

Wasteload Allocation Using Wastewater Treatment and Flow Augmentation

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Abstract A Wasteload allocation model, named Cost-Flow-Augmentation Model involving wastewater treatment and flow augmentation as a method of pollution abatement has been developed. The cost functions for wastewater treatment were developed as power functions of biochemical oxygen demand (BOD) removal using the regression module of the SPSS10 software. The cost function for flow augmentation was also developed using a regression between cost of dam/barrage and corresponding flow released from upstream reservoir for downstream water quality improvement. The response of wasteloads and flow augmentation on the water quality was quantified in terms of transfer coefficient calculated using the QUAL2E water quality simulation model. The performance of these models is demonstrated on the 22-km-long Delhi stretch of river Yamuna, India. Optimal solutions of the formulated models were obtained using the Web-based interactive non-differentiable interactive multiobjective bundle-based optimization system software. The optimal solutions obtained reveal that flow augmentation is not an economically feasible pollution abatement option for the Delhi stretch of river Yamuna.

Keywords Wasteload allocation · Wastewater treatment · Flow augmentation · Cost function · Optimization

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1 Introduction

Regulation of stream flows needed to assimilate wasteloads is known as flow augmentation. The critical stream flows (at low-flow conditions) can be augmented by releasing additional water from reservoirs, which are located up-stream of a water quality monitoring station. This additional flow may improve the critical flow conditions by increasing the flow in streams, reducing temperatures and increasing the reaeration rates [17]. Extensive work has been reported on optimal reservoir operation for downstream water quality improvement [23, 24, 28, 29]. However, these studies have focused on flow augmentation only without the inclusion of additional pollution abatement methods such as wastewater treatment, artificial aeration, etc. Furthermore, most published optimization models for regional water quality management address the problem by determining the optimal BOD removal using the wastewater treatment only without a joint optimization of pollution abatement methods such as wastewater treatment and flow augmentation. However, a very few studies have focused on developing wasteload allocation models with a combination of different pollution abatement methods.

Paraska [24] examined the cost effectiveness of low-flow augmentation and mechanical aeration in Miami River. The study showed that flow augmentation can appreciably decrease the cost of meeting water quality standards. Hass [13] developed a waste load allocation model with treatment and flow augmentation as the pollution abatement methods. The model was applied to the Miami River basin and solved using the Dantzig–Wolfe nonlinear decomposition technique. Grantham et al. [10] used system analysis to investigate the economic implications of low-flow augmentation. A hydrologic flow simulator and a water quality linear programming model were interfaced to develop a procedure for

determining a lower bound on the value of flow augmentation. Chia and Meier [2] presented a water quality–quantity management approach which incorporated the costs of water pollution control and water treatment, the benefits of water-based activities, and the trade-off between low-flow augmentation and water quality improvement. The analytical framework was based on the decomposition procedures using dynamic programming. McNamara [18] presented a nonlinear programming model using geometric programming technique for application to the Hudson River. The model considered wastewater treatment, bypass piping, flow augmentation, and artificial aeration for determining the least-cost solution. Harb Harboe [12] presented two examples of water resources to illustrate the applicability of approximation method from monthly to daily basis. These were: (a) a reservoir operated for hydroelectric power production and low-flow augmentation provided on a daily basis on the Lech River System in Germany and (b) a reservoir operated optimally for daily low-flow augmentation at a control gauge downstream on the Wupper River System in Germany.

The literature review reveals that since the late 1970s or so, not much has been reported on flow augmentation in the archival literature. This long hiatus in studies on flow augmentation as a means of water quality control can largely be attributed to scarce fresh water availability in the catchments because of increasing demands of population. Moreover, augmenting the flow also increases the velocity which displaces the DO profile downstream in rivers. Such a situation is undesirable in river stretches near urban areas as the positive effect on the DO sag curve would be shifted some distance downstream of the urban centers. Regardless of the above limitations, the overall effect of flow augmentation is positive [33]. Taking a leaf out of this hypothesis, the present study was undertaken for investigating the optimal combination of flow augmentation and wastewater treatment for a highly critical Delhi stretch of river Yamuna, India. A WLA model, named Cost-Flow-Augmentation Model (CFAM) was developed to find the optimal combination of wastewater treatment and flow augmentation for improving the water quality in a river system. The study is different from previously reported literature in the following aspects: (a) use of nonlinear cost function for wastewater treatment and flow augmentation, (b) use of transfer coefficient-based linked simulation–optimization approach, (c) probably the first study in India to consider the joint optimization of wastewater treatment and flow augmentation, (d) probably the first attempt in India to assess the environmental flow using simulation and optimization models, and (e) Application of simulation (QUAL2E) and optimization (non-differentiable interactive multiobjective bundle-based optimization system (NIMBUS)) models on the highly polluted Delhi stretch of river Yamuna, India.

