environmental sciences proceedings

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Proceedings Paper

Application of Epanet 2.0 Software and Jal-Tantra Web System for Optimal Hydraulic Design of Water Distribution System for University of Kashmir †

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† Presented at the 7th International Electronic Conference on Water Sciences, 15–30 March 2023; Available online: https://ecws-7.sciforum.net.

Abstract: Availability of the portable water in sufficient quantity and standard quality is one of the basic requirements of a civilization. The study area of the present work is the University of Kashmir where the existing WDS has served its function for more than 60 years. The UOK is a growing campus and the existing WDS is unable to meet the demands of growing population and suffers from excessive leakage losses and insufficient pressure head at the nodes. In this work it has been attempted to replace the existing WDS by an optimally designed WDS for the University of Kashmir, so that the objective functions of maximum efficiency of performance and minimum capital cost in terms of pipe diameters are met. From the literature available EPANET has been found to perform the hydraulic and water quality modelling with fair degree of accuracy, ease of use and free of cost. Therefore, for the hydraulic analysis EPANET 2.0 has been used. To augment the assessment of efficiency of performance of the WDS, the TPI (technical performance indices) for pressure and velocity at the hour of peak demand have been evaluated. Finally, the cost optimality of the network in terms of the pipe diameters has been validated by using JAL-TANTRA web system developed by IIT Bombay.

Keywords: optimal hydraulic design; (WDS) water distribution system; EPANET 2.0; JalTantra web system; performance analysis; (TPI) technical performance indices

Citation: Ajaz, M.; Ahmad, D. Application of Epanet 2.0 Software and Jal-Tantra Web System for Optimal Hydraulic Design of Water Distribution System for University of Kashmir. *Environ. Sci. Proc.* **2023**, *5*, x. https://doi.org/10.3390/xxxxx

Academic Editor(s):

Published: 15 March 2023

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1. Introduction

A well designed, efficiently performing WDS is a prerequisite for both urban and rural civilizations. Various softwares are available for modelling a WDS. The public domain softwares which are available free of cost include EPANET, Pipes and Loops, Branch, etc. and the paid ones available include Water cad, Watergems, Synergy water etc [1]. The paid versions are found to be to be more efficient in reference to the functionality, compatibility to different computational systems, graphical user interface, searching and optimizing algorithms [2]. However, EPANET is a freely available software which offers fair degree of accuracy and ease of use, when the cost is a constraint [3]. EPANET 2.0 was used by various researchers to design the WDS for different areas [4,5]. The design includes the layout of the WDS, modelling the pressure head at nodes and the velocity of flow in the links. However, no consideration was given to maximisation of the efficiency of performance and minimisation of the capital cost of the network.

The design of a WDS by EPANET 2.0 must be such that it serves as an "optimal" solution, which can be obtained by setting out objective functions of maximum efficiency of performance and minimum capital cost of the network [6,7]. The decision variables and the constraints for the optimality of the network design have to be decided after conducting a study of the factors affecting the optimality of the design and consulting various codes and manuals laying down the standard range of values of those parameters [8,9]. The decision variables for optimal hydraulic design include the pressure head at nodes, velocity of flow in the links, staging height of the storage tank. The check for cost optimality of the network formulated by EPANET 2.0 can be conducted using JalTantra web system [10,11]. The optimality of the network can further be checked by carrying out the performance analysis by various indices [12–14].

The objective of the current study is to provide an optimally designed WDS for the University of Kashmir using EPANET 2.0 software and JalTantra web system. The existing distribution system has already served for more than 60 years and is unable to meet the requirements of the growing population of the campus. Moreover, the worn out links and other components of the network lead to increased pipe and joint losses, reducing the efficiency of the existing distribution system. So there is an immediate need to redesign the water distribution network such that the design is an optimal solution meeting the objective functions of the maximum efficiency of performance and the minimum capital cost of the network, rather than just simulation of the pressure head and velocity of flow.

2. Methodology

2.1. Study Area Description

The study area for current work is the University of Kashmir is located at Hazratbal Srinagar, Jammu and Kashmir, India. It is a residential campus comprising of academic blocks, administrative blocks, hostels, medical unit, cafeteria and staff quarters. It has got a residential and floating population of 5324 and 1050, respectively for the Naseem Bagh Campus and 8066 and 1720, respectively for Hazratbal campus.

