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Z-Transform Based Instantaneous Unit Hydrograph for Hilly Watersheds

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Abstract

Present study emphasizes the applicability of linear theory concept onto hilly watersheds. For this purpose, Z-transform technique was used to derive the instantaneous unit hydrograph (IUH) from the transfer function of autoregressive and moving average (ARMA) type linear difference equation. Parameters of the ARMA type rainfall-runoff process were estimated by least-squares method. The derived IUH from Z-transform (i.e. ARMA-IUH) has been used to compute the hydrologic response i.e. direct runoff hydrograph (DRH). Further, the superiority of the proposed approach has been tested by comparing the results through the results obtained from the Nash-IUH. Analyzing the results obtained from ARMA-IUH and Nash-IUH for the two hilly watersheds of North Western Himalayas shows the applicability of the linear theory concept even in turbulent flow conditions which are frequently encountered in hilly terrains under similar conditions of flow.

Keywords: IUH, ARMA Process, Z-Transform, Nash Model, Direct Runoff Hydrograph, Hilly Watershed

1. Introduction

The rainfall-runoff process is nonlinear and dynamic with spatially distributed inputs and outputs. Watershed response is inherently spatial, non-linear and time-variant. However, linear models are frequently used for analysis of watershed response to rainfall, as they are mathematically more convenient to handle than non-linear models. The input-output mathematical models based on linear theory of hydrologic systems attempt to establish a link between two or more observed phenomena without detailed description of physical process under investigation. In hydrological context, the basin is regarded as the system in which an input of effective rainfall is transformed into an output of discharge at the basin outlet. Spolia and Chander [1] presented a discretely coincident form of the equal-reservoir cascade model [2]. A discrete linear cascade model was developed for hydrology using the cascade concept of the Auto Regressive Moving Average (ARMA)-type difference equation and derived the unit impulse response function as a discrete time function for a family of discrete-parametric models [3–4]. Wang and Wu [5] showed that discrete input data could be represented by means of unit step functions. Wang et al. [6] developed a rainfall-runoff model for small watersheds and an applied discrete excess rainfall-runoff model to calculate the hydrograph of a watershed from the excess rainfall under the concept of linear system.

Hilly terrains are generally encountered in many countries. Planning of water resources is equally important for such watershed. Often, very little attention has been given to these watersheds because of poor availability of hydrological data due to inaccessible terrains. In this work, data of two hilly watersheds are subjected to analysis using Z-transform technique, with the objective to study the rainfall-runoff process. Normally, rainfall-runoff process is treated as a linear system. However, it is perfectly not known whether this linearity also holds good for hilly watersheds. Thus, the intention is to analyze the data and see the applicability of using linear system concept in the modeling of rainfall-runoff in hilly terrains.

Hilly terrains normally possess larger roughness in comparison to the plane watersheds. This may be because of the nature of surface over which overland flow can take place. Piece of boulders, gravels is frequently encountered in the hilly regions. Also, a dense forests and scrubs may add to the roughness. Thus, it is equally important to test the linear behavior of the system even in conditions of terrains having normally higher roughness. In hilly terrains, due to higher roughness and large velocities of flow, the regime of flow is generally turbulent and applicability of linear system concept on hilly
terrains remains unexplored.

Therefore, the present study has been carried out with an objective to test whether the system behaves in a linear manner even under several extreme and complex conditions of terrains and nature of flow.

2. Derivation of IUH

Since inception [7], the unit hydrograph approach has been very well established as a linear theory concept in surface water hydrology and is continuously used by the researchers. For more generalized form, an instantaneous unit hydrograph (IUH) approach has achieved considerable momentum [2,8–10]. Other than the conceptual models [2,10], the researchers investigated wide range of methodology to derive IUH [11]. Also, many transform techniques (viz., Harmonic series, Fourier transform, Laplace transform etc.) either in continuous time domain or discrete time domains have been successfully used in the derivation of IUH. The Z-transform method constitutes one of the transform methods that can be applied to develop the response functions as a discrete time function of linear difference equations [10–15]. The technique works under the premise that the rainfall-runoff process behaves as a linear system for which Z-transform of the transfer function of the Z-transform of the transfer function and the effective rainfall. They have used higher order polynomial to analyzed the single storm event and derive the unit hydrograph ordinates by root selection from Argand diagram, which is a complicated and time consuming process. Therefore, in the present study analytical derivation of instantaneous unit hydrograph from the transfer functions of ARMA type difference equation using the Z-transform (ARMA-IUH) has been presented. The derived ARMA-IUH is then used to apply for the computation of direct runoff hydrographs. The relative performance of the proposed method has been tested by comparing it with the Nash-IUH model.

Therefore, the procedure for the derivation of ARMA-IUH and Nash-IUH has been represented in the following section.

2.1. ARMA-IUH (p, q) Model

The current outflow at the watershed outlet would generally be expected to depend on inflows (excess rainfall) and outflows (direct runoff) of several time units back. Therefore, an autoregressive and moving average (ARMA) process of excess rainfall-direct runoff can be used to determine the transfer function of the watersheds. The autoregressive and moving average of order (p, q) (ARMA (p, q)) process of rainfall excess and direct runoff can be given as [16]:

\[ Q(t) = a_0 Q(t-1) + a_1 Q(t-2) + \cdots + a_p Q(t-p) + b_0 I(t) + b_1 I(t-1) + b_2 I(t-2) + \cdots + b_q I(t-q) \]  \hspace{1cm} (1)

in which Q(t) is the direct runoff in m³/sec, I(t) is the excess rainfall intensity in m³/sec, a’s and b’s are the discrete time invariant parameters to be estimated and p and q are the order of the autoregressive and moving average (ARMA) processes respectively. For hydrologic applications, values of p and q must be selected through model identification and are generally less than four [17].