2 Development of the Model

2.1 Problem Statement and Assumptions

Suppose there is a river system with various polluters (in the form of sewage treatment plants (STPs) or drain outfalls) located at the bank discharging treated and untreated waste water/effluent into it. The upstream of the river stretch has a reservoir from which fresh water can be released for assimilating/diluting the waste loads coming from the STPs located along the river system. This will help in improving the river water quality in the river downstream. The following assumptions are made in model formulation:

- a) The proposed STPs are assumed to be located just before the outfall of the drains such that they directly discharge their effluents into the river.
- b) The STPs/drains are assumed to enter at the beginning of a reach.
- c) The STPs/drains divide the study stretch into reaches.
- d) The amount of discharge, the DO concentrations, and the raw BOD loadings of all the drains are assumed to be steady and known.
- e) The parameters of a water quality simulation model (the QUAL2E in this study) are assumed to be known.
- f) The stream flow in each reach is considered as deterministic and known.
- g) The desired water quality standard in terms of the minimum dissolved oxygen concentration in the study stretch is known.

2.2 Formulation of the CFAM

In general, there is a pollution control agency whose role is to look after the treatment of wastewater (taking into account the assimilative capacity of river water) before discharging the effluent into the river in a manner such that the water quality of the river is within permissible limits [11]. The developed model (CFAM) considers determination of the optimal BOD removals by the STPs and optimal flow release from the upstream reservoir in order to minimize the total cost of wastewater treatment and flow augmentation while satisfying the water quality standard in all downstream river reaches. The decision variables, the BOD removal fraction at each treatment plant are selected from the feasible range (between the upper and lower bounds). Other decision variable is the optimal flow release from an upstream reservoir, which in combination with the optimal BOD removal would give an appropriate optimal combination. The problem involves the use of an optimization model where the objective function comprises a function for cost of wastewater treatment and flow augmentation cost. The constraints comprises

water quality standards, bounds on BOD removal, and flow release from upstream reservoir. Thus, if the cost function for wastewater treatment is denoted by $C_i(r_i)$ and that for low-flow augmentation by $C_f(y)$, the CFAM can be formulated as follows (Eqs. (1) to (4)):

$$\text{Min } F = \sum_{i=1}^{NS} C_i(r_i) + C_f(y) \tag{1}$$

Subject to

a) Water quality improvement constraints.

$$\sum_{i=1}^{N_j} \phi_{ij} W_i r_i + b_j y \geq S_j \quad \forall j, = 1, 2, \dots, N_j \tag{2}$$

b) Inequality constraints (BOD removal constraints or bounds).

$$r_i^{(L)} \leq r_i \leq r_i^{(U)}, \quad \forall i = 1, 2, \dots, NS \tag{3}$$

c) $y_u \geq y \geq y_l$ (4)

Where, the notations are defined in Table 1.

Equation 1 defines the objective function representing total cost of wastewater treatment and flow augmentation. The first term in this equation is the cost function for wastewater treatment which was formulated by developing a regression relationship between annualized cost of wastewater treatment and fractional BOD removal. The second term in this equation represents the cost function for flow augmentation which was formulated by developing a relation between cost incurred to build and operate a reservoir/headwork to release flow required for downstream water quality improvement and the corresponding flows. Equation 2 defines the water quality response, relative to the water quality standard. These constraints require that the supply of available oxygen for the decomposition process in each reach (that available over and above the quality standard requirement) must be equal to or exceed the demand imposed by BOD loads discharged into that reach and all reaches preceding it; i.e., for each reach j we require,

$$\phi_{i1} W_1(1-r_1) + \phi_{i2} W_2(1-r_2) + \dots + \phi_{iNS} W_i(1-r_{NS}) \leq y b_j \tag{2a}$$

or,

$$\phi_{i1} W_1 r_1 + \phi_{i2} W_2 r_2 + \phi_{i3} W_3 r_3 + \dots + \phi_{iNS} W_i r_{NS} \geq S_j \tag{2b}$$

where,

$$S_j = \phi_{i1} W_1 + \phi_{i2} W_2 + \phi_{i3} W_3 + \dots + \phi_{ij} W_i \tag{2c}$$

Table 1 Notations of the CFAM model

| Notations | Description |
|-------------|---|
| C_i | Cost of wastewater treatment by plant i |
| r_i | BOD removal by the i th plant |
| $C_i(r_i)$ | Cost function for wastewater treatment |
| $C_f(y)$ | Cost function for low-flow augmentation |
| NS | Total number of treatment plants/dischargers |
| i | Source number |
| j | Reach number |
| m | Total number of reaches |
| N_j | Number of dischargers upstream of j (highest i upstream of j) |
| b_j | Improvement in dissolved oxygen at point j due to removal of r_i in BOD at all sources upstream of j . In fact, it represents the improvement in water quality because of treatment and flow augmentation, i.e., amount of DO available for decomposition (total available less standard requirement in reach j) |
| y | Additional flow needed for water quality improvement in a reach |
| ϕ_{ij} | Transfer coefficient (it estimates the improvement in DO at j per unit increase in treatment or reduction in BOD load at i) |
| $r_i^{(U)}$ | Minimum allowable treatment level/lower bound |
| $r_i^{(L)}$ | Maximum allowable treatment level/upper bound |
| W_i | Waste loading from source i (kg BOD ₅ /day) |
| y_l | Lower bound of flow augmentation |
| y_u | Upper bound of flow augmentation |

