DESIGN AND OPTIMIZATION OF DRINKING WATER SUPPLY UTILITY

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Design and Optimization of Drinking Water Supply Utility

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CERTIFICATE OF ORIGINALITY

I hereby declare that this thesis is my own work and that, to the best of my knowledge and belief, it reproduces no material previously published or written, nor material that has been accepted for the award of any other degree or diploma, except where due acknowledgement has been made in the text.

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Abstract

Engineering project management with respect to optimization analysis has become a crucial concern amongst water supply utility sector to address resource availability, purification, supply, and optimum distribution of drinking water. Significant changes that has been observed over the past decade is because of the growing awareness about the reliability aspect within the water industry due to climate change and increasing population. However, assessing the technical design aspects of a water utility structure is not an easy task to address as it needs to be regulated to a special point of view which are not always quantifiable due to several crucial factors involved in it.

A variety of concepts and criteria can be included into a suitable framework which is considered in this thesis which presents an organized methodology to the qualitative and quantitate investigation of drinking water and to the meta-heuristic techniques for the optimization of the water distribution system. The proposed methodology is based on the forecasting and optimization of the river water intake location considering demand management and its allocation which can be considered as an extension to the existing engineering consideration and modelling techniques. The measures are calculated using the results of the steady state and the unsteady state of pumping in a riverbank aquifer. For the planning and design, the only necessary thing to know is the desirable discharge and the aquifer characteristics to modelled the river-aquifer system.

The traditional way of using optimization methods, e.g. stochastic meta-heuristic algorithms have come along with various constraints to explore an optimum solution. In this study, a newly developed meta-heuristic algorithm called Simple

Benchmarking Algorithm (SBA) has been used to optimize the pipe size. A modified approach with SBA having interfaces with EPANET 2.0 hydraulic simulation model has been used to compute the minimum cost of Two-looped network and Hanoi benchmark WDN. Results show that SBA is more efficient in obtaining the least possible cost with fast convergence ability. SBA's compatibility with the pipe hydraulic related engineering problem is explored in this thesis. Several areas of water utility system are analyzed in detail and some advances have been made to the analysis and modelling procedures that are currently in use. This includes cost optimization-oriented modelling by selecting optimum pipe diameter without violating the hydraulic requirement, water quality modelling (forecasting safe distance of a pumping well, considering biological pollutants) and water quantity modelling (attenuation of pollutants in infiltration gallery). In addition, analytical methods devised herein which are based on Logistic Function, have been applied to the hydrogeological environment variables which are perceived appropriately representing the aspects being assessed.

This thesis compendium is a comprehensive effort (comprehensive summary of a large project, in this case, the drinking water utility sector) to analyze and quantify the computational performance of the optimization technique. To improve the understanding of such methodology, a verity of fields has been analyzed in an innovative and precise way. The framework which is employed in this thesis provides a shift in the way engineering problems are formulated in water supply system planning, allowing greater control of the analysis objectives and improved serviceability. It is believed that the approach developed herein can effectively address the problem by studying a wide range of planning and operating conditions. This research is helpful to water industry technical managers, planning and design engineers, regulators, civil engineers, and water resource academics/researchers.

Drinking water quality and minimum cost estimation are important issues in a water supply project while designing and planning. Despite extensive research on the subject, most studies have not been able to provide simplified methodologies with efficient tools for optimization. It is inevitable that such physically deficient models may suffer from inappropriate mapping, substantial uncertainty inherent in the modelling and inadequate optimization during benchmarking. The major contribution of this research is the development of analytical models to efficiently and explicitly simulate the groundwater flow in the riverbank filtration facility. An intelligent approach to cost optimization of a water distribution network has also been demonstrated and this thesis advocates the use of SBA algorithm for the optimization problem in the water distribution system.

Publications

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DEDICATION

This thesis is dedicated with metta

To my adopted mother

Yasue Inaba

(11th September 1942 – 14th January 2018)

Who passed away before I fulfil her wish to complete my study in The Hong Kong Polytechnic University. Without her enormous motivation for higher education, I would have never become the individual that I am today.

I had promised to make her proud by the achievement of this monumental academic goal and I hope that I have fulfilled that promise. I wish that she could still be alive today to share with me the celebration and the success of my graduation with a Doctor of Philosophy Degree.

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Abbreviations

C_{HW}	Hazen-William roughness constant
C_d	Coefficients
C	The concentration of pathogen at time t
C_o	The initial concentration of a pollutant in river water
Cp	Static penalty
D	Depth of aquifer
D_h	Hydrodynamic dispersion
D_i	The diameter of i^{th} pipe
D_{min}	Minimum existing commercial diameters
D_{max}	Maximum existing commercial diameters
D_h	Hydrodynamic dispersion
ESR	Elevated service reservoir
E_s	Solution space of the ecological system
GA	Genetic Algorithm
HGL	Hydraulic gradient line
H_{min}	Minimum required pressure head
$H^{\min}$	Minimum hgl at node
H^{des}	Desirable head at node
H^{avl}	Available head at node
H_j	The downstream node of pipe x
H_n	Pressure head at node n
H_r	Reservoir head
H_f	Fixed or known head in the network
IOA	Intelligent optimization algorithm

K_{att}	Attachment rate coefficient
K_{det}	Detachment rate coefficient
L_i	Length of the i^{th} link
M	Numbers of source nodes
N	Total number of links
N_L	Total numbers of loops in the distribution system
NHA	Node head analysis
N_j	Total number of nodes in the system
OF	Objective function
P_e	Péclet number
P_j	The pressure in node j
P_{max}	Maximum pressure points in the node
P_{min}	Minimum pressure points in the node
Q_{ext}	External demand at the node
Q_{in}	Inflows of the node
Q_p	Pumping rate
Q_{out}	Outflows of the node
Q_j	Flow in link j
R	Retardation factor
R_x	Resistance constant of pipe
RBF	Riverbank Filtration
SBA	Simple Benchmarking Algorithm
T	Transmissivity of aquifer
$\overline{v}$	Average velocity
V_i	Flow velocity in the i^{th} pipe

V_{max}	Maximum velocity in the pipe
V_{min}	Minimum velocity in the pipe
WDN	Water distribution network
WDS	Water distribution system
WSDNS	Water supply and distribution network system
WSS	Water supply system
X	Number of pipes
c	Cost per unit length of the link having
c_i	The objective function coefficient
d	Pipe diameter
erf	Error function
h_x	Water lever at distance x from the river
h_p	Head supplied by the pump
i_w	i^{th} location drop of water carrying pollutant
j	The upstream node of pipe x
k	Hydraulic conductivity of aquifer material
m	Number of constraints
n	Number of cycles (log cycles)
n_f	Time step
n_i	Total number of pipes in the distribution network
p	Penalty multiplier
q_j^{req}	Required demand
q_j	Known demand at node j
q^{avl}	Available flow
q^{req}	Required flow

r	Reproduction rate
r_w	The radius of pumping well
s	Drawdown at wellpoint
t	Time
t_r	The travel time of a parcel of water
t_{rr}	Time duration by which the concentration reduces to $c_o 10^{-n}$
$t+1$	Next iteration after current (t^{th}) iteration
u	Darcy velocity
x_w	The distance of abstraction points from the bank of the river
α	Longitudinal dispersivity
β	Aquifer diffusivity
σ	Step rise in stream stage
ε	Very small time
η	The porosity of the aquifer
μ	Detachment rate coefficient
v	Seepage velocity
ω	Dimensionless conversion factor
λ	The decay rate of pollutant (*e. Coli* pathogen)
Δt	Change in time; and
ΔQ	Loop flow corrections

Chapter 1 : Introduction

1.1 Research Background

The traditional approach adopted by the water industry planner exhibits a poor level of service during supply and distribution. It includes meeting people's minimum requirement, less capable design to fulfil the water quality aspect, expensive construction cost involved due to inaccurate planning, etc. The need to cope with the increasingly competitive corporate environment and the tight cost-effectiveness constraints while satisfying the current customer's reference guidelines implies that it is the overall performance of the system that needs to be addressed at all stages of the planning, design and operation tasks. Even though the traditional approach follows the main objective of minimization of investment cost considering various operational constraints, it has been seen that it often failed to maintain the desired level of efficiency due to underestimated hydraulic design.

In recent years, the conventional analysis methods for the optimized design of the water supply system (WSS) has been improved significantly because it considers the efficient analysis of the network to perform its distribution task. Such analysis plays a significant role in the case of urban water distribution systems which has complicated layouts, myriad demand points, on the other hand frequently over simplistic operations.

It is easy to identify water systems that have been inadequately designed or are facing functional and operational difficulties originating in hydraulic shortcomings. Due to the significant population growth of urban areas, there is a strong need of improvement in the types of tools that support planning and evaluation of the overall management of a system to make decisions themselves and having least dependency on experienced professionals.

The efficient use of various models in system analysis is a robust solution to the problem. For example, water network simulators are an invaluable aid in the assessment of the response of the system to the alternative demand and operational scenarios. However, the output of such models may have complex results which may be unrealistic to comprehend, and the results are frequently inferior to that of the objective function, and it is difficult to compare different solutions. The present methodology and improved optimization strategy aim to solve this problem and bridge the gap, which can be applied to a variety of water supply utility with a complex distribution system.

Measuring the quality of water and assessing the level of services provided by the water distribution leads to a criterion of a network which is not a straightforward task, given the multiple factors and viewpoints involved. The concept most commonly associated with the modelling / optimization of WDN concerns the adequacy of the supply in hydraulics terms, consisting of the quality of pumped water supplied and the cost of the distribution network in both qualitative and quantitative terms. Despite the availability of techniques providing much of the relevant information in each of these specific aspects, integrated methodologies that allow for flexible use in engineering tasks are not yet widespread.

This thesis attempts to systematize the issue of water quality and quantity along with cost analysis of the water supply and distribution by putting together a flexible framework based on an array of each measure devoted to a special aspect. The principal requirements for such a methodology are: (1) it should allow a certain degree of standardization in order to facilitate and indeed validate a multidisciplinary approach where various aspects to be considered may be brought down to the same quantified basis; and (2) it must be quantitative and numerically based – the envisaged tool should be translatable computationally in order to afford intensive use, either from within or as a post-processor to the current modelling technique. Even though it is to a certain degree possible to deal computationally with non-numerical information, it would be desirable for the sake of simplicity to find a method which would allow numerical treatment, especially it would be integrated with the current analysis and modelling techniques.

Some important objectives are addressed in this thesis by means of a simple methodology, which is applied to a variety of engineering aspects in water supply and distribution, selected while reviewing the three main areas viz: pumped water quality, demand satisfaction and cost optimization.

1.2 Research Objective

The main objectives pursued in this thesis are summarized as follows:

1. To examine a suitable meta-heuristic optimization model for design in water supply and distribution and to identify areas of study which may lend themselves to an engineering approach.
2. To develop a systematic and quantifiable approach to evaluate performance that can be used as a common methodology when tackling different areas of water distribution analysis. The method should also be designed as an engineering tool to complement the existing modelling and analysis techniques.
3. Finally, to analyze each selected area of study in depth; in order to identify what

aspects may be suitable for the approach mentioned in (2) and develop a method to model and quantify them; to apply the design methodology and analyze the system optimization based on those aspects.

1.3 Methodology and layout

This thesis is organized into five main chapters. After an introduction to the subject of water distribution design and optimization, a literature review is presented in Chapter 2 focused on the development of design optimization from the engineering viewpoint framework utilizing a genetic algorithm approach. The basic optimization parameters of the SBA application of the study for the present work are identified and outlined based on the GA approach.

In Chapter 3, Genetic Algorithm framework is developed and discussed. A GA framework is applied in the operation and technical management environment of water supply and distribution utilities, and as a complement to existing modelling analysis capabilities. A standardization procedure of GA is presented in such a way that is used to define the various parameters considering optimum design. The methodology is then applied at the elementary level and subsequently generalized to the whole domain. The various components of the algorithm are described with their assumption and simplification. The requirements and desirable properties for the measures contemplated by the meta-heuristic scheme are discussed, as well as the various ways of presenting the results using the penalty approach for constraint violation. The methodology supported by GA principles is then developed to adopt SBA and it is then compiled in code (computer) to run the analysis. The developed model is called SiBANET.

Chapter 4 covers a design on the improvement that is required for fast convergence strategy to be explored in an evaluation of the water distribution system using SBA algorithm. The hydraulic requirements have been fulfiled externally and supported by EPANET. In this chapter, the methodology called as ‘SiBANET’ is applied by taking into consideration of various hydraulic properties.

After introducing the subject of hydraulic modelling and optimization using SBA algorithm in water supply and distribution network in Chapter 4, an analytical model is presented in Chapter 5 and Chapter 6 where the general formulation governing the groundwater flow modelled in riverbank filtration are described. Subsequently, the two types of intake selection in which the hydraulic variables to be included in the evaluation are presented. Finally, illustrative examples are then given, and the use and potential application of the methodology is explored.

In Chapter 5, the quality of pumped water is discussed and also described in Chapter 6. It has been observed frequently that water utility companies do not follow the service drinking water quality standards to consider the equal category of customers. According to the WHO guidelines on drinking standards, only a certain minimum number of microbial counts along with other physicochemical parameters should be fulfilled when reaching the end user. Therefore, it is recommended that water utilities should use the purification system like riverbank filtration to abide by the standards. It is scientifically proved that the water quality will often improve in space and time during riverbank filtration with improvement in aesthetic properties – odour, taste, colour, turbidity, temperature and in its physicochemical properties and microbiological content, thus minimizing the danger of contamination. In Chapter 6, the application to a methodology for better water quality in the event of a flood in the stream is presented. In contrast to the hydraulic performance measures, for which there are well known and widely available models e.g. Harvey and Garabedian (1991), water quality modelling in riverbank filtration using analytical approach with more simplified framework is a less explored domain.

Chapter 6 presents analytical approach which includes the design of a water quality model to attenuate pathogen shock load. Its development and implementation are described after a detailed review of existing methodologies covered in this chapter. A complete description of the explicit finite difference scheme is also discussed with respect to a widely referred one-dimensional hydrodynamic dispersion and decay model. The emphasis is given on numerical accuracy; therefore, the explicit finite difference scheme was used to develop this analytical model. The model is extended not only to carry out the modelling of constituent concentration but also the consider water travel time with respect to the source water quality. The code for the analytical model is developed in MATLAB to study the different phenomenon of the riverbank filtration processes, and the model is called as LIFI-PATRAM. This model is then referred to the field examples for input of hydraulic parameters to illustrate the proposed methodology. The suitability of the placement of an infiltration gallery for water quality and quantity is then analyzed and concluded upon.

In Chapter 4, Chapter 5 and Chapter 6, the most palpable aspects for the water utility planner to establish an effective water supply and distribution system are analyzed and discussed. These aspects answer most of the objective of the water utility to satisfy all the demands with adequate and wholesome water while at the same time ensuring minimum project and operation cost.

Finally, the conclusions of all the chapters based on SiBANET and LIFI-PATRAM methodology for the design of drinking water utility system are presented in Chapter 7. This research investigation also proposes few areas for further examination considering optimization and analytical techniques.

Chapter 2 : Literature Review

The historical development of water distribution optimization and suitable (water) supply system optimization with intelligent optimization algorithms (IOA), has been covered. The history of water supply and distribution network optimization tool i.e. Genetic Algorithm (GA) and its variant along with the application of an intelligent optimization algorithm called simple benchmarking algorithm (SBA), with its background have been briefly covered.

2.1 Introduction

The main purpose of water utility is to meet its user's requirement for water demand with adequate pressure. The water distribution system is composed of three main components: pumping stations, storage tanks and distribution piping. These systems are loaded according to the hydraulic conditions i.e. pressure and discharge at nodal points.

In the usual network analysis methods, it is presumed to fulfil all the requirements at all the time and the corresponding service reservoir will serve with adequate capacity. This gives a network analysis for steady state, i.e. static conditions and is known as static analysis. However, in the actual practice, both the conditions are difficult to meet at the same time throughout the service period of the planned project. Typical diurnal demand for residential community area consisting of 1000 thousand residents as observed by Arsene (2017) is shown in Figure 2.1. Naturally, water levels in the reservoirs would also change, and this change would depend upon whether the water is withdrawn from a reservoir or is supplied to it from an external source or by the distribution network itself if the reservoir is floating on the system. Furthermore, when a part of a large network is considered, the inflows from and outflows to the surrounding pressure zones connected with the area under study would also change.

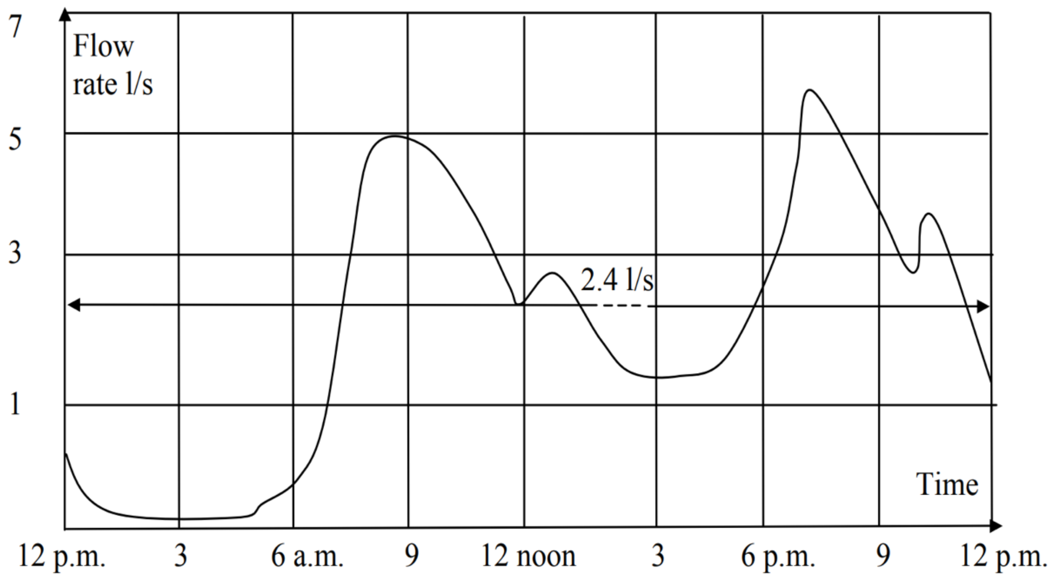


Figure 2.1 Typical variation in nodal demands (Arsene, 2017)

To fulfil the consumer's requirements, proper operation design and planning of the water supply system is necessary. This aspect of has been studied by various scholars (Ormsbee and Lansey, 1994; Mackle, et. al., 1995; Kazantzis, et. al., 2002; Van Zyl, et. al., 2004; Behandisha and Wu, 2014; Ibarra and Arnal, 2014) in past 2 decades to minimize operational cost. To reduce the environmental effects caused by the energy production, recently, the issue of global warming has also drawn the attention of researchers to incorporate greenhouse gas emissions (Wu, et. al., 2012; Sadatiyan Abkenar, et. al., 2014; Stokes, et. al., 2014) in the planning stage of the utility. The service provided should ensure (1) adequate pressure and discharge throughout the service period of the project, and (2) proper storage facility to balance water distribution for efficient operation. The knowledge of the impact of demand fluctuations on the level of service would help to control operation. Whereas, from a planning point of view, it would be necessary to evaluate the adequacy of storage to effectively meet the expected increase in the total demand. Both objectives can be achieved by carrying out the extended period analysis of the network over a period of 24-48 hours under changing demand patterns (as shown in Figure 2.1), reservoir water levels, pump operation, leakages and boundary conditions. This dynamic characteristic for the analysis for the water distribution networks was recognized by Shamir and Howard (1968), Tart (1973) and Demoyer et. al., (1973). Later, Rao and Bree (1977) suggested a general procedure for performing dynamic analysis of WDN.

In the subsequent sections, available methods for static analysis and extended period analysis are briefly described. The available heads (H_j) at each node corresponding to the required demand (q_j^{req}) at that node is explained in detail.

2.1.1 Formulation of Equations

In static analysis, the unknown parameters are the pipe discharges '*Q*' and its material resistance '*R*', respective nodal flow '*q*' and its corresponding head '*h*' which are some of the basic unknown parameters. Once the values of the basic unknown parameters are known, other unknown parameter's value is easily determined for a network of '*X'* pipes having source node '*M*' for demand node as '*N*' in the proposed network consist of '*C'* number of loops. The total number of unknowns is: $X + N$. Thus $X + N$ equations are formulated.

The equations are formulated using the basic unknown parameters, and the formulated equations are designated by these unknown parameters. Hence, for unknown pipe discharges the equations formed are known as Q equations.

In a simplified way, the two basic equations can be rewritten as:

- The node-flow continuity relationship for the demand 'nodes j':

$$\sum_{\substack{x \text{ connected} \\ \text{to } j}} Q_x - q_j = 0, \quad j = M+1, \ldots, M+N \tag{2.1}$$

- The general pipe-head loss relationship for "pipe x':

$$H_i - H_j = R_x Q_x^n, \quad x = 1, \ldots, X \tag{2.2a}$$

or

$$H_i - H_j = \frac{k L_x Q_x^n}{C_{HW}^n D_x^r}, \quad x = 1, \ldots, X \tag{2.2b}$$

in pipe x; H_i= heads at upstream node and H_j = heads at downstream nodes; Q_x = discharge; R_x = pipe resistance; q_j = node demand at j and n = exponent, k=10.68 general value; and C_{HW}= Hazen William' constant for pipe x.

Q equations:

By applying the node-flow continuity relationship, the basic unknown parameters of Q linear equations are formed. Whereas, non-linear equations for the general loop-head loss relationship can be framed as:

$$\sum_{x \in c} R_x Q_x^n = 0, \quad \mathrm{c} = I, \ldots, \mathrm{C} \tag{2.3}$$

H equations:

By applying the node-head continuity relationship, the basic unknown parameters of H linear equations are formed. Whereas, non-linear equations for the demand node flow continuity relationship can be framed as:

$$\sum_{\substack{i \text{ connected to} \\ j \text{ through } x}} \left(\frac{H_i - H_j}{R_x} \right)^{\frac{1}{n}} - q_j = 0, \quad j = M+1, ..., M+N \qquad 2.4$$

Four methods are commonly used to analyze a WDN. Brief descriptions of these methods are given in section 2.1.2.

2.1.2 Solution Methods

All the above methods use the iterative solution procedure. Some usual methods widely referred in the designing of WDN are: (a) Hardy - Cross method; (b) Newton-Raphson method; (c) Linear Theory method; and (d) Gradient method

A short description of each method is given below for a brief understanding of the enumeration involved in the hydraulic design.

2.1.2.1 Hardy-Cross method.

Hardy Cross (Cross 1936) suggested an iterative procedure for network analysis. His approach is primarily based on loop-flow correction equations. The principle to nodal-head correction equations was developed. This approach is also known as a method of balancing flow and head.

2.1.2.2 Newton-Raphson method

For solving a set of non-linear equations, the Newton-Raphson method is the most general scheme in practice. Since the number of ΔQ equations is less than that of Q-equations, Newton-Raphson method is preferred for ΔQ equations as compared to Q-equations. The number of H or ΔH equations are to be solved to obtain a solution. However, as done in the Hardy-Cross method, the effect of all adjacent loop is considered in each iteration. All the correction equations are solved simultaneously. Therefore, convergence in the Newton-Raphson method is achieved in a smaller number of iterations than that required for the Hardy-Cross method. Newton-Raphson method simultaneously solves the nonlinear equations by linearizing them through partial differentiation.

2.1.2.3 Linear Theory method

Like the Newton-Raphson method, this theory also solves all the equations simultaneously. Linearized forms of Q or H equations are solved here. The non-linear equations are linearizing whereas in the linear theory method it is performed by merging.

2.1.2.4 Gradient method.

To obtain Q and H values simultaneously, Todini and Pilati (1987) used the Newton-Raphson method. It is later commonly known as the gradient method. The gradient method simultaneously improves nodal heads and pipe discharges and is observed to converge faster than other methods of analysis.

This study has utilized the potential of EPANET 2.0 toolkit (Rossman 2000) which used a gradient method for simulation. It can also have the capability to perform an extended period of hydraulic simulation and water quality modelling in the pressurized pipe system.

2.1.3 Network Analysis Software

Fewer software is available to analyze a WDN considering various hydraulic parameters. Some of notable applications are: (a) EPANET (b) LOOP (c) KYPIPE (d)

PIPE 2006 (e) WaI CAD (f) CyberNET (g) H_2ONET (h) SynerGEE water, (i) RealPipe etc.

EPANET is presented in detail because it is used in this study. Its application is illustrated with a simple network for simulation purpose.

2.1.3.1 EPANET

EPANET (USEPA, 2002) is widely used software that models the piping networks developed by USEPA. It is a public domain software freely used and distributed and has the capability to do a static and extended period analysis of water distribution with pressurized pipe networks. The user manual describes the use of the program on different platforms. The hydraulic simulations are analyzed using the DLL of the toolkit.

2.1.3.2 Analysis of Simple Serial Network using EPANET

The serial network as shown in Figure 2.2 (Gupta and Bhave, 1996(b), Ang and Jowitt, 2006) has one source node labelled 0, four demand nodes labelled 1–4, and four pipes labelled 1–4. The diameters of pipes 1, 2, 3 and 4 are 400, 350, 300 and 300 mm respectively. Elevation of a node is the actual withdrawal points of that node. Each pipe has a length of 1000 m. Demand nodes 1, 2, 3 and 4 are considered as actual withdrawal points with required demands q_j^{req} of 2000, 2000, 3000 and 1000 L/min and elevations of 90, 88, 90 and 85 m, respectively. Source node 0 is an elevated service reservoir (ESR) with a constant water level of 100 m. *H-W* co-efficient for each pipe is of 130.

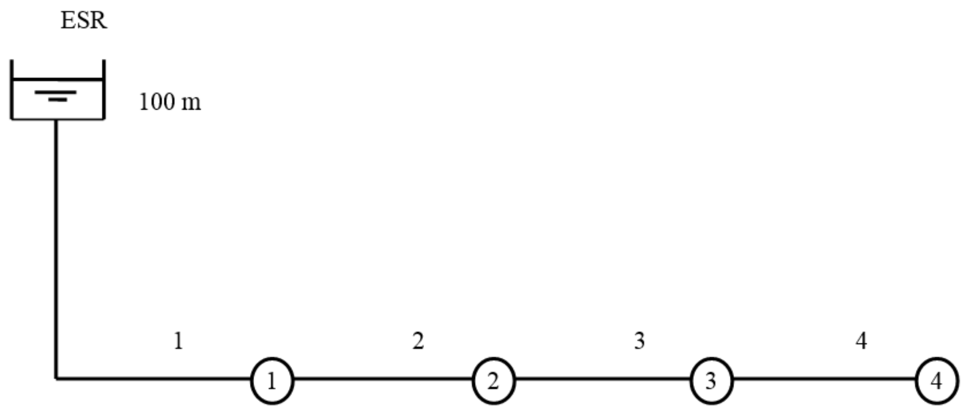


Figure 2.2 Schematic of Serial Network

2.1.3.3 Output Results:

The network is analyzed using EPANET 2.0 computer program. The available heads (H_j) at demand nodes are node 1 (97.30 m), node 2 (94.27 m), node 3 (91.24 m) and node 4 (91.01 m).

2.2 Dynamic analysis

Static analysis presumes that the nodal demand and the reservoir tank levels remain constant throughout. The assumption of constant water demands and reservoir water levels can be valid for a short period. However, it has been observed that in actual practice and operation of WDN, nodal demands and corresponding elevated service reservoir water levels never remain constant for any instant of observation (even within 24 hours). The nodal demands fluctuate during the day depending upon the type of demand, adequate flow rates and pressure. Water levels in the reservoirs changes and this change depending upon whether the water is withdrawn from a reservoir or is supplied to it from an external source or by the distribution network itself if the reservoir is floating on the system.

The objectives can be achieved by carrying out the analysis of the network for a period of 24-48 hours under changing demand patterns, reservoir water levels, and boundary conditions. Such analysis of the network to balance the supply and distribution is called an extended period simulation, extended period analysis or simply dynamic analysis.

The iterative approach to dynamic analysis is as follows:

2.2.1 Iterative Method

The dynamic analysis of a water distribution network basically depends upon the fundamental water-balance principle: For a particular time interval, the difference between the water supplied to a system and the water taken out of it is equals to the change in the water stored in the system. The application of this principle to a single-reservoir water distribution network without any boundary conditions is quite simple. The demand curve gives the total volume of water supplied to the consumers. Thus, the corresponding change in its water level can be determined with respect to the initial reservoir level. Therefore, at the end of the time interval, the change in its water level can be estimated. For a multiple reservoir water distribution system, this type of condition is complex in nature to optimize the network.

Considering various analysis methods studied so far, the static analysis of the network cannot be directly performed as head and discharge is unknown. Even with different formulations of equations and their solutions, the problem persists due to fluctuation in reservoir level.

Identifying this difficulty, Rao and Bree (1977) suggested a superior analysis procedure for the prediction of water level in the reservoir. Using the known water levels at time t and the predicted changes in the water levels during the time interval Δt, the water levels in the reservoirs at times $t + \Delta t$ are predicted. In this process, the time interval is split into several sub-intervals and the solution at the end are linked with each other through an iterative integration procedure. In this way, a static solution is obtained for a particular instant. Following this static solution procedure, the reservoir discharge at the end of time 't' can be estimated. By assuming the constant flow rate, the change in the volume of all reservoirs can be predicted. This will also help to plot head-discharge rating curve for a given period. Since the flow rates at the reservoirs obtained from the static analysis for time $t + \Delta t$ would be different from those used in the predictor calculations, a correction would be necessary. Some correction is applicable in the static solution predicted discharge in the reservoir is different.