The Equation (1) can also be written using the back shift operator such that, \[ B[Q(t)] = Q(t-1) \] as [17]:

\[ (1 - a_1 B - a_2 B^2 - \cdots - a_p B^p) Q(t) = (b_0 + b_1 B + b_2 B^2 + \cdots + b_q B^q) I(t) \]  \hspace{1cm} (2)

Equation (2) can be expressed in the form of transfer function as:

\[ H(t) = \frac{Q(t)}{I(t)} = \frac{(b_0 + b_1 B + b_2 B^2 + \cdots + b_q B^q)}{(1 - a_1 B - a_2 B^2 - \cdots - a_p B^p)} \]  \hspace{1cm} (3)

where, H(t) is the transfer function of ARMA(p, q) process of excess rainfall-direct runoff of the linear system.

2.1.1. Definition of Z-Transform

The Z-transform is one of the transform methods applied to the solution of linear difference equations. Difference equations are functional equations that define sequences and are the discrete counterparts of the differential equations. In many systems, the outputs are measured at discrete values of time, usually at \( nT \), \( n = 0, 1, 2, \ldots \), where \( T \) is the fixed positive number, usually referred to as the sampling period (it could be unity). Consider such a sequence \( \{(nT)\} = 0, 1, 2, \ldots \), which can be thought of as arising from a continuous waveform sampled at times \( nT \), \( n = 0, 1, 2, \ldots \). The Z-transform of this sequence is defined as [18]:

\[ F(Z) = Z \{f(nT)\} = \sum_{n=0}^{\infty} f(nT)Z^{-n}, \text{ for } |Z| = 1/R \]

\[ = f(0) + f(T)Z^{-1} + f(2T)Z^{-2} + \cdots \] \hspace{1cm} (4)

where, \( Z = \exp(\text{i}\omega T) \), \( R \) is the radius of convergence of the infinite series, \( Z \{f(nT)\} \) is the Z-transform of the sequence \( f(nT) \), \( i = \sqrt{-1} \) and \( \omega \) is the angular frequency.

Here, in the manuscript the derivation of IUH was presented for the ARMA (2, 2) and ARMA (1, 1) processes using Z-transform technique according to the watersheds considered for the study.

2.1.2. IUH from ARMA (1, 1) Process [ARMA-IUH (1, 1)]

The transfer function of ARMA (1, 1) process of rainfall-runoff described by Equation (3) can be written as:
\( H(t) = Q(t) / I(t) = (b_c + b_l B) / (1 - a_r B) \) 
\[ (5) \]

The Z-transform of \( H(t) \) is:
\[ H(Z) = \frac{Q(Z)}{I(Z)} = \frac{b_c + b_l Z^{-1}}{1 - a_r Z^{-1}} \]
\[ (6) \]

Division of Equation (6) throughout by \( Z \) gives:
\[ H(Z) / Z = (b_c Z + b_l) / (Z - a_r) \]
\[ (7) \]
in which \( Q(Z) \) and \( I(Z) \) are the Z-transforms of \( Q(t) \) and \( I(t) \) sequences and \( H(Z) \) is the Z-transform of transfer function. The inverse Z-transform of \( H(Z) \) gives the unit impulse response function as:
\[ h(t) = A \delta_r + B a_r \]
\[ (8) \]
where:
\[ A = \frac{b_c Z + b_l}{Z - a_r} \]
\[ (9) \]
and:
\[ B = \frac{b_a a_r + b_l}{a_r} \]
\[ (10) \]

It should be noted that for \( t = 0 \), the unit impulse response function will be zero. Thus, substituting \( t = 1, 2, \ldots, n + 1 \) in the right side of the Equation (8), the unit impulse response function, \( h(t) \), of the watershed can be written as:
\[ h(t) = A \delta_r + B a_r \]
\[ (11) \]

In Equation (11) which \( h(t) \) is the unit impulse response function (ARMA-IUH) at discrete time \( t \) and \( \delta_r \) is the Dirac delta function which is defined as:
\[ \delta_r = \begin{cases} 1, & \text{for } t = 1 \\ 0, & \text{for } t \neq 1 \end{cases} \]
\[ (12) \]

2.1.3. IUH from ARMA (2, 2) Process [ARMA-IUH (2, 2)]
The ARMA (2, 2) process of rainfall-runoff in its transfer function form described by Equation (3) can be written as:
\[ H(t) = Q(t) / I(t) = (b_c + b_l B + b_2 B^2) / (1 - a_r B - a_r B^2) \]
\[ (13) \]

The Z-transform of Equation (13) is:
\[ H(Z) = \frac{Q(Z)}{I(Z)} = \frac{b_c + b_l Z + b_2 Z^2}{1 - a_r Z - a_r Z^2} \]
\[ (14) \]

Equation (14) can be simplified as:
\[ H(Z) = \frac{Q(Z)}{I(Z)} = \frac{b_c Z^2 + b_l Z + b_2}{1 - a_r Z - a_r Z^2} \]
\[ (15) \]

