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Expansion and Upgradation of Intermittent Water Supply System

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ABSTRACT

Giving face lift to the existing system to meet out increased demand or introducing the new water supply scheme by discarding the existing one needs complete study on feasibility of rehabilitation in a scientific and cost effective manner. This paper addresses certain issues pertaining to the optimal upgradation and expansion of existing water distribution system. A comparative study of existing system from a town in India is presented for the design of water supply system using present design practices and an optimal design method. The attempt has been made using hydraulic simulation along with differential evolution based optimisation module. It is proved that optimisation method is useful for the expansion and upgradation design of drinking water systems. For overall effective management of a water supply scheme, the operational management issues are also discussed along with the suggestions.

Key words: Water supply, pipes and pipelines, infrastructure planning

INTRODUCTION

There is growing need to rehabilitate, replace and expand the existing water distribution systems in many counties. Water distribution systems require major investments from the municipalities and hence economical design is important. Water distribution network problems can broadly be categorised as: (1) Designing a new network, (2) Modifying or expanding an existing network and (3) Operating an existing network. During the life time of a water distribution network, it experiences various functional stresses due to the extraction of water more than designed demand, increased roughness of pipe surface due to aging, breakage of components, expansion of service area and increased service connections than actually estimated. In view of these, the distribution system may require rehabilitation and expansion to improve the efficiency. Due to the economical growth, the urban development is taking place both horizontally and vertically. While the vertical development requires rehabilitation or upgradation of the network, the horizontal development requires the expansion of existing water distribution system. While a network is expanded, it should be economical as well as reliable. As far as the economical and reliable design is concerned, the traditional trial approach through repetitive analysis is followed in many places. Though a network designed by a trial approach is satisfying the requirements, it is not guaranteed that the net benefit is optimal. At the same time, there is a very good development in the field of operations-research in the recent years. Hence it is required to bring the application of operations-research tools in the application of water distribution network analysis and design.

Walski (2001) highlighted the possibilities of misusing the optimization principles in the design of water distribution networks. He suggested the use of maximisation of net benefit rather than the minimisation of network cost as objective function. Due to uncertainty in demand and other parameters, some extra capacity above the capacity required for normal operation of the network, is always preferred by the water supply agencies. While designing a network apart from the initial network cost, the life cycle cost including the assessment of the effects caused by emergency conditions, like the component failure, should be considered. In the decision making process the economic data related to the effects caused by emergency conditions are essential. However, such economic data are not available or difficult to arrive at meaningful estimates. Hence, meaningful surrogate indicators that would enable the comparison of alternate designs are used. Some of the surrogate indicators are the different forms of reliability, vulnerability, resilience etc. Bhave (2003) reviewed various analytical and simulation methods in detail on the reliability estimation of water distribution network. Various analytical methods include (1) Loss-of-load-expectation analysis, (2) Frequency-duration analysis, (3) Markov chain analysis, (4) Contingency analysis, (5) State enumeration method, (6) Network reduction method, (7) Path enumeration methods, (8) Ccut-set method, (9) Fault tree analysis, (10) Reachability method and (11) Nodal availability method. Most of above concepts for network reliability estimation were adopted from similar fields in electrical and mechanical engineering which experience either 'working condition' or 'shutdown condition'. Thus, working-state and shutdown-state of a water distribution network are considered and the intermediate partial working state is not considered. The two state functioning is not applicable to the water distribution network as a whole though it may be applicable for individual components like pumps, values etc. There are other methods which consider partial supply at demand nodes; however these most of these methods require pressure-driven network simulation. Under pressure deficit state, conventional approach of demand-driven analysis (demand is assumed satisfied always) cannot predict the behavior of water distribution network. To overcome this weakness of demand-driven analysis, Bhave (1981, 1991) introduced 'node flow analysis' method. Further, several models were developed to include pressure-driven flow into hydraulic analysis and network reliability were estimated (Wagner *et al*., 1988; Reddy and Elango, 1989; Chandapillai, 1991; Jowitt and Xu, 1993; Gupta and Bhave, 1996; Fujiwara and Li, 1998; Tanyimboh *et al*., 2001; Ang and Jowitt, 2006; Wu *et al*., 2009).

Lai and Schaake (1969) presented a model based on linear and dynamic programming for optimal expansion of New York City water supply problem; subsequently many other also applied different optimization techniques for optimal expansion of the same network. To sustain any single link failure, Ormsbee and Kessler (1990) suggested a linear programming based method and later Lingireddy *et al*. (2000) presented a genetic algorithm based method. Dandy and Engelhardt (2001) used a genetic algorithm to find an optimal schedule for the replacement of water supply pipes. Dandy and Engelhardt (2006) developed trade off curves between cost and reliability for pipe replacement decisions for a water distribution network. Jayaram and Srinivasan (2008) proposed a multi-objective formulation with "minimization of life-cycle cost" and "maximization of performance" as objectives for the optimal design and rehabilitation of a water distribution network. Neelakantan *et al*. (2008) proposed a methodology for the design of water distribution networks incorporating the pipe-break and the replacement cost economics. Recently network resiliency (power) based designs are reported by Todini (2000), Prasad and Park (2004), Jayaram and Srinivasan (2008) and Vasan and Simonovic (2010). Many more research outputs are published in the area of reliability based optimisation of water distribution networks (Goulter and Coals, 1986; Su *et al*., 1987; Bao and Mays, 1990; Fujiwara and Tung, 1991; Cullinane *et al*., 1992; Prasad and Park, 2004; Seifollahi-Aghmiuni *et al*., 2011).

