Analysis of pipe flow & head loss of a modeled network based on EPANET in a Water Treatment Plant at Raipur, West Bengal

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Abstract

The study has been carried out in two phases. In the first phase, a model of pipe network is developed based on EPANET (Version 2.0) in such a way that the modeled flow data for various zonal overhead reservoirs takes almost equal fill up times by adjusting the flow control valves (FCV) and changing some other parameters such as flow parameter of flow control valves, loss coefficient of throttle control valves (TCV), percentage of opening of pressure reducing valves (PRV) etc. In the second phase, a lot of modifications have been done by adjusting different parameters of TCV, PRV and FCV for optimizing the head losses in pipes and valves in the modeled system so that the desired water inflow to the zonal overhead reservoirs is achieved. It has been found that in this network, the time requirement taken for filling different overhead reservoir fully are not similar at all. Some of these fill up early and few other take a long duration. This will cause differential water supply and the result may be indicated insufficient supply to the local consumers. The results conclude that 60% and more reduction of losses have occurred in the pipe lines and valves (average head loss of 90 valves out of 95 is less than 24 m.). The validation of the model also reveals that 92.31% success (22 overhead reservoirs out of 24 reservoirs and two booster stations, are filled up completely after 5.5 hr & 6 hr respectively for maintaining the same fill up times of all the overhead reservoirs.

Key words: Overhead Reservoir, Gradient Method, EPA NET 2.0, Head loss, Control valves

1. INTRODUCTION

1.1 Brief description of the process in the Water Treatment Plant (WTP)

The rising main of the long pipe line for the Water Supply Scheme under Public Health Engineering Department (PHED), Government of West Bengal of the Dakshin Raipur Water Treatment Plant, South 24 Parganas, West Bengal, India has been considered in this study area. The network comprises of a Clear Water Reservoir (CWR) from which two pumps are supplying water to twenty four (24) numbers of Zonal Overhead Reservoirs (OHR) and two Ground Level Reservoirs (GLR) (BS-I & BS-II for feeding 27 more overhead reservoirs) are connected through the rising main. Intake raw water pump house is situated at 1.5 km away from Dakshin Raipur Water Treatment Plant which is situated at the bay of the Hooghly River. The intake jetty is designed to abstract 152 MLD of water from the river Hooghly. The river water is not fit for human consumption because of high turbidity & other chemical & bacteriological contamination and needs thorough treatment. The water from river Hooghly is sucked into the raw water pump house by 3 vertical pumps of 335 HP each having capacity of discharge 3410 m^3/hr.

Water is delivered to the inlet well of the treatment pump complex. As the water flows through the inlet channels chemicals such as solutions of Chlorine, Alum & Poly-electrolyte are added. The preparation of Alum & Poly-electrolyte solution and dosing the same is another important feature of the treatment. The alum cakes are transported into the alum tray and water is spread over it. The alum solution is passed into the dosing tank where it is thoroughly mixed with the help of agitator. The required alum or poly-electrolyte dose is pumped out by the metering pumps.

Water flows into the flash mixtures where chemicals are mixed thoroughly by rotating paddles. Water then enters into the center of the clarifloculator. In the clarifloculator zone flocks are formed by gentle rotation of the rotating paddles inside. In the clarifier zone the water is comparatively clean, because of deposition of sludge at the bottom. Water is then collected into the Launders. Now at the filter beds here the microphylls and other organism are removed in the sand beds. The filtered water is then collected into the underline drain. It is collected to clean water channel. Here the disinfection is carried out adding chlorine into the clear water, which is collected to the under water reservoir.

The disinfection of water is of utmost importance of the treatment process. The dose of gaseous chlorine is regulated by the chlorinator and its accessories before pumping out clear water. The clear water is pumped out by 5 numbers 1100 HP pumps having 6.6 kV HT motor situated to clear water pump house. When filter bed is clogged, it is backwashed by air and water from the bottom of the filter bed. The waste water is collected in the waste water channel and conveyed to the sludge pump. Finally the waste water is pumped out in the sludge pond.