Equation (15) has been written after division by \( Z \) as:
\[ \frac{H(Z)}{Z} = \frac{b_c Z^2 + b_l Z + b_2}{Z (1 - a_r Z - a_r Z^2)} \]
\[ (16) \]
where:
\[ r_i = \frac{1}{2} \left( a_r - \sqrt{a_r^2 + 4 a_l^2} \right) \]
\[ (17) \]

and:
\[ r_2 = \frac{1}{2} \left( a_1 - \sqrt{a_1^2 + 4 a_2^2} \right) \]
\[ (18) \]

The partial fraction expansion of Equation (16) is:
\[ H(Z) / Z = \frac{A}{Z} + \frac{B}{Z - r_i} + \frac{C}{Z - r_2} \]
\[ (19) \]

Therefore, the unit impulse response function (ARM-A-IUH), \( h(t) \) will be expressed as follows.
\[ h(t) = A \delta_r + B a_r + C \]
\[ (20) \]

where:
\[ A = \frac{b_c Z^2 + b_l Z + b_2}{Z^2 - a_r Z - a_r} \]
\[ (21) \]
and:
\[ C = \frac{b_c Z^2 + b_l Z + b_2}{Z (Z - r_i)} \]
\[ (22) \]

The inverse Z-transform of Equation (19) is as follows.
\[ h(t) = A \delta_r + B a_r + C \]
\[ (23) \]

2.1.4. Parameter Estimation of ARMA (p, q) Process
Methods of fitting mathematical models to numerical data have been presented in a number of references [17, 19, 20]. The least-square method was used to fit the model parameters of the ARMA (p, q) process from input (excess rainfall) and output (direct runoff) data. This method seeks estimators which minimize the sum of the squared residual or errors between the observed and calculated \( Q(t) \). Let the residual be \( e(t) \), then:
\[ e(t) = \hat{Q}(t) - Q(t) \] for \( t = 1, 2, \ldots, m \)
\[ (24) \]
in which \( \hat{Q}(t) \) and \( Q(t) \) are the observed and calculated value of the direct runoff data. The Equation (26) may be written in matrix form as follows.
\[ \hat{\varepsilon} = \hat{Q} - A \hat{\beta} \]
\[ (27) \]

The components of the Equation (27) is given as follows.
\[ \hat{\varepsilon} = [e(1), e(2), \ldots, e(m)] \]
\[ (28) \]
\[ \hat{Q} = [\hat{Q}(1), \hat{Q}(2), \ldots, \hat{Q}(m)] \]
\[ (29) \]
\[ \hat{\beta} = [a_1, a_2, \ldots, b_0, b_1, \ldots] \]
\[ (30) \]
The least-square estimate of $\bar{\beta}$ is the solution of Equation (27), that is:

$$\bar{\beta} = (A^T A)^{-1} A^T Q \quad (32)$$

2.2. The Nash’s IUH

Nash [2,8] considered watershed as consisting of a series of $n$ identical reservoirs and proposed a conceptual model by routing an instantaneous inflow through a series of linear reservoirs in the following form of the instantaneous unit hydrograph equation.

$$u(0,t) = \frac{1}{K \Gamma(n)}(t/K)^{n-1} \exp(-t/K) \quad (33)$$

In above relationship, $u(0,t)$ is the ordinate of instantaneous unit hydrograph (1/hr) at time $t$, $K$ is storage constant (hr), $t$ is the time in hours after the beginning of direct runoff (hr), $\Gamma$ is the gamma function such that $\Gamma(n) = (n-1)!$ and $n$ is the shape parameter. Equation (33) in terms of time to peak ($t_p$) can be written as follows.

$$u(0,t) = \frac{(n-1)^n}{t_p \Gamma(n)}\left((t/t_p) \exp\left(-t/t_p\right)\right)^{n-1} \quad (34)$$

In the above relationship, $t_p$ is the time of peak flow ordinate.

2.2.1. Estimation of Shape Parameter $n$

The shape parameter, $n$ was estimated for the corresponding values of dimensionless recession constant using the curve (Figure 1), the relationship between dimensionless recession constant and the hydrograph parameter, $n$. The dimensionless recession constant was estimated by using the following equation followed by plotting the recession curve of the actual direct runoff hydrograph on semi-logarithmic paper, with direct runoff hydrograph on the logarithmic scale, it was possible to fit a straight line to the part of the curve immediately following the crest segment of the hydrograph [21].

$$K_1/t_p = \frac{t_i - t_0}{2.3 t_p \log(Q_i/Q_0)} \quad (35)$$

where $t_i$ is the time to peak, and $Q_0$ and $Q_i$ is the two values of the discharge and $t_i$ and $t_0$ are the corresponding two values of the time on the straight line in the semi-logarithmic plot.

3. The Hilly Watersheds

Two watersheds (viz., Arki and Chaukhutia) of different topographic and land use conditions from North Western Himalayas have been picked up to test the concept of linearity. The Arki watershed (31° 08′ 58″ and 31° 12′ 58″ N latitude and 76° 56′ 50″ and 76° 59′ 50″ E longitude)
Figure 3. Drainage map of Chaukhutia watershed.

