcy-Weisbach or Mannings formulae.

It has the capability in considering minor losses from bends, fittings, etc.

It also can duplicate demands which vary over time.

It can also handle for different demand patterns for each node

1.8 OBJECTIVES

Keeping in view the literature, following objectives is planned.

1. To analyze the existing water distribution system using EPANET and to suggest some measures if present network does not fulfil the present and future demand.
2. To study the water distribution network of loops.
3. To collect pipe report and junction report of the network.
4. To analyze the data by using EPANET software.
5. To check the discharge and pressure head in looped network.

2. METHODOLOGY

3.1 Principle

The working flow diagram has been illustrated in figure 1. At first, the design and necessary data of the water distribution system of Deulgoan Gada was collected. The data was provided by the Gram panchayat deulgoan Gada. The length and diameter of the pipes, the capacity of the overhead tank were collected from the gram panchayat and the demand of the nodes was estimated through field investigation. The elevations of the nodes were found by using ArcGIS 10.4 and Google Earth Pro. After creating the model using this data, the model was simulated and its reliability was checked for the present demand. Then the model was simulated again with the expected growth in demand as per future master plan for Deulgoan Gada. Finally, the reliability of the water distribution system was checked for the future demand as per future master plan for Deulgoan Gada.

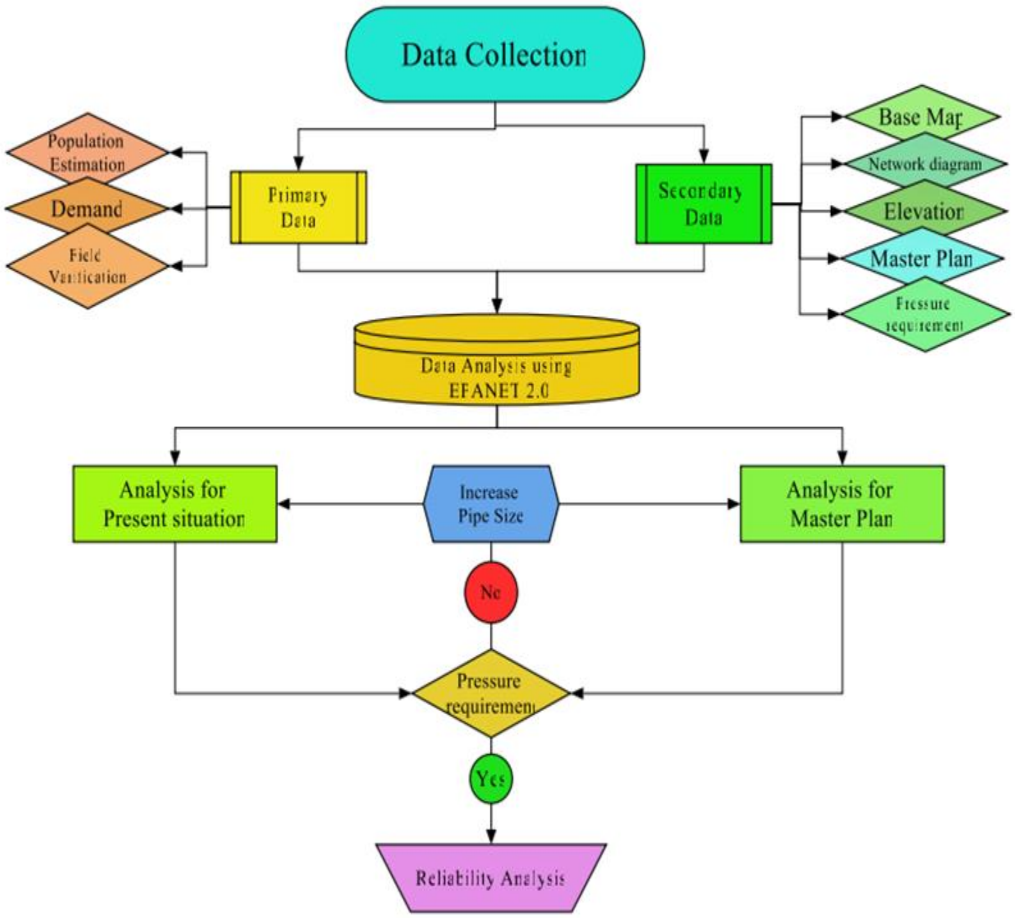

Fig. 2: Methodology of water distribution network

3.2 Preparation of water distribution network layout

The map of the existing water distribution network of Deulgoan Gada was collected from the gram panchayat Deulgoan Gada. The map contains the present water distribution layout of Deulgoan Gada which includes length and diameter of the pipe network with their relative positions.