2.2. Methodological Approach

A quantitative research approach was employed to arrive at an optimal design of the WDS for University of Kashmir by use of various soft wares. Study of the literature related to the work was carried out and the standard books and codes related were consulted. Preliminary survey of the study area was succeeded by the division of the study area into two parts and provision of separate WDN for the two parts (Hazratbal campus and Naseem bagh campus). The source of water for the networks is the ground water reserve within the campus, which is pumped by means of the intermittently operating pumps into the overhead water tanks.

2.3. Methods of Data Collection

Secondary, quantitative data of the existing population of various academic blocks, hostels, family quarters and administrative blocks was obtained. The maximum depth of water in the bore wells below the ground level is 10 m. In addition, the campus plan of the study area was obtained. The data availability was facilitated by engineering wing of the UOK. Elevation data was obtained using Google earth pro.

2.4. Methods of Analysis

2.4.1. Evaluation of the EPANET 2.0 Input Parameters

A dead end type of distribution system was provided for both parts of the campus. The population forecast for the design period of 30 years, considering a constant increase of 5% per decade in both residential as well as the floating population was carried out [15,16]. The values of ADD for both parts of the network were evaluated using water demand as 135 lpcd for residential population and 45 lpcd for the floating population [17], the MDD was evaluated using a peak factor of '1.8' [18], a peak factor of '3' was used to calculate the MHD from the AHD [18]. Provisions to meet the fire demand were kept as per Indian standards [19]. Base demand multiplier equals the coincident draft in liter per person per minute; (Table 1). Two different demand patterns were used for the network nodes, one for the hostels and quarters (p1) and the other for academic building and administrative blocks (p2). The capacity of the storage tank was evaluated by the mass curve analysis, for both parts of the network. Discharge to be provided by the pump was worked out by considering 12 h of pumping per day, staging height of the tank above the ground level was selected after iterations to run the software successfully and achieve standard pressure heads at the nodes. The economical diameter of the rising main was obtained using Dupit's expression. The pressure head to be developed by the pump has to be equal to the sum of the delivery head, suction head and the frictional head loss in the rising main. Input power required to run the pump equals (ωQH/n), the efficiency of the pump (n) was taken as 75%. Energy required for 12 h per day operation of the pump was evaluated to arrive at the cost of the electrical energy @ Rs 5 per unit per day. Design flow rate of the Pump was taken equal to the MHD in litre per minute; (Table 2). After entering the EPANET 2.0 input data [20] and successful run of the software, hydraulic modeling of the networks was carried out. Simple controls (obtained after iterations to run the software successfully and achieve standard pressure heads at the nodes) as given in Table 3a,b were used to perform the hydraulic analysis.

Table 1. Demand calculations for WDS part I and WDS part II.

S NO	PARAMETER	WDS PART I	WDS PART II
1	Forecasted Residential Population	5324	8066
2	Forecasted Floating Population	1030	1720
3	Avg. Daily Demand (ADD)	891,000 litre	1,350,270 litre
4	Max Daily Demand (MDD)	1,603,800 litre	$2.43 * 10^6$ litre
5	Avg. Hourly Demand (AHD)	37,125 litre	56,262 litre
6	Max Hourly Demand (MHD)	111,375 litre	168,786 litre
7	Fire Demand	216,000 litre	216,000 litre
8	Coincident Draft	1,107,000 litre	1,566,270 litre
9	Base Demand Multiplier	0.104	0.1

Table 2. Storage tank and pump characteristics.

S NO	PARAMETER	WDS PART I	WDS PART II
1	Total storage capacity of the OHT	6.17 * 105 litre/day	8.24 * 105 litre/day
2	Volume of the OHT	617 m^3	824 m^3
3	Height of the OHT	10 m	12 m
4	Diameter of the OHT	9 m	9.5 m
5	Discharge to be provided by the Pump	0.037 cumec	0.056 cumec
6	Staging Height of the Tank	30 m	30 m
7	Economical diameter of the Rising main	250 mm	300 mm
8	Pressure head to be developed by the Pump	51 m	53 m
9	Input power for the Pump	24.7 kw	37.4 kw
10	Cost of electrical energy/day @ Rs 5 per unit	1482 rupees	2245 rupees
11	Design flow rate of the Pump	1856 lpm	2813 lpm

Table 3. (a, b) Simple control for WDS part (I, II).