is a sub-watershed of Satluj river catchment comprising an area of 24.60 sq km and lies in Solan district of Himachal Pradesh (India) as shown in Figure 2. The watershed is more or less rectangular in shape and has a mean length of 7 km and width of 3.5 km. The maximum and minimum elevations of the watershed above mean sea level are 1828 m at the upstream end of Arki river and 1060 m at the gauging station near the Arki town respectively. The watershed lies in the upper Shivaliks and mid hills and has sub-temperate climate. The total annual rainfall recorded at different locations of the watershed varies from 800 mm to 2000 mm and about 78 percent of the total annual rainfall occurs during the monsoon season (mid June to mid September). The watershed has hilly terrain with extremely undulating and irregular slopes ranging from relatively flat in valleys to quite steep slopes towards ridges with average slope of about 9 percent [22]. Whereas, the Chaukhutia watershed, a sub-watershed of Ramganga river catchment, a spring fed river originating from the mid-Himalayan ranges in Chamoli district of Uttarakhand (India) covering an area of 452.25 sq km and is located between 29° 46' 15" to 30° 6' N latitude and 79° 12' 15" to 79° 31' E longitude as shown in Figure 3. The Chaukhutia watershed is also approximately rectangular in shape and elongated in

Figure 4. Comparison of observed and computed direct runoff hydrographs of sample storm events of Arki watershed.
north-south direction, has a maximum length (north-south) of 30 km and width (west-east) of 16 km. The maximum and minimum elevations of the watershed are 3114 m at the upstream end and 929 m at the gauging station respectively. The slopes in the valley vary from 8-10 percent. The moderate hills lie between valley and steep hills with slopes varying from 10-50 percent while the slopes in the steep hills vary from more than 50 percent to almost vertical hills. The annual average precipitation in Chaukhutia watershed varies from 1084 mm to 1679 mm at different locations with mean annual precipitation of 1384 mm. From the total annual rainfall, about 75 percent occurs during the monsoon season from southwest monsoon. The climate of the Himalayan sub-watersheds varies from sub-tropical to sub-temperate with mean annual temperature of about 22°C. The mean annual minimum and maximum temperatures are 18°C and 30°C respectively. The monthly mean daily maximum temperature is highest (40°C) in the month of April whereas it is lowest (23°C) in December. The monthly mean daily minimum temperature is lowest (2°C) in January and highest (20°C) in August. The three distinct seasons in the area are: winter (October to March), summer (April–mid June) and monsoon (mid June–September). Severe frost occurs during nights from mid-December to mid-February when winter rains are deficient and damage fruits and vegetable crops grown in the watersheds.

Sixteen storm events that produced single peaked runoff hydrographs for the years 1993 to 1997 for Arki watershed and twenty storm events for the years 1976 to 1984 were analyzed to estimate the model parameters representing the watershed response. Almost all these events encountered a flow regime of turbulent flow around occurrence of peak flows. Direct runoff hydrographs were obtained by separating base runoff from total runoff hydrographs using the convex method suggested by Chow [23]. The volume of excess rainfall was determined by using the Φ-index method. The Φ-index method determines the horizontal line on rainfall hyetograph by iterative procedure such that the total depth of rainfall above it equals the resulting direct runoff [24].

As stated the objectives of the work is to confirm the applicability of the linear system concept for hilly watersheds in the runoff generation process. Towards this, the complexity of the system has been identified by the study of flow regime (i.e. laminar or turbulent flow) followed by the application of the methodology. For the application of the models, the data has been randomly divided into the calibration events and validation events. The parameters were estimated on the basis of the storm events used in calibration. The procedural details are discussed in next section for the Arki watershed and subsequently the results are presented for Chaukhutia watershed.

![Figure 5. Comparison of observed and computed direct runoff hydrographs of sample storm events of Chaukhutia watershed.](image)
Table 1. Comparison of observed and computed peak flows for Arki watershed.

| Date of Storm Event | Effective Rainfall (cm) | Observed Peak Flow (m³/sec) | ARMA-IUH | Nash-IUH |
|---------------------|-------------------------|-----------------------------|----------|----------|
| August 6, 1993      | 0.0280                  | 0.75                        | 0.747    | 0.82     |
| February 2, 1994    | 0.2422                  | 6.04                        | 6.461    | 4.08     |
| July 8, 1994        | 0.5677                  | 16.88                       | 15.13    | 16.61    |
| July 19, 1994       | 0.4450                  | 11.53                       | 11.87    | 13.02    |
| 'July 20, 1994      | 0.3556                  | 9.40                        | 9.4845   | 10.40    |
| 'August 2-3, 1994   | 0.1536                  | 4.35                        | 4.085    | 4.49     |
| August 23, 1994     | 0.0422                  | 1.10                        | 1.126    | 1.23     |
| September 5, 1994   | 0.2256                  | 6.05                        | 6.017    | 6.60     |
| 'September 8, 1994  | 0.1403                  | 4.12                        | 3.746    | 4.10     |
| 'June 6, 1996       | 0.5174                  | 12.48                       | 13.81    | 15.14    |
| 'June 17, 1996      | 0.0564                  | 1.37                        | 1.504    | 1.65     |
| 'June 30, 1996      | 0.4602                  | 11.50                       | 12.27    | 13.46    |
| August 2-3, 1996    | 0.0821                  | 2.31                        | 2.19     | 2.40     |
| September 2, 1996   | 0.1298                  | 3.75                        | 3.463    | 3.79     |
| 'August 12, 1997    | 0.2267                  | 6.35                        | 6.048    | 6.63     |

Table 2. Average estimated values of statistical measures for the models.

| Statistical Measures | Arki Watershed | Chaukhutia Watershed |
|----------------------|----------------|----------------------|
|                      | ARMA-IUH | Nash-IUH | ARMA-IUH | Nash-IUH |
| Coefficient of Efficiency (CE) | 0.9723 | 0.9190 | 0.9778 | 0.9438 |
| Relative Error in Estimated Peak (EP), % | 5.293 | 9.604 | 4.2771 | 4.6232 |