Equation 2b states that the supply of dissolved oxygen created by treatment and flow augmentation must exceed that demanded by the gross wasteloads. Equation 3 represents the bounds on the decision variable, the BOD removal. Equation 4 represents the bounds on flow augmentation. The CFAM model provides an optimal combination of BOD load removals by STPs along the river and the flow release from the upstream reservoir to satisfy the desired DO goals. The decision variables, the BOD removal fractions at each treatment plant are selected from the feasible range (between the upper and lower bounds). The optimum solution or combination consists of those values of r_i and y that satisfy the constraint Eqs. (2) to (4).

2.3 Optimization Using NIMBUS Software

The CFAM model formulated in the previous section was solved using the NIMBUS software. NIMBUS is a multiobjective optimization system capable of handling both differentiable and nondifferentiable multiobjective and single objective optimization problems subject to nonlinear and linear constraints with bounds for the decision variables. It is not a usual computer application, because it has only a

WWW (World-Wide Web) interface. This means that the software is used via a Web browser. The problems to be solved are of the form:

Optimize.

$$\{f_1(r), \dots, f_k(r)\} \tag{5}$$

Subject to.

$$g_j(r) \leq 0, \quad j = 1, \dots, m_i \tag{6}$$

$$g_j(r) = 0, \quad j = m_i, \dots, m \tag{7}$$

$$\phi_{ij}(r) \leq b_1 \tag{8}$$

$$\phi_{ij}(r) = b_2 \tag{9}$$

$$r_i^{(L)} \leq r_i \leq r_i^{(U)}, \quad i = 1, 2, \dots, n \tag{10}$$

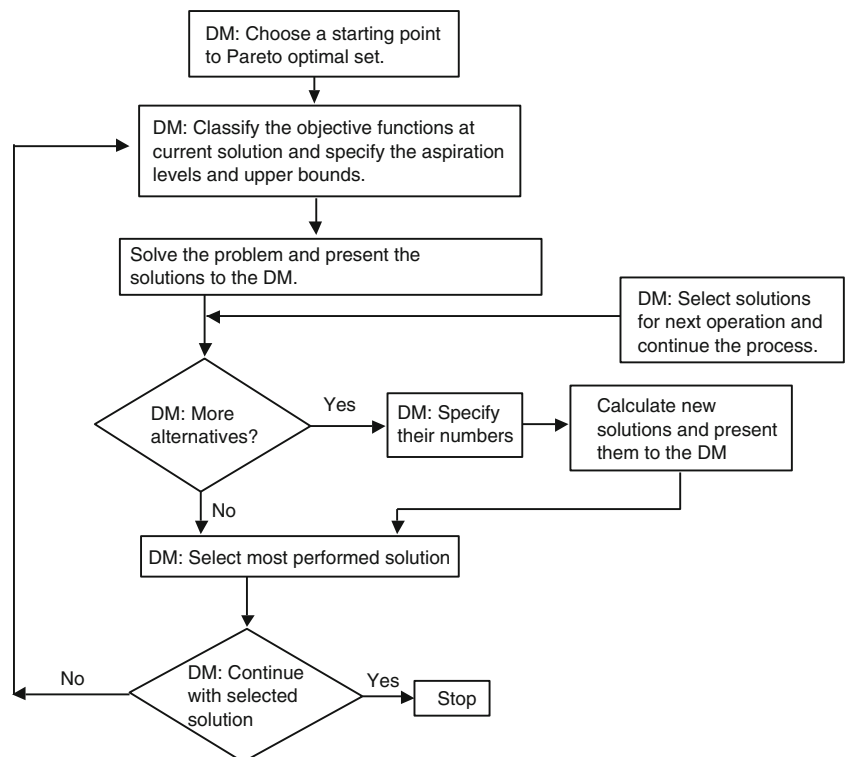
where k is the number of objective functions, m is the number of the nonlinear constraints, decision variables, r and A matrix of linear constraint coefficients. The idea in the NIMBUS system is that the user examines the values of the objective functions calculated at a current solution (optimal solution or Ideal criterion vector) and divides the objective functions into up to five classes. These are: the value of

objective function should be decreased, should be decreased till some aspiration level, is satisfactory at the moment, is allowed to increase till some upper bound, and is allowed to change freely. At each iteration, the decision maker is shown the values of the objective functions. After the classification, a new subproblem is formed and solved. The improvement in the Pareto optimal set in any objective function values is possible only by allowing impairment in some other objective function. The search procedure stops if the decision maker does not want any further improvement in the value of objective function. The problem is solved either by opting the multiobjective proximal bundle method or genetic algorithms. The NIMBUS algorithm is depicted in the form of a flow chart in Fig. 1. For more details reader may refer to Miettinen [20].