2.2.2 Direct Method

In the predictor-corrector iterative procedure described in the earlier section, $q_r(t + \Delta t)$ values are taken as $q_r(t)$ for the first predictor-corrector integration iteration. These values are then successively corrected so that the predicted reservoir HGL values $H_{rp}(t + \Delta t)$ and the predicted reservoir flows $q_{rp}(t + \Delta t)$ can be co-realtered with the corresponding corrected head and discharge. This correlation has been observed by Bhave (1988), who demonstrated that above mentioned predicted iterative procedure can be avoided. With the help of this approach, head and discharge can be obtained directly from the static analysis for time t.

2.3 Optimal Design of Water Distribution System

The water distribution network is said to be the least costly whose optimized design solution fulfil nodal demand through the shortest route. In general, optimization is a notion of obtaining optimum solutions (Rao 1999, Pant 2004, Taha, 2003) and serve maximum benefits. Water distribution networks are generally designed to serve various objectives, such as; head, discharge, pressure at a delivery node point, water quality, minimum operational cost etc. To serve all the crucial objectives, the engineer/designer must have enough understanding of the various hydraulic phenomena and their uncertainty.

The highest priority in WDN optimization is always given to reliability of a system against failure in any condition. Therefore, suitable identification of pipe sizes and its appropriate combination in the network is a crucial challenge (Mays, 2000). The following section describes in detail the various techniques used in optimization.

2.3.1 Pipe Size Optimization

In the early era of pipe sizing optimization, classical methods were extensively used for the design of a water distribution network. These methods include linear programming, dynamic programming, nonlinear programming etc.

Alperovits and Shamir (1977) are amongst those researchers who developed the approach of linear programming using the gradient method for the design of WDN. Later, several researchers followed this design approach with suitable modification in the methodology. The incorporation of alternative derivations to the linear programming-based gradient expression improve the optimum solution (Quindry et. al., 1981, Fujiwara et. al., 1987, Lansey and Mays 1989, Fujiwara and Khang 1990). In addition, Xu and Goulter (1999) proposed a fuzzy linear program optimization approach.

The only problem with the modified design was that the optimized solution comprises of pipe segments which are unrealistic and difficult to find into the commercial market. To overcome this difficulty, Walski (1985) presented a state-of-the-art principle for optimization of pipe networks which was realistic to serve the design purpose. A basic constraint handling model which considered energy conservation and reliability was proposed by Su et. al., (1987). However, these models had the limitation of discrete pipe diameters which required rounding off to the appropriate sizes for practical implementation. This approximation of pipe diameters affected the feasibility of an optimal solution.

To address the problem of pipe size approximation, a heuristic-optimization approach was proposed by Fujiwara and De Silva (1990) and resulted in a least-cost solution of WDN. The heuristic method comprised multiple objectives like the constraint of reliability over the design period. The design of WDN also examined with non-linear programming technique of optimization (Fujiwara and Khang 1990, Fujiwara and Khang 1991) in which the diameter was treated as a continuous variable. Park and Liebman (1993) followed the non-linear programming technique considering redundancy-constrained to get the minimum-cost design solution for more optimized design.

Due to the improvement in stochastic optimization theories in the ’90s, the design of WDN was being formulated as nonlinear mixed integer problems. The introduction of Genetic algorithms (GA) brought the change to the design procedure using computer code. GA has been used by Murphy et. al., (1993), Simpson et. al., (1994), Dandy et. al., (1996), Savic and Walters (1997), Keedwell and Khu (2005), and Keedwell and Khu (2006). The rehabilitation of a WDS has been applied by Wu and Simpson (2001) using a GA to obtain the optimal design for a real network. Espinoza et. al., (2003) evaluated a self-adaptive hybrid genetic algorithm (SAHGA) on different test function including constrained optimization problems to estimate the performance.

In addition to GA, the simulated annealing technique has also demonstrated the promising results which were proposed by Cunha and Sousa (1999) and Cunha and Sousa (2001) in the optimum design of water distribution system to obtain least-cost. The applications of various other techniques of evolutionary optimization algorithm are reported in the literature of the design of the WDN. It comprises Cross-entropy optimization (Rubinstein 1999, Rubinstein and Kroese 2004), Shuffled Frog Leaping Algorithm (Eusuff and Lansey 2003), Ant Colony Optimization (Maeir et. al., 2003), Tabu Search heuristic (Cunha and Ribeiro 2004), Scatter Search (Lin et. al., 2007), Particle-Swarm (Suribabu and Nilkanthan et. al., 2006, Montalvo et. al., 2008), Harmony Search (Geem 2009), mine-blast algorithm (Sadollah, et. al., 2015) etc.

Multi-objective design optimization has also been explored by various researchers by applying evolutionary algorithms to the water distribution network design. Various least cost design solution is reported (Halhal et. al., 1997, Walters et. al., 1999, Prasad and Park 2004, Kapelan et. al., 2005) that take full advantage of the benefits by fulfilling the constraints.

A constrained problem of WDN optimization design was presented by Farmani et. al., (2005) using a self-adaptive fitness function. The beauty of the method is to handle infeasible solutions to remain fit during a low objective function value. The main advantage of this method is that it handles multiple objectives together. To performs such multi-objective optimization, it does not require an initial solution to the location global optimal solution. At the same time, Vairavamoorthy and Ali (2005) proposed a GA based methodology to minimize the capital cost deprived of bargaining adequate with pressures even in peak demand. The global solution search was performed by a pipe index vector. Several benchmark networks investigated in the literature were verified using this method to realize a robust and efficient solution in search of a global optimum. A new approach has been proposed by Babayan et. al., (2005) to quantify the influence of demand uncertainty. For this, an original stochastic model was restructured and clubbed with GA having a deterministic approach. The solution obtained has a robust performance while identifying for a global solution. The suitability of the Harmony Search based optimization model was tested by Geem (2006). This methodology was tested on five water distribution networks. The designed results showed superiority in global search.

A similar approach was adopted by Khu and Keedwell (2005) obtained solutions using GA considering a multi-objective optimization technique. Solutions for the design of a WDN were compared with single-objective GA and a two-objective algorithm are called Non-dominated sorted GA-II (NSGA-II).

Shuffled Complex Evolution (SCE) algorithm was tested for the design of WDN as a powerful optimization tool (Liong and Atiquzzaman, 2004). It was linked with the hydraulic solver EPANET 2.0 to handle hydraulic constraints and solve network design problems. A similar method was proposed using NSGA-II by Atiquzzaman et. al., (2006) to help decision makers by suggesting a best alternative network design. The proposed solution was designed to consider limited funds. It must also tolerate a pressure deficit condition in the entire distribution system. The optimization was externally supported by the EPANET hydraulic solver to deal with the pipe size simulation and to estimate pressure.

Design of water networks for minimum cost was also investigated using the branch and bound integer linear programming method (Samani and Mottaghi, 2006). The optimized network was subjected to various constraints to be satisfied, such as; pipe sizes, flow velocities, reservoir levels and nodal pressures. An optimization technique using scatter search (SS) heuristic proposed by Lin et. al., (2007) has been used in the network optimization problem of WDS. Solutions were obtained with the three sample networks which indicate the robustness of algorithm in comparison with the other algorithms in the literature. A combinatorial optimization method called heuristic cross-entropy approach was used by Perelman and Ostfeld (2007) to obtain the optimal design of WDS. Chu .et. al., (2008) proposed another heuristic approach to obtain the optimal design of the network. This approach was inspired by the defence process of the biological immune system called Immune Algorithm (IA).

The performance of various meta-heuristic methods was evaluated by Reca et. al. (2008) in the design optimization of the water system. It comprises GA, Simulated Annealing, Iterative Local Search, Tabu Search etc. The results indicated the robustness of GA to handle medium size network. Reca et. al. (2008) evaluated the performance of meta-heuristic techniques such as GA, Simulated Annealing, Tabu Search (TS) and Iterative Local Search used in the design optimization of the water distribution networks. It was observed that the Genetic Algorithm was more efficient when dealing with a medium-sized network, but other methods outperformed it while dealing with a real complex network. A modified Harmony Search Algorithm was studied by Geem (2009) incorporating 'Particle Swarm' method in the design, whereas, Di Pierro et. al. (2009) optimized a WDN multi-objective problem using two hybrid algorithms namely ParEGO and LEMMO. A new Memetic Algorithm (MA) was analyzed by Baños et. al., (2010) for the optimal design of the water distribution systems to test the performance of the network.

So far, we have covered various optimization techniques experimented by various researches. The next section will cover the network design considering pipe sizes to suit the appropriateness of the distribution network.

2.3.2 Joint Consideration of Network Layout with Pipe Sizes

In the design of WDN, size of the pipe is an important constraint to be handled. Rowel and Barnes (1982) addressed this issue as a joint problem design of WDN. The network layout was obtained for single and multiple sources WDS. Least expensive layout design was proposed by Bhave and Lam (1983) with a dynamic programming approach. The model consists of two linked LP formulations. In this approach, one linear program determined the least cost layout and other LP program determined the least cost component design for the initial pressure distribution. Goulter and Coals (1986) developed a reliability-based least-cost design of WDN considering two quantitative approaches. The Linear Programming (LP) technique was used in both the approaches to obtain an optimal layout of WDN. Awumah et. al., (1989) proposed a model for the layout optimization of the water distribution networks under single loading. The pipes forming the network were selected using a zero-one integer programming model considering redundancy and hydraulic requirements. The solution was then refined to obtain a layout for the pipe sizes and pressure heads.

A methodology for a branched pipeline irrigation WDS was developed by Lejano (2006) to obtain the optimal layout. He considered the spatial distribution of potential clients and their respective demands. An empirically derived objective function was designed using a mixed integer LP (MILP). Afshar (2007) used a GA approach to find the simultaneous layout and pipe size optimization of WDNs (Afshar et. al., 2005, Afshar 2006).

Tanyimboh and Setiadi (2008) designed WDN for joint layout, pipe size and reliability optimization by means of a multi-criteria maximum entropy approach. The main criterion considered was the capital cost of the system. A new multi-objective evolutionary optimization approach was developed by Saleh and Tanyimboh (2013) to the simultaneous layout and pipe size design of WDS. They developed a discrete optimization technique considering capital cost and a unified feasibility measure accounting nodal pressures and network topology.

2.4 Heuristics

Deb and Agrawal (1995) proposed a non-dominated sorting GA (NSGA-II) to the design WDN as a constrained problem. For the optimization problem of a continuous function having multiple objectives, this approach was widely recognized, in which, a binary representation has been used in concurrence with traditional genetic operators. Those operators were point mutation and one-point crossover. A real-value representation was suggested for continuous function optimization problems. In turn, requiring representation of specific genetic operators such as polynomial mutation and simulated binary crossover (SBX).

A multi-objective evolutionary algorithm (MOEA), was in practice since the 1980s (Farmani et. al., 2004). It was found that MOEA can accurately deal with several objectives and several linear and non-linear constraints. In the search process, the constraints are handled separately which generally require no penalty coefficient in the solution process (Nicolini 2004). Therefore, an MOEA produces a set of optimal solutions that can improve the decision-making procedure in the design of pipe network problems.

The following section further elaborates the application of NSGA-II in the optimum design of the WDN.

2.4.1 Application of NSGA-II in the least cost design of WDN

NSGA-II offers a trade-off between the various objectives considered. A comprehensive decision to substitute options is largely depends on trade-off information. However, to reduce the computing time, parallel computing is the most promising way to faster convergence with adequate solutions when efficacy and efficiency are of concern (Artina et. al., 2012). The results to the optimal design of small and medium-size WDN are promising for different parallel applications of NSGA-II.

As the population diversity progressively reduces, the MOEA based on the NSGA-II approach may have chances to be trapped in the local optima (Chen et. al., 2016). Therefore, some improvement by multiple recombination operators has been incorporated in NSGA-II, which is then called c-NSGA-II (Chen et. al., 2016). The study suggests a computational search strategy to maintain the diversity of the candidate solutions which results in a global optimum solution.

From a set of the solution, size based on the market availability for pipes, a set of the optimal pipe diameters is chosen. This minimizes the total network cost satisfying the various hydraulic constraints. The cost of the pipe networks can be used as one of the objective functions for the optimization of the distribution network. The EPANET platform along with an NSGA-II is clubbed together to solve the optimization problem (Li et. al., 2012). A multi-objective optimization algorithm (NSGA-II) can be coupled with the WDN hydraulic simulation software EPANET toolkit to provide the much-needed Pareto front of the cost and the nodal pressure deficit.

2.5 Research Gap

GA has been successfully used to solve various types of water distribution network systems design problem. Several modifications have been suggested in GA to increase their efficacy and efficiency in identifying an optimal solution in the past two decades. It was observed that various meta-heuristic approached even along with hybridization techniques has difficulty to locate the global optimum solution with very few iterations. The search becomes more extensive when GA and their various variants had to handle large networks. This is because of the excessive use of probability operators which are configured to global search. In addition, the search is depending on several parameters followed by hydraulic constraints and several methods of penalty application are some of the important parameters to assess the global optimum solution. Low penalty values may allow several infeasible solutions to remain in the population and participate in the crossover to generate new strings. On the other hand, high penalty values may result in immature removal of infeasible solutions from the population. Self-organizing or Self-adaptive penalty methods are favoured over other approaches of penalty applications as they provide more importance to solutions on the border of the infeasible and feasible front (Wu and Walski, 2005). The said approach has been explained with details in Chapter 3 to elaborate on various research gaps.

The main constraints in water distribution network design problems are satisfying the nodal demands with the desired pressure. Therefore, researchers have considered applying a penalty in proportion to the maximum head violation at any node in the network by considering that demands are satisfied. To satisfy the nodal demands, Kadu et. al., (2008) suggested a self-organizing penalty method based on equivalent energy charges. By assuming that the demands are satisfied at the nodal, the deficiency in the nodal head is achieved. However, demands are not completely satisfied at all of the nodes for an infeasible solution.

Therefore, high penalty values are calculated for such infeasible solutions and may result in immature expulsion from the population. The infeasible solutions on the boundary of the infeasible-feasible front should have an appropriate penalty to keep them in population and provide nearly equal opportunities to improve, just as those provided to a feasible solution on this front (Wang et. al., 2014). Deb et. al., (2000) investigated a Penalty-parameterless constraint handling method for single-objective optimization design. Their investigation demonstrated that a tournament selection-based algorithm can also be used to handle constraints. In another study by Sayyed et. al. (2015), authors have proposed a node flow approach to achieve the least functional evaluation using a non-dominated sorting genetic algorithm technique. The methodology proves that their technique is better than the numerous researches in WDN design.

Numerous investigation adopted NSGA-II approach, especially for the multi-objective design of WDN. A study by Wang et. al., (2017) proposed a genetically adaptive leaping algorithm (GALAXY) hybrid MOEA which comprises of six search operators. By maintaining a better balance between local and global search, they improve the efficiency and effectiveness during the optimization. Despite this efficient proposal, the necessity of a few function evaluations for finding the global optimum solution was expressively high.

A newly proposed SBA algorithm (Xie, 2018b) shown some ability to avoid getting trapped into a local optimum or global optimum infeasible solution. The principle to explore global solution is based on organizing tactics of benchmarking principles. The investigation elaborated in this thesis proves that SBA does not require any type of penalty application as it takes care of all the constraints. Therefore, a more efficient global search heuristic has been tried to solve the optimization of WDN and developed a methodology called SiBANET to prove its effectiveness. Herein, a more appropriate approach is suggested to obtain global optimal feasible solution satisfying hydraulic constraints of the available head and available flow at each node in the benchmarked network. The SiBANET methodology shows superior intelligent approach to deal with an infeasible solution on the boundary of feasible-infeasible solution front to remain in the population and improve further with its benchmarking principle protocol.

2.6 The scope of the present work

The study was carried out with consideration of following objectives:

1. To develop a simplified methodology by using a freely available and widely used hydraulic solver EPANET and carry out hydraulic analysis of WDNs.
2. To club a simple benchmarking algorithm and modify the EPANET methodology for its application to design WDNs and compare the effectiveness of the method with respect to other cases in the literature.
3. To compare the performance of SBA and with another IOA algorithm.
4. To investigate the state-of-the-art hybrid framework and identify its advantage.

Chapter 3 : Genetic Algorithm for Design of WDN

In the recent historical development of various metaheuristics optimization techniques, most of them were generated from very simple concepts of the nature of evaluation. From the evolutionary concept point of view, optimization is done by evolving an initial group of random solutions by evolutionary algorithms. One of the most popular algorithms in this type is the Genetic Algorithm (GA) which is a member of a club of search algorithms based on artificial evolution (Holland, 1975).

GA has been widely used in WDNs for optimization problems. Any literature review would not be exhaustive to possibly contain all the information available. Throughout this chapter, an example of related problems pertaining to WDN design will be covered wherever possible.

Current state of the art research into GA has been covered into the following section to elaborate on the optimization purposes involved in it. A brief explanation of GA's actual working shall be presented. It is important to discuss GA in this chapter to understand Simple Benchmarking Algorithm (SBA) which follows the basic design aspects as practised within GA. The probabilistic optimization of SBA is similar to that of GA, but the constraints handling strategy and organizing tactics which make SBA superior to other IOA used for WDS design. The intelligence principle of SBA strategy with suitable selection of its parameters to tune with EPANET has been illustrated in Chapter 4.

3.1 Introduction

For exploring developments in the field of GA's, *the genetic algorithm tutorial* (Davis, 1996; Whitely, 1994) and *good textbooks* (Goldberg, 1989) was used as a reference in earlier works (Back, 1995; Michalewicz, 1996; Mitchell, 1996; Deb, 1996).

Some applications of GA in the field of WDNs optimization for pipe diameter selection/optimization is proposed by Dandy (1996), Savic and Walters (1997), Morley et. al., (2001); Kahraman (2003). For the optimum location of control valves on a network, optimization of valve control, calibration of WDN (Creaco and Pezzinga, 2015), and hydraulic management of the WDS are some important work.

The possible solutions are first encoded into chromosome-like strings for optimization of a function such that the genetic operators can be applied. Mutation and Crossover are the two main genetic operators which are based on natural biological evolution. A population of randomly generated solutions is generally used to start GA. To form a newer solution as children, two solutions selected by the crossover operator are acted as parents. In the current population, parents are selected from a pool of all the solutions. Stochastically biased selection takes place in the direction of solutions for better objective function value. In evolutionary terms, such solutions are known to have higher fitness. The fitness evolution is determined by the goals of optimization (e.g. lowest cost or highest reliability in WDN) is based on the trial solution. Therefore, it is appropriate to say that Darwin's theory (1859) of 'survival of the fittest' is followed by GA.

The randomly selected solution is been modified to some extent by Mutation operator to form a new solution. After a certain number of crossovers and mutations, new solution replaces some of the solutions in the old population to perform one generation of the algorithm. Until a stopping criterion is met, these generations are kept on repeat. To optimize the real-life problems several additional features are necessary to preserve the best solution in the process of generation. One feature is Elitism strategy to preserve the best solution throughout the generations. For a robust performance, additional fitness scaling or ranking is often necessary. In case of WDN problems which are constrained in nature, application of GA (were originally intended for unconstrained problems) is based on different approaches of the user to locate the optimum solution. This problem shall be covered through the various sections in this chapter.

3.2 Advantages of GA

The simple nature of optimization is a main advantage of Genetic Algorithm. Differences that separate GA from the more conventional techniques could be defined as follows:

- The coding of the parameter set is used by GA instead of the parameter itself.
- It searches the entire feasible solution space with multiple starting points and therefore it has more chance of providing a global optimum solution.
- It moves randomly in search of a better solution and does not require any auxiliary information
- GA directly use the commercially available pipe diameter for WDN optimization problem and hence solutions are more field application oriented
- It also provides several near-optimal solutions through the probabilistic transition rules (stochastic operators) for the choice of the field practitioner

3.3 Elements of the Genetic Algorithms

The various parameters involved in GA search for an optimal solution to WDN are as follow:

3.3.1 Representation Schemes

Individuals or feasible solutions in evolutionary computation approaches are being represented by a genetic representation scheme. The genetic representation may be used to encode physical. Qualities, behaviour and the appearance of individuals. In evolutionary computation, the most difficult problem is to design a decent genetic representation scheme.

One candidate solution which generally represents as the block of memory is called an 'individual'. Whereas, 'population' is a subset of all the possible solution to a given problem. A 'chromosome' in general terms is referred to as a data block. One such solution to a given problem is called 'chromosome'. The data in that block is called a 'chromosome' (one such solution to a given problem is a 'chromosome'). Figure 3.1 articulates the representation of GA. As shown in the figure, each chromosome consists of a gene which is its element position. The possible values of a particular gene are called alleles which gene takes from a particular chromosome.

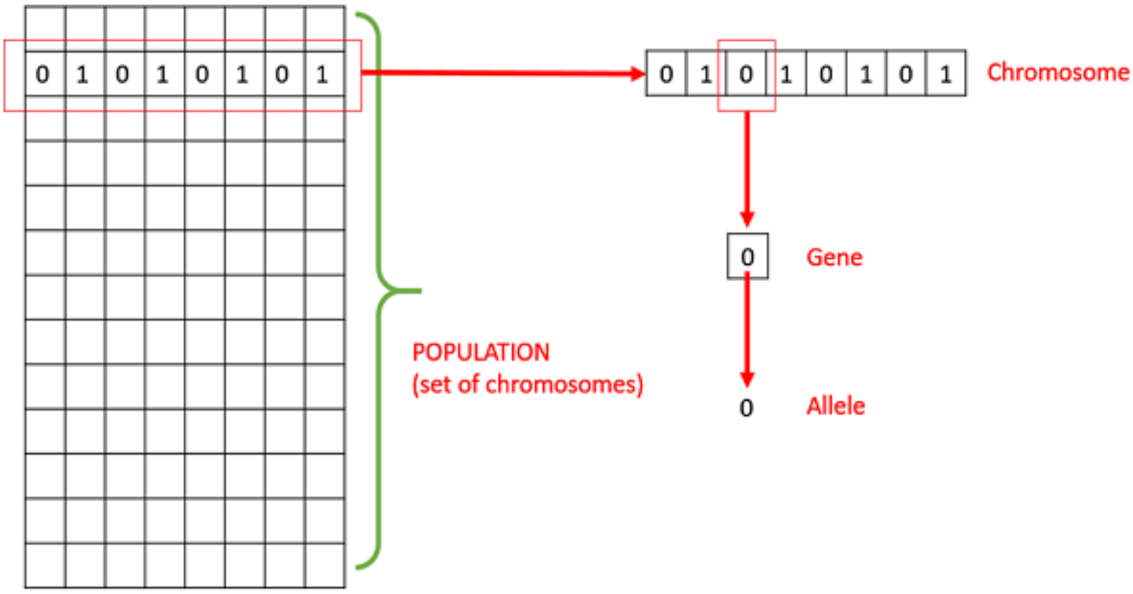


Figure 3.1 Basic terminology used in the genetic algorithm (Source: www.tutorialspoint.com)

In simple GA technique for water distribution network optimization, each commercial pipe size is coded using a suitable representation scheme to form the alleles. For WDNs, the alleles representing the pipe diameter contains randomly selected commercially available diameter joined together to form one individual solution. Using a binary, gray, integer and real coding, all the individuals of a population can be represented. These are briefly explained below.

3.3.1.1 Binary coding scheme

The binary coding scheme uses 0s and 1s to represent a commercially available pipe size. Therefore, a solution string prepared by joining alleles will contain binary information. The bit size to represent the entire commercially available diameter will depend on the number of commercial diameters available. However, some substrings that may be redundant due to the schema are assigned combining larger diameters. The binary scheme to represent 14 diameters will require 4-bit substring i.e. $2^4 = 16$ will be able to accommodate these pipe diameters. Two substrings will be redundant in this scheme. Whereas, 6 diameters of the pipe will be represented by 3-bit substring. Just like the previous scheme, two substrings will be redundant in this scheme as well. In this way, the total length of the individual string will be the product of several pipes in the water distribution network. The section of the bit-size is more important to accommodate the various commercial diameters of pipe. For example, a network containing 20 pipes and 9 commercial diameters will have an individual solution string containing $20 \times 4 = 80$ bits.

3.3.1.2 Gray coding scheme

The gray coding is similar to the binary coding except that the representation differs slightly. In the case of gray coding, the adjacent decision variable coded substring differs by one bit or is separated by a Hamming distance of 1 (Caruana and Schaffer, 1988). It is specified in Table 3.1 along with other coding schemes. As gray coding eliminates the Hamming cliff of the binary coding (Caruana and Schaffer, 1988), the performance of GA might improve. However, length of the individual string remains same as that of binary coding scheme.

3.3.1.3 Integer coding scheme

In integer coding, pipe sizes are represented by using the digits 0, …, 9 and one digits represents one bit as in binary coding. If the number of commercially available diameters is less than or equal to 10, it is suitable. If the available pipe sizes are more than 10, then the redundant substrings will be very large in number. As 100 pipe sizes can be accommodated within only two-digit substrings. Table 3.1 illustrations the use of integer coding scheme for the sample size considered herein.

Table 3.1 Different coding schemes

Number	Pipe size (mm)	Coding scheme			
		Binary	Gray	Integer	Real
(1)	(2)	(3)	(4)	(5)	(6)
1	100	000	000	0	100
2	150	001	001	1	150
3	200	010	011	2	200
4	250	011	010	3	250
5	300	100	110	4	300
6	350	101	111	5	350
7	400	110	101	6	400
8	−	111[a]	100	(7-9)[b]	−

[a] redundant substring; [b] mapping of a pipe size is not required in the real coding scheme

3.3.1.4 Real coding scheme

The real coding scheme uses discrete available pipe size directly to form a solution string, as shown in Table 3.1. The problem of redundant substring does not arise in real coding. It is also advantageous, as it does not require any decoding of the parameters for the evaluation of the fitness, thereby saving the time of computation. Evaluation of all the representation schemes shows that real coding may be the best (Wardlaw and Sharif, 1999). It produces a smoother curve while plotting maximum fitness versus the generation number. It has some advantage over other schemes of representation such as the higher maximum fitness and the smoother convergence towards an optimal solution.

The next section covers various operators of a genetic algorithm which influence the minimum solution in the design of WDN. It comprises initialization, reproduction, crossover, mutation, fitness function, termination condition, elitism and penalty.

3.3.2 Initialization

Three primary operators of each genome are initialization, crossover and mutation. The population has simply initialized the GA using initialization operator. The population carrying pre-specified genetic information which it evolves to produce all solutions. The initialization operator is randomized for a wide search space which depends on the developer's desire. However, it takes more time to execute if the parallel implementation of such wide search spaces is not adopted (Goldberg, 1989). The wider search space is possible through more randomized initialization. Different coding schemes were evaluated by Wardlaw and Sharif (1999) and they concluded that the real coding scheme is the best. It produced a smoother curve while plotting maximum fitness versus the generation number. The binary coding made erratic progress towards better fitness.

The real coding scheme sustained its longer rate of improvement due to a slower rate of improvement in fitness somewhere in the mid-generation period of the GA run. Therefore, to reach the optimum solution with real coding in binary and Gray coding schemes require more generations. This scheme has many advantages over the binary scheme and the Gray coding schemes. The advantages include: (1) higher maximum fitness; (2) smoother convergence to the optimal solution; (3) higher average fitness of the population; and (4) reduced execution time to obtain the optimal solution.

3.3.3 Reproduction

It is one of the first operator applied to the population of solutions (= *P*). It selects good strings from the population with respect to the fitness of the solution to form a mating pool. Therefore, it is sometimes known as the *selection operator*. The essential idea in it is that some solution strings would be selected more than once.

The objective function value (maximization or minimization) takes fitness of a solution string. For WDSs in general, fitness of the solution string can be expressed as:

$$f_i = \frac{1}{C_i}, i = 1, \ldots, pop - size \qquad 3.1$$

where, f_i = fitness of solution *i*; C_i = total cost of the system represented by string *i*. In this process, the worst solution die off, whereas the best solution strings get more copies and the average stay even (Michalewicz, 1992). Thus, the solution strings in the mating pool, containing *P* solutions are selected in a probabilistic manner. The probability of selection of string *n*, i.e., p_i given by:

$$p_i = \frac{f_i}{\sum_i f_i} \qquad 3.2$$

Thus, *the survival of the fittest* principle is abode by reproduction scheme, wherein more fit strings make more copies for mating than less fit strings.

3.3.3.1 Roulette Wheel Selection

Here, a roulette wheel is used for random selection. Its circumference is marked for each string of the population in proportion to the fitness of the string. The string element is chosen by the wheel pointer during each wheel spun (*P* times). Thus, *P* solutions in a mating pool are selected. The roulette wheel is expected to make a copy of a string according to the above equation. The roulette wheel selection is also called a proportional selection.

It has been observed that, in selecting good individuals, the roulette-wheel selection makes noise. Even though this scheme is easier to implement, more stable version, known as the *stochastic remainder selection* is sometimes used. Copies of the solution are sent to the mating pool which is precisely equal to the mantissa of the expected count after the expected count for each selection is calculated. The decimal part of the expected count of each solution is used for the regular roulette-wheel probability selection. The stochastic remainder selection reduces the noise in selecting good individuals.

3.3.3.2 Rank Selection

In the rank selection scheme, the computed fitness values are used to order the population. The best individual is ranked one and the rank of the worst individual is *P*. According to the individual rank in the population, the parents are selected based on probability (Whitley 1989, Michalewicz 1992). This selection scheme ignores the relative fitness of an individual in the population as a constant selection differential is maintained between the best and the worst individual.