(**a**)
link Pu1 closed if node T1 above 10
link Pu1 open if node T1 below 4
(**b**)
link Pu1 closed if node T1 above 12
link Pu1 open if node T1 below 6

2.4.2. Evaluation of the Technical Performance Indices (TPI) [13,14]

The behavior of the nodes and pipes of the WDS can be evaluated using the TPI (Technical performance indices), which are an indicative of the performance of the network with respect to the hydraulic parameters like pressure head and flow. Two types of the TPI are used, $TPI_{pressure}$ [14] and $TPI_{velocity}$ [14]. The TPI values range from 0 to 1, '0' for the poor service and '1' for efficient service. The TPI values for both parts of the network were evaluated as given below and compared with the standard values.

$$TPI_{pressure} = \Sigma\ (Q_i * TPI_i)/\Sigma\ Q_i$$

where, Q_i = nodal demand and (i) iterates over all the nodes of the network.

$TPI_i = 0,\quad P_i < P_{min}$

$= 1,\quad P_{min} < P_i < P_{ma}$

$= 1 - \{(P_i - P_{max})/(P_{max} - P_{min})\},\quad P_{max} < P_i < 100$

$= 0,\quad P_i > 100$

Here, P_i = prevailing pressure head at the node, P_{min} = 17 m, P_{max} = 70 m, TPI_i = TPI for the node i

$$TPI_{velocity} = \Sigma(Q_j * TPI_j)/\Sigma Q_j$$

where, Q_j = flow in pipe (j)and (j) iterates over all the pipes of the network.

$TPI_j = 0,\quad V_i < V_{min}$

$= (V_i - V_{min})/(V_{mean} - V_{min}),\quad V_{min} < V_i < V_{mean}$

$= (V_i - V_{max})/(V_{mean} - V_{max}),\quad V_{mean} < V_i < V_{max}$

$= 0,\quad V_i > V_{max}$

Here, V_i = prevailing flow velocity in pipe j, V_{min} = 0.2 m/s m, V_{max} = 3 m/s, TPI_j = TPI for the pipe j, $V_{mean} = (V_{min} + V_{max})/2$

2.4.3. Input Parameters for JalTantra Web System

The input data for nodes (elevations, base demand and minimum pressures) and pipes (start nodes, end nodes, length and roughness) is entered into the Jal-Tantra system. The commercially available pipe diameters along with their Cost per unit length according to the prevailing market rates is also provided. The input data is given in the Table 4a,b.

2.3.4. Optimization of the Hydraulic Model

Objective functions:

The first objective function is the maximization of the efficiency of performance of the proposed WDS in terms of the hydraulics. The second objective function is the minimization of the capital cost of the network in terms of the pipe diameters and valves.

Decision variables:

The decision variables are:

- P_i = prevailing pressure head at the node.
- V_i = prevailing flow velocity in pipe.
- H = Staging height of the storage reservoir.

Constrains:

- $17 \leq P_i \leq 70$ (meter) [18].
- $0.25 \leq V_i \leq 3.0$ (m/s) [18].
- H = 30 m (obtained after iterations to achieve standard values of hydraulic parameters)

Table 4. (a): General input data; (b): Commercial pipe data.

(**a**)

Network Name	WDS Part I	WDS Part II
Organization Name	UOK	UOK
Minimum Node Pressure	17	17
Default Pipe Roughness 'C'	100	100
Minimum Head loss per KM	0.010	0.01
Maximum Head loss per KM	65.000	65.000
Maximum Water Speed	3.000	3.000
Maximum Pipe Pressure	70.000	70.000
Number of Supply Hours	24	24
Source Node ID	31	55
Source Node Name	Tank (T1)	Tank (T1)
Source Elevation	1624.00	1623.00
Source Head	1628.00	1629.00