3 Application

The model developed was applied to the Delhi stretch of the river Yamuna. The river Yamuna is a 1,376 km long tributary of river Ganges and has a catchment area of 366,220 km². The river traverses for around 600 km before it enters Delhi through the Wazirabad barrage. From the Wazirabad barrage, water is diverted for drinking water supply of Delhi city. During dry season, no water is allowed to flow downstream of the Wazirabad barrage as the available water is not adequate to fulfill the demand for water supply to Delhi. After

Fig. 1 Flow chart of the NIMBUS algorithm (source: Miettinen and Makela [21])



22 km downstream of the Wazirabad barrage, there is another barrage, the Okhla barrage through which the water is diverted into the Agra Canal for irrigation. The 22-km-long stretch between the Wazirabad Barrage and the Okhla Barrage has 15 drains discharging partially treated or untreated wastewater into it. Thus, the 15 drains divide the 22-km stretch into 16 reaches. Figure 2 presents the schematic sketch of the Delhi stretch of the river Yamuna along with all the drains. Table 2 presents the BOD and discharge details of the 15 drains discharging treated, partially treated, and untreated wastewater in the Delhi stretch of the river Yamuna. The total pollution load in terms of BOD, disposed off into the river by these 15 drains is about 164.05 tonnes/day [4]. Such high BOD load in a small 22 km-long Delhi stretch results in very poor water quality in the river. For the purpose of this study, the proposed STPs at the drains which outfall

along the river have been assumed to run between 0 and 96 % efficiency in removing BOD so that the effluent when discharged into the river does not violate the stream DO standard of 4 mg/l.

The model was aimed to determine the optimal BOD removal and the flow released from the upstream reservoir for water quality improvement on the downstream river system. The BOD loads were obtained from CPCB [4]. It was assumed that the amount of waste generated by the drain will not change as an efficient taxation system comes in place. In the present application to the Delhi stretch of river Yamuna, a time span of 3 months in a particular year, namely April, May, and June was considered as these months are the months of lowest flow. The flow pattern during the rains in recent years (especially August and September 2012) in the Delhi stretch of the river Yamuna reveals that approximately 25 lacs cusecs flow can be stored in the upstream reservoirs for release during dry periods.

Although the designated best use criteria for dissolved oxygen in the Delhi stretch of river Yamuna is 5 mg/l (class B), in this study a DO standard of 4 mg/l (Class C) was assumed. The background DO concentration of 5.5 mg/l was used. The stream flow and temperature conditions used as input to the QUAL2E water quality simulation model were based on low-flow and high-temperature conditions from the summer period (March–June 2004) [8]. The water quality response in the form of transfer coefficient was obtained using the QUAL2E simulation model [35] and fed into the optimization model. The optimization formulation contained differentiable objectives, linear constraints, bounds, and other inequality/equality constraints. Solutions of the formulated multiobjective optimization models were obtained using the NIMBUS Web-based optimization package [20]. The development of cost functions for wastewater treatment and flow augmentation is discussed in the following sub sections.

3.1 Cost Function for Wastewater Treatment

Cost functions are the main input to water quality management models. The cost function to be used in Wasteload allocation modeling is generally a function of the BOD removal [13, 18]. In this study, the cost data obtained from various sources were amortized at 4 % interest rate to find the present cost of the plant. The amortized cost was added to the annual operation and maintenance (O&M) cost to find the annualized cost of wastewater treatment. The corresponding data related to BOD removal by plants were obtained from various sources [3–5]. The regression analysis was carried out using the SPSS Version 10 [30] to obtain the cost as a function of the percentage BOD removal. A number of best-fitting equations were attempted while carrying out regression analysis. Results reveal that all the equation forms show good results. However, since all the