4. Results and Discussions

Eight out of sixteen storm events of Arki watershed were used to calibrate the model parameters. Analysis of these data revealed that the ARMA (2, 2) process was best fitted for Arki watershed. The average values of the parameters of ARMA (2, 2) process viz., $a_1$, $a_2$, $b_0$, $b_1$, and $b_2$ for Arki watershed were estimated to be 1.05042, -0.25591, 0.117016, 0.156270 and 0.126858, respectively. These parameters were then used to develop an IUH based on ARMA (2, 2) [i.e. ARMA–IUH (2, 2)] and IUH is as follows.

$$ h(t) = 0.49571 \delta_{t,n} + 1.50369 (0.66642)^{t-1} - 1.88239 (0.38402)^{t-1} $$

(36)

In the above relationship, $t$ is the unit time step. The developed ARMA-IUH has been used to compute the direct runoff hydrographs using the convolution technique. The comparisons of computed direct runoff hydrographs along with the observed hydrograph for sample storm events are shown in Figure 4. The hydrograph parameter i.e. peak flow rate for all the storm events used in the analysis are given in Table 1. Along with the visual assessment of proposed model, the following statistical criteria have been employed to test the performance of the approach.

1) Coefficient of efficiency (CE) [25]

$$ CE = 1 - \frac{\sum_{t=1}^{n} (\hat{Q}(t) - Q(t))^2}{\sum_{t=1}^{n} (Q(t) - Q_{av})^2} $$

(37)

where $\hat{Q}(t)$ is the computed discharge, $Q(t)$ is the observed discharge, $Q_{av}$ is average value of the discharge during the storm.

2) Relative error in estimated peak (EP)

$$ EP = \frac{\hat{Q}_p - Q_p}{Q_p} \times 100 \% $$

(38)

where $\hat{Q}_p$ is the computed peak discharge and $Q_p$ is the observed peak discharge.

The average estimated values of the CE and EP for Arki watershed is given in Table 2.
Table 3. Comparison of observed and computed peak flows for Chaukhutia watershed.

| Date of Storm Event | Effective Rainfall (cm) | Observed Peak Flow (m³/sec) | Computed Peak Flow |
|---------------------|-------------------------|----------------------------|--------------------|
| 1976                | 1976                    | 1976                       | 1976               |
| July 23, 1976       | 0.1150                  | 55.0                       | 58.31              |
| August 23, 1976     | 0.1166                  | 55.0                       | 59.12              |
| July 21-22, 1977    | 0.2690                  | 135.0                      | 136.39             |
| *July 22-23, 1977   | 0.2591                  | 140.0                      | 131.38             |
| August 11, 1977     | 0.1914                  | 100.0                      | 97.05              |
| August 18-19, 1978  | 0.2816                  | 150.0                      | 142.78             |
| *September 2, 1978  | 0.2573                  | 129.0                      | 130.46             |
| *June 21-22, 1979   | 0.1456                  | 72.0                       | 73.832             |
| July 21-22, 1979    | 0.1428                  | 74.0                       | 72.41              |
| August 31, 1980     | 0.2134                  | 102.0                      | 108.20             |
| August 2-3, 1981    | 0.2643                  | 124.0                      | 134.01             |
| *July 23, 1982      | 0.1650                  | 80.0                       | 83.66              |
| July 31, 1982       | 0.5034                  | 250.0                      | 255.25             |
| *July 17, 1983      | 0.2109                  | 105.0                      | 106.94             |
| August 21-22, 1983  | 0.1856                  | 95.0                       | 94.11              |
| *September 2, 1983  | 0.1890                  | 90.0                       | 95.83              |
| *September 5-6, 1983| 0.1882                  | 85.0                       | 95.43              |
| June 8, 1984        | 0.2174                  | 106.0                      | 110.23             |
| June 25, 1984       | 0.2190                  | 106.0                      | 111.04             |
| *July 15, 1984      | 0.2384                  | 120.0                      | 120.88             |

* Validation events

Similarly, the rainfall-runoff records of Chaukhutia watershed have been divided into two sets. Twelve out of twenty storm events were randomly selected for the calibration of the model parameters. Analysis of these data shows that the ARMA (1, 1) process found to best fitted. The average values of the parameters of ARMA (1, 1) (i.e. $a_1$, $b_0$ and $b_1$) using the least-squares method (Equation 6) were found to be 0.60585, 0.04921 and 0.37349, respectively. Using these parameters, the derived ARMA-IUH for Chaukhutia watershed is given as follows.

$$h(t) = -0.61647 \delta_{t-1} + 0.66568 (0.60585)^{t-1}$$ \tag{39}

The developed ARMA-IUH has been used to compute the direct runoff hydrographs using the convolution technique. The comparisons of computed direct hydrographs along with the observed one are shown in Figure 5. The hydrograph parameter i.e. peak flow rate for all the storm events used in the analysis are given in Table 3. The average estimated values of two statistical criteria (i.e. CE and EP) are given in Table 2.