Though so many reliability or other surrogate performance measures are available, no single measure is suitable in all situations at all places. However, the network is expected to work satisfactorily in the emergency loading conditions like the fire demand, component failure and insufficient storage capacity. Pipe break in a water distribution system is common and the number of customers affected depends on the presence and location of valves in the system. Despite the isolation of portion due to breakage, the demands may be met without seriously affecting most of the customers, if the network is looped well with adequately sized pipes. Mere existence of loops with small pipes does not guarantee the alternate path for sufficient flow during a pipe break and hence the effect of losing an individual pipe segments is to be tested. Bouchart and Coulter (1989) defined the 'link importance measure' for a link as the ratio of the total deficit to the total demand in the network when the link has failed. However, a network can be designed such that there will not be any deficit in supply due any one failure at a time.

The present study is in this direction with multiple objectives: (1) To optimize the cost of expansion of an existing branched network to cover new service area in which the expansion is also meant converting the network to a looped network and (2) To arrive at a design which will not to suffer with deficit in supply with a single link failure. The second objective is a achieved by considering it as a explicit constraint. The notions of Walski (2001) have been incorporated in the present model through the constraint that the network should not suffer due to single link failure. The network may suffer with deficit if more than one link fails simultaneously; however, the probability of simultaneous failure of more than one link is very small and hence ignored. The objective of this research is to explore the advantages of optimisation based expansion of water distribution networks along with converting from branched to looped system. In this study, a typical water distribution network expansion design is analysed scientifically and also in the manner conventionally the designs are carried out by the concerned agency and the results are compared. A water distribution system in the Thanjavur town in Tamilnadu state, India is taken for case study. Apart from the optimal expansion, the possibilities of further expansion which may be required in a decade are also analysed.

STUDY AREA AND IMPORTANCE OF THE STUDY

Thanjavur town (Fig. 1) located in Tamilnadu state in India is famous for the 1000 years old Big Temple made of stones and fine arts. The population of the town is about 0.23 million within the municipal limits and the town is growing fast beyond the municipal limits. The recent growth in number and size of the educational institutions attracts more people to the town. For example, the number of residential buildings in a rectangular region (260×1110 m) between (10°45'31"N, 79°0604"E) and (10°44'57"N, 79°06'14"E) comprising Rahman Nagar which is abutting the municipal border on the outside, was only about 26 in March 2001, increased to 125 in November 2007 and increased to 208 in June 2009.

In the town of Thanjavur, water is supplied through municipal pipe network only for about 2-3 h a day. The source for municipal water is both river-bed and aquifer. As in most developing countries, groundwater is one of the major sources for drinking water. Many households are having their own private bore wells in the house premises. Generally the depth of bore well ranges between 50-120 m. When the round-the-clock municipal supply is not assured and sinking a bore well is not