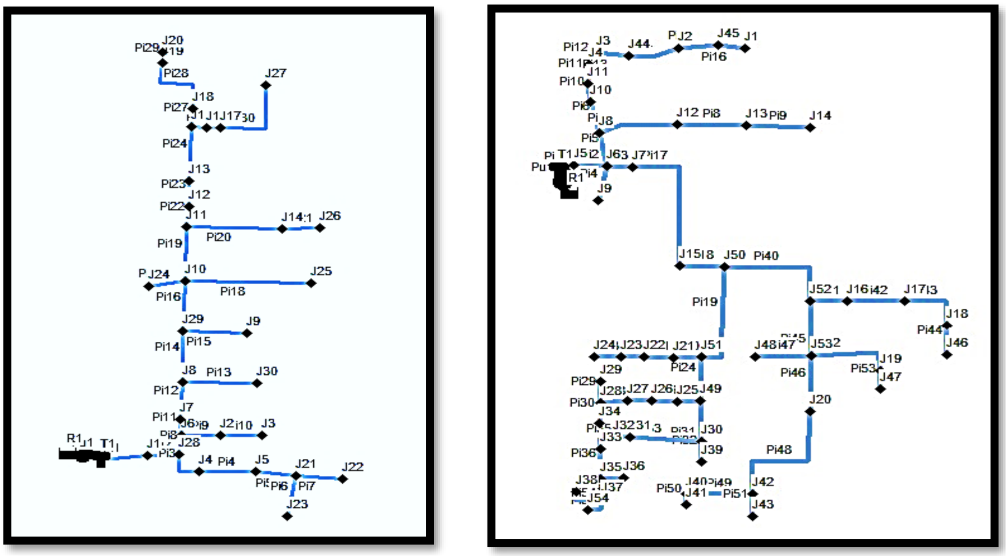
(**b**)

Diameter (mm)	Cost(Rs)
15	123
20	155
25	225
32	281
40	320
50	439
65	553
80	712
100	1031
125	1392
150	1626
200	2760

3. Results and Discussions

3.1. Distribution Network Layout

A dead end type of distribution network was drawn for both parts of the study area using EPANET 2.0, as shown in Figure 1a,b. The input data for nodes; elevation, base demand and demand pattern, was provided for each node and the input data for pipes; pipe diameter, roughness coefficient and length, was provided. The pipe network was laid for both the WDS, keeping in view the geometry of the study area the dead end type of distribution system was used for both the parts of the study area. Moreover, the basic assumption of the JALTANTRA web system, of having no loops in the network has to be satisfied.

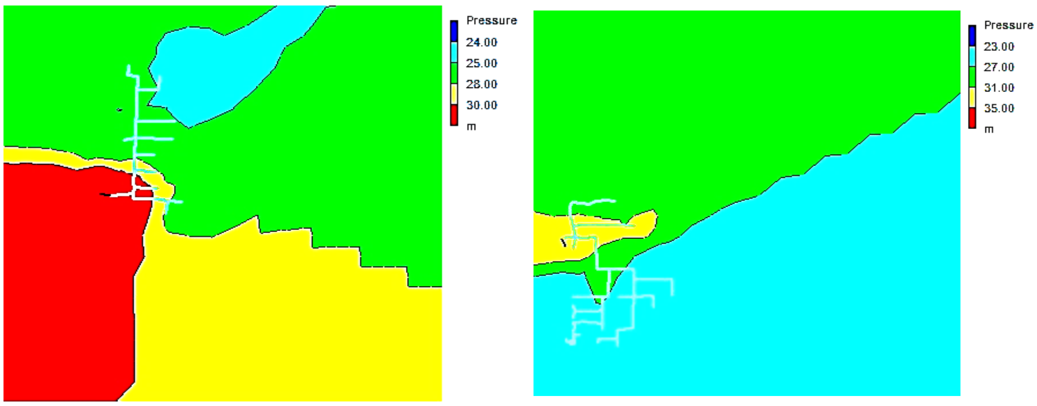


(**a**) (**b**)

Figure 1. (**a**,**b**): Distribution Network layout for WDS part I and part II.

3.2. *Contour Plan of Pressure Head at Nodes at 9 am (Peak Demand Hour)*

The contour plans given in Figure 2a,b show the distribution of pressure head across the layout of the network part I and II. Different colours representing the range of the pressure head is depicted by the scale provided in each map. The analysis indicates that the pressure head at all the nodes is greater than 17 m, thus the water at each node can reach up to a minimum of three stories of the building [18]. It also indicates that there is no need to use the PSVs. Moreover, the pressure head is also less than 70 m at each of the nodes of both parts of the WDS, thus there is no need of installing the PRVs at any of the nodes.