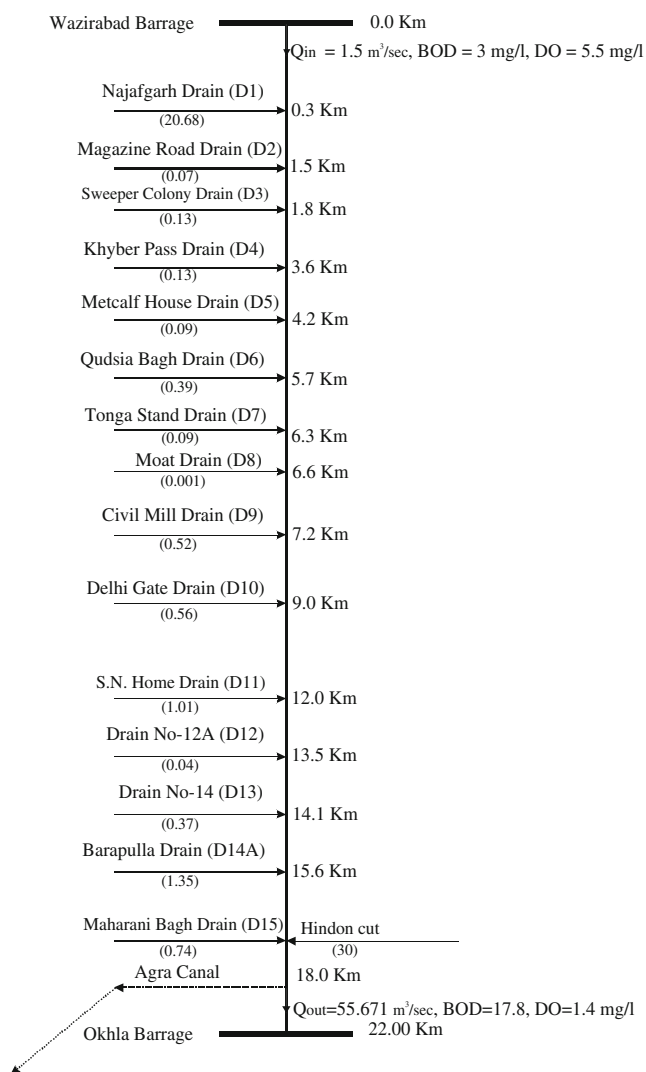


Fig. 2 Schematic sketch of drains in the Delhi stretch of river Yamuna (shown in parenthesis are discharge values in cubic metres per second)

Table 2 Details of BOD load and discharge of drains

| Name of drain | Flow (m ³ /s) | BOD (mg/l) | DO (mg/l) | Temperature (°C) | Percentage treatment |
|------------------------------|--------------------------|------------|-----------|------------------|----------------------|
| Najafgarh Drain (D1) | 20.68 | 56 | 0.0 | 28 | 0.0 |
| Magazine Road Drain (D2) | 0.07 | 333 | 0.0 | 28 | 0.0 |
| Sweeper Colony Drain (D3) | 0.13 | 236 | 0.0 | 28 | 0.0 |
| Khyer Pass Drain (D4) | 0.13 | 136 | 0.0 | 28 | 0.0 |
| Metcalf House Drain (D5) | 0.09 | 73 | 0.2 | 28 | 0.0 |
| Mori Gate Drain (D6) | 0.39 | 134 | 0.2 | 28 | 0.0 |
| Tonga Stand Drain (D7) | 0.09 | 96 | 0.4 | 28 | 0.0 |
| Moat Drain (D8) | 0.001 | 62 | 0.3 | 28 | 0.0 |
| Civil Mill Drain (D9) | 0.52 | 171 | 0.0 | 28 | 0.0 |
| Delhi Gate Drain (D10) | 0.56 | 103 | 0.0 | 28 | 0.0 |
| Sen Nursing Home Drain (D11) | 1.01 | 183 | 0.0 | 31 | 0.0 |
| Drain no. 12A (D12) | 0.04 | 105 | 0.2 | 31 | 0.0 |
| Drain no. 14A (D13) | 0.37 | 116 | 0.3 | 31 | 0.0 |
| Barapulla Drain (D14) | 1.35 | 135 | 0.0 | 32 | 0.0 |
| Maharani Bagh Drain (D15) | 0.74+28.0 ^a | 48 | 1.5 | 32 | 0.0 |

^a Flow through Hindon Cut

proposed treatment plants at the outfall of the drains might be required to treat wastewater up to the advanced treatment level, nonlinearity beyond primary treatment was introduced into the cost functions by developing them as power functions. Thus, the cost function obtained was of the form given below:

$$C_i = pr^q \quad (11)$$

Where,

C_i =annualized cost of wastewater treatment.

P =coefficient of BOD removal in the cost function.

q =exponent of BOD removal in the cost function.

On the basis of varying flow range, all the 15 drains were categorized into seven different ranges and cost functions developed. These are presented in Table 3. It is evident from this table that developed functions have R^2 value more than 0.70 thereby implying good correlation.

Table 3 Developed cost functions for wastewater treatment

| Range of flow (in MLD) | Drain nos. | Cost function (C) | R^2 |
|------------------------|------------------------|------------------------|--------|
| 0–5 | D8 and D12 | $0.000010282r^{3.525}$ | 0.9949 |
| 6–20 | D2, D3, D4, D5, and D7 | $0.0011654r^{2.7795}$ | 0.8882 |
| 21–35 | D6 and D13 | $0.1548r^{1.8462}$ | 0.9399 |
| 36–50 | D9 and D10 | $0.4524r^{1.6944}$ | 0.7406 |
| 51–70 | D15 | $0.2352r^{1.9171}$ | 0.975 |
| 71–100 | D11 | $0.8314r^{1.5179}$ | 0.7326 |
| ≥100 | D1 and D14 | $0.001395r^{3.4277}$ | 0.9395 |

Where r is the percentage of BOD removal

3.2 Cost Function for Flow Augmentation

The net costs of flow augmentation can be defined as the minimum costs necessary to increase the flows during the period of low flows for the sole purpose of water quality management [17]. The cost function for such flow augmentation involves a relation between cost incurred to build and operate a reservoir/headwork to release flow required for downstream water quality improvement and the corresponding flows. Such a cost function is normally nonlinear [13, 18] with the power form being the most reliable. The cost function for flow augmentation used in this study is of the form given below:

$$C_f = g(y)^h \quad (12)$$

Where,

C_f =annualized cost incurred to build and operate a reservoir/headwork to release flow required for downstream water quality improvement.

y =flow corresponding to C_f .