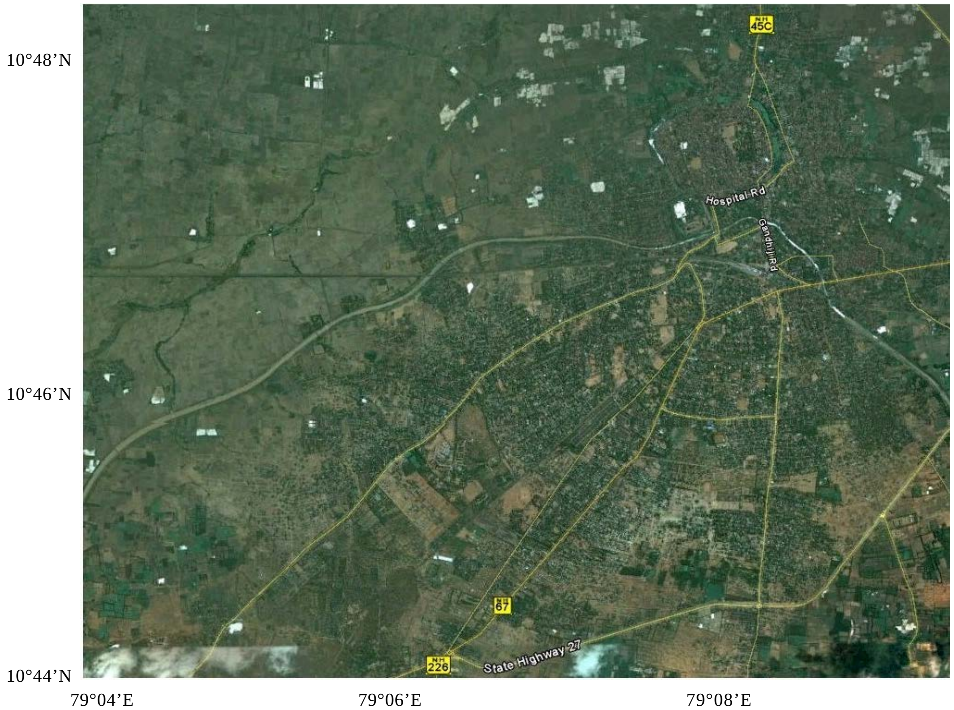

Fig. 1: Location map of study area (Courtesy: Google Earth)

to expensive, a building construction normally starts with sinking a bore well. Generally the municipal water supply connection is obtained by the residents after construction is over and hence with the two sources of water, water availability is more reliable. Over a period of time, when an area is gradually occupied by new buildings and groundwater extraction increases, the groundwater piezometric surface or water table lowers. This situation leads to a race on sinking of deeper bore wells. In many cities in Tamilnadu state India, this type of race is experienced and over a period of time, the initial wells which are relatively shallow, dry and the residents should sink another deeper well and depending on the depth and water availability, type of pump also requires a change. Due to this, the duration of pumping and cost of pumping are also increasing. The legislation or regulation on sinking the wells is neither strong nor seriously monitored and implemented. Hence ultimately over a period of time, the dependence of groundwater reduces while the piped municipal water supply becomes more important.

EXISTING SYSTEM

The main source of water for Thanjauvr municipal supply is the river Cauvery. Water is pumped from the river bed at two locations, one on the main river Cauvery at Tirumanur and on the river Vennar (a branch of river Cauvery) at Vennatrankari which are about 25 and 5 km, respectively from Thanjavur. Water is pumped from Tirumanur through the water head works at Vennatrankari. From the water head works at Vennatrankari, water is pumped to sumps or over-head tanks/reservoirs and further distributed by gravity flow. One such sump located at Ganapathy Nagar (hereafter referred as GNagar) is connected with 5 downstream overhead tanks (OHT's), with a combined storage capacity of 3500 m^3. In the present optimal expansion study, the water supply downstream of GNagar sump is considered. At present each of the OHT is filled at about 60-80% of its capacity once in a day. Hence in the design, expecting a raise the demand is taken nearly one filling per day. In this study for the purposes of hydraulic modelling, each OHT is treated as a demand node with a daily demand equal to the capacity of the OHT. The locations and capacities of these OHTs are detailed in Table 1.

At the pumping station in the Gnagar, two numbers of 90 hp pumps are in service while one operated at any given time. The pumps can work at a flow rate of 6 m^3 per minute at a pressure head of 40 m. This was used as a single point pump-curve data to model the pump in the hydraulic analysis. The Thanjavur water supply pumping mains downstream of the GNagar sump are arranged in a branch layout and the layout is indicated in Fig. 2, Table 2 and 3.

Table 1: Location and capacity of over-head tanks

East latitude	North longitude	Location	Existing/proposed	Node No. in Fig. 2	Capacity of OHT or daily demand (m^3)
79°07'40"	10°46'34"	Stadium	Existing	3	100
79°07'54"	10°46'21"	VOC Nagar	Existing	4	400
79°08'03"	10°46'02"	Arulananda Nagar	Existing	5	1000
79°06'52"	10°45'25"	Elisa Nagar	Existing	8	1000
79°06'30"	10°45'50"	East end of Medical College	Existing	9	1000
79°06'04"	10°45'31"	West end of Medical College	Proposed	10	1000
79°06'49"	10°45'01"	East end of New Bus Stand	Proposed	11	1000
79°08'12"	10°45'19"	Mahalakshmi Nagar	Proposed	12	400