4.1. Comparison of ARMA-IUH with Nash-IUH

The validity of the proposed approach has been tested by comparing the results obtained through the Nash-IUH Model (Equation 34). For this purpose, the shape parameter (i.e. $n$) has been estimated adopting the procedure given by Wu et al. (1964) and time to peak (i.e. $t_p$) has been obtained from the available storm event data of the two hilly watersheds. The calibration set of data have been used to estimate the value of $n$. The average estimated values of shape parameter $n$ and the time to peak $t_p$ for Arki and Chaukhutia watersheds were determined to be 3.75 and 1.50 hours and 5.307 and 2.00 hours, respectively. Finally the obtained relationship of IUH for Arki watershed using Equation (34) is obtained as follows.

$$u(0,t) = 6.9997(0.667t e^{-0.667t})^{2.75}$$ \tag{40}

In a similar fashion, the relationship obtained to define the Nash's IUH (Equation 34) for Chaukhutia watershed is given as follows.

$$u(0,t) = 30.136(0.5 t e^{-0.5 t})^{4.307}$$ \tag{41}

The derived relationships of instantaneous unit hydrographs using Nash model have been used to compute the direct runoff hydrographs for the available storm events of the two hilly watersheds. The comparison of the resulting direct runoff hydrographs from Nash model with
the observed direct runoff hydrographs along with the DRH obtained from ARMA-IUH are shown in Figures 4 and 5 for Arki and Chaukhutia watersheds, respectively. The hydrograph parameter i.e., peak flow rate for all the storm events used in the analysis are given in Tables 1 and 3. Along with the visual assessment of proposed model, the statistical criteria (i.e. CE and EP) have been used and the average estimated values of these errors are given in Table 2.

5. Summary and Conclusions

In the present study, an attempt has been made to seek the applicability of linear theory on the complex hilly watersheds. For this purpose, transfer function derived from the ARMA type difference equation has been used for the derivation of IUH applying the Z-transform technique. The proposed ARMA-IUH has been used for computation of direct runoff hydrographs for two hilly watershed viz. Arki and Chaukhutia watersheds. Further, the superiority of the proposed approach has been tested by comparing the responses obtained from the Nash-IUH. From the present investigation, following conclusion can be drawn.

1) Since, both the models have been developed from the linear theory concepts, i.e. ARMA-IUH is derived from the transfer function of the linear ARMA type difference equation and Nash-IUH was derived from the cascade of linear reservoirs, therefore, it is clear that the concept of linear theory is applicable to the hilly watersheds of complex hydrologic system.

2) It has been clearly observed from the Figures 4 and 5 as well as from Tables 1 through 3 that the proposed ARMA-IUH reproduced responses very close to the observed responses in comparison to that of Nash-IUH.

6. References

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## Notations

| Symbol | Description                  | Symbol | Description                  |
|--------|------------------------------|--------|------------------------------|
| \( \tilde{Q}_p \) | Observed peak flow rate      | \( H(t) \) | Inverse transform of \( H(Z) \) |
| \( T_p \)       | Time to observed peak flow   | \( I(t) \) | Input (effective rainfall) at time \( t \) |
| \( \delta \)    | Dirac delta function         | \( I(Z) \) | Z-transform of \( I(t) \) |
| \( \tilde{Q}(t) \) | Observed direct runoff ordinate at time \( t \) | \( K \) | Storage constant |
| \( a_1, a_2 \)  | Time-invariant parameters in discrete system | \( K_1 \) | Recession constant |
| \( B \)        | Backward shift operator      | \( n \) | Shape parameter |
| \( b_0, b_1, b_2 \) | Time-invariant parameters in discrete system | \( Q(t) \) | Computed direct runoff ordinate at time \( t \) |
| \( CE \)       | Coefficient of efficiency    | \( Q(Z) \) | Z-transform of \( Q(t) \) |
| \( EP \)       | Relative error in estimated peak | \( \tilde{Q}_p \) | Estimated peak flow rate |
| \( H(Z) \)     | Z-transform of transfer function | \( T_p \) | Time to simulated peak flow |
| \( h(t) \)     | Impulse response function at time \( t \) |
A Diffusion Wave Based Integrated FEM-GIS Model for Runoff Simulation of Small Watersheds

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Abstract

In this paper, an integrated model based on Finite Element Method (FEM) and Geographical Information Systems (GIS) has been presented for the runoff simulation of small watersheds. Interception is estimated by an exponential model based on Leaf Area Index (LAI). Philip two term model has been used for the estimation of infiltration in the watershed. For runoff estimation, diffusion wave equations solved by FEM are used. Interflow has been simulated using FEM based model. The developed integrated model has been applied to Peachtree Creek watershed in USA. Sensitivity analysis of the model has been carried out for various parameters. From the results, it is seen that the model is able to simulate the hydrographs with reasonable accuracy. The presented model is useful for runoff estimation in small watersheds.

Keywords: Diffusion Wave Model, GIS, Interception, Interflow, Philip Infiltration Model, Runoff Simulation

1. Introduction

Watershed is the fundamental geographical unit for the planning and management of water resources. Various hydrological processes occurring in the watershed are very complex in nature. A careful representation of the hydrological processes is necessary for the hydrologic modeling as it promises better estimates of hydrologic variables for management decisions [1]. Recent advancements in computing and database management technologies offer better physically based hydrological modeling which in turn provide better estimates of hydrologic variables such as infiltration, runoff etc.

Usually interception can be deducted as some percentage of rainfall. If required data is available for estimation of interception, it is better to incorporate it in the rainfall-runoff model. Aston [2] developed an exponential model for calculation of interception loss based on Leaf Area Index (LAI). Jetten [3] has used LAI and cover fraction based interception method in LISEM (LImburg Sediment Erosion Model). Kang \textit{et al.} [4] observed good agreement between the measured interception and interception obtained from linear regression model based on plant height and LAI.

Infiltration is an important hydrologic process, which must be carefully considered in the hydrologic models. Philip model is one of the commonly used approximate infiltration model. Luce and Cundy [5] used Philip infiltration model to calculate rainfall excess in their overland flow model. Jain \textit{et al.} [6] used the Philip two term infiltration model, to compute the infiltration in their model.