The treated water travels along a distance of 50 km and is again stored into two underground reservoirs at Boosting Stations I & II. From here the water is further boosted with pumps. Chlorine is further administered here for effective disinfection of water at the consumer point. The potable water is finally conveyed to 55 (currently 51) zonal overhead reservoirs from where it gets distributed to the consumers. Times of supply of water to the local consumers are 6.30 am to 8.00 am, 12.00 noon to 1.30 pm & 3.30 pm to 5.00 pm

In a water treatment plant (WTP) there are a large number of zonal overhead reservoirs. Efficient design of clear water pipe network system of the rising main is critical for this plant, as each overhead reservoir requires different rate of inflow for filling up almost simultaneously. The filtered water is supplied through a network of pumps, pipes, valves and other equipments in such a way that each of the tanks receives the designed water flow. However, during the plant operation, it is often seen that the different tanks in the network, get much less or much more than the required amount of water (demand) during a certain interval. This may be due to improper design of the network or additional head drops in the valves. Under such a situation, any corrective action can only be taken, if one calibrates the measured flow of the existing network of the water treatment plant with a computer model like EPANET (Version 2.0), which is gradient algorithm based software that has been invented by Rossman (1994) and developed by the Water Supply and Water Resources Division of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory. It is public domain software

The main objective of the study is not only to develop a model for pipe network based on EPANET but also to optimize the head losses in pipes and valves in the modeled system in such a way that the desired water flow in the model network can be achieved.

1.2 Methodology

The temperature of the modeled pipe network is considered as 25°C. To begin with, each pipe diameter is taken as equal to its internal pipe diameter. All the pipe diameters have been taken as equivalent pipe diameter, but the lengths remain unchanged. Equivalent internal pipe diameters and lengths are calculated due to lesser number of pipes in the equivalent network model. All tanks are assumed to be fully filled twice a day (during 12 hours operation daily). For the existing network, the hydraulic head at the supply point is taken as 81.8 m and that at the two booster reservoirs (CWR-I & CWR-II) as 0.5 m. Elevation of all the tanks above a common datum of the bottom shell is assumed as 25 m. Initial level (height in feet of the water surface above the bottom elevation of the tank at the start of simulation), Minimum and Maximum level (the minimum and the maximum height of the water surface above the bottom elevation of all the tanks) are 0.5 m, 0.5 m and 5.0 m respectively. Tanks are not allowed to drop below minimum level and above maximum level. Sluice valve, Butterfly valve & Air valves (which are present in the actual plant layout) are considered as FCV, TCV & PRV respectively. Setting parameter (pressure) in all PRVs are 75m and loss coefficients for all TCVs are 0.5. Firstly, an iteration scheme is used in arriving at the final values of the flow at each tank in such a way that all the arriving times remain same. Finally, the system is modeled to optimize the head losses in pipes and valves so that desired water flow in the network is achieved. The velocity (V) has been calculated using Hazen Williams's formula. Thereafter, the loss coefficient (K) is evaluated by equating the corresponding pressure drop data with $KV^2/(2g)$. The friction loss has been calculated using Hazen Williams Equation using C = 130 (for MS Pipes), 110 (for CI, DI Pipes). The changes in the network are incorporated in the model until the pressure is optimized and maintaining the flow values by the flow control valves as given in their settings. The iterative scheme has been terminated when the flow values matches reasonably well.

1.3 Analysis of pipe flow by Gradient Method using EPANET

The method used in EPANET to solve the flow continuity and head loss equations that characterize the hydraulic state of the pipe network at a given point in time can be termed a hybrid node-loop approach. Todini and Pilati (1987) and later Salgado et al. (1988) chose to call it the "Gradient Method". Similar approaches has been described by Hamam and Brameller (1971) (the "Hybrid Method) and by Osiadacz (1987) (the "Newton Loop-Node Method"). The only difference between these methods is the way in which link flows are updated after a new trial solution for nodal heads has been found. Because Todini's approach is simpler, it was chosen for use in

EPANET.