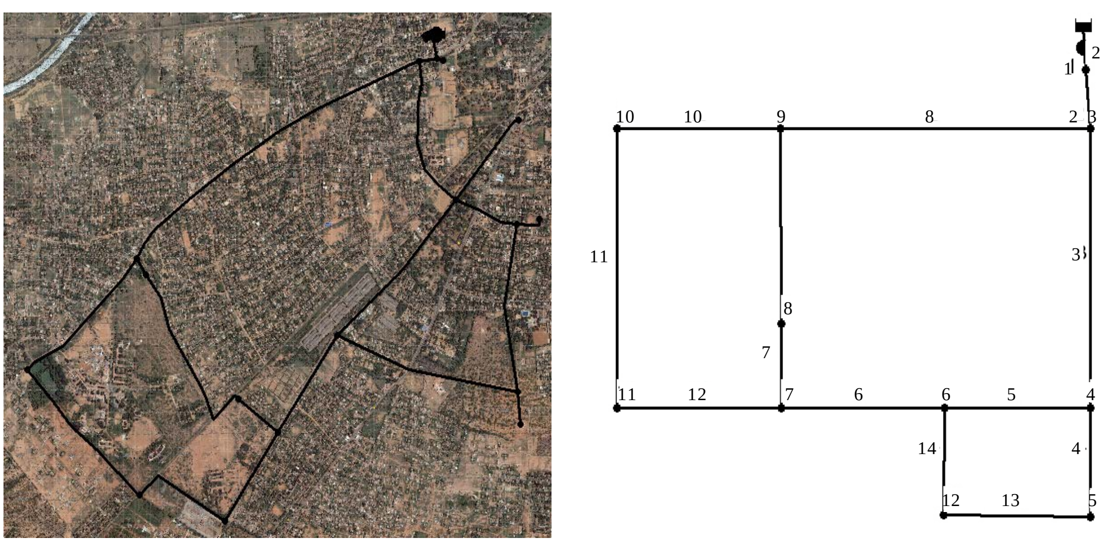

Fig. 2: Water distribution network on the map (Courtesy: Google Earth) and the simplified nodes

Table 2: Ground elevation and demand at nodes of the water distribution network

Node No.	Ground elevation (m)	Demand (L min^{-1})
1[a]	65	-4130
2	65	0
3	68	70
4	75	280
5	73	700
6	76	0
7	79	0
8	78	700
9	73	700
10	70	700
11	80	700
12	71	280

[a] Node 1 is the ground level reservoir at GNagar

Table 3: Link details of the water distribution network

Link No.[a]	Length (m)	Existing diameter (mm)
2	167	450
3	1095	450
4	555	250
5	1206	450
6	733	450
7	360	280
8	2391	450
9	1375	Proposed
10	1040	Proposed
11	1130	Proposed
12	1491	Proposed
13	1110	Proposed
14	1365	Proposed

[a]Link 1 is a dummy link provided to connect a pump between node 1 and 2

PROPOSED EXPANSIONS

To cater for the increasing demand of the expanding settlements in the south-western suburban areas of Thanjavur, it is proposed to add two numbers of 1000 m^3 and a 400 m^3 capacity overhead tanks to the water supply infrastructure. The river Cauvery is discharging about 4000 million m^3 of water during a normal year though this region and hence source of water is available. To add some redundancy to the system and thus to increase the reliability of supply, the proposed pipelines would form looped network. The layout of the proposed extension is shown in Fig. 2.

In the above table, the nodes upto 9 are connected in the existing network and the demands are due to the OHTs. Five OHTs of 100, 400, 1000, 1000 and 1000 m^3 capacity are located at nodes 2, 3, 4, 8 and 9, respectively. Nodes 10, 11 and 12 are the proposed nodes to be connected to the existing network. The OHTs are proposed at nodes 10, 11 and 12 with capacity of 1000, 1000 and 400 m^3, respectively. The links from 2 to 8 are existing and the pipes are made of cement concrete. However, for the proposed links PVC pipes are considered as PVC is generally considered

Table 4: Cost of installing commercially available PVC pipes

Pipe diameter (mm)	Cost per metre length (Rs.)
110	415
125	520
140	629
160	801
180	1006
200	1267
225	1589
250	1946
280	2426
315	3047

economical. The OHT at node 11 is to be situated on the eastern side of the new Bus Stand, the OHT at node 10 is to be situated on the western end of the Medical College, the OHT at node 12 is to be situated at Mahalakshmi Nagar.

The network has 14 links and 12 nodes. The links from 1-8 are already existing and 9-14 are proposed. The existing network is not a looped system and hence when a link fails, water cannot be supplied in the downstream areas of failure. To avoid this, the new links are proposed in such a way that the network is converted into a looped system. Hence during a link failure, the water can be rerouted in the looped network and there may not be failure to supply or shortage of supply. The diameter for each of the proposed links (9-14) is to be set such that during failure of a link, the nodes should get sufficient water at sufficient pressure through the rerouting in the loops. For the design purpose, both the existing and proposed links are set with Hazen-Williams roughness value of 100 as the existing links are also of young age. The cost data used for this study is explained in Table 4 which are based on the prevailing rates at the study area.

OPTIMISATION PROCEDURE

A combined simulation-optimisation model is developed in which the optimisation model is outer driver model and simulation model is the inner model. Differential Evolution Algorithm is used for optimisation. The EPANET (Rossman, 2000), United States Environmental Protection Agency (USEPA) developed freeware analysis software, is used for the simulation. A computer code is written for Differential Evolution based optimisation in C programming language and the simulation model EPANET is linked to the optimisation code through the EPANET Toolkit.