In steep humid catchments, interflow (through flow) is more likely to be the main form of drainage. Interflow travels laterally through the upper soil layers until it reaches a conveyance system. Jayawardena and White [7, 8] developed a finite element model suitable for catchments where through flow is dominant and applied to two experimental catchments. Sunada and Hong [9] presented a numerical runoff model describing interflow and overland flow on hill slopes.

St. Venant equations of continuity and momentum are the basic equations to simulate runoff routing in a watershed. A complete solution of St. Venant equations may not be worthy at all times, in view of the large computational efforts. The approximations of St. Venant equations like diffusion wave equations or kinematic wave equations may give most appropriate results with reasonable computational efforts. Morris and Woolhiser [10] showed that diffusion wave equations are adequate for highly sub-critical flow and where the downstream boundary conditions are important consideration. Hromadka II and Yen [11] developed a finite difference based diffusion hydrodynamic model and applied it to different civil engineering drainage problems. Hromadka and DeVries [12] and Ponce [13] in their papers have
discussed the limitations of kinematic wave modeling and advantages of diffusion wave modeling over kinematic wave modeling.

The Finite Element Method (FEM) is an efficient way to transform partial differential equations in space and time into ordinary differential equations in time. Finite difference schemes may then be solved for the time dependent solution of the system [14]. Blandford and Ormsbee [15] used diffusion wave equations to develop a finite element model for dendritic channel network with trapezoidal and rectangular channel geometries.

The physical parameters, which influence runoff routing, such as slope and Manning’s roughness vary spatially over the watershed. Spatial variation of these properties can be better incorporated into the model by using Geographical Information Systems (GIS). Sui and Maggio [16], Garbrecht et al [17] and Vieux [14] discussed the use of GIS in watershed modeling. Jaber and Mohtar [18] developed a GIS interface for the overland flow model by using the ArcView 3.2. In this paper an integrated hydrologic model based on FEM and GIS is presented for the runoff simulation and to analyze its application to a small watershed.

2. Model Formulation

In this paper, finite element based rainfall-runoff model using GIS is described for the simulation of event based surface runoff. Diffusion wave equations are used in the model development. Galerkin FEM technique has been used in the solution of the governing equations. Interception has been estimated by an exponential model based on Leaf Area Index (LAI). Philip two term infiltration model is used for the estimation of infiltration. Interflow has been estimated using FEM based model. Finite element grid has been prepared for the watershed by using GIS. Spatially distributed information for model inputs such as slope and Manning’s roughness are provided for each node of FEM grid using GIS.

2.1. Interception Model

Interception is estimated based on the equations used in the LISEM model [3]. In this model, canopy of crops and vegetation is regarded as simple storage. The cumulative interception during an event is given as [2]:

\[ I_c = c_p \cdot S_{c,\text{max}} \left[ 1 - e^{-c_{vd} \cdot \frac{P_{cum}}{S_{c,\text{max}}}} \right] \]

where \( I_c \) is the cumulative interception, \( c_p \) is the fraction of vegetation cover, \( c_{vd} \) is the correction factor for vegetation density and is given as \( c_{vd} = 0.046 \times \text{LAI} \), \( P_{cum} \) is the cumulative rainfall, \( S_{c,\text{max}} \) is the canopy storage capacity and \( \text{LAI} \) is the Leaf Area Index. Canopy storage capacity \( S_{c,\text{max}} \) is calculated by equation developed by Von Hoyningen-Huene [3] as follows:

\[ S_{c,\text{max}} = 0.935 + 0.498 \times \text{LAI} - 0.00575 \times \text{LAI}^2 \] (2)

The interception loss \( (I) \) for every time step is calculated as follows:

\[ I = (I_c)^{t+\Delta t} - (I_c)^{t} \] (3)

where \( t \) is the time and \( \Delta t \) is the time step. Interception rate from the interception loss has been calculated and is deducted from rainfall intensity to get the effective rainfall intensity which is used to calculate infiltration and subsequent runoff.

2.2. Philip Two-Term Infiltration Model

The Philip two term infiltration model is used for computing the infiltration. The rate of infiltration given by Philip in 1957 [19] is as follows:

\[ f = \frac{1}{2} s_i \cdot \frac{r}{\lambda}^{1/2} + K \] (4)

where \( f \) is the infiltration rate as a function of time, \( S_i \) is the infiltration sorptivity and \( K \) is the hydraulic conductivity which is considered equal to the saturated hydraulic conductivity \( (K_s) \). Infiltration sorptivity \( s_i \) [20] can be expressed as follows:

\[ s_i = 2(1 - s_{ini}) \left[ \frac{5 \eta K_s \Psi \Phi(d, s_{ini})}{3 \lambda \pi} \right]^{1/2} \] (5)

where \( s_{ini} \) is the initial (uniform) soil saturation degree in the surface boundary layer, \( \Psi \) is the saturated matrix potential of the soil, \( \Phi(d, s_{ini}) \) is the dimensionless surface sorption diffusivity of the soil, \( \eta \) is the effective porosity of the soil, \( \lambda \) is pore size distribution index and \( d \) is the diffusivity index.