In order to normalize the pricing of goods and labor, the wholesale price index [20] was applied to calculate the present cost. After price conversion, the total annual cost was calculated by the following equations [27].

Total annual cost.

$$(TAC) = CRFX IC + OMC \quad (13)$$

where, CRF is the capital recovery factor, IC is the initial cost and OMC is the operation and maintenance cost. The CRF is given by,

Table 4 Results of regression for various forms of cost equation for flow augmentation

| Data details | No. of data sets | Linear | Quadratic | Power | Cubic |
|----------------|---------------------------|--------|-----------|-------|-------|
| Whole India | 192 | 0.916 | 0.92 | 0.935 | 0.948 |
| Ganga basin | 79 | 0.657 | 0.721 | 0.955 | 0.954 |
| Final equation | $C_y = 439.364y^{0.7717}$ | | | | |

$$CRF = I(1+i)^n / (1+i)^n - 1 \tag{13a}$$

where, *i* is the interest rate and *n* is the life of reservoir/dam/barrages.

The cost and flow data of 192 dams/barrages/weir constructed during the period 1953 to 2005 were considered [6, 7, 31, 32, 34]. The development of cost function for flow augmentation requires the cost of headwork only and not for the whole irrigation scheme. However, the literature survey carried out revealed that in India, the available cost data was for the whole irrigation scheme (inclusive of construction cost of headwork, canal, power house etc.) and not the

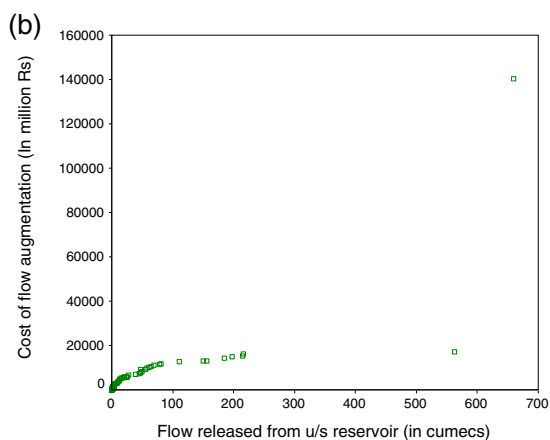
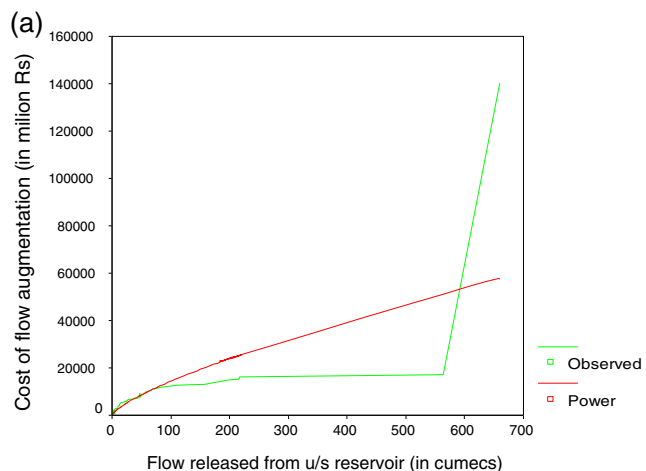


Fig. 3 **a** Profiles of the observed and computed cost curves for flow augmentation using Ganga basin data sets (79 numbers). **b** Trend of observed flow augmentation data

Table 5 Optimal solution for the CFAM

| Drain no. | Name of the drain | BOD removal (<i>r</i>) | |
|--------------------------|-------------------|--|----------------------------------|
| | | Case I (with minimum flow release of 1.5 m ³ /s from μ/s) | Case II (with zero flow release) |
| D1 | Najafgarh | 0.855 | 0.855 |
| D2 | Magazine Road | 0.15 | 0 |
| D3 | Sweeper Colony | 0.15 | 0 |
| D4 | Khyber Pass | 0.15 | 0 |
| D5 | Metcalf House | 0.15 | 0 |
| D6 | Mori Gate | 0.15 | 0 |
| D7 | Tonga Stand | 0.15 | 0 |
| D8 | Moat | 0.15 | 0 |
| D9 | Civil Mill | 0.25 | 0.46 |
| D10 | Delhi Gate | 0.25 | 0.35 |
| D11 | Sen Nursing Home | 0.0 | 0 |
| D12 | Drain no. 13 | 0.0 | 0 |
| D13 | Drain no. 14A | 0.0 | 0 |
| D14 | Barapulla | 0.0 | 0 |
| D15 | Maharani Bagh | 0.53 | 0.59 |
| <i>y</i> (in cumecs) | – | 1.8 | 0.00001 |
| Total cost of WWT and FA | | Rs. 725.71 million | Rs. 679.82 million |

headwork alone. Thus, as per the guidelines available [1, 14, 22, 26, 34] and personal discussions with the officials of UP Irrigation Department, 15–35 % of the total cost, depending upon the size of project was assumed to be the cost of headworks/dams/barrages.