3.3.3.3 Tournament Selection

Random selection from the population takes place by first picking a few individuals with (or without) replacement. Then it selects the best of the chosen individuals for inclusion in the next generation. Until the appropriate number of individuals is selected for the new generation, this procedure is kept on repeating.

All the above mention reproduction schemes were compared by Goldberg and Deb (1991) and indicated a preference for tournament selection.

3.3.4 Crossover

Here, information exchange between two strings is implemented using probabilistic choices in a systematic way. In this development, from the mating pool, two newly reproduced strings are arbitrarily selected to exchange their corresponding portions of binary strings. Depending on the set of crossover points, better qualities are united in a crossover process can be implemented in a variety of ways among the preferred good strings. There are two ways of recombination that have been implemented in the process of crossover; these are, Real-valued recombination and Binary-valued recombination.

3.3.5 Mutation

A genome may undergo mutation after recombination. In order to truly initiate the genetic process, a mutation operator needs to be incorporated which imitates the random mistake as committed by Nature. New genetic material is introduced in the population using mutation. The mutation operator introduces small random changes into chromosomes and thus prevents irreversible loss of certain patterns (Wall, 1996).

Being trapped at local maxima is prevented by mutation operator in GA. However, the mutation process usually occurs with a small probability and may abruptly spoil the opportunity of the current generation. As a common practice, the mutation rate is set to $1/n$. Here 'n' is the number of variables (dimensions) in the genome. Fast converging GAs can be achieved by dynamic crossover and mutation probability rates (Sheble and Britting, 1995) to that of constant probability rates. Mutation heavily depends on the genome representation as it affects their behaviour depending on the implementation. For example, with a given probability, a binary string genome flips each bit in the string (Wall, 1996). On the other hand, the mutator would swap sub-trees with a given probability in a genome represented by a tree.

Some alternative solutions in the genetic pool may disappear in the algorithm process which may have the potential to lead to the final solution. Such alternatives get a chance through mutation operator to be in the population pool.

3.3.6 Objective (fitness) function and generation transition

The better solution to the problem is used by fitness function to identify the genome and assigned a 'fitness' score in each generation. The genomes that score higher fitness survived to the next generation process of reproduction or mutation. It is possible to play with fitness values which can be normalized, scaled, shared, or left unchanged to get the superior solution. How new genomes will replace from the previous generation must be specified by the reproduction algorithm. During this process, the population must remain constant in each generation. This is important to avoid an exponentially increase in search breadth. Occasionally, it may happen that the entire population is replaced in each cycle, and sometimes only a subset of the population is replaced. A generation gap value decides the percentage of a population to be replaced.

In addition, to remain in the population, a certain number of best individuals can be guaranteed to keep some of the best-fitted chromosomes as they are in the next generation without losing their identity. This is called ‘elitism’ (Varsek et. al., 1993). Elitism is generally used to quickly increase the performance of GA. It is further discussed in the next section.

Some of the following metrics are used while evaluating the above-mentioned selection schemes:

- **Bias**: Absolute difference between normalized fitness of an individual and it’s likely probability of reproduction
- **Loss of diversity**: The proportion of individuals of a population that is not nominated during the selection phase
- **Selective pressure**: The probability of the best individual being selected as compared to the average probability
- **Spread:** A variety of likely values for the number of offspring of an individual

3.3.7 Termination conditions

GA will typically run repeatedly unless a suitable termination condition is allocated. After a fixed number of generations have been simulated, the ‘terminate-upon-generation’ condition halts the program. At this point, the best existing solution is returned. On the other hand, a pre-set threshold level trigger for ‘terminate-upon-convergence’ to takes place if the best existing solution scores a fitness level limit. These two strategies can also be combined for termination.

3.3.8 Elitism

Selection schemes are based on statistics in elitism. The fittest members are more likely to be chosen than the fewer fit members. However, if the population size is small, this situation may not essentially occur. Sometimes, selection for reproduction may not happen or it may cross with an incompetent member and vanish from the population. The fittest member of the population may not be selected for reproduction or it may cross with an unfit member and disappear from the population. Even though there is a continuous increase in the average fitness of the population, a member with those same characters may not emerge for hundreds of generations. This situation unreasonably extends the optimization process. Elitism limits this type of problem to ensure that they will survive until better members replace. This happens by copying one or few of the fittest members of each generation into the next generation.

3.3.9 Penalty

As discussed earlier, GA search ability is influenced by various parameters such as coding scheme, fitness function, population size, penalty method, the probability of mutation '*pm*', the probability of crossover '*pc*', selection and crossover operators and hydraulic simulation technique adopted. For handling the nonlinear constraints in the design, a penalty function is generally designed to penalize the infeasible solutions to reduce their fitness. To suit the application of evolutionary computing techniques, the constrained optimization model can be converted to an unconstrained model. In the optimization design, constraint violations and a penalty multiplier (p) are generally assigned as a function of the penalty cost. The multiplier p is often defined in such a way that a greater constraint violation results in a higher cost of the penalty. For effective implementation of GA as an optimization tool, many constraint-handling techniques were developed. Nonlinear constraints in GAs are handled through the application of penalty function. Therefore, it is the most important component in the design of a water distribution system which is generally unconstraint problems. In order to deteriorate the optimality of an infeasible solution, a penalty function is planned by adding a penalty cost to its objective function. In this way, by constructing a weighted sum of the penalty cost and the original objective function, non-constrained type optimization problem can be formulated. Therefore, this type of optimization method such as a genetic algorithm is more likely to help to get the solution.

GA is usually blind during a WDN optimization process to judge for a feasible solution. Therefore; the quality of the optimum solution is only determined as the sum of the objective value and penalty cost. That's the reason why it is very important to study a penalty approach in network design. It can be also understood that the optimization efficiency of GA is treated as best if the total cost in any design for least.-cost optimization is the smallest possible value.

Any infeasible solution using excessive penalty cost is quantified as a less-favourable solution. This solution required a large penalty factor to guarantee a feasible solution. Therefore, the cheapest feasible solution is less than the sum of the penalty cost and the objective value. An effective penalty factor depends on the constraints and the dimensions of the optimization problem, which generally varies from one optimization study to another study approach. Many penalty-function methods (Michalewicz and Schoenauer, 1996) have been proposed so far within the framework of the GAs approach for solving Non-linear constrained optimization problems which require tuning the penalty factor. Different penalty factors will produce a diverse range of penalty costs for the same solution. Thus, penalty factors greatly affect the rate of convergence and the solution optimality.

Effective penalty factors to a problem may be adjusted to get a specific optimum solution. The penalty factor is manually reduced to fine-tune the convergence rate which is a tedious and time-consuming process. Considering this tedious process, Deb and Agrawal (1999) developed a niched-penalty method in solving constrained type optimization problems in the genetic algorithm. Prasad and Park (2004) have effectively applied this method to get the optimal design of WDN. Even though the niched-penalty method avoids the penalty factor, it usually generates a constantly high selection pressure. This led the GA to search for an optimum solution within a feasible solution region. The niched-penalty rules were further improved by Coello and Mezura-Montez (2002). For solving single-objective constrained optimization problems, they introduced a concept of the selection ratio to maintain variety in the solution.

In addition to the above, the self- organizing penalty technique was proposed by Lin and Wu (2004) to resolve a constrained non-linear problem. Whereas, Kadu et. al., (2008) considered the additional pumping cost to formulate the self- organizing penalty. The maximum deficient head was referred to calculate the extra pumping cost to lift the water. Wu and Simpson (2002) proposed a boundary search method which is a key to improving the efficiency to get an optimized solution for WDN design. Their study demonstrated a robustness improvement and significant efficiency for a single-objective optimization problem.

The evolutionary algorithm which uses various penalty function methods are:

3.3.9.1 Death Penalty

This method simply rejects infeasible solutions (Back et. al., 1991; Michalewicz, 1995). Such rejection gives a poor performance as it avoids searches from approaching boundaries where the optimal solutions generally locate.

3.3.9.2 Static Penalty

This penalty method was proposed by Homaifar et. al., (1994). For constraints violation.

$$C_p(X) = \begin{cases} 0; & X \in F \\ \sum_{j=1}^{M} f_{ij}\,[g_j(X)]^2; & X \notin F \end{cases} \quad 3.3$$

where F is the region of the feasible solution and f_{ij} is the penalty factor for constraint 'j' at violation level 'i'. The level of constraint violation must be set earlier. Such penalty type varies with kind optimization problem. Therefore, it is often difficult for defining suitable violation levels for a non-linear constraint. Because of such a limitation, this method generally needs a large number of penalty coefficients despite a small size optimization network.

3.3.9.3 Dynamic Penalty

Joines and Houck, (1994) proposed the dynamic the penalty which is as follows:

$$C_p(X) = \begin{cases} 0; & X \in F \\ (c \times T)^2 \sum_{j=1}^{M} f_{ij}\,[g_j(X)]^3; & X \notin F \end{cases} \quad 3.4$$

where T is the number of generations /iterations and the value of c=0.5.

During the process of optimization, as the number of iterations increases, the penalty on an infeasible solution increases accordingly. By using such method in early generations, good results can be achieved. There is a chance of getting trapped into local minima during the search for optimality.

3.3.9.4 Annealing Penalty

Michalewicz and Attia (1994) proposed the annealing penalty as:

$$C_p(X) = \begin{cases} 0; & X \in F \\ \dfrac{1}{2\tau}\displaystyle\sum_{j=1}^{M} f_{ij}\,[g_j(X)]^2; & X \notin F \end{cases} \qquad 3.5$$

Here, τ is called as the temperature analogy. It shows that, as the temperature is cooling down penalty cost increases during the process of optimization. The range of temperature varies starting from 1.0 to a freezing temperature such as 0.00001. It can be understood from the formula that the temperature is an equivalent for the inverse of the penalty factor. The temperature can be differentiated in the penalty factor by dividing all the constraints into linear equations.

3.3.9.5 Niched Penalty

Deb and Agrawal (1999) proposed that during the genetic selection operation, constraints are satisfactorily handled. In this situation, the solutions are compared pairwise through tournament selection. Following these constraints, the criteria below must be fulfilled:

1. Any infeasible solution wins over any feasible solution
2. Based on constraint violations, two infeasible solutions are compared
3. Based on their objective function values, two feasible solutions are compared

To calculate the normalized Euclidean distance between two solutions, a niching technique is proposed. This will preserve the variety of solutions in one GA population. Such critical distance is important to measure for comparison of two solutions with their objective function values only when they are within critical range.

3.3.9.6 Self-Organizing Penalty

Lin and Wu (2004) proposed the self-organizing penalty. It is handled as:

$$C_p(X) = \frac{100+T}{100} \times \frac{1}{M} \times \sum_{j=1}^{M} f_j^T g_j\,(X);\; X \notin F \qquad 3.6$$

where, f_j^T is the penalty factor at T generation for the j^{th} constraint. The beauty of this method is that each constraint gets assigns to one penalty factor. Here, f_j^T is improved by satisfying the constraint j^{th}. Generally, a huge number of penalty factors are needed for a highly constrained optimization problem in WDN.

3.3.9.7 Self-Adaptive Penalty

Tessema and Yen (2006) proposed a self-adaptive penalty function and applied for constrained optimization in WDN design. As optimization of the network must fulfil many designs constraints, the 'maximum-violation penalty Function' for the constraint solution is calculated for penalty cost (*X*). It is defined as follows:

$$C_p(X) = \begin{cases} 0; & X \in F \\ f_p \underset{j}{argmax} |g_j(X)|^2; & X \notin F \end{cases} \qquad 3.7$$

Where, $C_p(X)$ is the penalty cost for optimum solution X during optimization and for the penalty factor f_p to be evolved.

3.4 Illustration for Penalty approach

With enough understanding of the theoretical background of GA and its components, an illustrative is attempted here to explain the application of penalties. For a better understanding of various discussed penalties (including a selection of candidate diameters), the potential of self-adaptive penalty for a single source water distribution as proposed by Kadu et. at., (2008) is illustrate as follows:

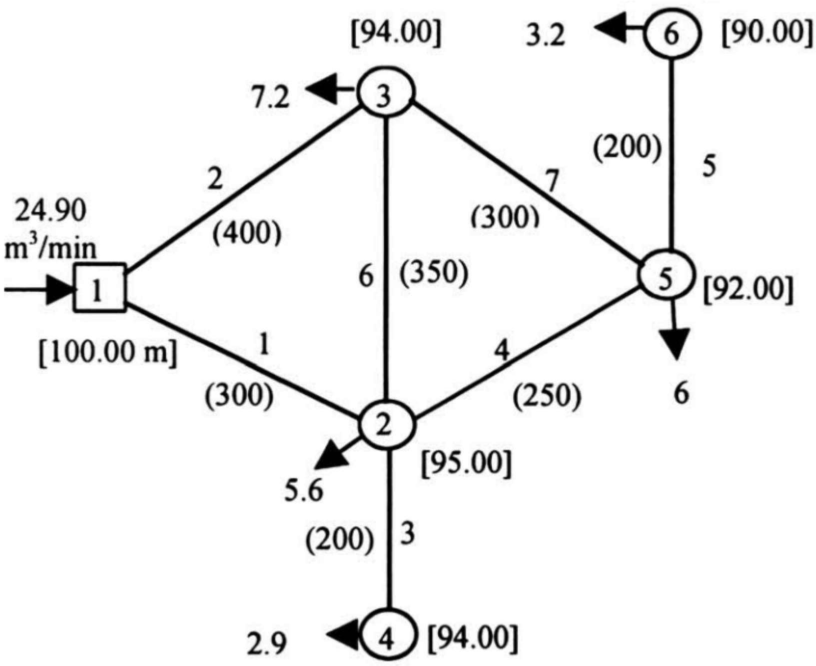


Figure 3.2 Single source water distribution network (Kadu et. al., 2008)

Figure 3.2 consisting of five demand nodes and seven-link WDS connected with the single source (Node 1) of supply. The source node is at HGL of 100 m (constant head). The least required HGLs in meter in marked in the bracket and the demand nodes are labelled 2–6. The nodal demands in *m^3/minute*. Links 1–7 and their lengths in meters given in parenthesis are shown along the links. Links 1–7 and their lengths in meters given in parenthesis are shown along the links. The pipe sizes (available in the retail market) and their respective unit price are given in Table 3.2.

Table 3.2 Pipe size and their unit cost

Sr. No.	Diameter	Cost (Rs)
1	150	1115
2	200	1600
3	250	2154
4	300	2780
5	350	3475
6	400	4255
7	450	5172
8	500	6092
9	600	8189
10	700	10670
11	750	11874
12	800	13261
13	900	16151
14	1,000	19395

Note: 100 Indian Rupees = 2.2 US Dollar

The head loss has been estimated by a Hazen-Williams formula in Eq. 4.5. The values of ω is chosen as an average value i.e. 10.667. Here the values of h and length L are in meters, Q in m^3/minute and D in millimeters. The value of ω becomes 2.234 x10^{12} in Eq. 4.5. The Hazen – Williams's coefficient of 130 has been considered for all the pipes.

The single source WDN was proposed by Kadu et. al., (2008) as shown in Fig. 3.2. This network was designed to demonstrate the modified GA approach which required very few numbers of function evaluations. This approach also reduces search space to locate the optimum solution. GA operators in the study are the probability of crossover=0.95; population size=60; a number of generations= 25 to 50 and probability of mutation=0.02–0.05. The same parameters are used herein under the application of various types of penalty strategies for comparison. The convergence characteristics of GA with cost optimizations as objective functions shown in Figure 3.3.

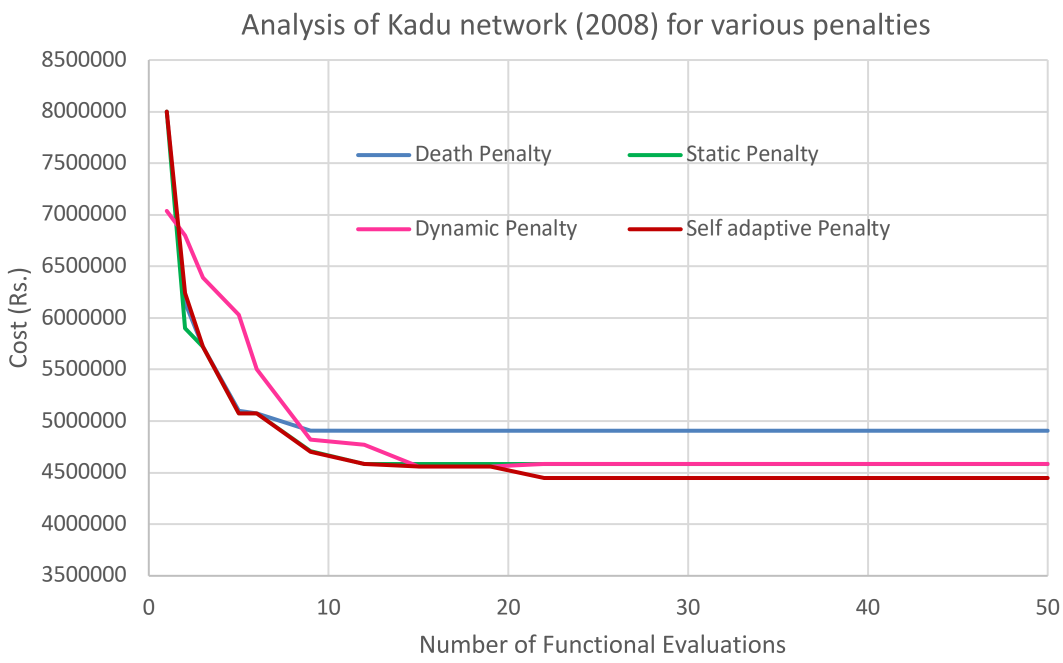


Figure 3.3 Convergence characteristics of various penalties

Death penalty was applied to eliminate the results which are lying in the infeasible region. The static penalty was applied considering the addition of fixed cost based on the number of nodes which are not meeting the head constraint. The dynamic penalty considered the number of generations in the calculation of penalty cost. From Figure 3.3, it is observed from the comparative analysis of various penalties that self-adaptive types of penalties are more promising in terms of cost optimization and helps the GAs to converge faster for a globally optimal solution within few numbers of functional evaluation.

Form the various studies, it is understood that the GA search is affected and influenced by several parameters such as penalty method, fitness function, population size, selection and crossover operator, coding scheme, the probability of crossover, and mutation. In addition to all the above-mentioned, the choice of the hydraulic simulator to handle various constraints has been demonstrated in the previous section with an illustrative example. In addition to these parameters, important parameters such as the size of the network and the number of available pipes sizes are having an influence on the GA search for an optimum solution.

In various studies, it was observed that the convergence to the optimum solution was often delayed when GA have to search across huge discrete space in case of some large networks. Therefore, to reach a global optimum solution along with the least required cost, it requires more computation time. It is possible to produce a feasible solution more rapidly within a few numbers of iterations, but for that possibility, a reduced search space is required. By doing this, it means that the number of trials required to get the solution is excessively high which makes the optimization process more tedious for relatively large size water distributions networks.

Optimum design of a water distribution system is a constrained non-linear optimization problem. A penalty function is often employed to transform a constrained into non-constrained optimization problem within the framework of genetic algorithm (GA) search. A penalty factor is used for defining the penalty function and calculating the penalty cost for the solution with constraint violation. Effective penalty factors vary form one optimization model to another. The analysis performed in this thesis was to verify the impact of accuracy level of 'self-adaptive penalty' which required least functional evaluation to explore global optimum solution. The results show that this approach is more effective than other penalty methods used for search of optimal and near optimal solution.

Finding the optimum solution for a water distribution design is sensitive to the penalty cost. Successful application of an optimization model to a real system requires trial-and-error evaluations for choosing an effective penalty factor. The analysis presented in this thesis shows that it is safe to choose a big penalty factor to start with, then manually reduce the penalty factor to lead the GA for the optimal and near-optimal solutions.

3.5 Further understanding of the research gap

Based on original IOAs methodology, many optimization strategies have appeared in improved versions in this decade. But still many of those newer versions of intelligence strategies unable to explore the optimum solution for different categories of the problem the domain. Various scholars have proposed numerous unique search patterns taking reference of the GAs framework and introduce a breakthrough in the traditional idea of the intelligent computing approach. A variety of encoding schemes have been used to map the problem using the population and individuals to search for the optimal solution. But it has been observed that the use of probability guidelines influences the enumeration of a global result. It affects the IOAs tendency i.e. the so-called 'intelligence' of search behaviour.

Even though GA firstly created the idea of using coded individuals to get the optimal solution via "intelligent enumeration", it has several limitations. Probabilistic approach confines the intelligent computational thinking boundary which is mostly based on a strict mathematical judgement. Therefore, it is important to apply the search strategies which do not completely rely on the stochastic approach. To present-day, virtually all the existing intelligent optimization algorithms (IOAs) have failed to escape the probabilistic model of traditional thinking.

The next chapter attempts to put forward new search ideas which prove superiority and robust performance during the application in the domain of water distribution network.

Chapter 4 : Simple Benchmarking Algorithm for Design of WDN

4.1 Introduction

The crucial characteristic of a water distribution network is to perform adequately under financial limitation. Traditional approaches in design are based on minimization of cost factors considering various hydraulic constraints. With the advent in computer-aided design, the application of meta-heuristic algorithms also called an intelligent optimization algorithm (IOA) for WDN design has shown promising success for finding the economic expenditure. Despite of all these developments, the guarantee to locate a global optimum solution is relatively less due to the stochastic approach. To ensure the solution evaluated is optimum, extensive trial runs are required. For a larger network, the computation time for excessive iterations is very high. Numerous schemes/strategies for WDN considering minimum price have been reported in the literature for optimum evaluation (Wang Qi et. al., 2017; Cardoso *et. al.,* 1994; Cunha & Sousa 1999; Savic & Walter 1997; Prasad & Park 2004; Suribabu & Neelkantan 2006a; Geem 2009; Sayyed 2017).

In the past 20 years of optimum design for WDN, verities of the computer-based optimization tools / strategies have been implemented to a diverse type of projects (Neelkanthan & Suribabu 2005; Wu *et.* al., 2001; Savic and Walters 1997; Eusuff & Lansey, 2003; Suribabu & Neelkanthan 2006a; Geem 2009; Mohan & Babu 2010; Sayyed 2017). It includes heuristic approaches e.g. Genetic Algorithm (GA), Shuffled Complex Algorithms (SCA), Honey-Bee Mating Algorithm (HBMA), Simulating Annealing (SA), Particle Swarm Optimization (PSO), Non-dominated Sorting GA(NSGA-II), etc. Some advantages and disadvantages are associated with such traditional approaches. The difficulties associated with formulation include ease, fast convergence, handling non-linearity, handling discrete diameter, etc. Such limitations have led researchers to use these algorithms with some deterministic math-based approaches. Various combination of GA, PSO, SA, NSGA etc. along with their modified versions have experimented in WDN optimization (Wu 2001; Neelkanthan & Suribabu 2005; Kadu *et.* al., 2008; Geem 2009; Wang *et. al.* 2014; Sayyed 2017). These investigations demonstrated that deterministic models are better in the computational efficiency point of view when compared with the various stochastic algorithm, but still the dominant factor of probability rules of operator affect the global search solution.

In very recent time, a novel method has been introduced to the optimization strategy which is called the Simple Benchmarking Algorithm (SBA). SBA (Xie 2018a, 2018b) adopts an intelligence strategy to find the best solution during a global search in which so-called 'intellect' is based on operational rules, probability equations and mathematical formulation. The individuals within the solution space learn from each other and emulate a good example according to the organizing tactic. A modified approach with SBA algorithm is introduced so-called 'SiBANET' to optimize the design cost of WDN by considering two well-established benchmark cases. For this purpose, SiBANET used a well know hydraulic simulation model in WDN design called EPANET 2.0 (Rossman 2000). The results are verified for the optimum cost and a number of iterations. The prime objective of the proposed investigation is to check the efficiency of SiBANET in search of a global optimum solution. Additional objectives are:

1. To apply SBA for WDN design methodology.
2. To compare the performance of SBA and other IOA techniques.

3. To investigate the state-of-the-art hybrid framework and identify its advantage.

SBA discusses several sets of configurations strategies for locating a local and global solution. In the SBA approach, the learning strategy is embedded in the domain of global and the local benchmarking framework. The global benchmarking is served by the global optimal in the current generation. In the same way, the local optimal individual is supported by the local benchmarking. The probability values as a set of random figures which lie between 0 and 1 for the global benchmarking, local benchmarking, and self-learning. The beauty of simple benchmarking algorithm is that it manages to maintain a unique balance between the exploration and the exploitation operation. The ability to perform a global search is generally reflected by an exploration method. This ability helps to develop new space. On the other hand, the ability to perform a local search and refining the improvement is reflected by an exploitation method.

It is presumed in optimization strategies that stronger exploration often followed by weaker exploitation. If the exploitation is too weak the search result is easy to diverge to locate the extreme points. This framework is not so helpful to find out the global optimal solution during optimization. In the same fashion, a local extreme point is affected by weak exploration. If the exploration is too strong and the exploitation is too weak, the search result is easy to diverge and easy to miss the extreme points. Ultimately, this is not so helpful to find out the global optimal solution. Similarly, weak exploration leads to a local extreme point.

It is important to see what strategy has been deployed by an intelligent optimization algorithm to balance its exploration and exploitation methods (Xie, 2018a). This help to estimates the performance of that specific IOA. While designing the algorithm, it is possible to achieve synergistic coexistence of exploration and the exploitation strategies even though they are complete opposite phenomenon. The automatic balance is possible to maintain during the search process.

For this purpose, the probability based hidden rules are required to be relaxed to balance the exploration and exploitation. SBA has ability to develop this strategy to maintain synergy. Such a type of search strategy refines the solution and brings more improvement in the process of optimization. The implementation of the global benchmarking and the local benchmarking during the iterative process is ensured by the guiding principle of SBA.

All the above-mentioned framework is explained in the following section.

4.2 The fundamental philosophy of algorithm framework

'Benchmarking' is a kind of management approach, comes from the field of business management. It is said that 'benchmarking' was proposed for the first time by Xerox Corporation's predecessor Halo–d Corporation. However, it is believed that as a management idea, benchmarking came from the traditional culture, especially the traditional management philosophy, because there are a lot of words in the ancient vocabulary of all ethnic groups. In traditional terms, benchmarking means that when you see a good man; you try to emulate him, and when you see a bad man; you search yourself for his fault.

The real meaning of the benchmark from a civil engineering point of view is a spot marked by the surveyor on a permanent object /structure of a fixed location and elevation used as a point of reference. It also means by a standard of measurement by which something can be umpired. In benchmarking in the field of enterprise management, the basic idea is to find the learning objects according to the existing accomplishments and improve the quality of the business via legal learning and effective matching. In other words, to improve its own performance, an enterprise compares against the performance of other companies for the overall operation of the company, or for a department, or for a business unit, or for a product, or for a particular process, and then learns and emulates the best practices.

The process of benchmarking can be divided into several stages, like setting a benchmark, benchmarking management, resetting the benchmark etc. In the actual implementation process, the management must be flexible to arrange and adjust due to different situations. Because of its practical effect, benchmarking has become a popular management idea and tool in the field of global business management since the seventies of the last century. However, benchmarking is not simply to learn from others, but includes four basic principles, which are four core values. This includes comprehensive quality (CQ), perfect process (PP), reference point (RP) and effective learning (EL). CQ means to achieve the customer's comprehensive satisfaction. PP means to cover the operation processes of the learning objects besides oneself. RP is to set up the common criterion on organizational functions as a basis for comparison. EL means to learn from others as well as self-learning.

As per the meaning of benchmarking as discussed above, there are numerous optimization rules hidden in the concept of this benchmarking philosophy. These guidelines can also pave references for the development of new types of IOAs. In commercial conglomerate, benchmarking is a procedure where it relate its own products and procedures to the outstanding companies in its field. The relative performance of the process is attempted to improve with this comparative analysis. In this way, the follower company which is trying to be successful, locates the gap and soon overcomes the inferior process by the study of the outstanding group of companies and consequently becomes one of those successful ones. It may also happen that they superseded their competitors and become the eminent company.

Thus, it has been observed that the best solution for an inferior company to become a superior is through the process of constantly resetting the benchmarks by copying the existing solution and gradually improve undergoing various iterations. Such a process can be translated into the modern computing technology with an optimization point of view in the framework of benchmarking principles.

4.2.1 The principle of setting the benchmark

There are several principles to be followed during the benchmarking process. The initial stage is determining a suitable rule based on the nature of the existing problem to be solved. In this process through benchmarking rules to be implemented, a relationship can be established between the decision variables and the evaluation objective, which easily can be translated as an optimization problem. The constraint range of each decision variables must be defined along with the evaluation criteria which ultimately becomes a vital part of a mathematical model.

4.2.2 The principle of studying and emulating

Every problem has some unique characteristics which must be studied first before starting the competition. In this respect, the competing company first carry out the self-benchmarking based on its own domain feature and evaluation criteria of the alternative scheme. This is the proven method in the corporate structure to study, emulate and improve the object.

Intelligent algorithms use the encoded individuals to get the optimal solution through an intelligent inventory. Therefore, whenever such schemes are adopted, it follows its own principle of studying and emulating to improve the structure and ultimately to lead to an optimized solution.

4.2.3 The principle of benchmark resetting

After successfully following the original benchmark to reach an improved stage, now it may realize two conditions; (1) the objective is fulfilled with an improvement and (2) inferior accomplishment of the objective. In both the situation, a new benchmark must be set by decision making to realize sustainable development and achieve desired results. In the same fashion, for the excellent performance of an IOA, next iteration is expected to be better regardless of the previous exercise. To achieve this, it is required to re-establish a new benchmark object conferring to the real situation of search space.