Assume we have a pipe network with N junction nodes and NF fixed grade nodes (tanks and reservoirs). Let the flow-head loss relation in a pipe between nodes i and j be given as:

$$h_{ij} = H_i - H_j = r\,Q_{ij}^{\,n} + m\,Q_{ij}^{\,2} \text{ - - - - - - - - - - (i)}$$

Where H = nodal head; h = head loss; r = resistance coefficient; Q = flow rate; n=flow exponent; m=minor loss coefficient

The value of the resistance coefficient will depend on which friction head loss formula is being used. For pumps, the head loss (negative of the head gain) can be represented by a power law of the form

$$h_{ij} = -\,\omega^2 [\, h_0 - r(Q_{ij} / \omega)^n \,]$$

Where h_0 = shutoff head for the pump; w? = a relative speed setting; r and n = pump curve coefficients

The second set of equations that must be satisfied is flow continuity around all nodes:

$$\sum_j Q_{ij} - D_i = 0 \ \text{ for } i = 1,\ldots..N \text{ - - - - - - - - - - - (ii)}$$

Where D_i is flow demand at node i and by convention, flow into a node is positive.

For a set of known heads at the fixed grade nodes, we seek a solution for all heads H_i and flows Q_{ij} that satisfy the equations (i) and (ii).

The Gradient solution method begins with an initial estimate of flows in each pipe that may not necessarily satisfy flow continuity. At each iteration of the method, new nodal heads are found by solving the matrix equation:

$$AH = F \text{ - - - - - - - - - - - - - - - - (iii)}$$

Where A = an (N×N) Jacobian matrix; H = an (N×1) vector of unknown nodal heads; F = an (N×1) vector of right hand side terms; The diagonal elements of the Jacobian matrix are:

$$A_{ij} = \sum_j P_{ij}$$

While the non-zero, off-diagonal terms are: $A_{ij} = -\,P_{ij}$

Where P_{ij} is the inverse derivative of the head loss in the link between nodes i and j with respect to flow.

For pipes, $P_{ij} = \dfrac{1}{nr\left|Q_{ij}\right|^{n-1} + 2m\left|Q_{ij}\right|}$, while for pumps,

$$P_{ij} = \frac{1}{n\omega^2 r(Q_{ij}/\omega)^{n-1}}$$

Each right hand side term consists of the net flow imbalance at a node plus a flow correction factor:

$$F_i = \left[\sum_j Q_{ij} - D_i\right] + \sum_j y_{ij} + \sum_f p_{if} H_f$$

Where the last term applies to any links connecting node i to a fixed grade node f and the flow correction factor y_{ij} is: $y_{ij} = P_{ij}(\, r\,|\,Q_{ij}\,|^n + m\,|\,Q_{ij}\,|^2)\ \mathrm{sgn}\,(\,Q_{ij})$ for pipes and $y_{ij} = -P_{ij}\, w^2\,(\,h_0 - r\,(\,Q_{ij}/W\,)^n\,)$ for pumps, where sgn(x) is 1 if x > 0 and -1 otherwise. (Q_{ij} is always positive for pumps.) After new heads are computed by solving Eq. (iii), new flows are found from:

$$Q_{ij} = Q_{ij} - [y_{ij} - P_{ij}\,(H_i - H_j)] \text{ - - - - - - - - (iv)}$$

If the sum of absolute flow changes relative to the total flow in all links is larger than some tolerance (e.g., 0.001), then Eqs. (iii) and (iv) are solved once again. The flow update formula (iv) always results in flow continuity around each node after the first iteration.

2. INPUT DATA & OUTPUT

The basic input data (obtained Direct from Treatment Plant) for the study has been taken from the following documents and subsequent communication with Public Health Engineering Department (PHED), Government of West Bengal of the Dakshin Raipur Water Treatment Plant, South 24 Parganas.