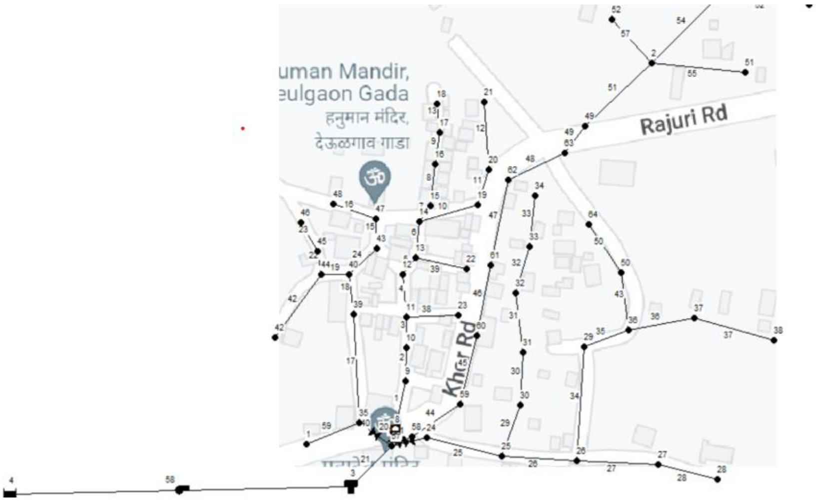

Fig. 3: Water distribution network layout

3.3 Assigning invert elevation data

Elevation data was taken using Google earth from Digital Elevation Model (DEM) data. The data was converted and adjusted with the available datum in Deulgoan Gada. Then the elevation data to each nodes were assigned and the contour map of the Deulgoan Gada found out as shown in figure. It has been found that the invert elevation varies from 0 to 8 m. As Deulgoan Gada is in the hilly region, there is much variation of elevations as a result the distribution network need special attention.

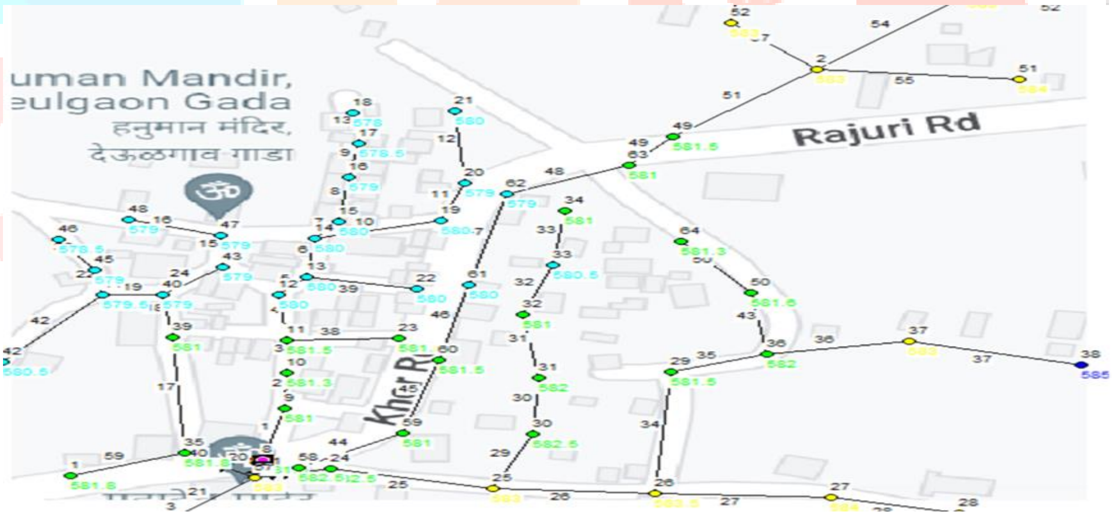

Fig. 4: Invert elevation data

3.4 Base demand estimation

The water base demand at the nodes has been estimated using the guidelines from BNBC 2015. Total houeses in Deulgoan gada have been considered. The buildings are classified as occupancy catagory. The numbers of population has been determined by field investigation and per capita demand has been taken from BNBC 2015. Total demand then converted to GPM and assigned to the nodes in of the distribution network. The contour plot of base demand are shown in figure