The optimal expansion is formulated as follows. The failure of any one link (from 3-14) should not affect the supply and pressure at all the demand nodes. Obviously the link 1 and 2 are the links connecting reservoir to the node 3 and they are not in the loop and hence failure of these links will result in 'no-supply' to the entire network. Hence failure of links 1 and 2 are not considered. Using the EPANET toolkit function, the link status is set 'closed' to simulate the link failure condition. When a set of diameters is proposed for links 9-14 (that is a set of 6 diameters), the network should work successfully when all links 'open' as well as when any one of the links from 3-14 is 'closed'. Hence this is an optimisation problem with 6 decision variables (diameters for links 9-14). The first constraint is that all the links from 3-14 are in 'open' condition. The other constraints are one on the links from 3-14 is in 'closed' condition while the remaining are in 'open' condition. These result in 12 constraints as there are 12 links (3-14) considered. The unit cost of pipe of different diameters is used to work the total cost of the proposed six links and hence the total cost is to be minimised. Penalty method is used to handle the constraints in the optimisation problem; that is a penalty, sufficiently large, is added with total cost when a constraint is violated.

Minimisation of the cost of the expansion may be expressed mathematically in Eq. 1:

$$\text{Minimize cost} = \sum_{i=1}^{N} Cp(D_i)L_i \tag{1}$$

where, $C_p(D_i)$ is cost of per unit length of new link i with diameter D_i, L_i is the length of new link i. Conservation of mass or mass balance at each node is taken as first set of constraints. Sum of flows into or out of any junction node is equal to zero. Mathematical representation of the constraints is presented in Eq. 2:

$$\sum_{i\in NP_{in,n}} (F_i \times Q_i) - \sum_{i\in NP_{out,n}} (F_i \times Q_i) = Q_{ext,n} \quad \forall n \in NN \tag{2}$$

where, Q_i is flow in link i, $Q_{ext,\,n}$ is demand at node n, $NP_{in,\,n}$ is set of links entering node n, $NP_{out,\,n}$ is set of links leaving node n, NN is the node set. The F_i takes the value '1' if the link is open and takes value '0' if the link is closed. The failure of any one link from 3-14 is permitted (12 links) and this has been implemented by the following constraint (Eq. 3):

$$\sum_{i=3}^{14} F_i = 11; \quad \forall F_i \in [0 \text{ or } 1] \tag{3}$$

The above constraint forces any one of the link to have zero flow which is equivalent to single link failure. The energy conservation forms the next set of constraints. The head loss around a loop is equal to zero or the head loss along a path between the two fixed head nodes is equal to the difference in elevation (Eq. 4):

$$\sum_{k\in loopj} K_{li} Q_i^p = \Delta E_j \qquad \forall\, j \in NL$$

Where:

$$K_{li} = \frac{\alpha L_i}{C_{HW}^p D_i^{4.87}} \tag{4}$$

where, K_{li} is the hydraulic resistance (function of diameter, material and roughness of pipes), α is a co-efficient, whose value depends on the units used, L_i is length of the link i, C_{HW} is Hazen-Williams co-efficient, D_i is diameter of the link i, NL is loop set, ΔE_j is the sum of energy losses around loop j that will be equal to 0 for closed loops and equal to the head difference between the ends of open loops and p is a exponent. The third set of constraints expresses minimum pressure requirements at each node (Eq. 5):

$$H_n \geq H_{min} \tag{5}$$

The fourth set of constraints belongs to commercial set of diameters (Eq. 6):

$$Di \in [D] \quad \forall i \in NP \tag{6}$$

Where:

[D] = Set of commercial diameters

DIFFERENTIAL EVOLUTION ALGORITHM

The evolutionary methods like genetic algorithm (Savic and Walters, 1997; Simpson *et al.*, 1994), differential evolution algorithm (Suribabu, 2010) and particle swarm optimisation algorithms (Suribabu and Neelakantan, 2006) are robust for solving complicated real-world optimisation problems. The gradient information or mathematical characteristics of the objective function are not required for these algorithms. These algorithms are inspired by biological and sociological behavior and are suitable to find the optimal solution in discontinuous and multi-modal response surfaces. Among the evolutionary methods, Differential Evolution (DE) algorithm is becoming more popular and has been used in many practical cases, mainly because its robustness and faster convergence properties compared to other evolutionary algorithms. Further understanding and implementing the differential algorithm is relatively easier. The DE algorithm was first introduced by Storn and Price (1995) for complex continuous optimisation problems and it is an improved version of genetic algorithm. Like genetic algorithm, the DE is also a population-based algorithm and uses crossover, mutation and selection operators. The DE explores the candidate solutions and uses those with better fitness iteratively until the stopping criterion is reached.