1. EPANET layout **(Fig. 3)** of Rising Main within Jurisdiction of South 24 Parganas Water Supply Division I.

2. Zonal overhead reservoir capacity (in m^3).

Zone No.	Capa City (m^3)	Inflow Rate (m^3/hr) (Output from EPA NET)					Zone No.	Capa City (m^3)	Inflow Rate (m^3/hr) (Output from EPA NET)				
		Required	**0** hr.	**3** hr.	**5.2** hr.	**6** hr.			Required	**0** hr.	**3** hr.	**5.2** hr.	**6** hr.
R1	800	133.3	133.3	133.3	133.3	0	R15	800	133.3	119.4	111.6	106.0	133.3
R2	800	133.3	133.3	133.3	133.3	0	R16	225	37.5	37.5	37.5	37.5	0
R3	550	91.7	91.7	91.7	91.7	0	R17	700	116.7	105.0	98.2	93.3	135.0
R4	900	150.0	150.0	150.0	150.0	0	R18	800	133.3	133.3	133.3	133.3	0
R5	900	150.0	150.0	150.0	150.0	0	R19	900	150.0	150.0	150.0	150.0	0
R6	450	75.0	75.0	75.0	75.0	0	R20	550	91.7	91.7	91.7	86.8	0
R7	700	116.7	116.7	116.7	116.7	0	R21	550	91.7	91.7	91.7	91.7	0
R8	700	116.7	116.7	116.7	116.7	0	R22	550	91.7	91.7	91.7	91.7	0
R9	900	150.0	150.1	145.6	142.2	0	R23	900	150.0	150.0	150.0	150.0	0
R10	700	116.7	116.7	116.7	116.7	0	R24	350	58.3	55.0	55.0	55.0	0
R11	450	75.0	75.0	75.0	75.0	0	Total	16777	2826.3	2800.0	2777.8	2448.2	268.3
R12	900	150.0	150.0	150.0	150.0	0	B.S. I	6150	1050.0	1050.0	1050.0	1050.0	1050.0
R13	567	94.5	94.5	92.2	89.2	0	B.S. II	6891	1160.0	1060.0	1160.0	1160.0	1160.0
R14	1135	189.2	189.2	189.2	189.2	0	Grand Total	29818	5036.3	4910.0	4987.8	4658.2	2478.3

B.S.-I (supplies R25 to R33) **Onwards & B.S.-II** (supplies R40 to R51) **Onwards**

Zone No.	**Capacity** (m^3)	**Zone No.**	**Capacity** (m^3)	**Zone No.**	**Capacity** (m^3)	**Zone No.**	**Capacity** (m^3)
R25	1000	R31	700	R41	450	R47	550
R26	350	R32	700	R42	700	R48	450
R27	800	R33	550	R43	800	R49	340
R28	550	Total	**6150**	R44	700	R50	151
R29	800			R45	800	R51	700
R30	700	R40	800	R46	450	Total	**6891**

3. Specification of each of the pumps:-

Type	At **Dakshin Raipur**	At **B. S.- I**	At **B. S.- II**
	Horizontal centrifugal	Horizontal centrifugal	Horizontal Centrifugal
No of pumps	5	5	5
Operating now	2	2	2
Capacity (CMH)	2184	650	650
Power (kW)	800	90	90
Speed (rpm)	1450	1450	1450
Efficiency (%)	75-76	75-76	75-76
Total head (m)	60	42	42
Individual head (m)	65	60	60
Pump elevation (m)	4	3	3

3. RESULTS AND DISCUSSION

The treated water travels along a distance of 124 km (only Rising Main within Jurisdiction of South 24 Parganas Water Supply Division I) from the treatment plant. At present CWR-I & CWR-II are closed. The treated water is again stored into two underground reservoirs at Boosting Stations I & II. From here the water is further boosted with horizontal centrifugal pumps. Chlorine is further administered here for effective disinfection of water at the consumer point. The potable water is finally conveyed to 55 (currently 51) zonal overhead reservoirs from where it gets distributed to the consumers. In the treatment plant and two booster stations horizontal centrifugal pumps are operating during the 12 hours of operation in every day. All the overhead reservoirs are filled up fully twice in a day. So every reservoir needs 6 hours to fill up completely. Here, "P" denotes Pump, "R" denotes Reservoir, "BS" denotes Booster Station and "CWR" denotes Clear Water Reservoir, "N" denotes node.