The design life of flow augmentation reservoirs/headworks was taken as 80 years. The rate of interest used in calculation of the CRF was taken as 6.5 % [1, 14, 22]. Accordingly, the value of CRF was calculated as 0.0655. The annual O&M cost was considered as 10 % of the cost of headwork. Once the annualized cost for flow augmentation was obtained using Eq. (13), the corresponding flow that can be released from these reservoirs for downstream water quality improvement was required. It would be pertinent to mention that in India, no such data is available [15, 36]. In fact, in India no guidelines are available for minimum flow requirement downstream of the dam for maintaining the ecology of rivers. Although this aspect has been recently studied for some river basins in India [9, 16, 37], no such data are available for any of the rivers in India. In the absence of this, certain assumptions were made to find the environmental flow that can be released from reservoir/dams for downstream water quality improvement. These are as follows:

- a) The total live storage in the reservoirs was assumed to be available for release for water quality improvement. For some reservoirs this was assumed to be released through

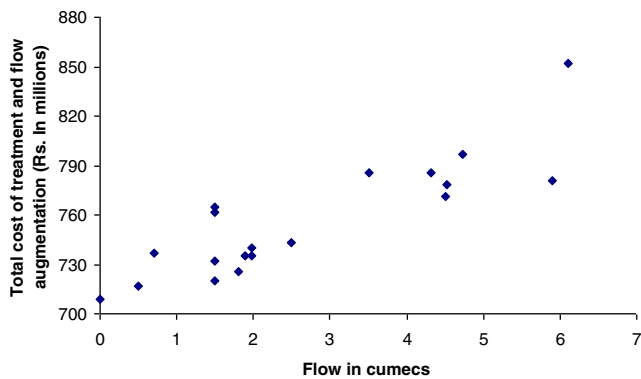


Fig. 4 Economic analysis of flow augmentation

out the year. However for reservoirs in Bundelkhand area of the UP state, this flow was assumed to release during 3 months only. Accordingly, the flow that can be released daily was calculated for all 192 headworks/reservoirs considered in this study.

- b) In reservoirs with hydropower projects, the flow from tail race was assumed to join the river on the downstream. This flow was assumed to be available for water quality improvement downstream of the headwork.

Once the flow data were collected, the cost function was formulated as a regression model. The cost of flow augmentation is the dependent variable and the corresponding flow released to meet the downstream water quality is the independent variable. Two separate data sets were attempted. One for the 192 data set of reservoirs through out India and other for 79 data sets of Ganga river basin alone. For both these cases, the regression analysis was carried out using the SPSS Version 10 [30]. A number of best fit equations were attempted, while carrying out the regression analysis. The results obtained from these trials are shown in Table 4. It is evident that although all equation forms show good results, the power function appears to be the best with a R^2 value of 0.955 for the Ganga basin. Figure 3a, b shows the profiles of observed and computed cost for the power form of cost function for the Ganga basin.

4 Results, Discussion, and Conclusions

The CFAM model consists of a single objective function formulated with the aim of minimizing the cost of wastewater treatment (of all 15 plants) and the cost incurred in releasing water from an upstream reservoir for water quality improvement on the downstream subject to water quality constraints. The input consists of the objective function of cost minimization, bounds of 16 decision variables (BOD removal and flow available for augmentation), 15 linear

equality constraints, and 17 nonlinear inequality constraints. The decision variables, the BOD removal at each of the treatment plants can vary between the lower and upper bounds of 0.0 and 96 %, respectively. Other decision variable is the optimal flow to be released from the reservoir for downstream water quality improvement. The minimum and maximum flow of 1.5 and 90 m^3/s was chosen, respectively. The maximum value of 90 m^3/s was chosen based on the results using the simulation model. The details of the same are presented elsewhere [25]. The water quality response was obtained using the QUAL2E water quality simulation model in the form of linear equality constraints as presented in Eq. (2). Since NIMBUS is capable of handling only the “less than or equal to” constraints, the LHS and RHS of the linear constraints have been multiplied with -1 . Thus, all the coefficients in the linear constraints are negative. Since the flow release from the up-stream reservoir can vary depending upon its availability in the basin, two separate cases were considered. These are:

- Case I: minimum flow release of 1.5 m^3/s (same as considered in simulation)
- Case II: with no flow release from upstream reservoir.

Case I In case I, it was assumed that under the baseline conditions, the minimum flow released continuously from the upstream reservoir is 1.5 m^3/s . Thus, the transfer coefficients as used in the simulation model were used in this case also. As mentioned earlier, the starting point of 0.0 was chosen as the initial solution to start the optimization search process. The option of “Differentiation by the system” was chosen and the input submitted. NIMBUS took 40705 function and sub-gradient evaluations to obtain the optimal solution. The solution obtained in this case was $r^*=(0.855, 0.15, 0.15, 0.15, 0.15, 0.15, 0.15, 0.15, 0.25, 0.25, 0, 0, 0, 0, 0, 0.53)$ and $y=1.8 \text{ m}^3/\text{s}$. The value of objective function obtained was 725.71.