Differential evolution is a stochastic and population based search algorithm. The DE utilises the same operators of genetic algorithm and the main difference in constructing better solutions rely on mutation rather than cross-over operation. The evolution process starts with an initial population containing individuals or vectors (candidate solutions) according to size of the population and gene values (decision variables or diameters to be decided) of each individual are generated randomly. The fitness value (total cost of the links) of each individual are calculated according to the fitness function (diameter-unit cost relationship). Three individuals or vectors are selected randomly from population and differences of genes of two vectors are calculated and it is multiplied by weighing factor called mutation (0-1). The resulting weighted genes are added with the corresponding genes of third vector and it is called noisy vector. A vector called trial vector is created by performing crossover between noisy vector and a target vector selected form the population; the fitness of both are compared; and the vector which is having better fitness is taken to the next generation. The above procedure is repeated many times with different vectors of the population and a new population is obtained for next generation. The entire procedure is repeated for a number of generations. The gene values of the best vector of the last generation are taken as the solution to the problem.

Consider a population $P^{(G)}$ of generation G. The population size s is the number of vectors or individuals considered in each generation. Each individual or vector is made up of n number of genes or decision variables (x). In the present case, there are n number of links in the network for each of which diameter is to be decided. A decision variable takes a value from the commercially available set of diameters. The population can be specified as below by a matrix (Eq. 7):

$$P^{(G)} = \begin{bmatrix} x^G_{1,1}, x^G_{2,1}, x^G_{3,1}, \ldots\ldots, x^G_{s,1} \\ x^G_{1,2}, x^G_{2,2}, x^G_{3,2}, \ldots\ldots, x^G_{s,2} \\ x^G_{1,3}, x^G_{2,3}, x^G_{3,3}, \ldots\ldots, x^G_{s,3} \\ . \\ . \\ . \\ x^G_{1,n}, x^G_{2,n}, x^G_{3,n}, \ldots\ldots, x^G_{s,n} \end{bmatrix} \tag{7}$$

In the above matrix, each column represents a vector or solution, where, $x^G_{i,j}$ is the diameter for jth link in ith solution or vector. In a generation, s number of solutions or vectors are considered and hence the value of i varies from 1 to s. Each solution or vector is made up of diameters (decision variables) of n number of links hence and the value of j varies from 1 to n. Mathematically a vector or a solution can be resented as:

$$X^G_i = \left[x^G_{i,1}, x^G_{i,2}, x^G_{i,3}, \ldots x^G_{i,n}\right]^T$$

Using the following expression, an initial population P^0 is generated with the candidate diameters within the predefined lower and upper bounds (Eq. 8):

$$x^0_{i,j} = x^{(L)}_j + r^G_{i,j}(x^{(U)}_j - x^{(L)}_j), \quad \forall i = 1 \text{ to } s, \quad \forall j = 1 \text{ to } n \tag{8}$$

where, $r^G_{i,j}$ is a uniformly distributed random value within the range from 0.0-1.0, $x^{(U)}_j$ and $x^{(L)}_j$ are upper and lower limits of the variable $x^{(G)}_{i,j}$. From the first generation onwards, the population (new vectors) of the subsequent generation is generated $P^{(G+1)}$by the combination of vectors randomly chosen from the current population by mutation. Two vectors namely A and B are randomly chosen from the population and the difference between them is found. This difference is multiplied with a user defined mutation constant F which is in the range of 0 to1. The weighted difference is added with another vector C chosen randomly from the population; and the resultant vector is called noisy vector. The noisy vector is then mixed with the predetermined target vector using cross-over which results in a trial vector. A gene in the trial vector is calculated by Eq. 9:

$$t_j = \begin{cases} x^{(G)}_{C,j} + F \times (x^{(G)}_{A,j} - x^{(G)}_{B,j}) & \text{if } r^G_{i,j} \le C_r; \\ x^{(G)}_{D,j} & \text{other wise} \end{cases} \tag{9}$$

Where:

$$A \in [1,2,\ldots,s], B \in [1,2,\ldots,s], C \in [1,2,\ldots,s], D \in [1,2,\ldots,s], \quad A \ne B \ne C \ne D$$
$$C_r \in [0 \text{ to } 1], \; F \in [0 \text{ to } 1], \; r^G_{i,j} \in [0 \text{ to } 1]$$

where, C_r is crossover constant defined by the user, based on which the diameters are selected either from the noisy vector or from the target vector to get the trial vector. In the case of the minimisation problem, the trial vector is carried to the next generation only if it reduces the objective function value; if not the target vector is taken in the next generation.

The population of the next generation $P^{(G+1)}$ is selected as follows:

$$X_i^{G+1} = \begin{cases} T & \text{if } f(T) \le f(X_D^G) \\ X_D^G & \text{otherwise} \end{cases} \tag{10}$$

where, f represents the objective or cost function value of the vector and $T = [t_1, t_2, t_3, \ldots, t_n]^T$. A detailed description of DE is provided by Storn and Price (1995) subsequently many used this algorithm. In water distribution network design optimisation, DE was applied by Suribabu (2010) and Vasan and Simonovic (2010).