4.2.4 The principle of teamwork

This is the most important principle in benchmarking philosophy which targets individual teams as the basic unit to perform better. This means achieving shared progress through teamwork during benchmarking. When a team has to be translated in IOA terms which have collective intelligence in searching and optimization, the search performance of the group is upgraded significantly rather than the individual performance.

4.2.5 The principle of diversity

During the implementation of benchmarking, a certain degree of diversity is required amongst various company/industry/business. This diversity of decision variables helps to maintain a certain degree of redundancy and stability. Therefore, it is recommended to have alternate schemes and benchmarking objectives with a certain degree of diversity. For a superior performance of any IOA, it must maintain the principle of diversity during an optimization process. Most of the existing IOA encounters premature phenomenon due to the incompetent search strategy and inability to maintain population diversity.

4.2.6 The principle of efficiency

Benchmarking philosophy must give attention to to efficiency during performance. The efficiency of the process is understood from the output result depending on the input given. For a corporate company to improve its benchmarking objective, the cost of input is a matter of a concern as the output is already set. Therefore, it invests a lot in people, material and resources to improve the desired performance at various stages of benchmarking. Although, it is equally important to take care of the cost-cutting to show profitable results.

In the same fashion, IOA also applies the same strategy in search of a good candidate solution which incurs a relative cost. Here, core refers to run time, the consumption of storage space and other computing resources.

4.2.7 Maintaining population diversity

The role of balancing exploration and exploitation is taken care by probability measures by keeping the population diversity in the process. All the traditional IOA including genetic algorithm and its versions follow this basic principle. This criterion is fulfilled by increasing the crossover rate or mutation rate. In the case of PSO and its various versions, the criteria for maintaining the population diversity are performed by using the inertia weight factor and the acceleration coefficients. While in the simple benchmarking algorithm, local and global search is aimed to reduce the differences. It forms a certain clustering effect in the search process which leads to a reduction of the population diversity (Xie, 2018b).

If an IOA framework succeeds in maintaining the population diversity by adjusting the operators during the process, the performance is bound to be superior.

4.3 Methods

The optimum design of a WDN has been examined to assess the suitable mixture of economically available pipe diameters which gives the least cost. Various hydraulic parameters at different nodes have been calculated with the help of the EPANET 2.0 hydraulic network solver.

The objective function is to optimize a distribution layout subject to the cost constraint. This is done by choosing various pipe sizes which are considered here as a decision variable. The optimum network cost to be achieved can be framed as:

$$\textit{Cost minimization} = f\,(D1, D2, \ldots\ldots Dn) \qquad (4.1)$$

In which; n= total number of links in the network; and D= pipe diameter.

The total cost of the WDN can be expressed mathematically as:

$$Total\ Cost = \sum_{i=1}^{n_i} L_i \times c(D_i) \tag{4.2}$$

Where; n=total number of links and $c(Di)$ is cost per unit length for the pipe of length L. As pipe lengths are fixed for a given network, the diameter of pipes are the decision variables. The basic hydraulic equations involved in EPANET hydraulic network solver are the loop energy balance and nodal mass balance which is expressed as:

$$Q_{ext} = \sum Q_{in} - \sum Q_{out} \tag{4.3}$$

Here; Q_{ext} is the external nodal demand, Q_{in} is inflow at node and Q_{out} is respective outflows.

For each closed loop, the conservation of energy can be expressed as:

$$\sum_{i \in loop\ p} h_{f_i} = \Delta H_i \qquad \forall p \in N_L \tag{4.4}$$

Where; h_{f_i} = frictional head loss, N_L = numbers of loops in the network and ΔH the = difference between node heads.

The relationship between pressure drop, flow rate, pipe length and pipe diameter can be described by various head-loss equations developed so far. Amongst these, the formula proposed by Darcy-Weisbach and Hazen-Williams (HW) is the most widely used equation developed for the flow in a pressurized pipe.

A form of Hazen-William which is used to estimate head loss due to friction (Savic & Walters 1997) is:

$$h_f = \omega \frac{L_i}{C_{HW}^{1.85} D_i^{4.87}} Q_i^{1.85} \tag{4.5}$$

In equation (4.5), C_{HW} is the Hazen – William roughness constant and Q_i is the i^{th} pipe flow. Here, a numerical conversion constant $'\omega'$ is a dimensionless factor that depends on the units of various components used in the equation. The higher range of head loss can be achieved by choosing the higher value of ω. But the optimum solution using higher conversion coefficient is more expensive than that of its lower value of the efficient. In WDN optimization literature, ω ranging from 10.431 to 10.903. The average value of ω $\text{i. e.}\ 10.667$ is used (Eusuff & Lansey 2001). The head loss in an i^{th} pipe which is located between j^{th} and k^{th} junction is:

$$\Delta H_i = H_j - H_k \tag{4.6}$$

Here; ΔH=0 if network's path is closed.

The pressure head at all demand nodes to meet the minimum pressure requirement of a network should be greater than the allowable minimum pressure head (H_{min}) and it can be expressed as:

$$H_n \geq H_{min} \tag{4.7}$$

Where; H_n is allowable pressure head at demand node n.

4.3.1 Formulation of a model

A simulation-optimization model, named here as SiBANET, is developed in this study to optimize the water distribution system. The single objective that is considered here is the network cost. The optimization is explored by SBA algorithm framework. The algorithm is externally supported by EPANET 2.0 hydraulic solver. Such support is required to satisfy certain hydraulic constraints e.g. Loop energy conservation, mass balance equation, etc.

The hydraulic constraints that are considered for the WDN design are as follows:

$$D_{min} \leq D_i \leq D_{max} \qquad i=1, \ldots,N_i \tag{4.8}$$

$$V_{min} \leq V_i \leq V_{max} \qquad i=1, \ldots,N_i \tag{4.9}$$

$$P_{min} \leq P_j \leq P_{max} \qquad j=1, \ldots,N_j \tag{4.10}$$

Where $D_i =$ available commercial diameters, V_i = flow velocity in the i^{th} pipe, P_j = pressure in node j in the water distribution network.

4.4 Simple benchmarking algorithm (SBA)

The Benchmark Learning Algorithm (Xie, 2014) theory that is typically used in business management leads to the improved version as SBA. Benchmarking is a kind of business management approach that consists of various implicit rules for optimization. It is a strategy in the corporate sector where various companies correlate its merchandise, processes, marketing, etc. with leading firms in a similar domain. In other words, the gaps are identified comparing with the best product and quickly made up for analysing and mimicking. In this way, the company will surpass the competition by adopting the best possible solution.

Therefore, when benchmarking philosophy is considered, the existing good solution will be implemented in the initial stage. This is an iterative process which involves studying and emulating gradually from the sub-optimal solution to the optimal and ultimately reaching the best solution. Repeated assessment of the progress for setting new benchmark is essential to achieve the best solution during optimization. Such searching process offers a certain degree of intelligence. SBA offers an intelligent approach whose framework depends on self-organizing learning strategies instead of excessive emphasis on probability and operational rules (Xie & Liu, 2017).

As per the philosophy and the guiding principles for the search of optimization by maintaining the diversity in the population, SBA demonstrates remarkable intelligent behaviours as compare to other IOA and their versions discussed in Chapter 2. The process is unique and therefore it has been chosen to observe the results (for improvement) in the water distribution network field.

The basis rule of setting the benchmark solution is depends on the initial possible solution which is treated as a benchmark. Therefore, determine the suitable rule of setting benchmark according to the characteristic of current problem to be solved is the **first step**. In the modelling process, it is necessary to establish the casual relationship between the decision variables (pipe sizes) and the evaluation objective (cost). **Secondly**, it is necessary to define the constraint range of each variables (hydraulic constraints e.g. head, pressure, discharge etc.). These two parts constitute the model which is also an evaluation criteria. For decision making hydride model (SiBANET), an evaluation criteria is composed of evaluation function and constraints.

The pseudo code of SBA is shown below:

- Initialization: np, N, $Grate'$, $Brate'$, $Brate'$ and other parameters if necessary.
- *For gen*=1: max_gen, *do*

(a) *For i*=1:np, do

- Evaluate fi_j and f_i,
- Find and record Pi_{best}, i.e., set the local benchmark

(b) Find out and record P_{best}, i.e., set the global benchmark

(c) Find out, record and update the best individual in the ecological system.

(d) Evaluate $fE = (\Sigma f_i)/np$

(e) For I=1:np, *do*

- {$maxf(x)$: $Ni = Ni + fi/f E - 1$ and $minf(x)$: $Ni = Ni - f- /f E + 1$
- If $N_i = 0$, then the niche population P_i die out

(f) *For i*=1:np, *do*

(g) P_{ij} conduct global benchmarking

(h) If f_{ij} is not improved by then, P_{ij} will conduct local benchmarking

(i) If f_{ij} does not get improved by then, P_{ij} will carry out self-learning

(j) If fE yet does not be improved or the best individual in the ecological system does not get replaced, then each niche population exchange their local benchmarks

- *Output the optimal solution(s)*

4.5 SiBANET method and hydraulic model application

In the domain of optimum design for WDS, during the past 3 decades, verities of the computer-based optimization tools / algorithm have been proposed by adopting various mathematical models that mimic the natures' biological processes to increase the efficiency. It includes an intelligent optimization algorithm framework of Genetic Algorithm (GA) and its various variants like Non-dominated Sorting Genetic Algorithm (NSGA) etc., Simulating Annealing (SA), Particle Swarm Optimization (PSO) and its variants, Shuffled Complex Algorithms (SCA), Shuffled LeapFrog Algorithm (SLFA), Honey Bee Mating Optimization (HBMO), Particle – Swarm Harmony Search (PSHS), etc.

A first global optimization approach using GA to the least cost pipe sizing decision was proposed by Goldberg & Kuo (1987). This optimization method was equated to other modified techniques by Simpson *et. al.,* (1994) and Savic & Walters (1997). In addition to this, Cunha & Sousa (1999) used a simulated annealing algorithm whereas Eusuff & Lansey (2003) demonstrated a shuffled frog-leaping algorithm (SLFA) for the optimization of the water distribution network. Another improved version based on GA proposed by Kadu et. *al.,* (2008) considered a critical path, a technique to reduce search space and is called as a modified GA (MGA). Cisty (2010) shows that the GA-LP hybrid approach is better than the complete stochastic approach. In addition to the genetic approach, Wu *et. al.,* (2001) investigated a fast- messy genetic algorithm (fmGA) for the cost-effective rehabilitation strategy in the water distribution system. Another nature inspired strategy was proposed by Mohan & Babu (2010) called honey-bee mating optimization (HBMO) strategy for pipe diameter optimization. A modified harmony search algorithm (PSHS) demonstrated by Geem (2009) used a particle swarm notion and demonstrated a fast convergence requiring less functional evaluation. Sayyed (2017) proposed a multi-objective optimization approach using NSGA-II which considered various penalty approaches and satisfies the hydraulic requirement with the least cost design of WDN.

SiBANET is a collective simulation-optimization model used in this study which has been tested for two benchmarks WDN cases available in the literature. The simulation was performed by EPANET as the inner driver model which is developed by The United State Environment Protection Agency (USEPA). EPANET toolkit (Rossman 1999) offers a source code continuing dynamic link library (DLL) function. This allows users to customize the hydraulic network based on specific needs. The SBA algorithm, on the other hand, is an outer driver model which works in conjunction with EPANET for network optimization. Figure 4.1 shows the development of SiBANET and it's processed in the form of a flowchart. MATLAB software was used to run the SBA algorithm in conjunction with the EPANET toolkit. SBA was deployed to compute the design cost of WDSs and update the input file with new diameters. This process was continued until the best individuals in the ecological system were identified as the best possible solution.

A systematic approach has been illustrated with Two Lopped Network (Alperovits & Shamir 1977) and Hanoi Water Distribution Problem (Fujiwara & Khang 1990). The optimized solutions were obtained by implementing the SiBANET methodology considering a single objective of costs optimization whereas pipe diameters were the only decision variables. The computation time required for a certain number of function evaluations was also noted to judge the efficiency of SiBANET methodology. The systematic way in which the simple benchmarking algorithm is deployed for the optimal WDN design is depicted in Figure. 4. as a flowchart. The network parameters are initialized which includes a minimum hydraulic head to be maintained at demand node, nodal elevation and demand, pipe layout, source head, length of the individual pipe, pipe size, and pipe cost. The hydraulic heads are computed using EPANET (Rossman, 2000) at every demand node.

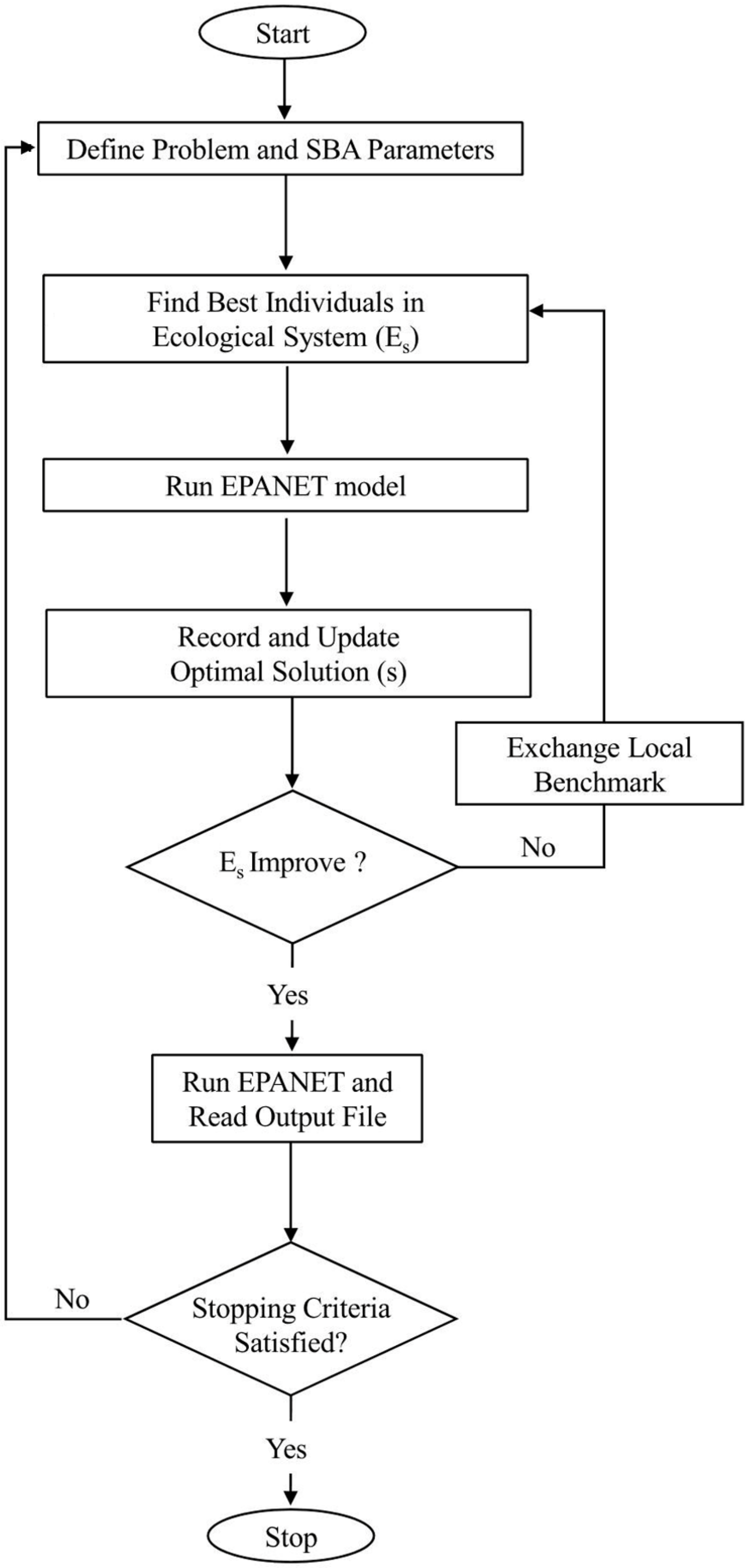


Figure 4.1 Flowchart for SiBANET methodology

The development of SiBANET methodology consists of the follows steps:

Step 1: Input initial condition (parameters like population, individuals, stopping condition etc.)

Step 2: Each individual is tested for pressure constraints on different nodes using EPANET

Step 3: Setting the benchmark according to the guiding principle of benchmarking

Step 4: Evaluate the fitness of each individual and the average fitness of each niche population.

Step 5: Local and global best are updated based on the fitness index of each individual

Step 6: Studying and emulating according to the specific method of benchmarking

Step 7: Fitness index of new individuals is computed, and local & global best are updated

Step 8: If termination criteria are not satisfied then Step 6 and Step 7 are repeated

SBA has some limitation to handle hydraulic parameters. Therefore, the hydraulic constraints of the water distribution layout have to be solved externally. Therefore, a joint simulation-optimization approach is presented in this investigation. To take care of the hydraulic simulation, EPANET (Rossman 2000) hydraulic solver as a reliable simulation model has been used by various researchers.

To customize the EPANET's computational engine, a dynamic link library (DLL) function in EPANET Programmer's Toolkit allows the user's specific needs to be implemented by developers. In this way, SBA is linked via the EPANET Toolkit in the MATLAB environment. The collective model called 'SiBANET' can be used to design new networks or redesign existing networks which do not have the most desirable performance.

While dealing with IOA for the optimization of the water distribution networks, a more robust parameter set must be considered which may vary to different kinds of distribution layouts in the design of WDN. It may include the choice of parameters guarantying a decent algorithm performance for any network layout. Even though the SBA algorithm is in its initial stage of development, excellent results have been evaluated in comparison to other IOA and its modified variant. Following are some of the crucial advantages of proposed methodology:

1. The SiBANET method depends on organizing tactics rather than probability rules of the operator.
2. No probability equations or mathematical formulations that need to be updated.
3. Achieved synergistic co-existence and automatic balance of exploration & exploitation.
4. Population diversity has kept during the algorithm run.
5. Inoperative and ineffective redundant operators are avoided in SiBANET

A new optimization strategy for WDN is proposed here using SBA as an intelligent method. This method follows a framework of an intelligent approach to locate an optimum solution which does not give more importance to the probability rule whole dealing with the population. In this way, a better balance is maintained for a balanced exploration and exploitation together at the time of execution. Such type of algorithm framework shows some intelligence; therefore, it is superior as compared to other

IOA's. The comparative statement is mention in Table 4.4 which shows the superiority of SBA. Some more explicit technical particulars have been adequately clarified by Xie (2018b) for the application in electricity distribution sector.

The evolution idea of continuously improving the solution space is perfectly followed by SBA. It is important for any algorithm structure to have a unique search strategy and show its robust performance. In addition to this uniqueness, cluster evolution and collaborative search have been well assessed by SBA. Another beauty of SBA is its ability to maintain the population diversity in the process of optimization search. During this development, many inoperable and unproductive repetitive operations are dodged resulting in less CPU computation time. The application of SiBANET is explained in the following sections.

4.5.1 The two-loop network

A well-known benchmark problem in WDS optimization study called Two-Looped Network (TLN) is used here for a demonstration of the SiBANET approach. TLN is a small size network generally referred to as a benchmark to test the water distribution system for optimization as shown in Figure 4.2.

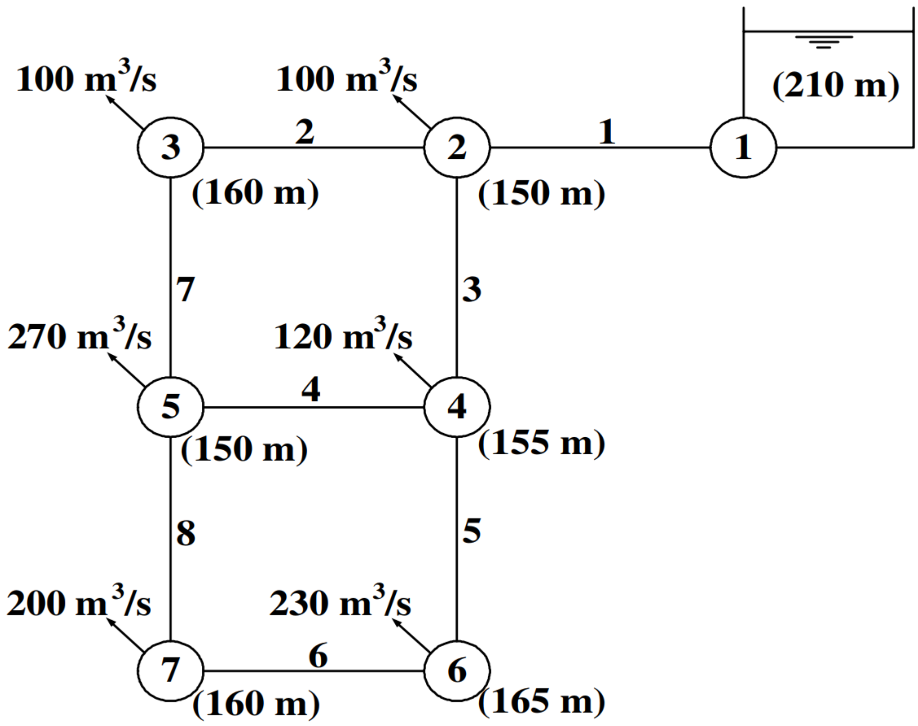


Figure 4.2 Two- loop benchmark problem for network optimization (Alperovits and Shamir, 1977)

All nodes are required to meet a minimum pressure demand for 30m. For the proposed network, Hazen-Williams coefficient of 130 is used. The nodes details of Two Looped Network along with demand and pressure requirement are given in Table 4.1. The respective elevation of nodes is also itemised.

Table 4.1 Two-looped Network node details (Alperovits and Shamir, 1977)

Node	Elevation (m)	Demand (m^3/hr)	Min. Pressure(m)
1	210	-1120	-
2	150	100	30
3	160	100	30
4	155	120	30
5	150	270	30
6	165	330	30
7	160	200	30

All the 8 links mentioned above are 1000m each in length. The minimum pressure required at all demand nodes is 30m. Hazen-Williams coefficient for the proposed network is 130 for all pipes (Alperovits and Shamir, 1977). The cost data for pipes is well documented in Alperovits & Shamir (1977) is shown below.

Table 4.2 Pipe size-cost table for Two-looped Network (Alperovits and Shamir, 1977)

Diameter (Inches)	Diameter (mm)	Cost (units)	Diameter (Inches)	Diameter (mm)	Cost (units)
1	25.4	2	12	304.8	50
2	50.8	5	14	355.6	60
3	76.2	8	16	406.4	90
4	101.6	11	18	457.2	130
6	152.4	16	20	508	170
8	203.2	23	22	558.8	300
10	254	32	24	609.6	550

In the case of TLN, which consist of 8 pipes, 14 discrete diameters sets are available for each pipe. Therefore, a search space consists of 14^8 (i.e. 1475789056 design combinations/number of outcomes) design combinations. At each node, EPANET 2.0 handled demand and pressure as decision variables implicitly. However; SiBANET works explicitly over 8 pipes as decision variables to explore the optimum cost of the network.

Figure 4.3 shows the graph of convergence characteristics for the two-loop network run in MATLAB R2016a.

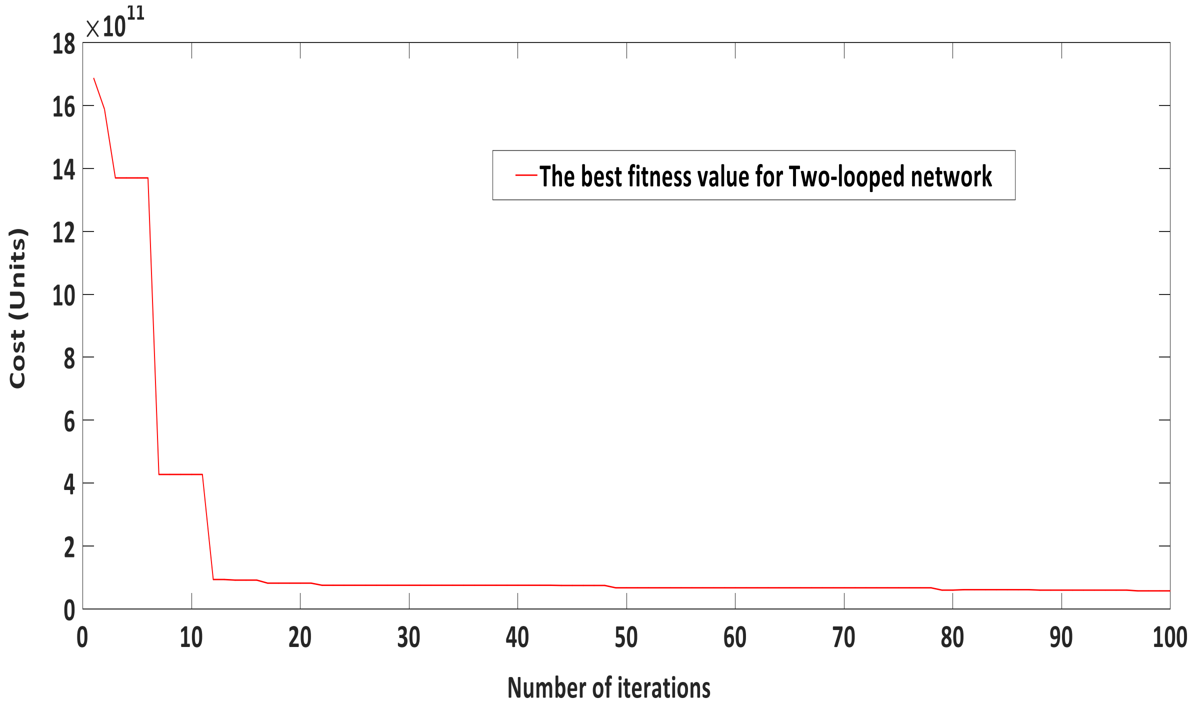


Figure 4.3 Evaluation of optimal design solution for TLN.

The least cost obtained using discrete diameter is 419,000 units with optimized pipe sizes for links 1 through 8 is represented in Table 4. 3.

Table 4.3 Evaluated optimum diameter for Two Loop network.

Link Number	1	2	3	4	5	6	7	8
Diameter (mm)	114.701	70.563	109.193	49.457	101.787	75.791	74.865	14.106

Various authors have also assessed the optimum pipe diameter solution for TLN using different optimization techniques as elaborated in Table 4.4. Pipe diameters are considered in inches as per Table 4.2.

Table 4.4 Comparison of pipe diameter solution for Two-looped Network

Pipe No.	Abebe and Solomatine (1999)	Savic and Walter (1997)	Wu Z Y (2001)	Afshar (2009)	Zhou (2016)	Present Work (2019)
	Traditional GA	Standard GA	fmGA	cGA	STA*	SiBANET
1	18	20	18	18	18	6
2	14	10	10	10	10	3
3	14	16	16	16	16	6
4	1	1	4	4	4	2
5	14	14	16	16	16	6
6	1	10	10	10	10	3
7	14	10	10	10	10	3
8	12	1	1	1	1	1
Cost (Units)	424000	420000	419000	419000	419000	419000
Evaluations	3381	250000	7467	3000	-	100

*STA= State transition algorithm

Note: the solutions are obtained using different numerical conversion constant for heal loss equation as given in Eq. (4.5).

The SiBANET method has also explored a similar cost solution for TLN within a few numbers of iterations. It is very clear from Table 4.4 that the pipe design optimization by SiBANET is the best-known optimized solution than the other techniques for the same hydraulic condition considering the average value of ω i.e. dimensionless factor for numerical conversion constant. The higher the value of ω, the greater is the head loss. The cost optimized solution is tabulated in Table 4.5 for further comparative assessment.

Table 4.5 Comparison of result for Two-looped Network

S. No.	Authors	Technique used	Optimal cost (Units)	Number of function evaluation	CPU time (seconds)
1.	Savic and Walter (1997)	Standard GA	419,000	65,000	600
2.	Cardoso (1994)	Non-Equilibrium SA	419,000	1,09,957	25
3.	Cunha and Sousa (1999)	SA	419,000	70,000	40
4.	Abebe and Solomatine (1999)	GA	424000	3381	Not available
5.	Wu (2001)	fmGA	419,000	7467	Not available
6.	Eusuff and Lansey (2003)	SLFANET	419,000	11,155	Not available
7.	Liong and Atiquzzaman (2004)	Shuffled complex algorithm	419,000	1,019	18
8.	Suribabu and Neelakantan (2006)	PSONET	419,000	760	2
9.	Suribabu and Neelakantan (2005)	MGA	420,000	58,380	86
10.	Geem (2009)	PSHS	419,000	204	Not available
11.	Afshar (2009)	cGA	419,000	3000	Not available
12.	Mohan and Babu (2010)	HBMO	419,000	1293	Not Available

13.	Zhou et. al., (2016)	Discrete state transition algorithm	419000	200	Not available
14.	Sayyed (2017)	NSGA-II, Penalty 2	419,000	400	10-25
15.	Present Work	SiBANET	419,000	100	3.28

Table 4.5 indicates the efficiency of SiBANET which took 3.28 seconds of CPU time with 100 iterations for function evaluation. The algorithm used the Windows 10 operating system on Intel i7-7700K CPU@4.20 GHz for computation. Different permutation and combination within the E_s have been performed. These trials involved a number of niches population, initial solutions, number of individuals within each niche population at the initial stage, and a maximum number of iterations within the cycle of environmental change. During several trial runs, it is noticed that the number of individuals in each niche and the size of the niche population performs a dynamic part in the accomplishment of the global optimal solution. The investigation was also repeated with numerous cycles of environmental change. However, this does not affect the overall performance of optimization. Comparing SiBANET results with other meta-heuristic optimization algorithms (see Table 4.5), it is evident that the proposed methodology has performed efficiently because it has an intelligent approach to locate the global optimum solution. Nowadays, it is important to discover more effective and fast converging algorithm procedures that can handle large size problem.