Fig. 3 shows the variations of head losses in all pipes & pressures in all nodes after 5hr. 20 minutes of operation in the rising main. Flow through all the pipes are within range, maintained by some of the control valves. But due to low pressure heads at J12-R15 and J10-R17, they are maintained less inflow than the requirement. There is no pipe where the inflow is lesser than the designed flow. In the network, all pressure regulating valves are opened at 75m or equal to & greater than 75 m. Throughout the operation, the head loss is maximum (65.22 m) in valve V3 (PRV). In other valves like 3b (TCV), 3e (FCV), 3f (TCV), 3j (TCV), 3n (FCV), 4k (FCV), 7e (TCV) having head losses 46.62 m, 32.27 m, 29 m, 40.98 m, 25.1 m, 29.63 m and 35.62 m respectively are found from the model output. These losses are little high by considering as equivalent pipe diameters other than in actual cases.

Fig. 4 (a), (b), (c), (d), (e) & (f) reveal that all reservoirs are filled up around 5hr. 30 min., excepting

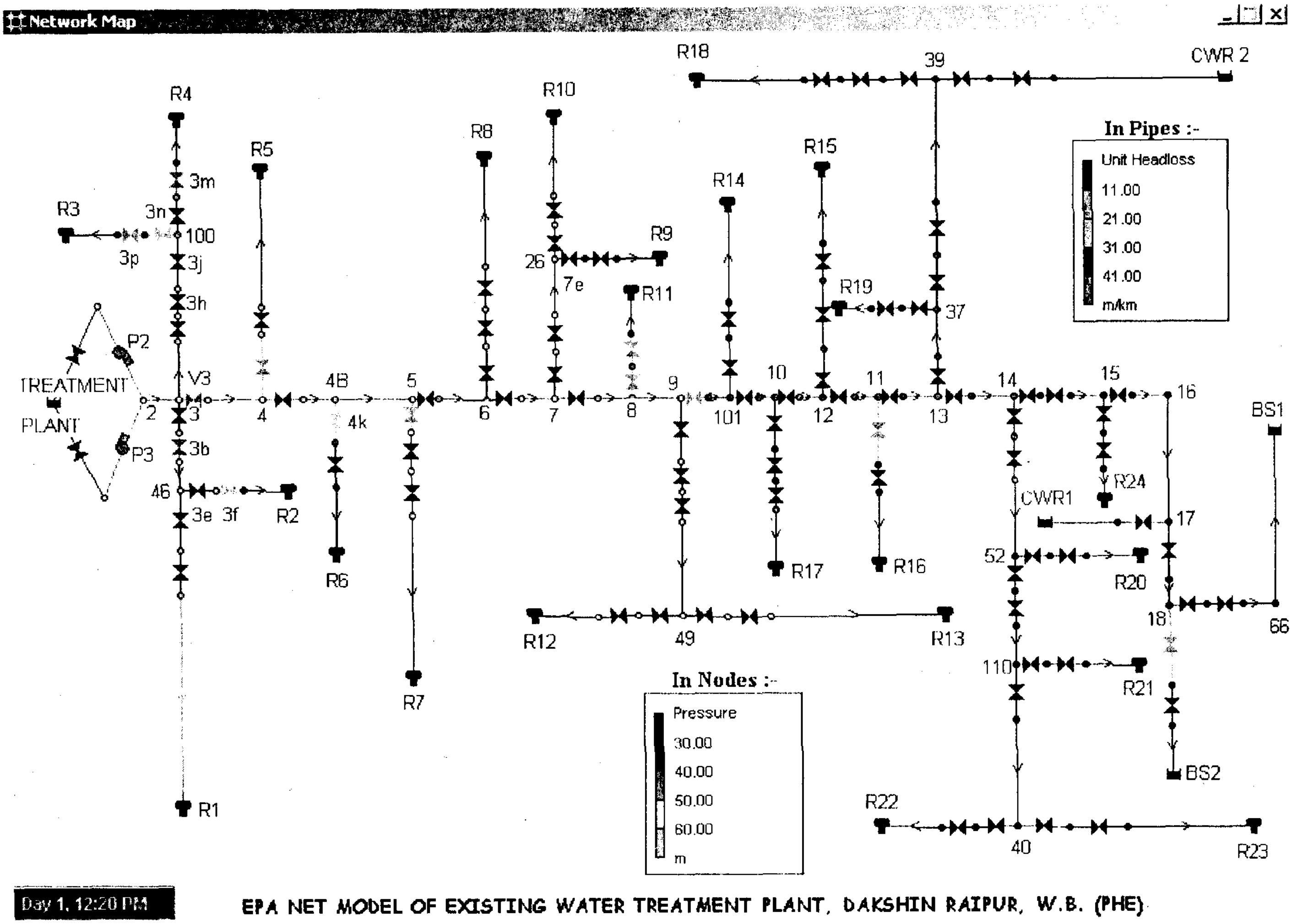

Figure. 3 : EPANET Model of existing Water Treatment Plant, Dakshin Raipur, W.B. (PHE)

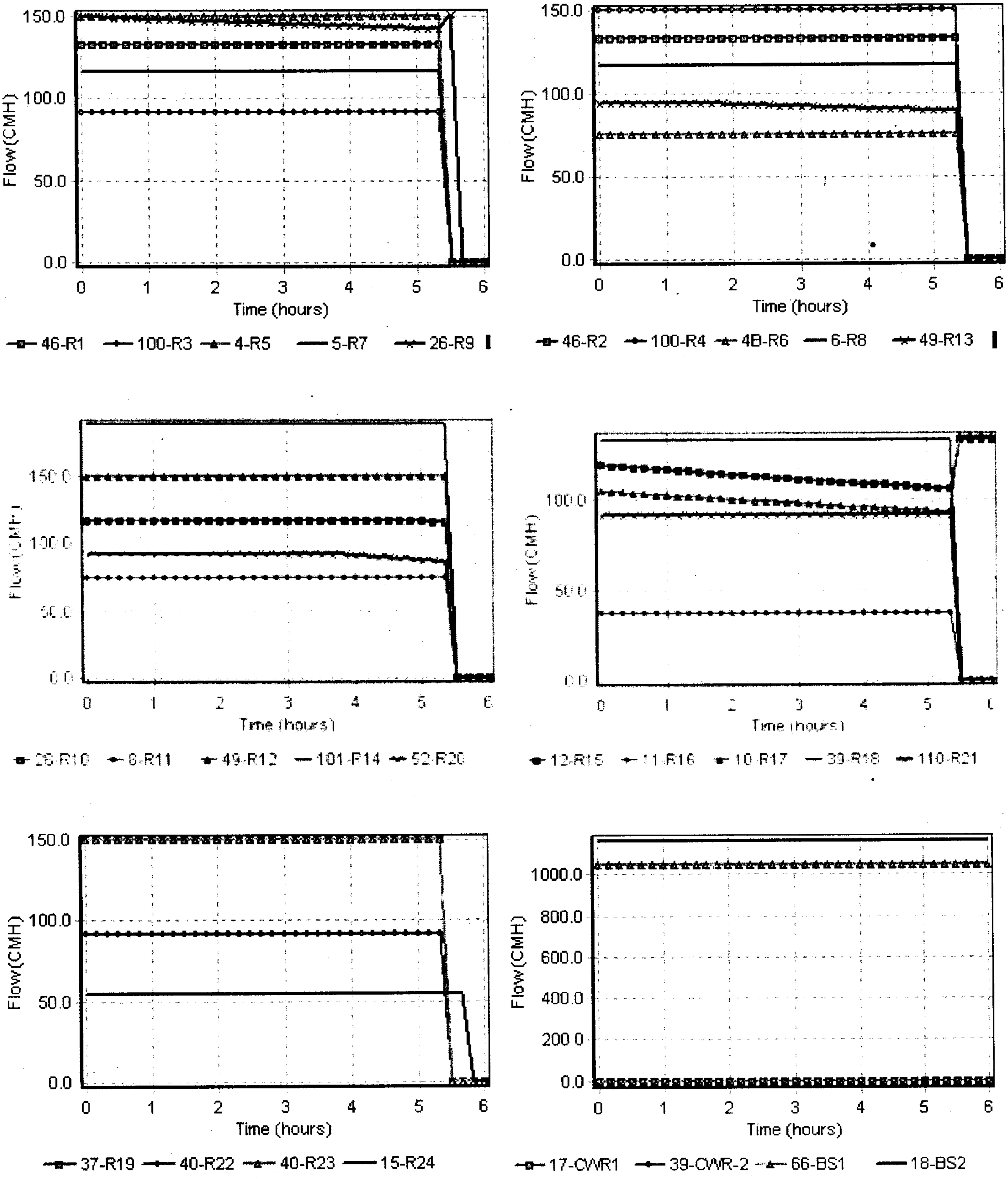

Figure. 4 (a), (b), (c), (d), (e) & (f) : Flow –Duration Curves of Reservoirs (R)