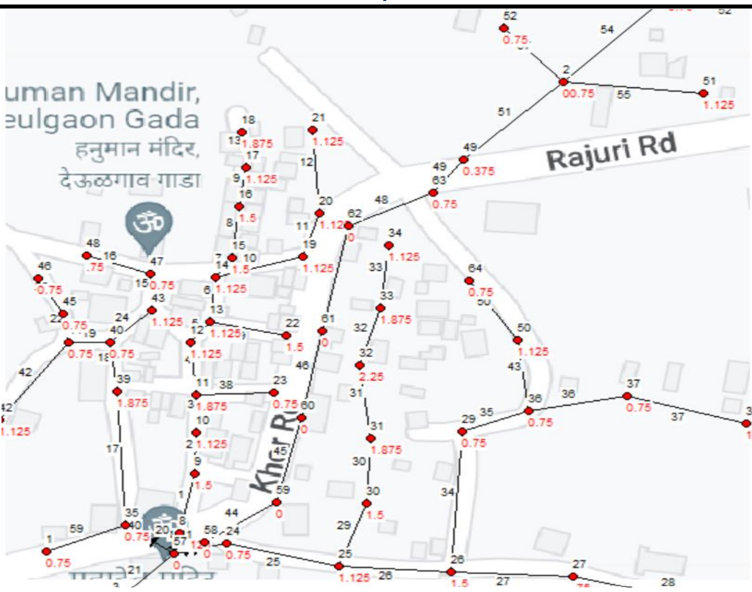

Fig. 5: Base demand

3.5 Assigning pipe size

Pipe length and diameter were assigned to the distribution model as shown in figure 5. The water distribution model consist of 138 numbers of pipe. Diameter and length of the pipe have been collected from Planning and development (P & D) department, CUET. The values are varified, adjusted and assigned to the model. It has been found that the supply mains of the water distribution system of CUET campus has a diameter of 4 inches, while the diameter of the submains has a diameter of 3 inches. The branch lines have diameter of 3, 3.5and 4 inches respectively.

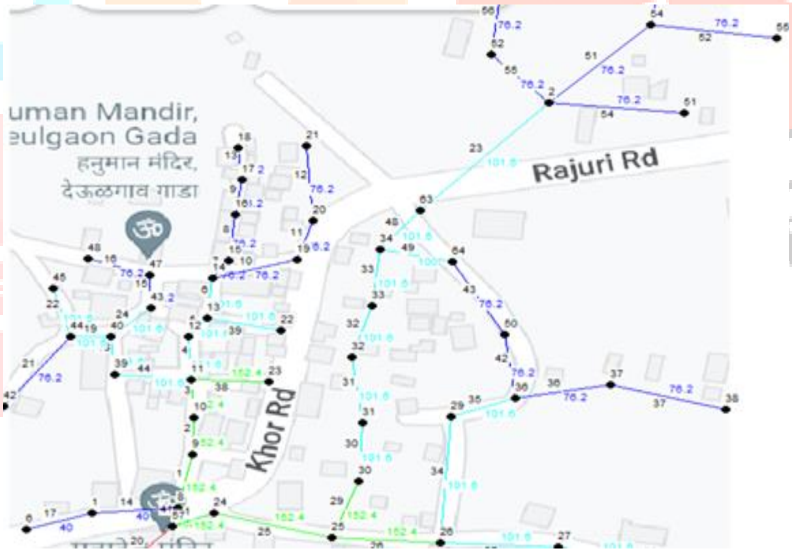

Fig. 6: Network pipe size

3.6 Future demand calculation as per master plan

After assigning all the necessary data, the model was completely prepared . Then the model was simulated, at first for present water demand, and then for future water demand. Validation of the model was done with respect to pressure in the nodes. After that, the result was obtained from the simulation and it was viewed in different formats. The diameter of the pipes was changed to find out the required diameter that results in pressure between the acceptable ranges if necessary. In this way, the reliability of the water distribution system was checked for present and future demand.

3.7 Population determination

Determination of population is one of the most important factors in planning, if the project has to serve the community for a certain design period. Normally, a design period of 20 to 40 years is selected. What will be the population at the end of design period is the basic question. This can be achieved by using various methods for population.