It has been observed that the optimal design of water distribution networks using SBA is a promising alternative in combinatorial optimization which is successfully demonstrated by the SiBANET methodology.

In the next section, a relatively larger size network has been configured for further investigation.

4.5.2 The Hanoi Network

The Hanoi benchmark network was proposed for Hanoi city by Fujiwara & Khang (1990). It is a three loop network having of 32 demand nodes, 34 links and 1 reservoir as depicted in Figure 4.4.

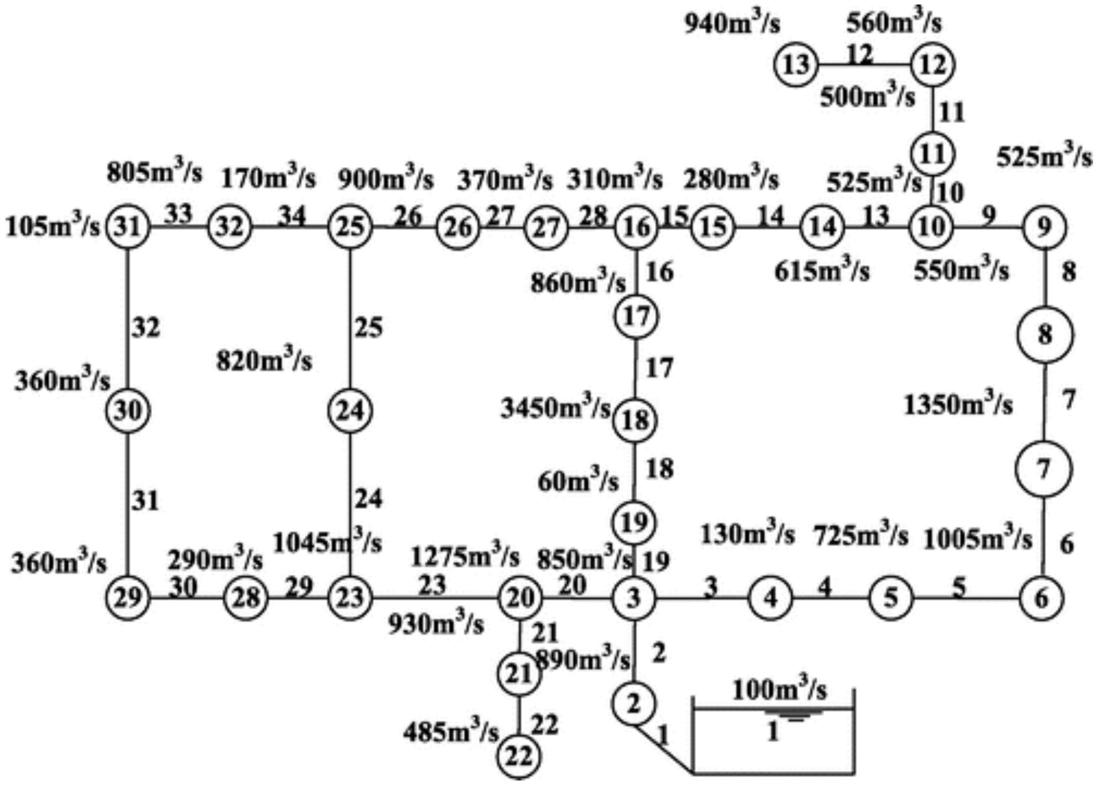


Figure 4.4 A network of the Hanoi water distribution system (Beygi et. al., 2014).

The Hanoi network is a widely used benchmark problem in large WDN design. It is used by several researchers for optimization (Cisty 2010; Prasad & Park 2004; Wu & Walski, 2005; Kadu et. al., 2008; Suribabu 2011; Haghighi et. al., 2011; Wang et. al., 2014 & Sayyed 2017). The hydraulic design of Hanoi water distribution network is restricted to six pipe diameters (for pipe cost data; See Table 4.6). Therefore, a huge solution space i.e. 6^{34} (2.87×10^{26} number of outcomes/design combinations) is available for optimization as compare to TLN. The unit cost of the pipe is given in Table 4.6.

Table 4.6 The unit cost of Hanoi Network (Fujiwara & Khang, 1990)

Pipe Type	Diameter (in)	Diameter (mm)	Unit Cost (\$ m^{-1})
1	12	304.8	45.7
2	16	406.4	70.4
3	20	508.0	98.4
4	24	609.6	129.3
5	30	762.0	180.8
6	40	1016.0	278.3

Overall 34 numbers of discrete decision variables are required to be handled by SBA. In the investigation of SiBANET, different permutations and the combinations of various components of the ecological system are performed with several trial runs. A similar methodology has been adopted from the previous example of Two Loop Network to assess the global optimal solution.

Table 4.7 Solution for Hanoi WDN with optimised candidate diameter (mm).

Link	1	2	3	4	5	6	7	8	9
Diameter	871.81	893.16	740.11	764.93	798.06	843.81	873.92	791.42	513.59

Link	10	11	12	13	14	15	16	17	18
Diameter	541.61	586.42	569.07	549.18	348.61	391.1	399.01	411.76	550.68
Link	19	20	21	22	23	24	25	26	27
Diameter	418.43	619.04	583.07	302.79	609.46	556.27	544.91	348.61	266.53
Link	28	29	30	31	32	33	34		
Diameter	241.54	311.06	237.42	199.67	299.96	311.06	332.53		

From these results (see Table 4.7), the optimum cost for Hanoi Network of $6.081 x 10^6$ (6081087 units) was obtained within 600 number of function evaluations. The comparative analysis is given in Table 4.8. Whereas, Table 4.9 shows the comparative analysis of the optimum pipe solution. The near optimum pipe diameters were selected by taking a reference to Table 4.5.

Table 4.8 Pipe diameters for optimal solutions of Hanoi network obtained by different techniques

Pipe Number	GA Savic et. al., (1997)	SA Cunha & Sousa (1999)	TS Sung et. al., (2007)	SiBANET (Present study)
1.	1016	1016	1016	1016
2.	1016	1016	1016	1016

3	1016	1016	1016	1016
4	1016	1016	1016	1016
5	1016	1016	1016	1016
6	1016	1016	1016	1016
7	1016	1016	1016	1016
8	1016	1016	1016	1016
9	1016	1016	1016	609.6
10	762	762	762	609.6
11	609.6	609.6	609.6	609.6
12	609.6	609.6	609.6	609.6
13	508	508	508	609.6
14	406.4	406.4	406.4	406.4
15	304.8	304.8	304.8	406.4
16	304.8	304.8	304.8	406.4
17	406.4	406.4	406.4	406.4
18	508	508	609.6	609.6
19	508	508	508	508
20	1016	1016	1016	762
21	508	508	508	508
22	304.8	304.8	304.8	406.4
23	1016	1016	1016	1016
24	762	762	762	609.6
25	762	762	762	609.6
26	508	508	508	406.4
27	304.8	304.8	304.8	406.4

28	304.8	304.8	304.8	406.4
29	406.4	406.4	406.4	406.4
30	406.4	304.8	304.8	406.4
31	304.8	304.8	304.8	304.8
32	304.8	406.4	406.4	406.4
33	406.4	406.4	406.4	406.4
34	508	609.6	609.6	609.6
Cost*	6.074	6.056	6.08 1	6.08 1

* Cost in million Dollar

Table 4.9 Optimum solution for Hanoi benchmarks WDS.

Sr. No.	Authors	Technique Used	Hanoi Network		
			Least cost solution (units)	No. of Funct. Eval.,	Time (s)
1.	Savic & Walter (1997)	Standard GA	6073000	1000000	10800
2.	Wu *et. al.,* (2001)	fmGA	6182000	113626	-
3.	Eusuff & Lansey (2003)	SLFANET	6073000	26987	-
4.	Liong & Atiquazzaman (2004)	SCE	6220000	25402	-
5.	Nilkanthan & Suribabu (2005)	MGA	6081087	1234340	1800
6.	Reca & Martinez (2006)	GENOME	6081000	50000	-
7.	Suribabu & Nilkanthan (2006a)	PSONET	6093470	6600	-
8.	Zecchin et. al., (2007)	ACO	6134000	85600	
9.	Sung et. al. (2007)	TS	6081000	40200	-

10.	Kadu et. al., (2008)	Modified GA	6190000	18000	468
11.	Reca et. al., (2008)	MENOME	6173000	26457	
12.	Geem (2009)	PSHS	6081087	17980	-
13.	Mohan & Babu (2010)	HBMO	6117000	15955	-
14.	Bolognesi et. al., (2010)	GHEST	6081087	16600	-
15.	Sheikholeslami et. al., (2014)	CSS	6081087	16440	-
16.	Sayyed (2017)	NSGA-II	6081087	18400	-
17.	Present study	SiBANET	6081087	600	236.78

GHEST= Genetic Heritage Evolution by Stochastic Transmission.

Table 4.8 shows that the optimization took about 237 seconds of CPU time for computation. Figure 4.5 represents the evaluation of the global optimal solution for the Hanoi network which shows fast convergence characteristics.

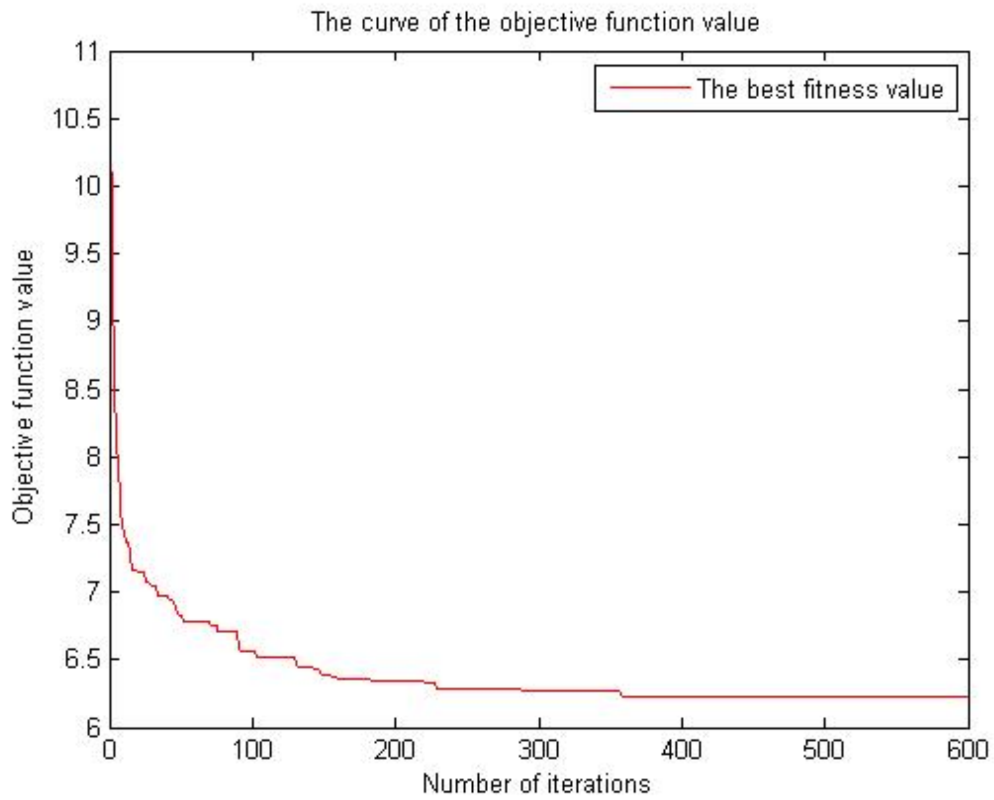


Figure 4.5 Convergence of optimal solution for Hanoi WDN design.

The solution obtained by the SiBANET methodology is slightly higher than that of Eusuff & Lansey (2003) for large network whereas there is similar cost computation for small size network. Furthermore, Geem (2009) evaluated the cost with the lowest number of iterations. However, then when it comes to a larger network of Hanoi, SiBANET converges was faster than all the listed IOAs.

As for randomness place a vital role, these independent results cannot be compared directly for reaching the optimum solution with respect to tiem required fro optimization. In comparison, the number of function evaluations and CPU time are significantly less in the case of SiBANET. While traditional optimization methods have a limitation towards NP-hard problem, the intelligence-based SBA algorithm is more promising due to its organizing tactics strategy.

4.6 Summary

In the domain of water utility management, WDN design is a constrained optimization problem for hydraulic requirements associated with the investment cost. It is necessary to incorporate new methodologies that ease the hydraulic design. When a meta-heuristic optimization technique is used to achieve an optimum solution, the constraints handling is a priority. An intelligent approach called SiBANET is formulated which comprise an EPANET 2.0 hydraulic solver. This new methodology was verified through the Two-looped network and the Hanoi network for assessing the minimum cost. The results of the analysis indicate that it is the most robust algorithm in handling discrete constraints wherein the global solution can be achieved within a few functional evaluations. It is also found that the CPU time is comparatively less than other IOAs. The SiBANET model has the desired results in WDN problems as it has the ability to handle the discrete pipe diameter along with fast convergence. The main reason for the lesser number of iterations is due to an intelligent exploration and exploitation strategy of SBA to locate a global optimum solution with its organizing strategies despite probability rules of the operator.

This study is a first effort to optimize the WDN utilizing an SBA intelligent approach in the meta-heuristic algorithm, hence only pipe-sizing optimization with a single objective was considered. Optimization of the complex looped networks with multiple objectives is highly recommended for further research. In the future, the proposed methodology can be prolonged to a large WDN by joining it with various decomposition approaches and various appropriate penalty function application to optimise the design. Form machine learning point of view, more learning method could be tested to improve the performance of SBA.

Chapter 5 : Forecasting Intake in Riverbank Filtration

5.1 Introduction

Researchers have confirmed and emphasised that increasing human interference with watershed and industrial growth leads to pollution of clean river water (Gavrilescu et. al., 2015; Islam 2015; Tsihrintzis 2013). The aquifer characteristics are also changing due to climate change and land use practices (Prasad et. al., 2016). Land disposal of human effluent and sludge is a major source of the toxic and hazardous pollutants to the surface water and groundwater. Many waterborne diseases outbreaks are caused by the consumption of groundwater that is contaminated by microbial pathogens (Beller et. al., 1997). One solution is to set up a pumping well or drainage gallery in the vicinity of the surface water body in order to take advantage of the riverbank filtration because it facilitates the removal of impurities, such as microbial contaminants, organic and inorganic chemicals, and turbidity (Prasad et. al., 2016; Ray et. al., 2002). The change in climate and land use practices has affected the available water quality, quantity, and demand. In addition, the overall management of the river and the sustainability of the riparian ecosystem in this dynamically changing scenario is a major challenge.

WHO (2008) recommends limiting the reduction of *Cryptosporidium*, *Campylobacter*, and *Rotavirus* in the filtered water within 4.2, 5.9, and 5.5 $\log_{10}$ units, respectively; because the presence of such pathogen beyond the recommended limit causes illnesses such as diarrhoea, vomiting, and intestinal infections. Scientists have proposed several methods to decide the suitable location of the groundwater abstraction point away from the contaminated water source, to contribute to sustainable water resource management. The pathogens need to be reduced to limiting values before they reach an abstraction point, otherwise, the treatment cost would escalate. To remove these contaminants, the riverbank filtration (RBF) is widely used because the technique is cost-effective (Derx et. al., 2013; Prasad et. al., 2016; Ray et. al., 2002). Although the RBF technique helps to reduce the pathogens, a problem with this technique is that the pathogens often do not get completely removed from the water (Pang 2009; Verstraeten et. al., 2003). The biological contaminants in the surface water body would enter the abstraction point if it is located near the water body in very close proximity, thus leading to high treatment cost. However, a well located far from the surface water source would decrease the specific discharge of the abstraction point. Consequently, the pumping cost to maintain the required withdrawal rate would increase and at the same time, the desired pumping rate may not be achievable (Gaur et. al., 2013; Prasad et. al., 2016).

Some literature reviews show that there is a specific distance between the abstraction point and the river which causes a reduction in the concentration of microbial pathogens by several orders of magnitude due to dispersion and decay of biomass (Ahmed and Marhaba 2016; Dalai and Jha 2014; Pang 2009; Schijven and Hassanizadeh 2000). Groundwater extracted beyond such specified distance from the river will satisfy the drinking water quality standards with optimal treatment cost. Thus, a feasibility assessment including pumping tests is often required to determine the optimum location of the production well although the pumping test is costly and time-consuming (Mategaonkar & Eldho 2012; Patra et. al., 2016). Researchers have developed several pathogen inactivation models under transport condition considering dispersion and decay for the assessment of water quality (Chrysikopoulos and Sim 1996; Harvey and Garabedian 1991; Pang 2009; Schijven and Hassanizadeh 2000;

Prasad et. al., 2016). The siting and design of the RBF system depend on the river hydrology, hydrogeological site conditions, and the aim of water withdrawal; however, for efficient and sustainable use of RBF, is that the river should be hydrologically connected to an adjacent aquifer.

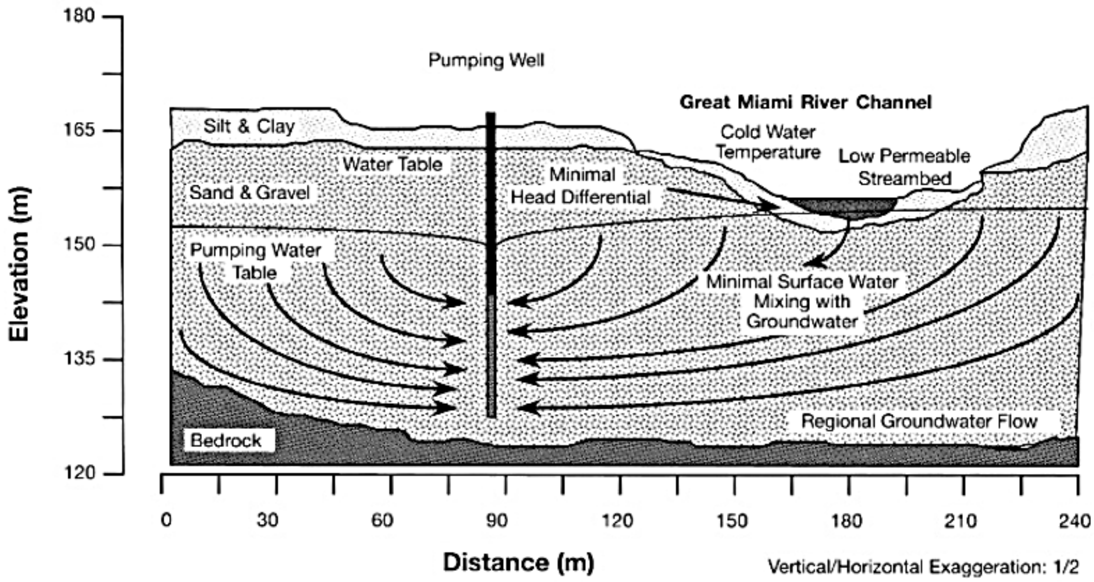


Figure 5.1 Condition for minimal induced infiltration (Ray et. al., 2002)

In the present study, the logistic differential equation was used to develop a decision tool to ascertain the optimum location of a pumping well. Verhulst (1838) introduced the logistic differential equation known as a logistic model (also referred to as inhibited growth model). The logistic model has been applied to several studies that focused on the population growth in the ecosystem (Chang et. al., 2016; Gershenfeld 1999; Pruitt and Kamau 1993). As per the literature survey, there is no direct application of the logistic model in the water resource sector. In this study, an analytic element Method (AEM) based flow model was developed and coupled with the Logistic model. Using the groundwater hydrodynamic flow equation, the travel time of a parcel of water was also derived. The safe distance of the abstraction point from a polluted river was then evaluated.

5.2 Methodology

In this study, a groundwater flow model was developed, which is based on the hydrodynamic equation and a pathogen transport model considering dispersion, reproduction, and decay. We called this model as logistic function-based pathogen transport model (LIFI-PATRAM). The flow fields were considered corresponding to the pumping rate Q_p (m^3/day), well location x_w (m) and various aquifer parameters. The initial concentration of the pathogen in the river water was known, and its allowable concentration was chosen as per drinking water quality standards (WHO 2008). From the specific values of decay rate and reproduction rate, the required number of log cycle reduction, *n*, in the concentration of the pathogen was determined. For the required log cycle reduction, *n*, with respect to the river water, the corresponding travel time t_r (days) by which the concentration will reduce from C_0 to $C_0 10^{-n}$ was ascertained. Based on the assessment of the travel time, t_r, the safe distance of the pumping well, x_w, was computed. A flowchart as shown in Figure 5.1 well demonstrates the proposed methodology.

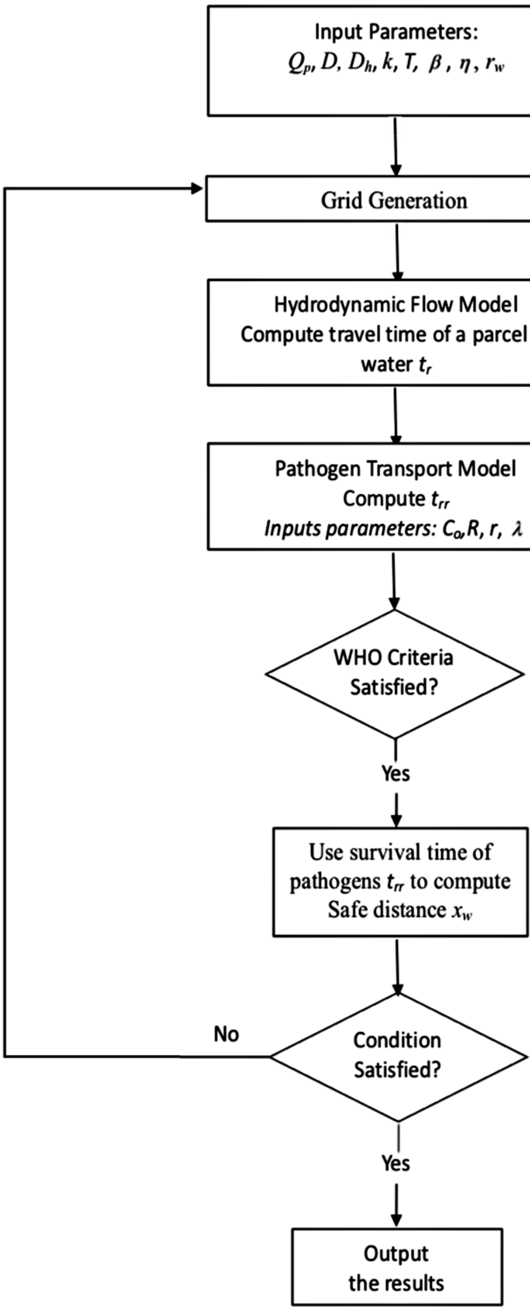


Figure 5.2 Flowchart for the proposed methodology.

5.2.1 Problem Statement

A pumping well located at a distance x_w from a fully penetrating straight river reach (bank) represented as y-axis is shown in Figure 5.2. A well is assumed to be fully penetrating the semi-infinite homogeneous isotropic aquifer. The objective is to determine the limiting (safe) pumping rate Q_p m^3/day so that the concentration of pathogens present in the river water decreases by 6 to 7 log cycle before the parcel of river water reaches the pumping well and attains the WHO prescribed limits. In other words, if a pumping well is to supply water at a given constant rate of Q_p m^3/day, the safe distance x_w of the proposed well from the river is to be ascertained to achieve the n log cycle reduction.

5.2.2 Model Formulation

The AEM-based flow model was used in this study. AEM is a computational method based on the superposition of analytic expression to represent a two-dimensional vector field. AEM can superimpose various analytic solutions to solve the groundwater flow problems and is capable of simulating streams, lakes, and complex boundary conditions (Strack 1989). In addition, the AEM flow solutions are inherently continuous over the flow domain and give a more accurate water budget for the area (Majumder and Eldho 2015; Matott et. al., 2006).

A stable-and-convergent numerical groundwater model is developed herein. Explicit models are subjected to a strict stability criterion which must be satisfied if the model is to simulate natural conditions in a realistic way (Douglas 1956; O'Brien et. al., 1949). A significant feature of this model is its simplicity which makes it attractive and useful for a wide variety of applications, including water quality modelling in riverbank filtration. The choice of an explicit model over an implicit model, in this case, is preferred. It is well known that implicit models are unconditionally stable but are subject to a convergence criterion which effectively places an upper limit on the time step. Examples of lack of convergence in water and sediment-transport modelling have been documented by Ponce *et. al.*, (1978; 1979). Therefore, when additional complexity is introduced, the implicit models may not be necessarily better than the explicit models. Here, the emphasis is on convergence, i.e., whether the numerical model can reproduce the differential equation with enough accuracy.

In pathogen transport movement, their growth and decay are governed by the structure and environment in the aquifer porous media. It is assumed that the aquifer material is spherical grains of uniform size *d*, and the material is under a rhombohedral dense cubic packing. The diameter of the spherical parcel of water, slightly less than $\left(\sqrt{d}-1\right)$, can pass through the aquifer material. Under the rhombohedral cubic packing condition, the diameter of the spherical parcel of water is $\left((2/\sqrt{3})-1\right)d$. For example, if *d*=1 mm, then the size of a spherical parcel of water is 155 microns. With this assumption, the mass transport equation is greatly simplified. Within a parcel of water which has a pathogen, the first order and second order spatial partial derivatives get eliminated and the equation reduces to the logistic equation. This reduced equation occurs when a source term is introduced due to the decay or elimination of the pathogen. The size of the indicator pathogen, e.g. an *E. coli*, is approximately 0.25-1.0μm in diameter and 2 μm in length (Reshes et. al., 2008). Therefore, when a group of' pathogen travel, each can remain confined in a spherical parcel of water. *E. coli* bacteria are considered here as non-pathogenic bacteria. However, *E. Coli* is the indicator organisms which are mostly found to be present along with the pathogenic bacteria. Moreover, *E.coli* is found to survive longer than the pathogenic bacteria and hence, in this study, *E. Coli* has been considered as the general notion for the pathogen.

5.3 AEM Flow Model

5.3.1 Steady State Flow Condition: Travel Time of a Parcel of Water

The travel time of a parcel of water was computed considering the earliest arrival of the pathogen through the shortest path from the river boundary to the pumping well. The river was considered as a line and the clogging of the river bed was ignored, by which the image well theory can be applied to compute the drawdown (Ferris et. al., 1962). The analytical solution for computing the drawdown is well documented (see Todd & Mays 2005).

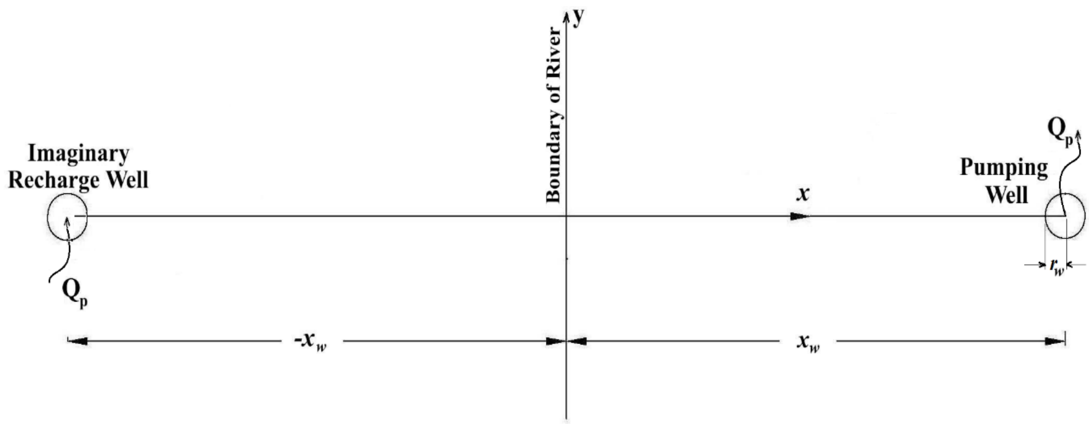


Figure 5.3 Schematic diagram of pumping well and image well

As shown in Figure 5.3, a well of radius r_w was constructed at a distance x_w in the vicinity of a fully penetrating river to pump the water at the rate of Q_p. The drawdown at a well point (x_w-r_w) along the x-axis under steady flow condition can be expressed as:

$$s_{(x_w - r_w)} = \frac{Q_p}{2\pi T}\{\ln(2x_w - r_w) - \ln(r_w)\} \quad (5.1)$$

where T= transmissivity of the aquifer medium.

The variable $S_{(x)}$ is then differentiated with respect to x to obtain an expression for Darcy velocity i.e., true velocity $u_{(x)}$ along the x-axis as given in Eq. (5.2).

$$u_{(x)} = k\frac{Q_p}{\pi T}\left\{\frac{x_w}{{x_w}^2 - x^2}\right\} = \frac{Q_p}{\pi D}\left\{\frac{x_w}{{x_w}^2 - x^2}\right\} \tag{5.2}$$

where D = thickness of the aquifer; and k is hydraulic conductivity.