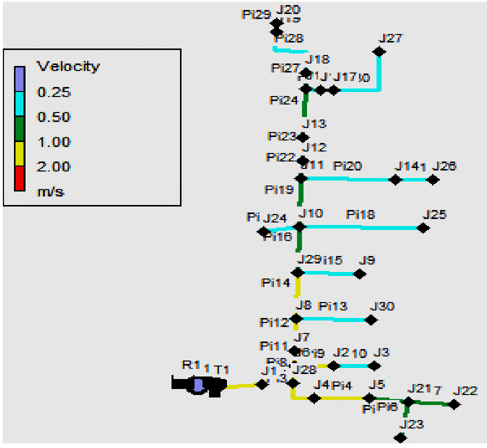


(**a**) (**b**)

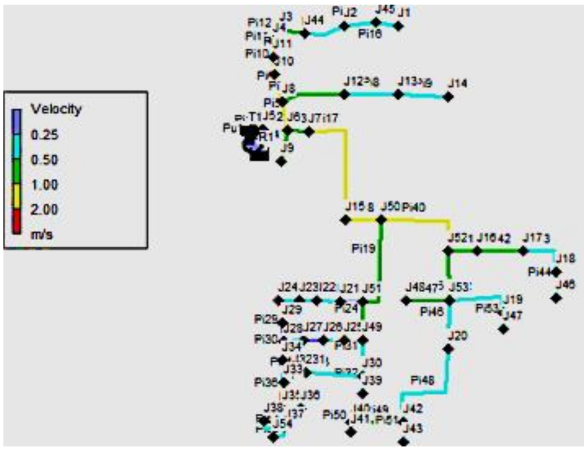
Figure 2. (**a**,**b**): Contour plan of pressure head at nodes at 9 am, for WDS part I and part II.

3.3. *Colour Coded Diagram of Velocity in Links at 9 am (Peak Demand Hour)*

The colour coded velocity diagram given in Figure 3a,b show the range of velocity of flow in each pipe of network part I & II. The scale showing the range of velocity represented by each colour is given for each of the networks. The analysis indicated that the velocity of flow for all the pipes in both the networks is in the range of 0.25 m/s to 3 m/s [18]. Thus there is no danger of erosion and deposition in the pipes at the hours of peak flow.



(**a**)

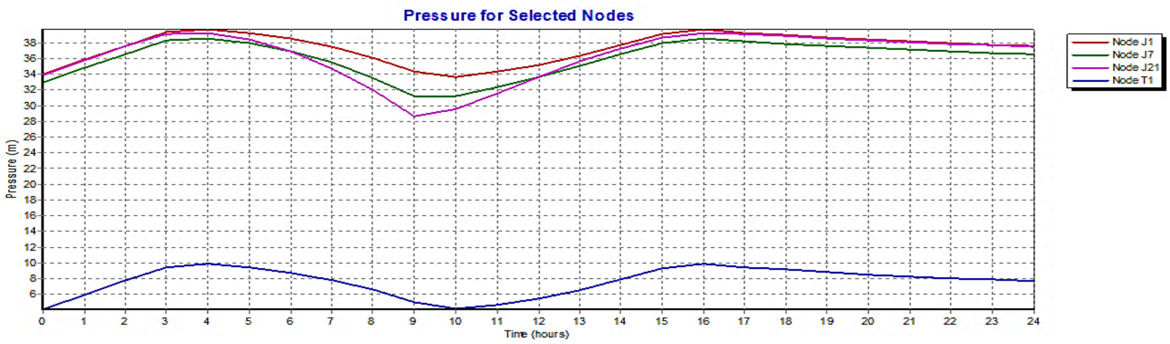


(**b**)

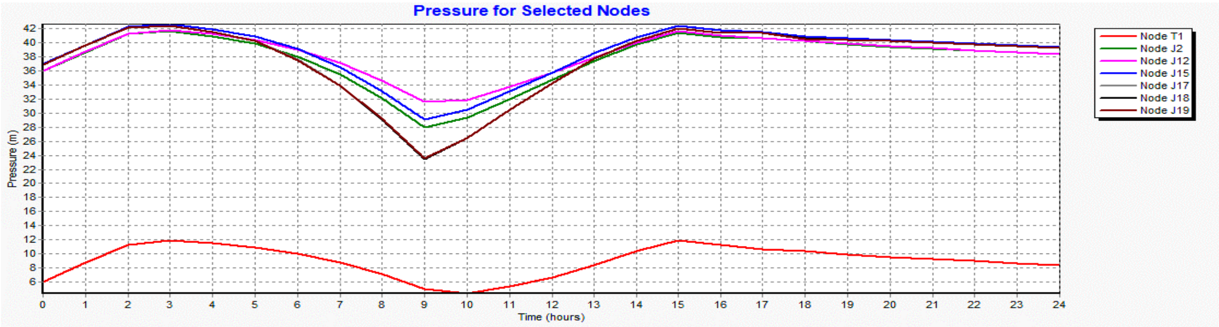
Figure 3. (**a**,**b**): Colour coded diagram of velocity in links at 9 a.m., WDS part I&II.