RESULTS OF OPTIMAL EXPANSION

The differential evolution based optimisation is used to find the optimal diameter for the links 9 through 14. The height of an OHT is generally fixed based on the topography, capacity and aerial extent of the distribution system catered by the OHT. In this study the height of OHTs are not fixed and instead the residual pressure expected is adjusted. The minimum residual pressure-head expected in the network is varied from 15-25 m and the corresponding results are presented in Table 5. Each optimisation would require about 15 h computing time if it is to be found by exhaustive exploration; however due to the differential evolution optimisation algorithm this time is reduced to about 20 min while using Intel Pentium dual core processor.

Few trial designs are presented in Table 6 which are selected based as the practicing engineers select with few trials (without proper optimisation) while satisfying the one-link failure condition. Generally the practicing engineers design the system with single diameter for all the links or they use few diameters due to purchase or maintenance convenience. The results in Table 6 show that the minimum residual pressure-heads obtained due to such designs and the corresponding cost. Comparison of Table 5 and 6, clearly shows that the designs presented in Table 5 are better. For example, the network with a cost of Rs. 9.516 million could give only a residual pressure head of 17.23 m as per the trial design. However, an optimal design at a cost of Rs. 8.846 million can provide a residual pressure head of 20 m.

Table 5: Optimal diameters for different minimum residual pressure-heads

Minimum residual pressure-head expected (m)	Optimal diameter for the link (mm)						Cost (Million Rs.)
	9	10	11	12	13	14	
15	200	200	160	180	140	160	7.256
16	200	200	180	200	160	160	8.068
17	225	200	160	200	160	160	8.279
18	225	200	160	200	160	160	8.279
19	225	200	180	200	160	160	8.511
20	225	225	180	200	160	160	8.846
21	250	200	200	225	180	160	10.004
22	250	225	180	225	180	160	10.044
23	250	250	200	225	160	180	10.763
24	280	225	225	225	160	180	11.415
25	280	280	250	250	180	180	13.449

Table 6: Trial designs used for comparison

Minimum residual pressure-head (m)	Optimal diameter for the link (mm)						Cost (Million Rs.)
	9	10	11	12	13	14	
10.59	180	180	180	180	180	180	7.556
17.23	200	200	200	200	200	200	9.516
23.78	250	250	250	250	250	250	14.616
25.20	280	280	280	280	280	280	18.222
25.20	280	280	280	280	200	200	15.353

ADDITIONAL SOURCE AND PUMPING STATION FOR FURTHER DEVELOPMENT

An additional water source and pumping station in the rapidly developing south-western areas of Thanjavur would be highly beneficial, in terms of both hydraulic efficiency and system reliability. The distance from the two overhead reservoirs adjacent to the Medical College and New Bus Stand, respectively to the GNagar sump is approximately 4 km and the Vennar River pumping station is 6 km up-pipe from the GNagar sump, with the headworks on the Cauvery River a further 20 km distant. Pumping over such distances, at pressures sufficient to supply all elevated storages, is energy intensive and the frictional head losses incurred during distribution will increase as the pipelines age. Additionally, shutdowns in pipeline operations, due to scheduled maintenance, or an unforeseen event such as a pipe burst at the upper reaches of the distribution system will adversely impact supply to all downstream reservoirs. In Thanjavur, some municipal groundwater pumping operations could partially offset the disruption caused by such an event, however such extractive capacity is limited and dependent on season.

The Grand Anicut (GA) canal runs from the Grand Anicut barrage across the Cauvery river approximately 40 km west of Thanjavur. The canal passes to within 2 km of the overhead reservoir at the Medical College, before skirting the grounds of the Big Temple and flowing further to the east for use in agricultural irrigation. The canal passes through the town and flowing further to irrigate the agricultural lands. The canal typically flows from July through to January and any water volumes extracted for the purpose of urban water supply to Thanjavur will be insignificant relative to the overall flow rate of the canal. During the periods where the canal does not flow, an agreement with the operators of the upstream dam and barrage could see water sufficient for urban supply released to the canal, similar to the current arrangement with the Vennar River.

The optimal location for the intake structure on this canal may be near the point (10°45’45”N, 79°04’37”E) which is near the junction of Pillaiyarpatti, Manojipatti and Vannarapettai villages which is at 3 km radial distance from the Medical College and suitable arrangement for the intake facility on the canal would resemble that already utilised on the Vennar River. There, a weir structure across the river ensures the availability of groundwater contained in the sands below, from where it percolates to an infiltration gallery at the bank of the river and is raised to a chlorination tank before being pumped to the distribution system. The capacity of the new pumping station could be sufficient to supply the entire network in the event of a disruption to the existing supply sources.

NEED FOR IMPROVED MANAGEMENT OF IMPROVED INFRASTRUCTURE

Previous sections of this study have considered optimised design measures to improve the reliability of water supply. However an increased investment in the physical infrastructure of the system will not have the maximum positive economic impact for the municipality unless they are supplemented by improvements in management practices.