From various field studies across the world (Ray et. al., 2002), following Table No. 5.1. Helped to choose various appropriate parameters to choose for the model which is formulated in this study.

Table 5.1 Various RBF systems with its hydraulic conductivity range (Ray et.al., 2002)

RBF System	River System	Aquifer Thickness (m)	Hydraulic Conductivity (m/s)
Henry, Illinois, United States	Illinois	15 to 20	2×10^{-3} to 3×10^{-3}
Jacksonville, Illinois, United States	Illinois	25 to 27	2×10^{-3} to 3×10^{-3}
Lincoln, Nebraska, United States	Platte	23 to 25	1.4×10^{-3}
Boardman, Oregon, United States	Columbia	13	3.7×10^{-3}
Casper, Wyoming, United States	North Platte	3 to 12	9×10^{-4} to 3×10^{-3}
Cedar Rapids, Iowa, United States	Cedar	12 to 18	7.5×10^{-5} to 1×10^{-3}
Cincinnati, Ohio, United States	Great Miami	~30	8.8×10^{-4} to 1.5×10^{-3}
Louisville, Kentucky, United States	Ohio	21	6×10^{-4}
Dresden-Tolkewitz, Germany	Elk	10 to 13	1×10^{-3} to 2×10^{-3}
Meissen-Siebeneichen, Germany	Elbe	15 to 20	1×10^{-3} to 2×10^{-3}

Totgau-Ost, Germany	Elbe	40 to 55	6x 10^{-4} to 2 x 10^{-3}
Auf dem Grind, Dusseldorf, Germany	Rhine	25 to 30	1 x 10^{-3} to 1 x 10^{-2}
Flehe, Dusseldorf; Germany	Rhine	10 to 12	3 x $10^{-3}$to 6 x 10^{-3}
Bockingen, Germany	Neckar	3 to 5	1 x 10^{-2}
Maribor, Slovenia	Drava	14	2 x 10^{-3} to 4 x 10^{-3}
Karany, Czech Republic	Jizera	8 to 12	4 x 10^{-4}

A parcel of water would travel with seepage velocity through the aquifer soil medium having porosity η. Thus, under convective transportation, the travel time, t_r of a parcel of water could be obtained using Eq. (5.3):

$$t_r = \eta \int_0^{x_w} \frac{dx}{u} = \frac{\eta \pi D}{Q_p x_w} \int_o^{x_w} \left(x_w^2 - x^2 \right) dx = \frac{2\eta \pi D}{3Q_p} x_w^2 \tag{5.3}$$

From Eq. (5.3) it is observed that the travel time is directly proportional to the squared distance, x_w, between the river boundary and pumping well and inversely proportional to the pumping rate, Q_p.

The travel time of a parcel of water t_r obtained from Eq. (5.3) should be more than t_{rr} (i.e. the time duration by which the pathogen concentration in that parcel of water reduces to allowable log cycle reduction). The groundwater extracted from the contaminated water body beyond such travel time will satisfy the drinking water quality standards prescribed by the WHO. However, if the well is located far away from the surface water source, the discharge per unit length would not be adequate to meet the required water demand.

5.3.2 Unsteady Flow Condition: Travel Time of a Parcel of Water

Depending upon the water requirement, a well is often pumped intermittently. This leads to unsteady flow field conditions. To formulate the Theis equation (Theis, 1941), the river is replaced by an imaginary recharge well as shown in Figure 5.2. The shortest distance $x_{w,}$ of the flow line, is along x-axis which joins the pumping well to the river boundary. The drawdown, $S_{(x,t)}$, at time t after the onset of the pump is obtained using Eq. (5.4):

$$s_{(x,t)} = \frac{Q_p}{4\pi T} \int_{\frac{(x_w - x)^2}{4\beta t}}^{\infty} \frac{e^{-u}}{u} du - \frac{Q_p}{4\pi T} \int_{\frac{(x_w + x)^2}{4\beta t}}^{\infty} \frac{e^{-u}}{u} du \tag{5.4}$$

The negative hydraulic gradient along the x-axis is obtained by using Eq. (5.4)

$$\frac{\partial s}{\partial x} = \frac{Q_p}{2\pi T} \left[\frac{1}{(x_w - x)} e^{-\frac{(x_w - x)^2}{4\beta t}} + \frac{1}{(x_w + x)} e^{-\frac{(x_w + x)^2}{4\beta t}} \right] \tag{5.5}$$

and the radial seepage velocity, $v_{(x,t)}$, along the x-axis is obtained using Eq. (5.6).

$$v_{(x,t)} = \frac{k}{\eta} \frac{\partial s}{\partial x} = \frac{Q_p}{2\eta \pi D} \left[\frac{1}{(x_w - x)} e^{-\frac{(x_w - x)^2}{4\beta t}} + \frac{1}{(x_w + x)} e^{-\frac{(x_w + x)^2}{4\beta t}} \right] \tag{5.6}$$

where β is aquifer diffusivity.

Euler's one-dimensional equation of motion is as given in Eq. (5.7).

$$\frac{dv}{dt} = \frac{\partial v}{\partial t} + \frac{\partial v}{\partial x} v \tag{5.7}$$

By applying Euler's equation to Eq. (5.6) and substituting $\frac{\partial v}{\partial t}$, $\frac{\partial v}{\partial x}$ and v in Eq. (5.7), the total acceleration of a parcel of water is derived in the subsequent section. From Eq. (5.6), the radial seepage velocity of a parcel of water at the river boundary i.e., at x=0 and at any time t can be expressed as Eq. (5.8).

$$v_{(0,t)} = \frac{Q_p}{\eta \pi D x_w} e^{-\frac{x_w^2}{4\beta t}} \tag{5.8}$$

At t=0, from Eq. (5.8), $v_{(0,0)}$=0. On differentiating Eq. (5.8), the acceleration of a parcel of water at any time t is obtained as,

$$\frac{dv_{(0,t)}}{dt} = \frac{Q_p}{\eta \pi D} \frac{x_w}{4\beta t^2} e^{-\frac{x_w^2}{4\beta t}} \tag{5.9}$$

At the commencement of pumping, a parcel of water carrying pathogens will start moving from the river boundary towards the well. It will follow the shortest path (i.e., along with the x-axis) with zero acceleration at the river boundary. In such a situation, the travel time of a parcel of water is computed hereafter. The time domain has been discretised with a uniform time step of size Δt. At $t = \Delta t$, the velocity at the river boundary can be obtained using Eq. (5.10)

$$v_{(0,\Delta t)} = \frac{Q_p}{\eta \pi D x_w} e^{-\frac{x_w^2}{4\beta \Delta t}} \tag{5.10}$$

at x=0, during $t = \Delta t$, the average velocity is given by Eq. (5.11),

$$\bar{v}_{(0,\Delta t)} = \frac{Q_p}{2\eta \pi D x_w} e^{-\frac{x_w^2}{4\beta \Delta t}} \tag{5.11}$$

and the average acceleration is further obtained as given in Eq. (5.12).

$$\frac{d\bar{v}_{(0,\Delta t)}}{dt} = \frac{Q_p}{2\eta\pi D}\left(\frac{x_w}{4\beta(\Delta t)^2}\right)e^{-\frac{x_w^2}{4\beta(\Delta t)}} \tag{5.12}$$

The distance travelled by a parcel of water for the first-time step i.e., in time Δt, is given by:

$$\Delta x_{(1)} = \frac{Q_p}{2\eta\pi D x_w} e^{-\frac{x_w^2}{4\beta\Delta t}}\Delta t + \frac{Q_p}{4\eta\pi D}\left(\frac{x_w}{4\beta(\Delta t)^2}\right)e^{-\frac{x_w^2}{4\beta(\Delta t)}}(\Delta t)^2 \tag{5.13}$$

The distance travelled by a parcel of water during the second time step for the velocity $v_{(\Delta x_{(1)},\Delta t)}$ was further analysed, wherein the velocity and associated total acceleration were considered. The velocity $v_{(\Delta x_{(1)},\Delta t)}$ is expressed in Eq. (5.14).

$$v_{(\Delta x_{(1)},\Delta t)} = \frac{Q_p}{2\eta\pi D}\left[\frac{1}{(x_w - \Delta x_{(1)})} e^{-\frac{(x_w-\Delta x_{(1)})^2}{4\beta\Delta t}} + \frac{1}{(x_w + \Delta x_{(1)})} e^{-\frac{(x_w+\Delta x_{(1)})^2}{4\beta\Delta t}}\right] \qquad (.5.14)$$

Therefore, by applying rectilinear motion, the distance travelled during the second- time step is:

$$\Delta x_{(2)} = v_{(\Delta x_{(1)},\Delta t)}(\Delta t) + \frac{1}{2}\frac{dv_{(\Delta x_{(1)},\Delta t)}}{dt}(\Delta t)^2 \tag{5.15}$$

It can be further expressed as $x_{(0)} = 0$; $x_{(\Delta t)} = \Delta x_{(1)}$; $x_{(2\Delta t)} = \Delta x_{(1)} + \Delta x_{(2)}$ and:

$$x_{(n_f\Delta t)} = \sum_{m=0}^{m=n_f}\Delta x_{(m)} = \sum_{m=0}^{m=n_f-1}\Delta x_{(m)} + \Delta x_{(n_f)} \tag{5.16}$$

Here, $x_{(n_f-1)\Delta t}$ locates the position of a parcel of water at a time $(n_f - 1)\Delta t$ and can be expressed as given in Eq. (5.17)

$$\Delta x_{(n_f)} = v_{(x_{(n_f-1)},(n_f-1)\Delta t)}\Delta t + \frac{1}{2}\left[\frac{dv_{(x_{(n_f-1)},(n_f-1)\Delta t)}}{dt}(\Delta t)^2\right] \tag{5.17}$$

When $x_{(n_f)}$ becomes equal to ($x_w - r_w$), $n_f\Delta t$ it represents the first arrival time of a parcel of water from the river boundary carrying pathogen with a reduction in the initial concentration.

5.4 Logistic Function Based Pathogen Transport Model (LIFI-PATRAM)

5.4.1 Population Growth of Pathogen Considering Safe Pumping Rate

Water treatment plants are often not able to remove all the organic and inorganic contaminants. Those fractions of contaminants enter into the water supply system. The formation of biofilm on the inner surface of the water supply pipeline is a safe place to hide for a harmful pathogen such as *E. coli*. If the pathogenic growth is too high, it can break off into the water flow, which at best can make water discoloured or taste unpleasant, and at worst can release more dangerous pathogen. By 6-7 log cycle reduction in riverbank filtration, the pathogen can be controlled in the water supply more effectively (Ramalingam et. al., 2013). Based on a detailed experimental study (Pekdeger, A., et. al., 1985, Yates et. al., 1985), the elimination rate constant and time to reduce the pathogen concentration by 7 log cycle in groundwater at 10°±1°C is given in Table 5.2.

Table 5.2 Elimination rates of the various pathogens for 7 log cycle removal

Pathogen	Survival time (days)	Elimination rate constant (per day)
Escherichia –coli	310	0.0522
Salmonella typhimurium	290	0.0562
Pseudomonas aeruginosa	33	0.484
Bacillus megatherium	6	2.61

As per Table 5.2, the survival time of *E. coli* pathogen is highest, i.e., 310 days in comparison with other pathogens which requires 7 log cycle reductions. The time required for the pathogen to reduce by 7 log cycle is herein termed as survival time. Therefore, the analytical solution was developed in this thesis by specifically targeting *E. coli*, which has the maximum survival time. Groundwater dilution was not considered while developing this model. Dilution influences the result of the log cycle reduction. If the dilution factor is considered, the desired log cycle reduction may need more setback distance because groundwater dilution mostly brings nutrients that feed the growth of pathogens.

The population growth of pathogens (Gershenfeld, 1999; Strogatz, 2001), in a parcel of water, can be expressed as:

$$\frac{dC}{dt} = r\left(1 - \frac{C}{C_0}\right)C - \lambda C \tag{5.18}$$

where C_0 = initial concentration of pathogen in a parcel of water at time t = 0 (number of microorganisms in unit volume of river water observed at river boundary); C = final concentration of pathogens (number of microorganisms in unit volume of water observed at pumping well); r= reproduction rate of a pathogen; and λ = decay rate constant of a pathogen. Assuming r and λ to be time-invariant, integrating Eq. (5.18) will reduce to:

$$\int_{C_0}^{C_{(t)}} \frac{dC}{(r-\lambda)C - r\dfrac{C^2}{C_0}} = \int_0^t dt = t \tag{5.19}$$

Let t_{rr} be the time duration by which the concentration of pathogen reduces from C_0 to $C_0 10^{-n}$. Hence, an expression for t_{rr} can be obtained as:

$$t_{rr} = \int_{C_0 10^{-n}}^{C_0} \frac{dC}{\left(\dfrac{r}{C_0}\right)C^2 + (\lambda - r)C} \tag{5.20}$$

Eq. (5.20) can be further simplified (Abramowitz, M., Stegun 1970, p.12), to obtain the expression for t_{rr}:

$$t_{rr} = \frac{2.303}{(\lambda - r)} \log_{10}\left\{\frac{r + \lambda 10^n - r10^n}{\lambda}\right\} \cong \frac{2.303}{(\lambda - r)}\left\{n + \log_{10}\left(\frac{\lambda - r}{\lambda}\right)\right\} \tag{5.21}$$

Furthermore, it is necessary that the travel time of a parcel of water $t_r = t_{rr} + \varepsilon$, where ε is very small time. Thus, substituting the appropriate terms in Eq. (5.3), the limiting (safe) pumping rate for the well located at distance x_w from the river can be rewritten as:

$$Q_p = \frac{2\eta\pi D}{3(t_{rr} + \varepsilon)} x_w^2 \cong \frac{2\eta\pi D}{3}\left[\frac{(\lambda - r)}{2.303\{n + \log_{10}((\lambda - r)/\lambda)\}}\right] x_w^2 \tag{5.22}$$

Thus, the travel time by which the concentration reduces to $C_0 10^{-n}$ is ascertained for the corresponding log cycle reduction n with respect to the river water concentration C_0.

5.4.2 Pathogen Transport in Groundwater Considering Dispersion and Decay

The model of Harvey and Garabedian (1991), which includes hydrodynamic dispersion, filtration, first-order decay of pathogen, and retardation for the solute transport in saturated, homogeneous porous media is expressed as:

$$R\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left(\alpha x_w v \frac{\partial C}{\partial x}\right) - v\frac{\partial C}{\partial x} - (\mu_l + \mu_s(R-1) + k_{att})C \tag{5.23}$$

where C = pathogen concentration at any time t; R= retardation factor; υ =average pore-water velocity; α = longitudinal dispersivity; μ_l = first order die-off rate for the free pathogen; μ_s = first order die-off rate for sorbed pathogen; and k_{att}= first order filtration rate coefficient. Eq. (5.23) can be further simplified as follows:

$$R\frac{\partial C}{\partial t} = D_h\frac{\partial^2 C}{\partial x^2} - v\frac{\partial C}{\partial x} - \lambda C \tag{5.24}$$

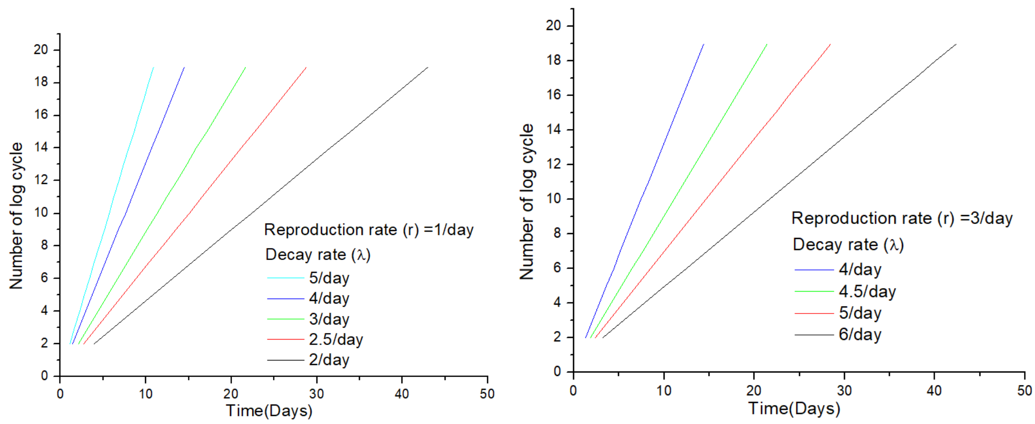
where D_h=hydrodynamic dispersion; and λ= first order total removal rate. In this case, the one-dimensional dispersion is assumed to be applied along the shortest flow path as shown in Figure 5.2. Equation (5.24) has the same form as that of the common convection-dispersion equation. The only difference between Equation (5.23) and Equation (5.24) is that the variable hydrodynamic dispersion coefficient, α, x_w, and v in Equation (5.23), has been replaced with the constant hydrodynamic dispersion coefficient D_h in Equation (5.24). The distance between the pumping well and river is discretized to a uniform grid size of Δx to formulate an explicit finite difference scheme. The grid size of Δx is so chosen such that the Péclet number, $P = (\Delta x v_{(i)}/D_h) \geq 4$. In case othe of the solute transport, if Péclet number is greater than 4, the concentration-time graph simulated by a Hybrid Cells in Series model matches with Ogata and Bank's solution (Ogata and Banks 1961). The hydrodynamic dispersion coefficient is taken as $D_{h_{(i)}} = \alpha\, v_{(i)}$. In this case, a grid size $\Delta x = 1$ m was considered and dispersivity of 0.2 m was assumed to find $C_{(x,t)}$. The concentration at the i^{th} node and j^{th} time step using an explicit finite difference method can be derived as:

$$C_{(i,j+1)} = C_{(i,j)} + \frac{\Delta t}{R}\left\{ D_{h\,(i)} \frac{C_{(i-1,j)} - 2C_{(i,j)} + C_{(i+1,j)}}{\Delta x^2} - v_{(i)} \frac{C_{(i+1,j)} - C_{(i-1,j)}}{2\Delta x} - \lambda C_{(i,j)} \right\} \tag{5.25}$$

The time step of size Δt is chosen. The velocity $v_{(i)}$ is computed for $x=i\Delta x$ and time $t=j\Delta t$. Because the velocity is infinite at the well point, Eq. (5.25) can be applied to the domain $0 \leq x \leq (x_w - r_w)$. The domain can also be expressed as $0 \leq i \leq (i_w - 1)$, where $i_w = x_w / \Delta x$. The boundary condition to be satisfied at the river aquifer interface is $C_{(0,j)} = C_0$. The boundary conditions that is convenient to be satisfied at the abstraction point is $C_{((i_w-1),j)} = C_{((i_w-3),j)}$. Numerical results are presented in this thesis by applying an explicit finite difference method which evaluates the safe distance of a pumping well.

5.5 Results and Discussion

By using the developed model called LIFI-PATRAM, pathogen concentration with respect to the distance measured from river boundary was obtained. A computer code was developed in the MATLAB_R2016b software package (Math Works 2016). The code couples the AEM model and developed LIFI-PATRAM. To test the performance of the model based on logistic function, it was analyzed over time laps with the various sets of reproduction rate (r) and decay rate (λ) of the pathogen. The variation of λ varies from 2/day to 6/ day and r from 1/day to 3/day was considered to test the behaviour. It was found that the concentration of pathogen during the time t_{rr} reduced to $C_o 10^{-n}$ over n log cycle. The variation is presented in Figure 5.4 (a and b) for the decay and reproduction rates.



(a) (b)

Figure 5.4 Time for different log cycles reduction in pathogen concentration for various reproduction and decay rates.

As per Eq. (5.22), the variation of t_{rr} with n log cycle follows the same nature as that of the growth curve phase (log phase) of *E. coli* pathogen in a closed system (Wang et. al., 2015).

5.5.1 Selection of flow condition

The travel time under steady and unsteady flow condition was calculated for a pumping well located at a distance from the river boundary i.e., x_w ranging from 25 to 100 m, the pumping rate Q_p ranging from 300 to 1000 m^3/day, thickness of the aquifer $D = 62.5\ m$, transmissivity $T = 500\ m^2/day$, $k = 8\ m/day$ and $\eta = 0.3$. Corresponding to Q_p and x_w, the co-relation between drawdown at the well point and related travel time were computed and is presented in Table 5.3.

Table 5.3 Travel time and drawdown at well under steady and unsteady flow field.

x_w *(m)*	Q_p *(m^3/day)*	*Drawdown* $s_{(x_w - r_w)}$ *(m)*	*Travel time* t_r (days) *Steady State*	*Unsteady state*
100	300	0.73	1309.95	1313.50
100	500	1.21	785.37	789.00
100	1000	2.42	392.68	395.25
50	300	0.66	327.24	327.50
50	500	1.10	196.34	196.50
50	1000	2.20	98.17	98.50
25	300	0.59	81.81	82.25
25	500	0.99	49.09	49.25
25	1000	1.98	24.54	24.75

A comparison of the travel time with respect to the drawdown reveals that the travel time of a parcel of water from the river boundary to the pumping well computed under unsteady flow condition is slightly higher than that computed under steady flow. This is because, in the case of unsteady state flow field condition, when pumping commences, the water is first taken from the aquifer storage and the contribution of the river takes place later in time. The results obtained for the unsteady state would further converge to the results that are obtained for the steady flow condition. Therefore, this analysis shows that the equilibrium radial flow i.e., steady flow condition can be confidently applied to find the safe distance and safe withdrawal from a pumping well. The dilution effect from the aquifer is not considered therefore, we are using the steady state condition for the evaluation point of view. From Table 5.3, it can be seen that the maximum drawdown, *s*, at the pumping well corresponding to various pumping rates is less than 15% of the aquifer thickness *D*. Therefore, the pumping rate Q_p=1000m^3/day can be safely withdrawn. The water entrance velocity from the river to the pumping well was found to be very large at the well boundary, whereas velocity was found to be near zero at the edge of the cone of depression.

Table 5.4 velocity of water with respect to distance from the river

Distance from river	Velocity (m/day)
0	0.14101818
5	0.1413773
15	0.14431641
20	0.14698855
25	0.15057069
30	0.15518935
35	0.16102088
40	0.16831012
45	0.17739975
50	0.18877753
55	0.20315551

60	0.22160888
65	0.24583412
70	0.27866556
75	0.32521205
80	0.39569891
85	0.51399396
90	0.75171205
95	1.46694747
96	1.82485967
97	2.42150621
98	3.61498566
99	7.19579111
99.5	14.3576739
99.55	15.9492144
99.6	17.9386422
99.65	20.4964806
99.7	23.9069347
99.75	28.6815741
99.8	35.8435376
99.85	47.7801494
99.9	71.653382
99.95 (well boundary)	143.273098

It is clearly demonstrated in Figure 5.5 that the values of the approach velocity at the river bank represent similar conditions as that of slow sand filters which generally have a filtration rate of 2 to 5 m/day (Ellis, 1985).

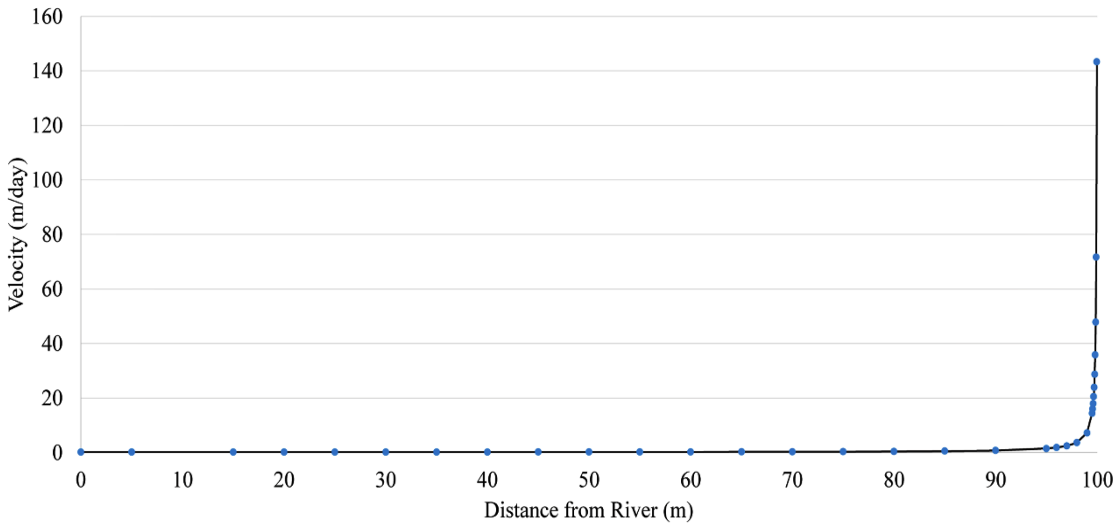


Figure 5.5 Velocity flow field from the river to the pumping well.

5.6 Model Application

The developed LIFI-PATRAM model was applied to forecast the safe distance of the pumping well located near the river carrying a pathogen. It was required to find the safe and continuous withdrawal from a pumping well with diameter r_w =0.1 m which was located at a distance x_W= 100m from the river boundary. The riverbank aquifer parameters were taken as k= 10 m/day; D=30 m, and porosity η=0.2. *E. Coli* pathogen was present in the river water. The survival time of *E. Coli*, corresponding to 7 log cycle reduction is 310 days and the decay (elimination or inactivation) rate $\lambda =$ 0.0522 /day (Pekdeger et. al., 1985).

Table 5.5 Site data for nine RBF systems in the United States and Germany (Ray et. al., 2002)

Items	SC WA, California	Boardman, Oregon	Lincoln, Nebraska	Cedar Rapids, Iowa	Louisville, Kentucky	Somersnorth, New Hampshire	Dresden-Tolkewitz, Germany	Meissen Siebeneichen, Germany	Torgau-Ost, Germany
Wells									
Number/Type	5H, 7V	2H	2H, 44V	2H, 53V	1H	2V+1V	71V	3V	42V
Maximum Capacity (m^3/d)	322,000	87,000	132,000	128,500	76,000	5,300	40,000	6,000	150,000
Screen Zone Below Land Surface (m)	24 to 30	15 to 15.6	12 to 18	18 to 24	24 to 30	12 to 16.5	15 to 19	12 to 17	32 to 52
Screen Zone (m)	6	0.6	6	6	6	4.5	4	5	20
Distance to River (m)	0 to 75	3 to 18	<30 to >800	9 to 245	<30 to 84	46	80 to 180	100 to 150	300
Travel Time (days)	4.9		<7 to >14	2 to 17	2 to 5	<55	25 to 50	50 to 100	80 to 300
River									

Discharge (m^3/s)	<2.8 to >1,40C	6,370 to	<50 to	4 to	6,300 to >28,000	N	120 to	120 to 2,000	120 to
Width/Depth	15 to 90/NA	4,000/3	300	225/2.5 to 3	600/10	12/N	120/2	140/2	130/2
Bed Sediments	Sand	Sand/Silt	San	Sand	Sand	Gravel	Coarse	Gravel	Gravel
Aquifer									
Type	Unconfined	Unconfined, Some Leaky	Unconfined to Leaky	Unconfined to Confined	Unconfined	Leaky	Unconfined	Unconfined	Unconfined
K (m/s)	2.4×10^{-4} to 4.3×10^{-4}	3.7×10^{-3}	1.4×10^{-3}	1.5×10^{-4} to 1.1×10^{-3}	6×10^{-1}	4.3×10^{-4}	1×10^{-3} to 2×10^{-3}	1×10^{-3} to 2×10^{-3}	6×10^{4} to 2×10^{-3}
Thickness (m)	8 to 26	15	21 to 26	15 to 20	0 to 40	15	10 to 13	15 to 20	40 to 55
Specific Yield (%)	NA	NA	15	10	NA	NA	20	20	20
Material	Sand and Gravel	Sand and Gravel	Sand and Gravel	Fine to Medium Sand on Coarse Gravel	Sand and Gravel with Silt and Clay	Sand with Some Gravel	Sand and Gravel	Medium and Coarse Sand	Medium and Coarse Sand
Heterogeneity	NA	Homogeneous	Few Clay Lenses	Silty Clay Lenses	Coarse Gravel and Pebbles	Fine Silt Lenses	Homogeneous	Few Fine Sand Lenses	Few Silt Lenses

Note: V= vertical well; H= Horizontal collector well; NA = Not available

Various aquifer parameters were selected taking reference of Table 5.5. for the simulation of LIFI-PATRM model.

Computing the reproduction rate $r = 0.00021$ /day from Eq. (5.21), the time required for 7 log cycle reduction concentration is t_{rr}=310 days. Assuming t_r=t_{rr} from the Eq. (5.22), the safe pumping rate is Q_p=405 m^3/day. The variation of log cycle reduction with respect to the distance from the river boundary for various travel time of a parcel of water after the onset of pumping is represented by Figure 5.6.