The obtained optimal solution for Case I is presented in Table 5. It is evident that a minimum cost of Rs 725.71 million is incurred to meet the dissolved oxygen of 4 mg/l at all points in the Delhi stretch of river Yamuna. To achieve this, the Najafgarh Drain (D1) and Maharani Bagh Drain (D15) are required to treat 85.5 and 53 %, respectively. For D2, D3, D4, D5, D7, and D8, only 15 % BOD removals are required. For D9 and D10, BOD removal of 25 % is required. It is seen that no treatment is required by drain D11 to D14 i.e., Sen Nursing Home Drain, Drain No. 12A, Drain No.14, and Barapulla Drain.

It may be noted that the obtained solution is Rs. 65.546 million higher than that obtained for the least cost model (presented in [25]). However, this does not have any relevance because in the least cost model (LCM), the cost of 1.5 m^3/s flow augmentation was not considered. If the cost of

1.5 m³/s flow augmentation is added to the cost obtained in LCM (only towards wastewater treatment), the total cost of LCM would have been Rs. 720.262 million. This value is quite close to Rs. 725.71 million obtained in CFAM. This implies that flow augmentation does not yield promising results as a pollution abatement measure for the Delhi stretch of river Yamuna. The wastewater treatment alone seems to be a more appropriate method especially from cost considerations.

Case II In case II, it is assumed that under baseline conditions (existing), no flow is released from the upstream reservoir. Accordingly, the linear constraints as used in the model were modified and NIMBUS run again. The solution obtained in this Case was $r^*=(0.855, 0, 0, 0, 0, 0, 0, 0, 0.46, 0.35, 0, 0, 0, 0, 0, 0, 0, 0, 0.59)$ and $y=0.00001$ m³/s. The value of objective function obtained was 679.82. This solution is Rs. 19.756 million higher than that obtained for the LCM. However, this also does not have any relevance because in LCM, the cost of flow augmentation was not considered.

The optimal solution obtained for case II are presented in Table 5. It is evident that a minimum cost of Rs 679.82 million is incurred to meet a dissolved oxygen of 4 mg/l at all points in the Delhi stretch of the river Yamuna. To achieve this, the Najafgarh Drain (D1), Civil Mill Drain (D9), Sen Nursing Home Drain (D10), and Maharani Bagh Drain (D15) are required to treat 85.5, 46, 35, and 59 %, respectively. For D2, D3, D4, D5, D6, D7, D8, D11, D12, D13, and D14, no treatment is required. In addition, a negligible flow of 0.00001 m³/s is required for release from the reservoir. This implies that flow augmentation of relatively small magnitude is hardly contributing to the water quality improvement in the Delhi stretch of river Yamuna. This finding was substantiated by the results obtained in simulation which are presented elsewhere [25] wherein low-flow augmentation (up to the statutory requirement of 10 m³/sec) did not yield significant improvement in dissolved oxygen.

Results obtained for cases I and II reveal that the flow augmentation is not an economically feasible pollution abatement option for the Delhi stretch of river Yamuna. However, its appropriateness cannot be totally ruled out, especially in view of the limited land availability for the construction of STPs on the bank of river Yamuna in Delhi. However, this aspect is beyond the scope of the present study. It is pertinent to mention that because of the nonlinearity of the cost function for wastewater treatment and flow augmentation, the number of function and sub gradient evaluation was very high, i.e., 40705. The number of function evaluation would have reduced significantly if the cost function for wastewater treatment and flow augmentation had been linear.

To assess the efficacy of flow augmentation as a method of pollution control, the economic analysis was carried out.

Such an analysis is aimed at investigating the tradeoffs among (a) expenditure on wastewater treatment at the drain outfalls, (b) the cost of measures to augment the “critical” low flow, and (c) the advantages to be gained by establishing higher stream use classifications. The analysis involved changing the flow release in the optimization model in the range 0.0 to 6.1 m³/s, and running NIMBUS time and again to obtain different values of objective function. Figure 4 presents these values as function of corresponding flow augmentation. For each low flow, a minimum cost reservoir and minimum cost treatment pattern may be determined. If augmentation is economic, the curve of total cost (flow augmentation and treatment) as a function of low flow should be minimum, for example, the point on the extreme left bottom presents possible solutions. Whether this occurs always will undoubtedly vary with the specific circumstances, but each case can be explicitly investigated for the trade off between augmentation and treatment; for example, the available treatment technologies at various discharge outfalls may not be feasible to provide the requisite treatment configuration/efficiencies.

Regardless of the above limitations, the optimal solutions and trade-off curves developed for the problem defined in the application can be used by the pollution control agency to identify an appropriate solution for implementation. Such findings would be useful to the decision makers in implementing policies and solutions for improving the water quality in the river Yamuna up to the desired level.

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