3.7.1 Arithmetical increase method

This is the simplest method of population forecast, though it generally gives lower results. In this method the increase in population from decade to decade is assumed constant. Mathematically, this hypothesis may be expressed as

$$\frac{dP}{dT} = K$$

From the census data of past 3 to 4 decades, the increase in population for each decade is found, and form that an average increment is found. For each successive future decade, this average increment is added. The future population Pn after n decades is thus given by

$$Pn = P + nI$$

Where

Pn = future population at end of n decades

P = Present population

I = Average increment for a decade

3.7.2 Geometrical increase method or uniform percentage growth method

In this method, it is assumed that the percentage increase in population from decade to decade is constant. From the population data of previous three or four decades, the percentage increase in the population is found and its average is found. If Ig is the average percentage increase per decade, or rg is the increase per decade expressed as ratio, the population Pn after n decades is given by

$$P^n = P(1+\frac{Ig}{100})^n = P(1+r_g)^n$$

3.7.3 Incremental increase method

This method combines both the arithmetic average method and the geometrical average method. From the census data for the past several decades, the actual increase in each decade is found. Then increment in each decade is found. Population in next decade is found by adding to the present population the average increase plus the average incremental increase per decade. The process is repeated for the second future decade, and so on. And it is expressed as:

$$Pn = P + nI + n\,(n + 1)/2$$

Where, P = present population

I = average increase per decade

r = incremental increase

n = number of decades

3.8 Water demand

Table 2: Water Requirement for Domestic Purposes

Sr. No.	Description	Amount Of Water In Liters Per Head
1	Bathing	55
2	Washing of clothes	20
3	Flushing	30
4	Washing the house	10
5	Washing the utensils	10
6	Cooking	5
7	Drinking	5
	Total	135

3.9 Water system losses

Losses from a water distribution system consists of Leakage and overflow from service reservoirs, Leakage from main and service pipe connections, Leakage and losses on customer premises when they get un – metered house hold supply, Under – registration of supply meters and large leakage or wastage from public tabs. In the case of well-maintained and fully metered water distribution system, the losses may hardly exceed 20% of the total consumption. In a system where supply is partly metered and partly – metered the losses may be up to 50% of the total supply.

3. RESULT

Water distribution system of Deulgoan Gada has been found as dead end or tree system. After assigning all the necessary data of nodes and pipes, the final simulation has been run for. EPANET 2.0 has provided pressure at different nodes and flow rate in the pipes.

Table 3: Network table - Nodes

Network Table - Nodes

Node ID	Elevation m	Base Demand LPS	Demand LPS	Head m	Pressure m
Junc 8	581.5	1.125	1.24	593.22	11.72
Junc 9	581	1.5	1.65	591.02	10.02
Junc 10	581.3	1.125	1.24	589.73	8.43
Junc 11	581.5	1.875	2.06	588.68	7.18
Junc 12	580	1.125	1.24	587.61	7.61
Junc 13	580	1.125	1.24	587.15	7.15
Junc 14	580	1.125	1.24	586.64	6.64
Junc 15	580	1.5	1.65	586.54	6.54
Junc 16	579	1.5	1.65	586.42	7.42
Junc 17	578.5	1.125	1.24	586.37	7.87
Junc 18	578	1.875	2.06	586.35	8.35
Junc 19	580	1.125	1.24	586.53	6.53
Junc 20	579	1.125	1.24	586.50	7.50
Junc 21	580	1.125	1.24	586.49	6.49
Junc 22	580	1.5	1.65	587.13	7.13
Junc 23	581.5	0.75	0.83	588.68	7.18
Junc 24	582.5	0.75	0.83	593.22	10.72
Junc 25	583	1.125	1.24	590.24	7.24
Junc 26	583.5	1.5	1.65	589.55	6.05
Junc 27	584	.75	0.83	589.52	5.52