3.4. Time Series Plots

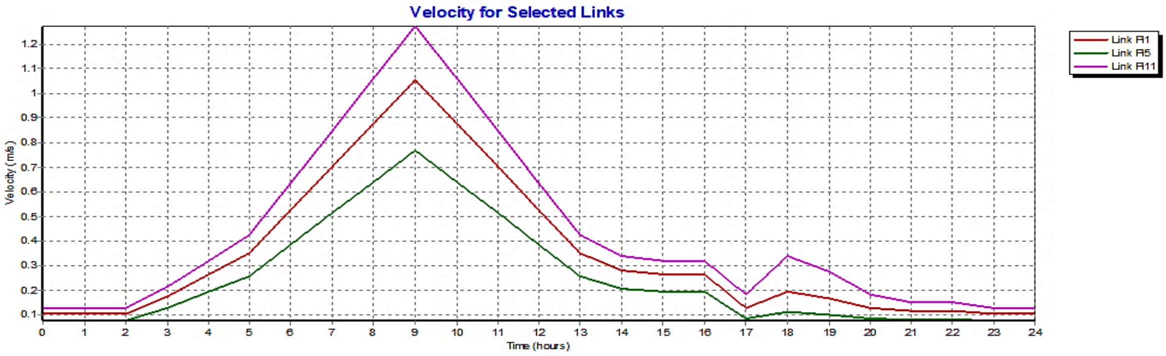
The temporal variation of pressure head at the peak demand nodes and the water level in the tank; Figure 4a,b, velocity in pipes feeding the peak demand nodes; Figure 4c,d is shown. It is indicated that the pressure head lies within the recommended ranges with respect to all the nodes shown throughout a day. However, it is seen that the velocity of flow in the pipes at the hours of minimum demand is less than 0.25 m/s, this can be attributed to the demand pattern adopted and the fact that the diameters have been selected to provide sufficient pressure head and cater to the demands at the peak hour. Moreover, the discharge in the pipes at these hours is less and thus there is no serious hazard of deposition in the pipes. However, the velocity less than 0.25 m/s may be adopted to meet the hydraulic standards only if there is a suitable method of scouring of the pipes [18].



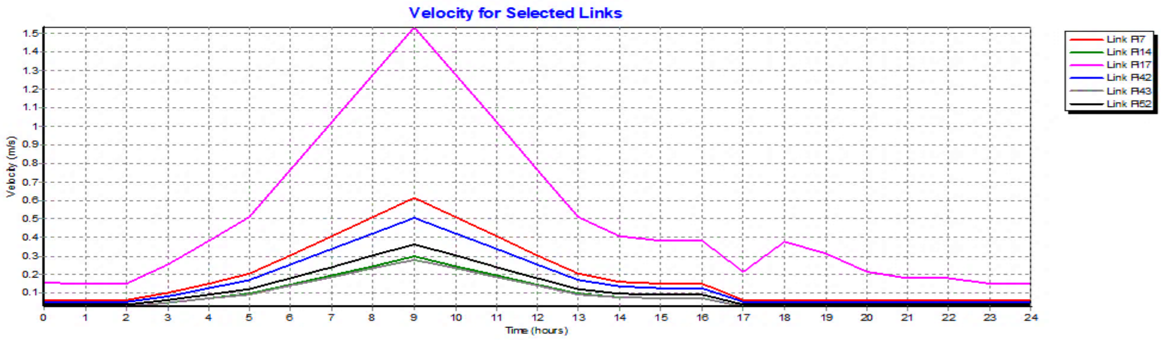
(**a**)



(**b**)



(**c**)



(**d**)

Figure 4. (**a**,**b**): Time series plot of Pressure at peak demand nodes and storage tank for WDS part I & II. (**c**,**d**): Velocity in links feeding peak demand nodes for WDS part I & II.