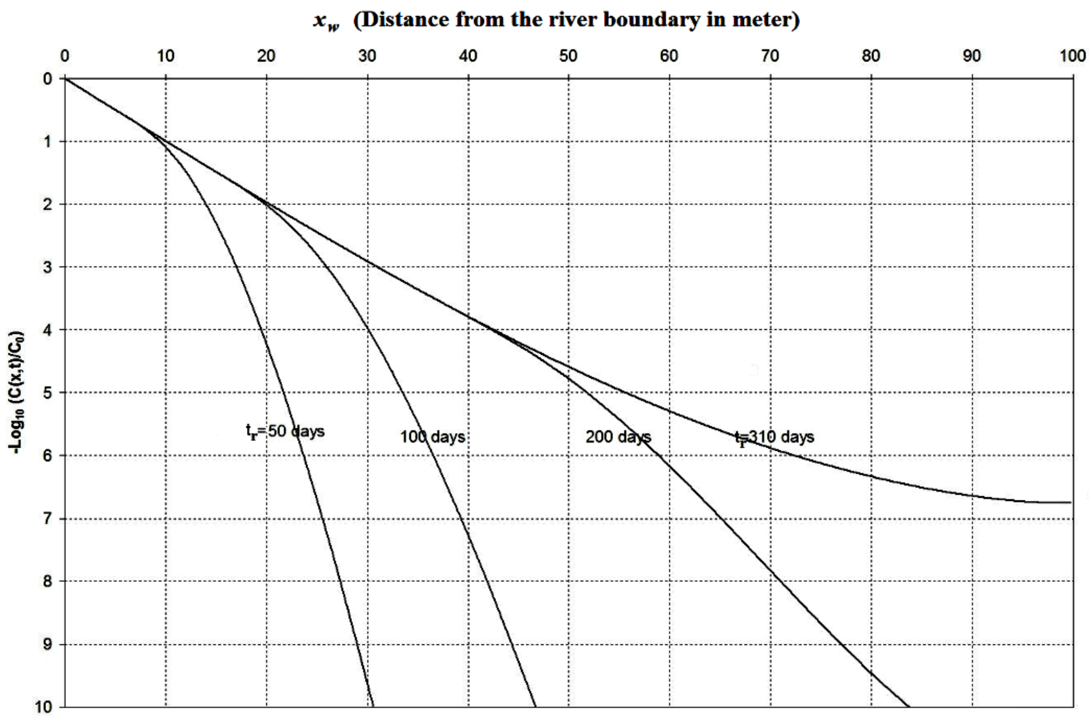


Figure 5.6 Variation of -Log10 C(x,t)/C0 with distance from the river boundary for various travel time of a parcel of water.

The linear part of the graph as shown in Figure 5.6 indicates that the dynamic equilibrium condition in the specific zone has been attained with respect to the solute transport which helps decide the safe distance of the pumping point away from the pathogen-bearing river. The concentration was found to be decreasing with the distance due to the decay and dispersion. Corresponding to the pumping rate $Q_P = 405$ m^3/day and the distance of the well x_W =100 m, at the end of 310 days of the travel time of a parcel of water, 6.7 log cycle reduction could be achieved.

The concentration of pathogen while passing through the aquifer medium has apparently declined with distance. This has important consequences for the prediction of contaminant removal, thus, for the calculation of effective setback distances to protect groundwater sources. This also assures, adequate treatment required for the infiltrated groundwater received at pumping well. Various field studies and laboratory experiments have also confirmed the safe distance considering pathogenic transport (Pang et. al., 2004; Prasad et. al., 2016; Schijven et. al., 2000; Wang 2003). The analytical findings are similar to Rice et. al., (1999) and Pang et. al., (2000). In comparison to other field and analytic studies, proposed solution, which is based on logistic function, is user-friendly.

5.7 Summary

In this study, the findings of analysis were reported to find a suitable location of a pumping well in a riverbank filtration facility. For this analysis, we developed an analytical method that we called LIFI-PATRAM. The model is based on the explicit finite difference scheme to estimate the decay of pathogen contamination during .riverbank filtration. Using LIFI-PATRAM, the log cycle reduction of *E. coli* pathogen was analysed. The travel time of the water was computed by using a hydrodynamic equation and was coupled with LIFI-PATRAM. The analysis was performed to find the optimal pumping rate to achieve the targeted log cycle reduction at the abstraction well. Based on the river water quality, the maximum and the minimum discharge limits of a pumping well were influential parameters and had to be selected properly.

The yield and distance of the pumping well were governed by the logistic function and therefore the developed model can be effectively applied to compute the safe yield and/or safe distance to achieve the desired log cycle reduction in pathogen concentration. Steady flow/analysis could be confidently applied to find the safe distance and safe withdrawal from an existing well. In the analysis, it was observed that under a dynamic equilibrium condition, the variation of log cycle reduction of the pathogen was linear with respect to distance, which helped to decide the appropriate location of the pumping well in the planning of riverbank filtration projects for an effective pathogen reduction. More importantly, the AEM-based LIFI-PATRAM model could find out the optimal location of a pumping well and could, therefore, be applied to various field problems by incorporating the different properties of the aquifer and source water quality for sustainable management of RBF.

Chapter 6 : Demand Satisfaction and Attenuation of Pathogen

6.1 Introduction

The biggest challenge of the 21st century is water security for every individual. As the population is growing at a fast pace, fulfilling the water demands, specifically for drinking, is a matter of urgent concern. To cope up with the drinking water requirement, new types of technologies were discovered in the twentieth century including water desalination, atmospheric water abstraction, artificial rains, wastewater treatment, etc. (Singh 2017). Groundwater abstraction is an old technique known since ancient civilizations. Nowadays, new ways of trapping groundwater resource (in RBF) have been developed which have a vulnerability to receive polluted stream water.

Researchers have confirmed and emphasised that increasing human interference with watershed and industrial growth tend to pollute clean river water (Gavrilescu et. al., 2015; Islam et. al., 2015; Tsihrintzis 2013). Aquifer characteristics are also changing due to climate change and land use practices (Prasad et. al., 2016) and disposal of human effluents and sludge is a major source of toxic and hazardous pollution in surface water and groundwater. Many waterborne disease outbreaks are caused by the consumption of groundwater contaminated by microbial pathogens (Beller et. al., 1997). The situation becomes more serious during the rainy season because all the watershed contaminants are drained to the stream (Philadelphia Water Department, 2003).

One solution is setting up a pumping well or infiltration gallery in the vicinity of the surface water body to take advantage of the river's bank filtration because a bank filtration facilitates the removal of impurities such as microbial contaminants, organic and inorganic chemicals, turbidity, etc. (Prasad et. al., 2016; Ray et. al., 2002). When it comes to meet the increasing demand of a growing population of a town, infiltration galleries are more powerful means to fulfil the water requirement. In shallow thin aquifer near a stream, an infiltration gallery is generally installed for tapping potable groundwater taking advantages of stream bank filtration (Ray et. al., 2002). If the stream water does not satisfy the water quality standard (WHO, 2008), a suitable location needs to be ascertained to trap the pollutant-free water for drinking. The influence of pathogens will be low if the infiltration gallery is placed some distance away from the stream. During the lean flow period, the stream receives water from the aquifer. However, during the passage of a flood wave, the property of the water entering the gallery is affected by stream water quality.

It is reported in various studies that the stream water quality is vulnerable during the rainy season (see Figure 6.1). This is due to the runoff contribution from the watershed catchment area which comprises agricultural fields and landfills for waste disposal (Tornevi et. al., 2014). Therefore, designers should take care of all these aspects prior to the construction activity for effective attenuation of the pathogen shock load.

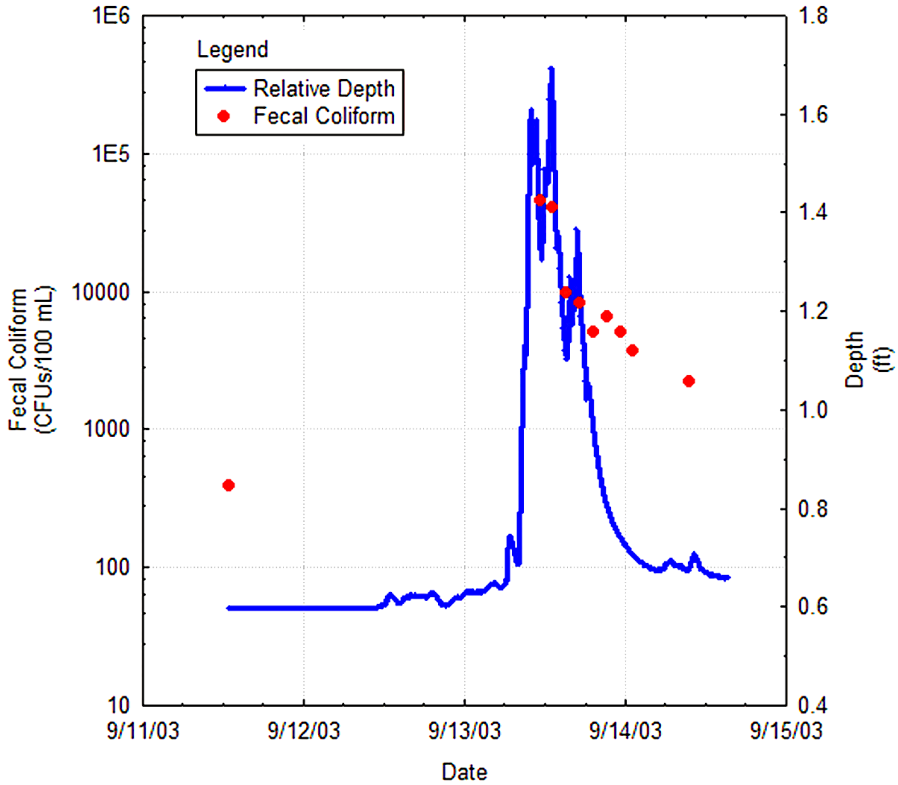


Figure 6.1 Water quality changes in a stream during a storm (Source: Phillyrivercast .org 2018)

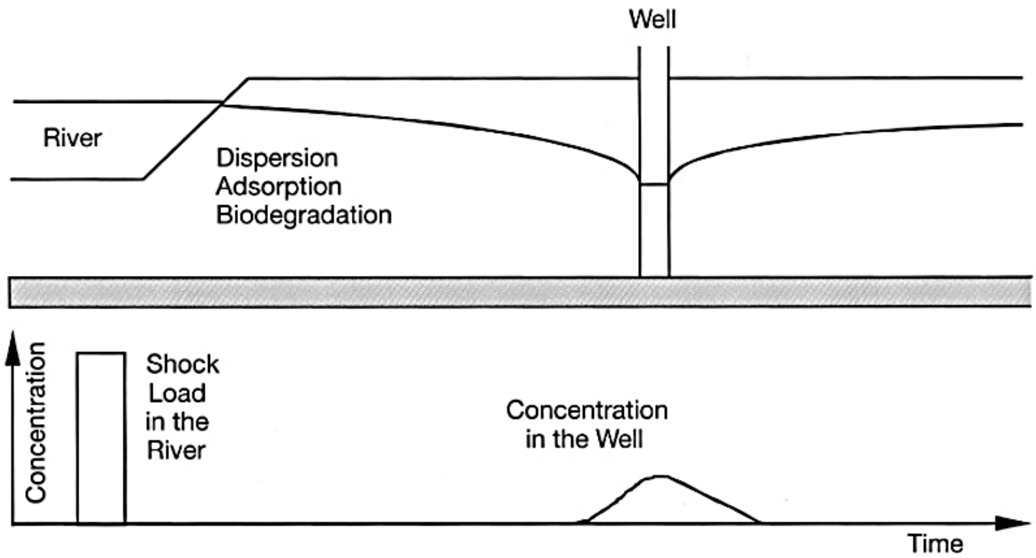


Figure 6.2 Delay and attenuation of shock load in the river by RBF (Malzer et. al., 2002)

The biological contaminants in the surface water body would enter the infiltration gallery if it is located near the water body in very close proximity, thus leading to high treatment cost and ultimately affecting the health and hygiene (Pang 2009; Verstraeten et. al., 2003). If the gallery is located far from the surface water source, the specific discharge would decrease and influence the drinking water demand. Some literature reviews show that there is a specific distance between the water abstraction location and the stream (Ahmed and Marhaba 2016; Dalai and Jha 2014; Pang 2009; Schijven and Hassanizadeh 2000). This specific distance of an infiltration gallery causes a reduction in the concentration of microbial pathogens by several orders of magnitude due to the dispersion and decay when the infiltrated water passes from the riverbank soil media (Pekdeger, A., et. al., 1985; Madema et. al., 2003). Several pathogen inactivation models under transport condition have been developed considering dispersion and decay for assessment of water quality (Harvey and Garabedian 1991; Chrysikopoulos and Sim 1996; Schijven and Hassanizadeh 2000; Pang 2009).

Numerical and analytical approaches are used to solve several problems arisen in RBF design. However, analytical solutions are more useful in quick estimation of concentrations, the simplicity of input parameters, and the absence of numerical errors. Recently, many mathematical modelling studies consider horizontal wells (infiltration galleries can also be referred to as horizontal wells) due to the larger supply capacity and suitability for installing in the shallow aquifer. Hantush and Papadopulos (1962) presented analytical solutions to simulate drawdown distribution. In order to approximate the drawdown, another analytical solution of flow to collector well was developed by Debrine (1970) proposing a uniform flux distribution along the laterals.

The results of this model study were preferred to that of the solutions of Hantush and Papadopulos (1962). Huisman and Olsthoorn (1983) have analyzed steady flow to a drainage gallery by drawing flow nets. Hunt (1983) applied Schwarz Christoffel conformal mapping technique. Zhan and Park (2003) have assumed uniform flux distribution along the horizontal well axis for solving unsteady flow to the well under various aquifer conditions. Weiss (2005) conducted monitoring for microbial study at RBF facilities located in The United State. Results of this numerical model demonstrated the potential for RBF to provide substantial reductions in microorganism concentration relative to the river water. In another study (Mohammed and Rushton, 2006), the flow in a shallow aquifer was analyzed to test the efficiency of the riverbank filtration to remove microorganisms. Anderson (2013) developed an explicit analytic solution to represent a 2D groundwater flow to horizontal well in an unconfined aquifer in a riverbank system. Generally, it is difficult to simulate the drawdown of water inside collector wells analytically due to its complex radial flow (Banerjee, 2011). Considering this difficulty, Shende and Chau (2018) developed an analytical solution to forecast the safe distance of a pumping well by simulating hydraulic head.

In this study, the logistic differential equation (Verhulst, 1838) was used (also referred to as a logistic model / inhibited growth model / sigmoidal function) to develop a decision tool to ascertain the optimum location of an infiltration gallery; specifically, to attenuate the pathogen shock load. The logistic model has been applied in several studies which focused on the population growth in the ecosystem (Shende and Chau 2018; Chang et. al., 2016; Gershenfeld 1999; Pruitt and Kamau 1993). Shende and Chau (2018) developed a logistic function-based pathogen transport a model called LIFI-PATRAM to forecast a safe distance of a pumping well. In this study, an analytical element method (AEM) based flow model was developed and coupled with the LIFI-PATRAM for the placement of an infiltration gallery. The travel time of a drop of water to reach the gallery during a step rise in the stream stage was analyzed and it was clearly demonstrated that the logistic function can be conveniently used to attenuate the pathogen shock load. This study presents the analytical deduction of the formulae to forecast the safe distance of an infiltration gallery considering shock loads during storms. The model application has been demonstrated with one theoretical example to verify the mathematical exercise. Results are presented by applying an explicit finite difference scheme, the method which evaluates the safe distance of an infiltration gallery to attenuate pathogens during stream sage rise.

6.2 Methodology

In this study, a groundwater flow model was developed using the analytical element method. The logistic function has been used to locate an infiltration gallery. The flow field was considered corresponding to the location of an infiltration gallery 'x_w' with respect to various aquifer parameters i.e. porosity, hydraulic conductivity, depth of aquifer, the transmissivity of the aquifer, aquifer diffusivity, etc. and pathogen's property of dispersion, reproduction and decay (see Figure 6.3).

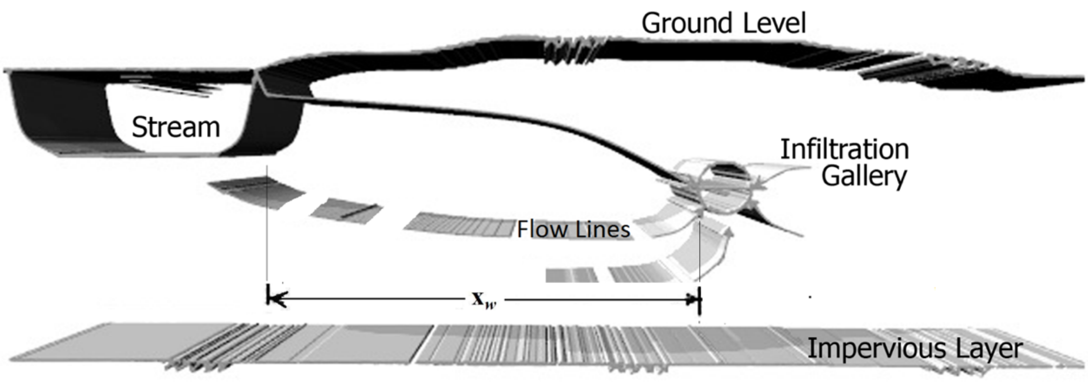


Figure 6.3 The hypothetical layout of an infiltration gallery in RBF schemes

The concentration of pathogens in the stream increases during the storm and the allowable concentration into the infiltration gallery was chosen as per drinking water quality standards (see Figure 6.1) with reference to the stream water quality. From the specific values of decay rate and reproduction rate, the required number of log cycle reduction 'n' in the concentration of pathogen was determined. For the required log cycle reduction 'n' with respect to stream water, the corresponding travel time t_r by which the concentration reduces from C_0 to $C_0 10^{-n}$ was ascertained. Based on the assessment of the travel time 't_r' corresponding to the decrease in the initial concentration and log cycle reduction, the safe distance of an infiltration gallery 'x_w' was computed.

6.2.1 Statement of the Problem

A single infiltration gallery is to be installed at a distance of 'x_w' from a fully penetrating stream (see Figure 6.3). The unconfined aquifer adjoining the stream has a transmissivity of T (m^2/day) and diffusivity of β (m^2/day). It is aimed to find the safe distance of an infiltration gallery during storm with unsteady flow condition. Before the stream stage rise, the aquifer was at rest condition (stable water level). A step rise in stream σ (m), takes place at time t=0. It is also required to find the corresponding travel time of a drop of water to reach the proposed location of an infiltration gallery to achieve the objective of safe drinking water.

6.2.2 Model Formulation

AEM based flow model was used in this study. AEM is a computational method based on the superposition of analytical expression to represent two-dimensional vector fields. It can superimpose various analytic solutions to solve the groundwater flow problems and is capable of simulating streams, lakes, and complex boundary conditions (Strack 1989). In addition, AEM flow solutions are inherently continuous over the flow domain and give a more accurate water budget for the area (Majumder and Eldho, 2015; Matott et. al., 2006).

A stable-and-convergent analytical groundwater model is developed herein using an explicit finite difference scheme. A significant feature of this model is its simplicity, which makes it attractive and useful for a wide variety of applications, including water quality modelling in riverbank filtration. The choice of an explicit model over the implicit model is preferred in this case. Explicit models are subjected to a strict stability criterion which must be satisfied if the model is to simulate natural conditions in a realistic way (Douglas Jr., 1956; O'Brien et. al., 1949). Even though it is well known that implicit models are unconditionally stable, they are subjected to a convergence criterion which effectively places an upper limit on the time step. Examples of the lack of convergence in water and sediment-transport modelling have been documented by Ponce et. al., (1978; 1979). Therefore, when additional complexity is introduced, implicit models may not be necessarily better than explicit models. Here, the emphasis is on convergence, i.e., whether the proposed model can reproduce the differential equation with sufficient accuracy. Explicit models have fast convergence and time accuracy with less computation time, therefore it is preferred to use in comparison with the implicit model.

6.3 AEM Flow Model

6.3.1 Stream Stage Rise

The infiltrated water to an infiltration gallery becomes more vulnerable for drinking during storm events. The concentration of pathogen is several times more during a storm than that of the steady-state condition of a stream in a normal situation (without flooding events). In this scenario, due to a higher concentration of pathogens in the stream, the distance of an infiltration gallery must be greater than the condition where the stream and the gallery are in equilibrium condition i.e. steady state condition. Here, the travel time of a drop of water was computed considering the earliest arrival of the pathogen from the stream boundary to an infiltration gallery. The stream is assumed to be fully penetrating and clogging of its bed is ignored. Stream stage rise is assumed to be less than 1/5 of the aquifer thickness. To maintain linearity, stream stage rise is assumed to be small relative to aquifer thickness.

Consequent to the stream stage rise, the rise in water level at distance 'x' at a time 't' after the onset of stream stage is given by (Knight and Rassam, 2007):

$$h_{(x,t)} = \sigma\left[1 - erf\left(\frac{x}{\sqrt{4\beta t}}\right)\right] \tag{6.1}$$

Where; β= aquifer diffusivity, σ= rise in stream stage, $erf(X) = \frac{2}{\sqrt{\pi}}\int_0^X e^{-\zeta^2}\, d\zeta$ is the error function whose properties are given by Carslaw and Jaeger (1959, pp. 482-484).

Differentiating $h_{(x,\,t)}$ with respect to x:

$$\frac{\partial h}{\partial x} = -\frac{\sigma}{\sqrt{\pi\beta t}}(e)^{\frac{-x^2}{4\beta t}} \tag{6.2}$$

The Darcy velocity at x is given by:

$$u_{(x,t)} = -k\frac{\partial h}{\partial x} = \frac{k\sigma}{\sqrt{\pi\beta t}} e^{\left(\frac{-x^2}{4\beta t}\right)} \tag{6.3}$$

Dividing the Darcy velocity $u_{(x,t)}$ by porosity η; the true velocity at x is:

$$v_{(x,t)} = \frac{k\sigma}{\eta\sqrt{\pi\beta t}} e^{\left(\frac{-x^2}{4\beta t}\right)} \tag{6.4}$$

The local acceleration is given by:

$$\frac{\partial v_{(x,t)}}{\partial t} = \left(\frac{x^2}{4\beta t^2}\right)\frac{k\sigma}{\eta\sqrt{\pi\beta t}} e^{\left(\frac{-x^2}{4\beta t}\right)} \tag{6.5}$$

Differentiating $v_{(x,t)}$ with respect to x:

$$\frac{\partial v}{\partial x} = \left(\frac{x}{2\beta t}\right)\left(\frac{-k\sigma}{\eta\sqrt{\pi\beta t}}\right) e^{\left(\frac{-x^2}{4\beta t}\right)} \tag{6.6}$$

Applying Euler's equation of motion:

$$\frac{dv}{dt} = \frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x} \tag{6.7}$$

Incorporating $v_{(x,t)}$, $\frac{\partial v}{\partial t}, \frac{\partial v}{\partial x}$ into Equation (6.7); the total acceleration of a drop of water at x and time t; during stream stage rise is derived as:

$$\frac{dv_{(x,t)}}{dt} = \left(\frac{x^2}{4\beta t^2}\right)\left(\frac{k\sigma}{\eta\sqrt{\pi\beta t}}\right) e^{\left(\frac{-x^2}{4\beta t}\right)} - \left(\frac{x}{2\beta t}\right)\left(\frac{k\sigma}{\eta\sqrt{\pi\beta t}}\right)^2 e^{\left(\frac{-x^2}{2\beta t}\right)} \tag{6.8}$$

6.3.2 Computation of Travel Time

The travel time of a drop of water has been computed considering the earliest arrival of the pathogen from the river boundary to an infiltration gallery. Let the time domain be discretised with uniform time steps each of size Δt. The distance $\Delta x_{(1)}$ travelled by a drop of water during Δt is derived as follows. The entrance velocity at the stream aquifer interface is:

$$v_{(0,t)} = \frac{k\sigma}{\eta\sqrt{\pi\beta}}\left(\frac{1}{\sqrt{t}}\right) \tag{6.9}$$

Where; β is aquifer diffusivity and k is hydraulic conductivity. The velocity at the river boundary is infinite at $t = 0$. At $t = \Delta t$, the average velocity during the first-time step is derived using the relation:

$$\bar{v}_{(0,\Delta t)} = \frac{1}{\Delta t}\int_0^{\Delta t} v_{(0,\Delta t)}\,dt = \frac{2k\sigma}{\eta\sqrt{\pi\beta\Delta t}} \tag{6.10}$$

Since at $x = 0; \frac{\partial v}{\partial x} = 0$; the acceleration is zero at t=0.

Hence, the distance $\Delta x_{(1)}$ traversed by a drop of water during the first-time step i.e. in time Δt is given by:

$$\Delta x_{(1)} = \frac{2k\sigma}{\eta\sqrt{\pi\beta}}\left(\sqrt{\Delta t}\right) \tag{6.11}$$

The velocity at $\Delta x_{(1)}$ and time Δt is:

$$v_{(\Delta x_{(1)},\Delta t)} = \frac{k\sigma}{\eta\sqrt{\pi\beta\Delta t}} e^{\frac{-(\Delta x)^2}{4\beta\Delta t}} \tag{6.12}$$

The total acceleration of a drop of water at $\Delta x_{(1)}$ and time Δt is:

$$\frac{dv}{dt} = \left(\frac{k\sigma}{\eta\sqrt{\pi\beta\Delta t}}\right)(e)^{\frac{-(\Delta x)^2}{4\beta\Delta t}}\left(\frac{(\Delta x)^2}{4\beta(\Delta t)^2}\right) - \left(\frac{k\sigma}{\eta\sqrt{\pi\beta\Delta t}}\right)^2(e)^{\frac{-(\Delta x)^2}{2\beta\Delta t}}\left(\frac{\Delta x}{2\beta\Delta t}\right) \qquad (6.13)$$

Therefore; by applying rectilinear motion, the distance traversed during the second-time step for the velocity of $v_{(\Delta x_{(1)},\Delta t)}$ is:

$$\Delta x(2) = \frac{k\sigma}{\eta\sqrt{\pi\beta\Delta t}} e^{\frac{-(\Delta x)^2}{4\beta\Delta t}} \Delta t + \frac{1}{2}\left\{\left(\frac{k\sigma}{\eta\sqrt{\pi\beta\Delta t}}\right) e^{\frac{-(\Delta x)^2)}{4\beta\Delta t}}\left(\frac{(\Delta x)^2}{4\beta(\Delta t)^2}\right) - \left(\frac{\Delta x}{2\beta\Delta t}\right)\left(\frac{k\sigma}{\eta\sqrt{\pi\beta\Delta t}}\right)^2 e^{\frac{-(\Delta x)^2}{2\beta\Delta t}}\right\}(\Delta t)^2 \qquad (6.14)$$

The distance travelled by the drop of water can be further expressed as:

$x_{(0)} = 0$; $x_{(\Delta t)} = \Delta x_{(1)}$; $x_{(2\Delta t)} = \Delta x_{(1)} + \Delta x_{(2)}$ and

$$x_{(n_f\Delta t)} = \sum_{m=0}^{m=n_f} \Delta x_{(m)} = \sum_{m=0}^{m=n_f-1} \Delta x_{(m)} + \Delta x_{(n_f)} \qquad (6.15)$$

Here,$x_{(n_f-1)\Delta t}$ locates the position of a drop of water at a time $(n_f - 1)\Delta t$ and can be expressed as:

$$\Delta x_{(n_f)} = v_{\left(x_{(n_f-1),(n_f-1)\Delta t}\right)}\Delta t + \frac{1}{2}\left[\frac{dv\left(x_{(n_f-1),(n_f-1)\Delta t}\right)}{dt}(\Delta t)^2\right] \qquad (6.16)$$

When $x_{(n_f)}$ becomes equal to x_w, then, $n_f\Delta t$ will be the first arrival time of a drop of water from the river boundary carrying a pathogen with reductia on in initial concentration. At this distance x_w, water that has entered from the contaminated stream during stage rise will satisfy the drinking water quality standards.

6.4 Logistic Function-Based Pathogen Transport Model

6.4.1 Population Growth of Pathogen using Logistic Function

Based on a detailed experimental study (Pekdeger, A., et. al., 1985, Yates et. al., 1985), the elimination rate is constant and the time to reduce the pathogen concentration by 7 log cycle in groundwater at 10°±1°C is given in Table 5.2. With reference to Table 5.2, the survival time of *E.coli* pathogen is the highest, i.e., 310 days in comparison with other pathogens. The time required for the pathogen to reduce by 7 log cycle is herein termed as survival time. Therefore, the analytic solution was used in this thesis by specifically targeting *E-coli* which has maximum survival time. Groundwater dilution was not considered while developing this model. Dilution influences the result of log cycle reduction. If the dilution factor is considered, the desired log cycle reduction may need more setback distance as groundwater dilution mostly brings nutrients to feed for the growth of the pathogen.

Population growth of the pathogen (Gershenfeld 1999; Strogatz 2001), in a drop of water, can be expressed as:

$$\frac{dC}{dt} = r\left(1 - \frac{C}{C_0}\right)C - \lambda C \quad (6.17)$$

Where, C_0 = initial concentration of pathogen in a drop of water at time t = 0 (number of microorganisms in unit volume of river water observed at river boundary), C = final concentration of the pathogen (number of microorganisms in unit volume of river water observed at the infiltration gallery), r= reproduction rate of a particular pathogen; and λ = decay rate constant of a particular pathogen. Assuming r and λ to be time-invariant, integrating Equation (6.17) will reduce to:

$$\int_{C_0}^{C(t)} \frac{dC}{(r-\lambda)C - r\frac{C^2}{C_0}} \quad = \int_0^t dt = t \tag{6.18}$$

Let t_r be the time duration by which the concentration of pathogen reduces from C_0 to $C_0 10^{-n}$. Hence, an expression for t_r can be obtained as:

$$t_r = \int_{C_0 10^{-n}}^{C_0} \frac{dC}{\left(\frac{r}{C_0}\right)C^2 + (\lambda - r)C} \tag{6.19}$$

Eq. 6.19 can be further simplified (Abramowitz, M., Stegun 1970, p.12) to obtain an expression for t_r:

$$t_r = \frac{2.303}{(\lambda - r)} log_{10} \left\{\frac{r + \lambda 10^n - r10^n}{\lambda}\right\} \cong \frac{2.303}{(\lambda - r)} \left\{n + log_{10}\left(\frac{\lambda - r}{\lambda}\right)\right\} \tag{6.20}$$

For the required log cycle reduction n with respect to the river water concentration C_0, the corresponding travel time by which the concentration reduces to $C_0 10^{-n}$ was ascertained.