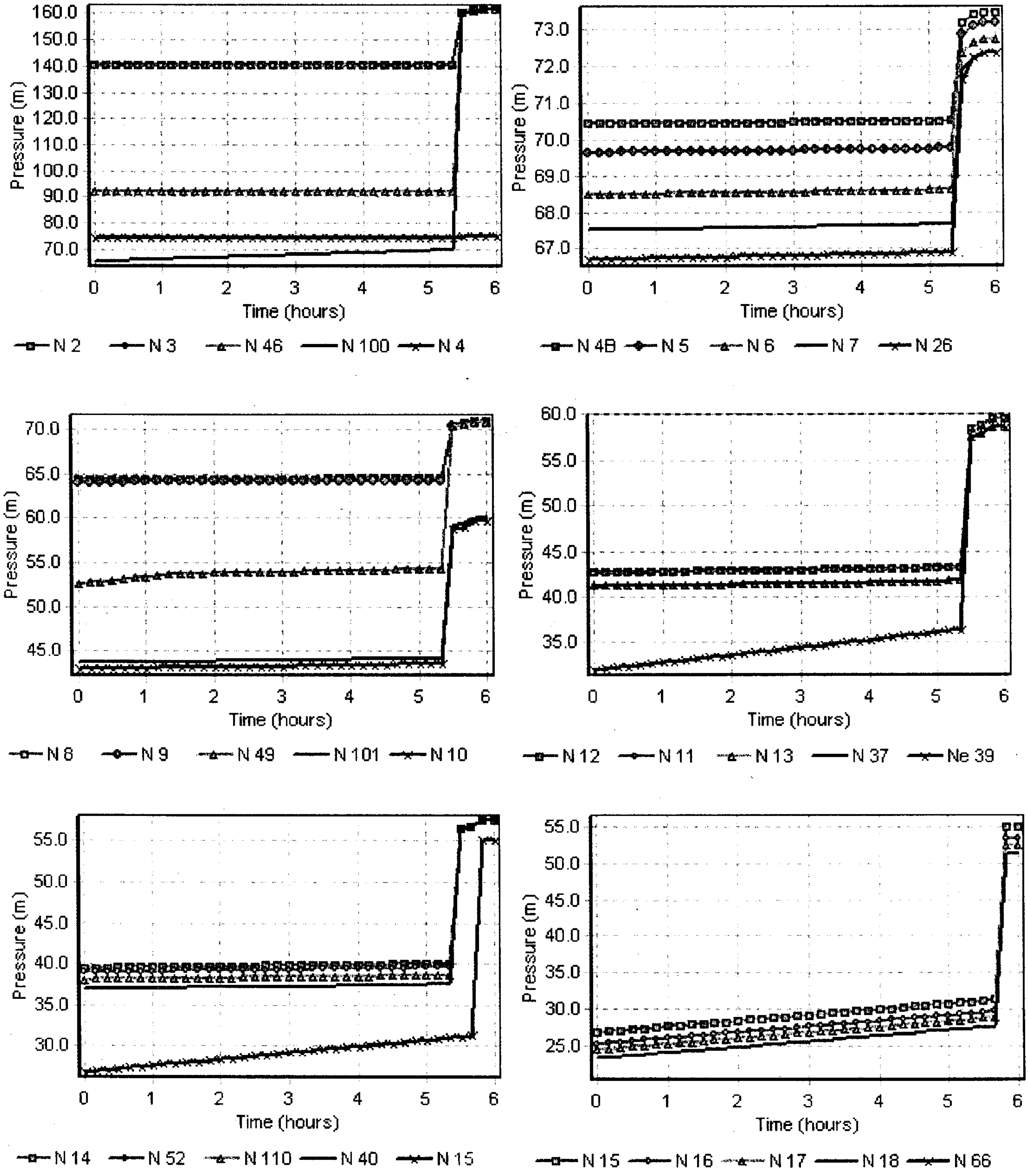

Figure. 5 (a), (b), (c), (d), (e) & (f) : Pressure- Duration Curves at different nodes (N) reservoir 15 & 17 (fills up after 6 hr.). So, there is a chance of huge pressure rise after that time. Our initial target was to fill up all the overhead head reservoirs almost after a same time domain without increasing the pressure head over pipes. From the figures it is clear that 22 overhead head reservoirs out of 24 overhead head reservoirs are filled up completely around 5hr. 30 min. So 91.67 % overhead head reservoirs are filled almost simultaneously. Now two Booster Stations also fill up fully around 6 hr. of operation. The flow rates to booster stations are maintained 2-2.5% higher as another 27 overhead reservoirs will be filled by the water taken from these two booster stations. So this negligible percentage of loss is assumed here. So, finally 92.31% success is achieved. Here R15 & R17 are filling fully after 6hr. 49 minutes & 6hr. 24 minutes respectively. So there is no chance of water hammer in those pipelines connecting the rising main with R15 & R17.

In actual case, the plant starts the water supply at 6.30 am to 24 overhead reservoirs and pump operations are started at 7 am and closed at 7 pm everyday. Now as the tanks are filling after 6 hrs so 8.33% volume is filling in every 30 minutes. It is considered that 8.33% of water is lost in every day. All overhead reservoirs fill up fully twice in a day (200%). Need of the water to the local people are 78%, 78% & 44% at 6.30 am to 8.00 am, 12.00 noon to 1.30 pm & 3.30 pm to 5.00 pm respectively. It is considered that 75% volume of water remains stored after 7 pm in each OHRs everyday. Therefore, the volumetric situation in all the overhead reservoirs (excepting R15 & R17) at 6.30 am, 7 am, 8 am, 12 noon, 1.30 pm, 3.30 pm, 5 pm & 7 pm are 75%, 49%, 13.66%, 80.33%, 27.33%, 60.67%, 41.67% & 75% respectively. Thus the results depict that there is no chance of water hammer in any of the pipe-line of the rising main.