Table 4: Network Table – Nodes at 1:00 hrs

Network Table - Nodes at 1:00 Hrs

Node ID	Elevation m	Base Demand LPS	Demand LPS	Head m	Pressure m
Junc 8	581.5	1.125	1.46	598.65	17.15
Junc 9	581	1.5	1.95	598.41	17.41
Junc 10	581.3	1.125	1.46	598.27	16.97
Junc 11	581.5	1.875	2.44	598.13	16.63
Junc 12	580	1.125	1.46	597.72	17.72
Junc 13	580	1.125	1.46	597.10	17.10
Junc 14	580	1.125	1.46	596.40	16.40
Junc 15	580	1.5	1.95	595.88	15.88
Junc 16	579	1.5	1.95	595.19	16.19
Junc 17	578.5	1.125	1.46	594.95	16.45
Junc 18	578	1.875	2.44	594.80	16.80
Junc 19	580	1.125	1.46	595.82	15.82
Junc 20	579	1.125	1.46	595.65	16.65
Junc 21	580	1.125	1.46	595.56	15.56
Junc 22	580	1.5	1.95	597.08	17.08
Junc 23	581.5	0.75	0.98	598.13	16.63
Junc 24	582.5	0.75	0.98	598.61	16.11
Junc 25	583	1.125	1.46	598.24	15.24
Junc 26	583.5	1.5	1.95	598.02	14.52
Junc 27	584	0.75	0.98	598.00	14.00

Table 5: Network Table – Links at 0:00 hrs

Network Table - Links at 0:00 Hrs

Link ID	Length m	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor	Status
Pipe 1	39.66	21.45	2.65	55.61	0.016	Open
Pipe 2	26.88	19.80	2.44	47.95	0.016	Open
Pipe 3	24.63	18.56	2.29	42.54	0.016	Open
Pipe 4	34.46	15.68	1.93	31.11	0.017	Open
Pipe 5	17.02	14.44	1.78	26.71	0.017	Open
Pipe 6	29.26	11.55	1.42	17.67	0.017	Open
Pipe 7	14.93	6.60	0.81	6.27	0.019	Open
Pipe 8	33.79	4.95	0.61	3.68	0.020	Open
Pipe 9	25.65	3.30	0.41	1.74	0.021	Open
Pipe 10	48.70	3.71	0.46	2.16	0.021	Open
Pipe 11	29.74	2.47	0.31	1.02	0.022	Open
Pipe 12	54.61	1.24	0.15	0.28	0.024	Open
Pipe 13	36.10	2.06	0.25	0.73	0.022	Open
Pipe 15	23.89	1.65	0.20	0.48	0.023	Open
Pipe 16	37.56	0.83	0.10	0.13	0.026	Open
Pipe 18	32.31	7.43	0.92	7.80	0.019	Open

Table 6: Network Table – Links at 1:00 Hrs

Network Table - Links at 1:00 Hrs

Link ID	Length m	Roughness	Flow LPS	Velocity m/s	Unit Headloss m/km	Friction Factor	Status
Pipe 1	12.09	150	35.59	1.95	19.71	0.015	Open
Pipe 2	8.19	150	33.64	1.84	17.75	0.016	Open
Pipe 3	8.41	150	32.17	1.76	16.35	0.016	Open
Pipe 4	9.58	150	18.52	2.28	42.38	0.016	Open
Pipe 5	17.02	150	17.06	2.10	36.40	0.016	Open
Pipe 6	29.26	150	13.65	1.68	24.08	0.017	Open
Pipe 7	14.93	150	7.80	1.71	34.68	0.018	Open
Pipe 8	33.79	150	5.85	1.28	20.36	0.019	Open
Pipe 9	25.65	150	3.90	0.86	9.61	0.020	Open
Pipe 10	48.70	150	4.39	0.96	11.95	0.019	Open
Pipe 11	29.74	150	2.92	0.64	5.64	0.021	Open
Pipe 12	54.61	150	1.46	0.32	1.56	0.023	Open
Pipe 13	36.10	150	2.44	0.53	4.02	0.021	Open
Pipe 15	23.89	150	1.95	0.43	2.66	0.022	Open

4. CONCLUSION

From above study it was concluded that; EPANET software is time saving and has no limitation for number of nodes, number of pipes or pumps to be modelled and analyzed in it so complex networks can be easily solved. As the number of iterations increase, the value of head loss becomes closer to zero and to verify the obtained answers, balancing of flows at each point is done. The results obtained using Hazen Willems method and EPANET software are nearly equal. Newton-Raphson method is quite difficult to for the analysis of large network, but it gives acceptable result in a smaller number of iterations.

The minimum water pressure should be maintained in the drinking water distribution network is 7m . If the diameter of the pipe or area of the pipe is changed with respect to length then the water pressure in the pipe is maintained.

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