6.4.2 Pathogen Transport in Groundwater Considering Dispersion and Decay

Adopting the model of Harvey and Garabedian (1991) which include hydrodynamic dispersion, filtration, the first-order decay of pathogen, and the retardation for solute transport in saturated, homogeneous porous media can be expressed as:

$$R\frac{\partial C}{\partial t} = \frac{\partial}{\partial x}\left(\alpha x_w v \frac{\partial C}{\partial x}\right) - v\frac{\partial C}{\partial x} - (\mu_l + \mu_s(R-1) + k_{att})C \tag{6.21}$$

Where C=Pathogen concentration. at any time t, R=retardation factor, v =average pore-water velocity and α =longitudinal dispersivity, μ_l= first order die-off rate for the free pathogen, μ_s= first order die-off rate for the sorbed pathogen and; k_{att}= first order filtration rate coefficient. The Equation (6.21) can be further simplified as follows:

$$R\frac{\partial C}{\partial t} = D_h\frac{\partial^2 C}{\partial x^2} - v\frac{\partial C}{\partial x} - \lambda C \quad (6.22)$$

Where; D_h=hydrodynamic dispersion and λ= first order total removal rate. In this case, the one-dimensional dispersion is assumed to be applicable. Equation (6.22) has the same form as that of the common convection-dispersion equation. The only difference between Eq. (6.21) and Eq. (6.22) is that the variable hydrodynamic dispersion coefficient, $\alpha x_w v$, in Eq. (6.21), has been replaced with the constant hydrodynamic dispersion coefficient D_h in Eq. (6.22). The distance between the infiltration gallery and the river is discretized to a uniform grid size of Δx to formulate an explicit finite difference scheme. The grid size of Δx is so chosen such that the Péclet number,$Pe = (\Delta x v_{(i)}/D_h) \geq 4$. In case of solute transport, if Péclet number is greater than 4, the concentration-time graph simulated by a Hybrid Cells in series model matches with Ogata and Bank's solution (Ogata & Banks 1961, Ghosh et. al., 2004). The hydrodynamic dispersion coefficient is taken as $D_{h_{(i)}} = \alpha\ v_{(i)}$. In this case, a grid size$\Delta x = 1$ m was considered and dispersivity of 0.2m was assumed to find $C_{(x,t)}$. The concentration at the i^{th} node and j^{th} time step using an explicit finite difference method can be derived as:

$$C_{(i,j+1)} = C_{(i,j)} + \frac{\Delta t}{R}\left\{D_{h_{(i)}}\frac{C_{(i-1,j)} - 2C_{(i,j)} + C_{(i+1,j)}}{\Delta x^2} - v_{(i)}\frac{C_{(i+1,j)} - C_{(i-1,j)}}{2\Delta x} - \lambda C_{(i,j)}\right\} \quad (6.23)$$

The time step of size Δt is chosen. The velocity $v_{(i)}$ is computed for $x=i\Delta x$ and time $t=j\Delta t$. Equation (6.23) can be applied to the domain $0 \leq x \leq (x_w)$. The domain can also be expressed as $0 \leq i \leq (i_w - 1)$ where; $i_w = x_w/\Delta x$. The boundary conditions to be satisfied at the river aquifer interface is $C_{(0,j)} = C_0$. The boundary conditions that is convenient to be satisfied at the infiltration gallery is $C_{((i_w-1),j)} = C_{((i_w-3),j)}$. Numerical results are presented in this thesis by applying an explicit finite difference method which evaluates the safe distance of an infiltration gallery to attenuate pathogens during stream stage rise.

6.5 Results and Discussion

In this study, the groundwater flow model and pathogen transport model (LIFI-PATRAM) considering dispersion and decay was used. The flow field was considered corresponding to the stream stage rise, aquifer parameters, stream water quality parameter (*E-coli* pathogen) and location of infiltration gallery. This will help find the pathogen concentration with respect to the distance measured from the river boundary. A computer code was developed in the MATLAB_R2016b which couples the AEM model and LIFI-PATRAM model. To test the performance of a population growth model based on logistic function over time analysis was carried out with various sets of reproduction rate (r) and decay rate (λ) of the pathogen. One set of decay rate and reproduction rate has been demonstrated in the model application section. It was found that the concentration of pathogen during the time t_r reduced to C_o10^{-n} over n log cycle. The findings are well documented in the article of Shende and Chau (2018). The initial concentration of pathogen in the river water was known, and its allowable concentration was chosen as per drinking water quality standards (WHO 2008). From the specific values of decay rate and reproductions rate, the required number of log cycle reduction in the concentration of pathogen was determined. For the required log cycle reduction

'n' with respect to polluted stream water, the corresponding travel time (in days) by which the concentration will reduce from C_0 to $C_0 10^{-n}$ was ascertained.

This study presents the analytical deduction of the formulae to forecast the safe distance of an infiltration gallery considering shock loads during storms. The model application has been demonstrated with one theoretical example in the following section to discuss the results.

6.5.1 Model Application

Logistic Function Case:

Results are presented for a safe distance of the proposed infiltration gallery from the polluted river during stream stage rise (σ) and aquifer parameters. For $\sigma = 3m$, k=8m/day, D=30m, porosity η=0.3, the distance traversed by a drop of water from the stream bank with time since onset of the flood is plotted in Figure 6.4. The initial concentration of *E-coli* in the stream during the storm has been taken as C_0=1x10^7 count /100 ml. For 10 log cycle reduction in bacteria concentration the time required is 204 days (see Eq. 6.20) corresponding to reproduction rate r =0.2/day and decay rate λ=0.3/day.

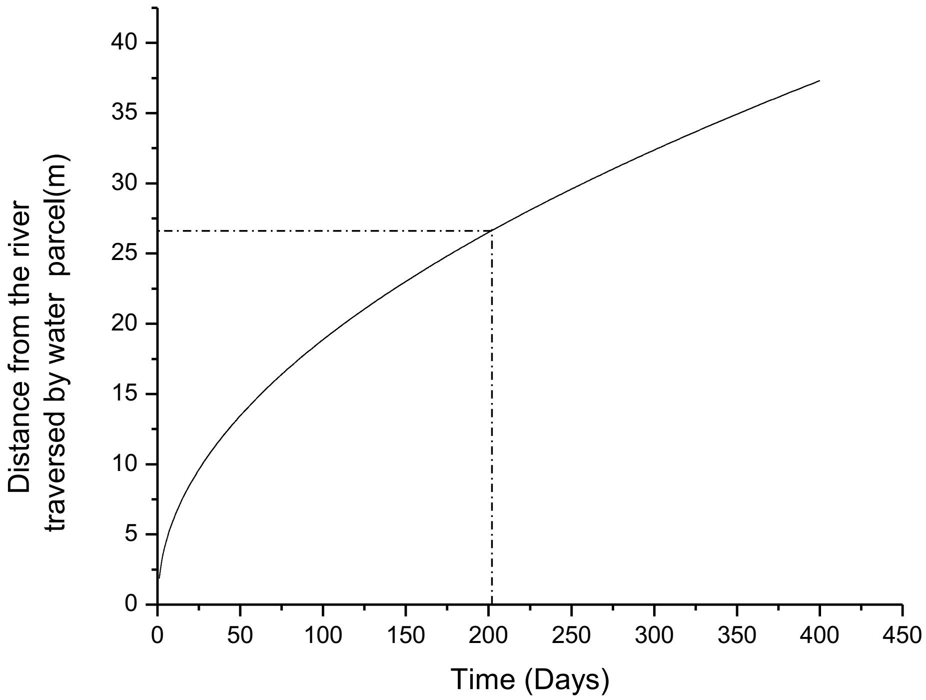


Figure 6.4 The distance of an infiltration gallery with respect to time.

For 10 log cycle reduction, the distance traversed by the drop of water in 204 days is 26.72m. Thus, if the gallery is located at 27m from the river, the bacteria concentration will be within the permissible limit during the storm. For a hydraulic understanding of the groundwater flow nature, the velocity of water has been plotted with respect to the distance of an infiltration gallery from the river bank (see Figure 6.5).

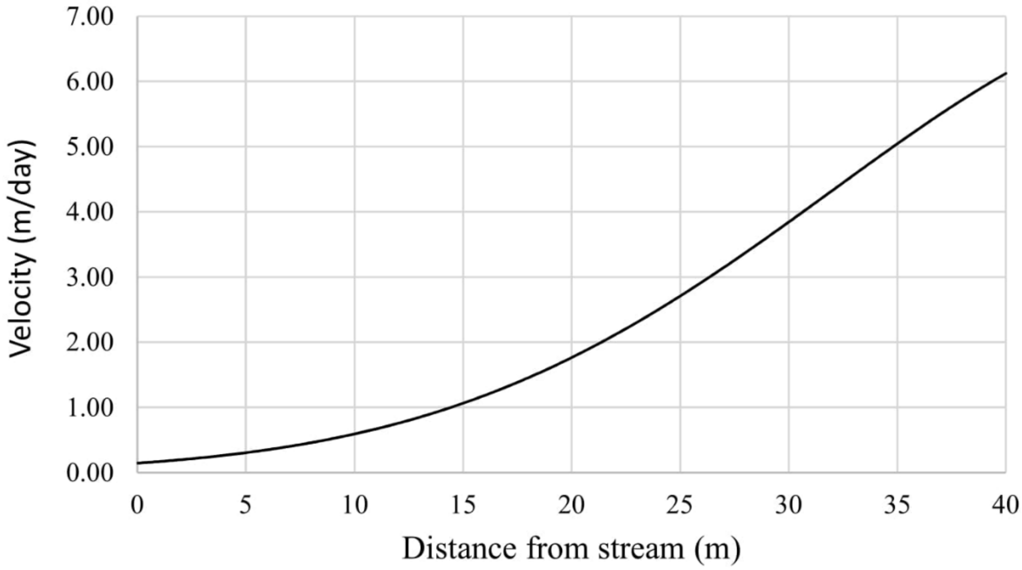


Figure 6.5 Velocity flow field from the stream to the infiltration gallery.

The velocities at different section towards an infiltration gallery are tabulated in Table 6.1. Only a certain section of the flow path is mentioned herewith.

Table 6.1 Computed velocity of infiltrated water.

Distance from a river (m)	Approach velocity of water (m/day)	Distance from a river (m)	Approach velocity of water (m/day)
0.1	0.1481	34.6	4.9503
0.2	0.1504	34.7	4.9740
0.3	0.1528	34.8	4.9976
0.4	0.1552	34.9	5.0212
0.5	0.1576	35	5.0447
0.6	0.1600	37	5.5025
0.7	0.1625	34.3	4.8792
0.8	0.1650	34.4	4.9029
0.9	0.1676	34.5	4.9267
1	0.1702	37.1	5.5247

1.1	0.1728	37.2	5.5467
18	1.4553	37.3	5.5687
18.1	1.4698	39	5.9274
30	3.8438	39.1	5.9475
30.1	3.8677	39.2	5.9675
30.2	3.8916	39.3	5.9874
32	4.3257	39.4	6.0072
32.1	4.3499	39.5	6.0268
32.2	4.3741	39.6	6.0464
32.3	4.3983	39.7	6.0658
34	4.8076	39.8	6.0850
34.1	4.8315	39.9	6.1042
34.2	4.8554	40	6.1232

Dispersion and Decay Case:

The pollutant transport from the stream considering dispersion and decay rate during step rise in stream stage is analyzed in the next section. An explicit finite difference scheme described by Equation (6.23) is adopted to get the variations of $C(x,t)/C_0$ and -Log_{10} ($C(x,t)/C_0$)), for retardation factor *R*= 2/day and decay rate $\lambda = 0.2/day$ and coefficient of dispersion D_h =0.02 m^2/day. The results are presented in Figure 6.6 for the rise in stream stage i.e. σ =3 m.

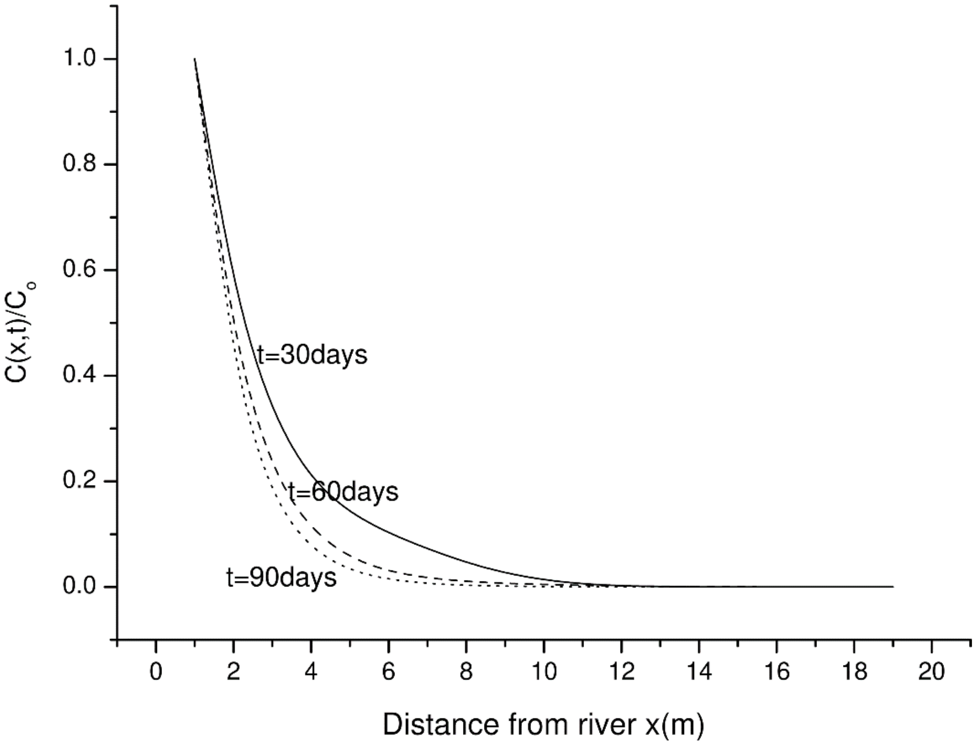


Figure 6.6 Variation of concentration with distance from the stream.

At a certain distance, the concentration decreases with time because of the decay factor. As time increases the entrance velocity at the stream-aquifer interface decreases and thus the inflow of pollutant diminishes. The variation of log cycle reduction is plotted in Figure 6.7 for R=2 and $\lambda = 0.2$ per day. For 10 log cycle reduction in concentration, the required distance is 28.5 m during the stream stage rise of 3 m.

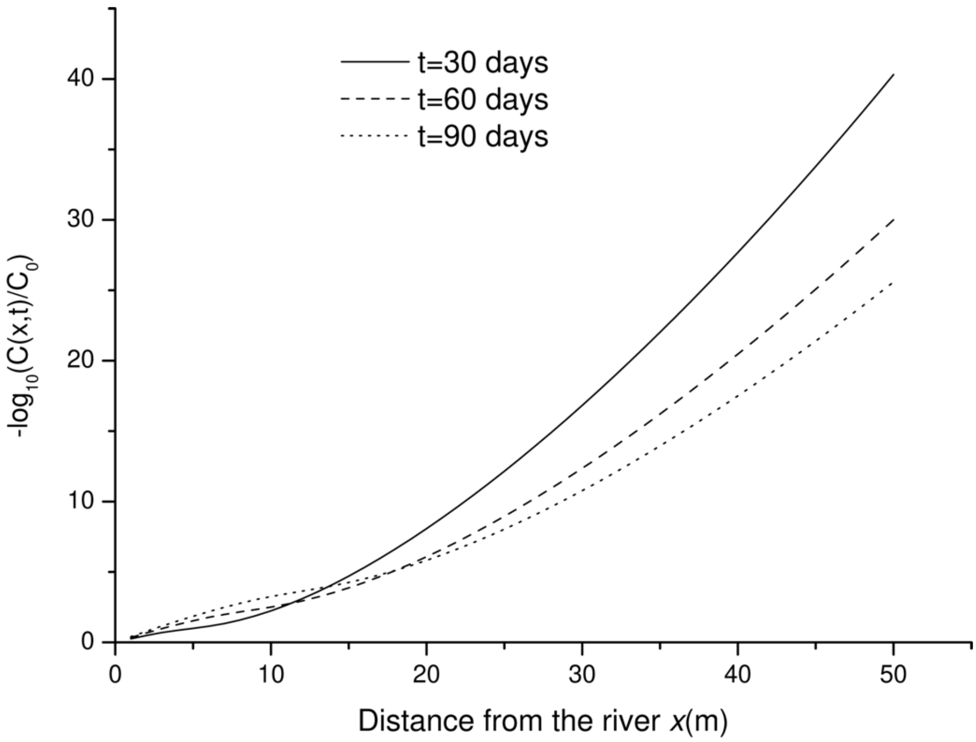


Figure 6.7 Log cycle reduction with distance from the stream.

In this analysis, it was also observed that the safe setback distance of an infiltration gallery during stream stage rise should be considered carefully, because, even though the concentration of pathogen shock load appears to be almost negligible at 15m from the stream boundary, the 10-*log cycle* reduction was only achieved beyond 28m (See Figure 6.6).

6.6 Summary

In this study, the findings of analysis were reported to forecast a suitable location of an infiltration gallery in riverbank filtration facility. For this analysis, we used an analytical model called LIFI-PATRAM to predict log cycle reduction of *E-coli* pathogen. The model is based on an explicit finite difference scheme which is used to estimate the change in pathogen concentration during stream stage rise. The applicability of this analytical model is demonstrated with an example. It is observed that the safe distance of an infiltration gallery is governed by the logistic function and therefore it can be effectively applied to compute the desired log cycle reduction in the pathogen concentration during riverbank filtration.

In this analysis, it was observed that the safe distance of an infiltration gallery should be considered carefully, because, even though the concentration of pathogen seems negligible at some distance from the stream boundary, the most desired log cycle reduction can be achieved from the analytical analysis presented in this study. More importantly, AEM based LIFI-PATRAM model can find out the optimal location and therefore can be applied to various field problems by incorporating the different properties of the aquifer and source water quality for sustainable management of RBF. In addition, more work is needed to understand riverbed dynamics and to use this information in modelling and management decisions. It is also important to conduct optimization and cost studies of RBF with other pre-treatment methods to show that RBF is a reliable and cost-effective treatment technique.

Chapter 7 : Conclusions and Future Work

The research work presented in this thesis attempts to constitute a comprehensive effort to analyze and quantify drinking water utilities. The methodology framework provides a shift in the way engineering problems are formulated in water supply utility, allowing for greater control of objectives analysis.

Three main areas in the drinking water utility system have been analyzed including the effective application of a new meta-heuristic modelling optimization algorithm, called ‘simple benchmarking algorithm’ (SBA), and the analytical assessment tool developed herein called ‘logistic function-based pathogen transport model’ (LIFI-PATRAM). It was found that this approach has the potential to drive any of the currently used engineering analysis and design tools for forecasting riverbank filtration (RBF) appropriateness and optimize the water distribution network. This proposed methodology has the potential to influence the technical relevance for the decision-making processes. Some precautionary measures have been also been analyzed considering the pathogen shock load during a storm for an infiltration gallery as an RBF component. A suitable setback distance of an infiltration gallery which supplies a significant quantity of water to a town was therefore forecasted.

7.1 SiBANET Optimization approach

This study aimed to analyze the optimized design of water supply and distribution network system (WSDNS) from an engineering point of view. Having the objective of developing a greater understanding of the fundamental design components, this study proposes a methodology for direct support to water engineers/policy planners considering the analysis with computer-aided design procedures. The WSDNS involves a considerable share of the total outlay of any water supply system, hence its optimal design is essential. Thus, the study sets out to assess the notions of optimization in the field of drinking water system utilities and selects those that lent themselves to an engineering application; develops a systematic and quantifiable approach that may be used to directly support water utility operation engineers; tests the applicability of the approach; analyzes each field in detail and selects the various elements necessary for the application. This can also include RBF facility as a drinking water supply utility.

By large, the objectives of the study have been fulfilled. The framework developed herewith satisfies the initially defined requirements and criteria. The main results achieved through the above steps are a clearer and deeper definition of the concepts of optimization of drinking water utility for hydraulic engineering perspective, and their standardized quantification in a systematic manner. The immediate consequence of the proposed methodology is; the possibility of reformulating the objectives traditionally employed in water engineering tasks; such as the optimal design of a water supply system, the general design of operational control and optimize distribution networks.

One of the major contributions of this research is to explore the applicability of an SBA algorithm for the optimized design of a WDN. For this purpose, a hybrid model has been developed called SiBANET. The pipe hydraulic solver called EPANET 2.0 was used for cost simulation with respect to hydraulic constraints. The effectiveness of the proposed SiBANET model has been verified by applying it to various benchmarking problems, ranging from small big sized networks consisting of numerous discrete sized pipe diameters to a large size network having huge solution space for optimization.

The non-linear head loss equation and the discrete nature of the commercially available pipes impose complexities in the design of WDNs. Several classical optimization techniques have been used by suppressing the requirements of commercial sizes to obtain a continuous pipe size. Although these methods are simple, they suffer from the drawback of providing a non-commercial size solution and cannot guarantee a globally optimal solution. With the advent of computers, many mathematical programming techniques and evolutionary algorithms have been developed and used for the optimization of WDNs. These techniques have been briefly discussed in Chapter 2. In most of the optimization techniques, the search for a better solution starts with some initial solution and deterministic transition rules or auxiliary information like the gradient of an objective function is used to move from one solution to another. The final solution depends on the initial solution and therefore, there is a possibility of it getting trapped into local optima. Therefore, several constraints handling strategies are made with different initial solutions and the minimum cost solution is considered as the global optimal solution.

Evolutionary computing techniques are multi-start random search techniques. They work on a population of solutions and search the entire feasible solution space, thereby increasing the chances of obtaining a global optimum solution. Several evolutionary techniques have been discussed in Chapter 3. These have been successfully applied to the WDN design problem. Even though they require several simulations runs, the numbers of simulations are far less than complete enumeration.

A simple evolutionary method founded on Darwin's theory of natural evolution was developed called a Genetic Algorithm. It can deal with an integer, real or mixed variables with relative ease. As in other evolutionary computing techniques, the random sampling capability increases the chances of obtaining the optimum global solution; however, because of the stochastic nature of the search an algorithm, it has a lesser assurance that the global solution will be found. GA uses a discrete diameter approach for the design of WDN and provides a set of single-sized practically favoured optimum solutions which may contain the global optimum. Since several near-optimal solutions are thus obtained, a wide choice of selection is available to the field engineers /policy planners.

GA has been tested on various benchmark design problems of WDNs. However, as discussed, GA suffers from a major drawback of computational exhaustiveness. The efficacy and efficiency of a GA are subject to on many constraints/strategies such as representation scheme, penalty method, fitness function, selection scheme, type of crossover and mutation operators, hydraulic analyzer, and more importantly the size of the search space. Many researchers have tried to improve the effectiveness and efficiency of GA (Dandy et. al 1996; Wu and Simpson 2001; Vairavamoorthy and Ali 2000; Wu and Walski, 2005; Vairavamoorthy and Ali 2005; Kadu et. al., 2008) to obtain a better solution in lesser evolutions. The modified GA suggested by Kadu *et. al.*, (2008) involving reduction in search space and application of penalty based on equivalent energy cost was found to reduce the number of evaluations drastically, from 1,000,000 (requiring 3 h run time) by Savic and Walter (1997) to only 18,000 (requiring 7.8 minutes). Still, the evaluations required were on the higher side as compared to some other evolutionary methods.

It is observed from the WDN literature study that even though the penalty evaluation method, based on a deficiency in the head, is good; the deficiency in the head is being obtained by considering the satisfaction of demand, higher than necessary penalty gets applied. Since an optimal solution lies on the boundary of the feasible and infeasible solution, the penalty method plays a big role in the computational effectiveness of GA.

EPANET, the most reliable hydraulic network solver available in the public domain, has been utilized by several researchers to carry out network analysis through repetitive runs. However, as seen in the literature, the faster converging meta-heuristic algorithms are still required hence, an improved methodology called ‘SiBANET’ for the design of WDN using EPANET 2.0 hydraulic solver for analysis of WDS is suggested in Chapter 4.

The next step was to couple the modified EPANET with the SBA Code. The SBA code proposed by Xie (2018b) was used. This new methodology was verified through the Two-looped network and Hanoi network for assessing the minimum cost with a lesser number of function evaluations. The results of the analysis indicate that SiBANET is the most robust technique in handling discrete constraints wherein the global solution can be achieved within a few functional evaluations. It is also found that the CPU time is comparatively lesser than other IOAs referred to in WDN design. SiBANET model has the desired results in WDN problems as it can handle discrete pipe diameter along with fast convergence. The main reason for the lesser number of function evaluations requirements is the intelligent exploration and exploitation strategy of SBA to locate a global optimum solution with its organizing tactics rather than the probability rules of the operator. The SBA based design application method is found better than other IOAs in reducing the number of evolutions.

This study is a first effort to optimize the WDN utilizing an SBA intelligent approach in the meta-heuristic algorithm, hence only pipe-sizing optimization with a single objective was considered. Optimization of the complex looped networks with multiple objectives is highly recommended for further research. In the future, the proposed methodology can be prolonged to a large WDN by joining it with various decomposition approaches and various appropriate penalty function application to optimise the design. Form machine learning point of view, more learning method could be tested to improve the performance of SBA.

7.2 Forecasting appropriateness of RBF utilities

The supply of pathogen-free fresh water to the optimized distribution network, as configured by SiBANET, has been investigated to the units considering RBF facility. An analytical solution is proposed in Chapter 5 & Chapter 6 and the methodology is applied taking into a reference of field data collected from various RBF sites worldwide.

RBF is a proven engineering technique for treating drinking water supply which is analogous to slow sand filters. There is enough experience in both the US and Europe that validates RBF in an array of environmentally different settings and under conditions that have rigorously tested this technology. Given enough flow-path length and time, microbial contamination can be removed/attenuated to levels protective for public health. Under optimal conditions, RBF can achieve up to 8 log cycle of pathogen removal over about 30 m of safe distance from the river. Greater removal efficiency may be expected for various other types of bacteria, protozoa and algae under the same conditions. These high removal efficiencies can be expected to protect public health to minimize risk levels considering appropriate travel time/retention time with respect to the pumping rate to fulfil the desired water demand.

Though RBF an engineering technique, but the design and appropriate construction depend on the personal experience of the designer, therefore, it has been observed that RBF efficiency diminishes with the short flow path length, high heterogeneity, coarse matrices, and steep gradient accompanying high velocities. These are some of the vital features which are common to many riverbanks filtrated water supply schemes. The deficiency in these features is well known and therefore many water treatments systems rely on additional treatment barriers, especially disinfection e.g. chlorination, etc. which makes RBF as a meaningless utility. Pathogen transmission which is investigated in Chapter 5 is little affected by disinfectants and therefore, requires greater attenuation if RBF is the only available water purification facility (e.g. rural areas of developing countries). The protective feature of RBF is both benefit and liability. The benefit is that RBF processes are always working to minimize various contaminant breakthrough concentration. The liability is that an RBF failure to completely remove microbial contamination will most likely result in difficultly recognizable, short periods with modest contaminant concentration. To deal with such situations, which generally occur during flooding conditions in the rainy season, the safer distance of a water abstraction location has been evaluated for pathogen attenuation as discussed in Chapter 6.

Those who are contemplating RBF, should move ahead and utilize its principles because this would not only add value to the water supplies but would also enhance their overall sustainability for future generations.

7.3 Suggestions and future work

In the line of investigation explained in this thesis, more research is needed to address the various challenges in RBF. The prediction transport model proposed herein can be used to simulate the complex groundwater flow fields. Due to the uncertainty of the aquifer pattern, there are some chances that the pathogens may enter the drinking water system. Considering such unforeseen difficulty, the water supply utility should use multi-barrier drinking treatment process to support RBF. Another approach may be studied by incorporating the ozonation unit along with activated carbon to remove the persisted contaminants.

Fairly speaking, before the publication of ‘Riverbank Filtration - Improving Source Water Quality’ by Ray et. al., (2002), there was no serious concern toward framing some standard guidelines for the management of RBF facilities. Now reference for the practical implementation of the collector well or infiltration gallery is available for an independent practitioner to set up an RBF utility effectively. However, it has been observed that there is a growing concern amongst developing countries to fulfil the drinking water demand with the utilization of less treatment cost and maintenance. In this connection, the proposed research work explained well in this thesis which can be used as a guideline. However, there are a wider verity of problems can be clubbed and address based on the fundamentals proposed herewith.

The scope for further research may include the following:

- Though RBF is an engineering technology, both its design and construction depend on personal experience/expertise.
- Validation of LIFI-PATRAM model for RBF sites to help utility managers to plan for emergencies, future expansion and other unforeseen events.
- Optimization and cost studies of RBF with other pre-treatment methods to show that RBF is a reliable and cost-effective treatment mechanism.
- More work to understand riverbed dynamics utilize this information in modelling and management decisions.
- More research is needed on simulating horizontal collector wells in shallow alluvial aquifers adjacent to surface waters, so that pathogen transport model prediction can be used to simulate these complex 3D groundwater flow fields.
- Further study on the temperature and turbidity of water is needed as it relates to both water travel time and pathogen growth.
- Exploring the effect of groundwater dilution on the performance of RBF.

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