While intermittent urban water supply is standard practice for cities in India, in Thanjavur most parts of the town the municipal supply is only operational for a comparatively brief 2 h day^{-1}, typically between 6 and 8 am. Intermittent urban water supply, a form of supply management, is an inefficient (Kumar and Managi, 2010; Dutta and Tiwari 2005; Woo, 1994) means of limiting water consumption through limiting supply. It is inefficient as it imposes a number of excess costs on to the consumer and water supply system operator. Principally among these costs are those associated with the degradation in water quality. The low pressures operating within the pipelines outside of service hours will facilitate ingress of foreign fluids with the result that a high quality, chlorinated water source can subsequently be contaminated within the distribution system. More details on the problems with intermittent water supply are given by Vairavamoorthy *et al*. (2007). The World Bank's Water and Sanitation Program, in conjunction with the Ministry of Urban Development published the guidance notes (McIntosh *et al*., 2009) for continuous water supply states "Tests in some areas in the country have shown that high levels of bacterial contamination are experienced in the first 10 min of re-pressurisation of a system and that, in some places, dangerous levels of contamination persist for up to 20 min".

To cope with intermittent and unreliable water supply, many consumers are investing in storage and treatment facilities and alternate supplies. In Thanjavur, many households, particularly in new developments, install private borewell and completely opt out of connecting to the municipal supply mains, representing a source of lost revenue to the Municipality.

A more efficient means of limiting consumption is to utilise demand management techniques within a continuous supply regime. Such techniques include the installation of automatic shut-offs in public taps and the promotion of low-flow fixtures in private dwellings. However the principal means by which demand is limited is through the application marginal pricing for every unit volume of water consumed. Such a pricing requires meters to be installed to each household connection which could be implemented on a staged basis, beginning with all new developments.

Whilst implementing a continuous 24×7 supply regime is likely to increase water quality, to achieve a degree of quality assurance, the improvement in distribution practice needs to be supplemented by a quality monitoring and sampling program. Urban water supply quality monitoring in India has been generally described by Kumar and Managi (2010) as 'haphazard', indicating a general failure of quality management and confidence in the water supply is unlikely to develop until the systems that monitor the quality can be considered good practice. In Thanjavur, chlorine is added to water at pumping stations before it reaches the over-head reservoirs. However, in the distribution system below the over-head reservoirs, monitoring the water quality is rare. Water supply in the distribution system is augmented by pumping the groundwater directly in the distribution network with the assumption that the groundwater quality is good enough for consumption.

Returns from investment in water supply infrastructure cannot be maximised without a management system to maintain the various assets that comprise the infrastructure. While it may seem obvious that an asset will only be maintained if its existence is acknowledged and location is identified, in practice this typically requires an asset register and accurate as-constructed drawings, particularly for buried assets such as pipelines and fittings. The asset database could be as broad or as detailed as necessary and may include various technical, costing and manufacturers' details on infrastructure items such as overhead tanks, pumps, pipes and fittings. Precise as-constructed information for assets whose details have been lost over time may not be available and in such an instance approximate indications of the general arrangements, with notes on the perceived accuracy of the details, can be recorded and updated as further information becomes available.

The asset register and as-constructed drawings will be crucial when planning and tracking a maintenance program. Each asset is requiring a particular maintenance regime and as maintenance records are generated, they can be entered and tracked against the relevant asset. Maintenance occurs in various forms and categories of infrastructure maintenance, listed approximately in increasing recurrence intervals.

The hydraulic models as produced for this study is not only of benefit in the design phase. It is also a valuable tool in the operations and maintenance of a water supply system once operational data is applied. There are no flow meters and few pressure gauges fitted to the distribution system except at the tanks or pumping stations. Enlightened management of the water supply system requires more sophisticated operational data collection and flow meters and pressure valves would facilitate such. By reconciling the readings of gauges and meters with the hydraulic model, a greater understanding of the system's behaviour will be gained. This system of feedback could lead to the earlier detection of impairments within the distribution system and thus faster repairs and an improved service to consumers.

There are a number of stakeholders associated with the Thanjavur urban water supply and effective management of these relationships can lead to both actual and perceived improvements in the water supply service. Reductions in water quality can be observed by consumers or, in more serious cases, by medical professionals. Clear lines of communication to and from local doctors, hospitals and the public can help to identify the occurrence of aberrations in water quality and thus simplify the identification of the cause.

CONCLUSION

The Thanjavur municipal water supply can benefit significantly through the greater reliability offered through looped networks and additional supply sources. Through the use of a hydraulic model and algorithms, the design of network extensions can be optimised. However the effective management of a water supply scheme is multi-faceted and well-designed and constructed improvements to the system will contribute little to the economic and social well-being of the community if it is not matched by effective water quality management, asset management and communication of the results of this effective management with stakeholders in order to build confidence in the system. However, all of these broad objectives will be excessively difficult to achieve with the continuation of an intermittent supply regime and the implementation of a continuous supply system may be worthy consideration for the municipality. Intermittent water supply is common in many developing countries and due to economic growth many are trying to improve the water supply. Hence the work reported here has relevance to many water distribution systems.

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