3.6. Performance Evaluation by TPI

The TPI $_{pressure}$ and TPI $_{velocity}$ for WDS at the peak hour, for both parts of the study area were evaluated. The TPI $_{pressure}$ for WDS part I = 1, indicating 100% efficiency of the network in terms of pressure head at the nodes at the peak hour. The TPI $_{velocity}$ for WDS part I = 0.615, indicating 61.5% efficiency of the network in terms of velocity of flow in the pipes at the peak hour. The TPI $_{pressure}$ and TPI $_{velocity}$ for WDS part II are 1 and 0.645, respectively. Indicating the efficiency in terms of pressure head at nodes = 100% and in terms of velocity in the links = 64.5%. The lower values of TPI $_{velocity}$ can be attributed the lower values of velocity at the off peak hours due to the demand pattern adopted.

3.7. Cost Optimization by JALTANTRA Web System

The optimized value of the cost of the WDS in terms of pipe diameters, was achieved by the minimization of pipe diameters while maintaining the minimum specified pressure head for the nodes. However, the total cost before and after the optimization is approximately same because minimization of diameters can be seen in fewer number of pipes and equals a maximum of 50 mm. Moreover, there is an increase in diameter of some pipe lengths by about 25 mm. The lesser difference in the costs indicates the cost optimal design of the WDS for the study area by EPANET.

Table 5. (a, b): Cost results for pipes, WDS part I and II.

(**a**)

Diameter	Length	Cost	Cumulative Cost
20.00	93	14,444	14,444
32.00	367	103,197	117,642
40.00	170	54,397	172,038
50.00	117	51,578	223,617
65.00	146	80,596	304,212
80.00	166	118,212	422,424
100.00	198	203,939	626,363
125.00	125	174,683	801,046
150.00	327	531,218	1,332,264
200.00	98	270,342	1,602,606
Total	1807	1,602,606	

(**b**)

Diameter	Length	Cost	Cumulative Cost
15.00	126	15,449	15,449
20.00	72	11,103	26,552
25.00	243	54,690	81,242
32.00	182	51,142	132,384
40.00	191	61,120	193,504
50.00	230	100,827	294,331
65.00	541	299,353	593,684
80.00	609	433,608	1,027,292
100.00	298	307,238	1,334,530
125.00	178	247,931	1,582,461
150.00	312	507,895	2,090,356
200.00	61	167,642	2,257,998
Total	3043	2,257,998	

4. Conclusions

In this work an "optimal" solution for the design of WDS for the University of Kashmir has been proposed to replace the existing and worn out WDS, which has already served its function for more than 60 years. The design not only includes the modelling of hydraulic parameters like pressure head and flow velocity but the components have been designed in such a way that the objective functions of maximum efficiency of hydraulic performance and minimum capital cost in terms of pipe diameters and valves are satisfied. The main highlights of the work indicating the optimal hydraulic design include the following:

- The WDS is designed to have a minimum pressure head of 17 m at each node throughout the 24 h of a day, so that the water can reach up to a minimum of the third floor level of all the buildings. The pressure head at all the nodes of both parts of the network is such that $17 \leq P_i \leq 70$ (meter), indicating the absence of the points of deficiency or excess of the pressure head in the network. As such there is no need of installing the pressure sustaining valves (PSV) or the pressure regulating valves (PRV), resulting in the further reduction in the capital cost of the network.

- The velocity of flow in all the pipes during the hours of maximum discharge is within the standard range of 0.25 ≤ Vi ≤ 3.0 (m/s) such that there is no danger of erosion or deposition during such hours. However, during few hours of least discharge the velocity of flow is less than 0.25 m/s in some pipes. Therefore, a scoring arrangement may be provided to remove the depositions if any.
- The TPI have been evaluated for both parts of the WDS. The performance at peak hour, with respect to pressure at nodes is 100% while the performance with respect to the velocity in pipes is 61.5% for WDS part I and 64.5% for WDS part II. The lower values of TPI velocity can be attributed the lower values of velocity at the off peak hours due to the demand pattern adopted
- Finally, the cost optimality of the network was checked by using JalTantra web system, where in the diameter of a few pipes reduced by a maximum of 50 mm, while it increased by about 25 mm for few pipes. The difference in the cost of the WDS as per EPANET modelling and that obtained by JalTantra is negligible, indicating the cost optimal design of the WDS by EPANET.

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