Fig. 5 (a), (b), (c), (d), (e) & (f) of pressure for selected nodes the pressure increases certainly between 5.20th hr. to 6.00th hr of operation. At nodes 2 & 3 pressure increases almost 20m during that period. But pressure at the node 4 remains constant throughout the 6 hr. of operation. This indicates that maximum amount of pressure increase due to the pipeline series R1-46-100-R4. Maximum amount of pressure increases at node 100 (obtained 100m), and then at node 46 (obtained 75m). Pressure at node 15 increases suddenly after 5.45 hr of operation and almost 55 m of head increment is there. All the pressure regulating valves will open when the pressure in the valve is more than 75 m. Maximum unit head loss is in pipeline 46- R1 (12.88 m/km) and head loss in all the other pipes are always less than 10 m/km.

In practical, there will be no such high increment of pressure heads in different nodes as water is distributed to the local consumer in a certain interval (thrice daily).

4. CONCLUSIONS

In the first part of analysis, a base network model has been prepared to maintain the fill up times of all the overhead reservoirs almost same by adjusting the flow control valves and changing some other parameters like flow parameter of flow control valves, loss coefficient of throttle control valves.

Second part of this study is to optimize the losses by adjusting the loss coefficients (K) of throttle control valves, flow parameters of flow control valves and pressure parameter of pressure regulating valves. This causes to achieve 60% and more reduction of losses in the pipe lines and valves (average head loss of 90 valves out of 95 is less than 24m.). This validation of the model reveals then 92.31% success for maintaining the same time for filled up of all the overhead reservoirs. It is also concludes the initial volumetric capacity in each overhead reservoir is maintained by (62-94)% then there is no chance of water hammer in any of the pipe-line of the rising main.

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