Here \( \Psi \), \( \Phi(d, s_{ini}) \) and \( d \) have been calculated based on the expressions given by Jain et al. [6]. Infiltration rate \( (f) \) has been estimated based on the equations given by Chow et al. [19]. The soil parameter \( S_i \) can be calculated using soil dependent parameters of \( \eta \), \( \lambda \), \( K_s \) and \( s_{ini} \). The standard values of \( \eta \), \( \lambda \), \( K_s \) are available in literature for various soil types. These are used as a first approximation in the model and optimum values can be estimated by calibration. The initial soil saturation degree \( s_{ini} \) has been assumed randomly for each rainfall event and optimal value can be estimated by calibration.

2.3. Governing Equations for Surface Runoff

In a watershed, surface runoff can be divided into overland flow and channel flow. Overland and channel flow
are formulated based on diffusion wave equations to route the runoff to outlet of the watershed.

2.3.1. One Dimensional Diffusion Wave Equation for Overland Flow

The continuity and momentum equations for diffusion wave for overland flow are given as [21]:

\[ \frac{\partial q}{\partial x} + \frac{\partial h}{\partial t} = r_c \]  \hspace{1cm} (6)

\[ \frac{\partial h}{\partial x} = S_o - S_f \]  \hspace{1cm} (7)

where \( q \) is the flow per unit width, \( h \) is the depth of flow; \( r_c \) is the excess rainfall intensity after interception and infiltrations loss, \( x \) is the variable representing space, \( t \) is the variable representing time, \( S_o \) is the slope of overland flow plane and \( S_f \) is the friction slope of flow plane. The flow per unit width is given as \( q = ah^3 \). \( \alpha \) and \( \beta \) can be derived by using Manning’s equation and are given as \( \alpha = \sqrt{S_f} / n_o \) and \( \beta = 5 / 3 \), where \( n_o \) is the overland flow Manning’s roughness coefficient.

The above governing equations are solved using initial and boundary conditions. Initial condition is of no flow condition and it is given as at time \( t = 0; h = 0 \) and \( q = 0 \) at all nodal points. Upstream boundary condition is assumed as zero inflow and it is given as \( h = 0; q = 0 \) at all times \( t \). Downstream boundary condition is of zero depth gradient [22] and it is expressed as \( (sh/ax) = 0 \) at all times \( t \). This condition can also be written with the end node \( M \) as \( h_M = h_{M-1} \).

2.3.2. One Dimensional Diffusion Wave Equation for Channel Flow

Diffusion wave equation for channel flow consists of continuity and momentum equations as:

\[ \frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} - q = 0 \]  \hspace{1cm} (8)

\[ \frac{\partial h}{\partial x} = S - S_{f_c} \]  \hspace{1cm} (9)

where \( Q \) is the discharge in the channel, \( A \) is the area of flow in the channel, \( S \) is the bed slope of channel and \( S_{f_c} \) is the friction slope of channel. \( Q \) in Equation (8) can be represented by the uniform flow equation such as Manning’s equation and is given as follows:

\[ Q = \frac{1}{n_c} R^{2/3} S_{f_c}^{(1/2)} A \]  \hspace{1cm} (10)

where \( R \) is the hydraulic radius \( (A/P)P \) is the wetted perimeter and \( n_c \) is the channel flow Manning’s roughness coefficient. By substituting for \( R \) in Equation (10) gives:

\[ Q = \frac{1}{n_c} \left( \frac{1}{P} \right)^{(2/3)} S_{f_c}^{(1/2)} A^{(5/3)} \]  \hspace{1cm} (11)

Initial condition is given as: at time \( t = 0; Q = 0, A = 0 \) and \( q = 0 \). Upstream boundary condition is of zero inflow and it is given as \( Q = 0 \) and \( A = 0 \). Downstream boundary condition is of zero depth gradient and it is similar to overland flow.

2.3.3. Finite Element Formulation for Overland Flow

Here, one dimensional line elements are used for spatial discretization. Applying the Galerkin finite element formulation [23] to Equation (6) gives:

\[ [C]^{(e)} \{h\}^{t+\Delta t} = [C]^{(e)} \{h\}^t - \Delta t[B]^{(e)} \{(1-\omega)q^t + \omega q^{t+\Delta t} \} \]

\[ +\Delta t\{f\}^{(e)} \{(1-\omega)(r_c)^t + \omega (r_c)^{t+\Delta t} \} \]  \hspace{1cm} (12)

where \( [C]^{(e)}, [B]^{(e)} \) and \( \{f\}^{(e)} \) are elemental matrices. The super scripts \( t \) and \( t + \Delta t \) indicate the variables at the previous time step and current time step. \( \omega \) is the factor that determines the type of finite difference scheme involved. Here Crank-Nicolson scheme with \( \omega = 0.5 \) is used. Equation (12) is applied to all elements in the domain and assembled to form a system of equations. The system of equations is solved by Cholesky scheme after applying the boundary conditions for the unknown values of \( h \). The solution of \( h \) requires iteration due to non-linearity of the Equation (12). Iteration is continued until the convergence is reached to a specified tolerance value \( \varepsilon \). After convergence on \( h \), the time step is incremented and the solution proceeds in the same manner by updating the time matrices and evaluating the new \( h \) values. During calculation of friction slope, the following formulation has to be applied in explicit finite difference form for the Equation (7) [6].

\[ (S_f)_i = (S_o)_i - \frac{h_k - h_i}{L} \]  \hspace{1cm} (13)

where \( i \) and \( k \) represent successive nodes in flow direction and \( L \) is the length of element.

2.3.4. Finite Element Formulation for Channel Flow

Here, continuity Equation (8) is approximated using Galerkin FEM. The final form of the FEM equation which will be used in the channel flow model is as follows:

\[ [C]^{(e)} \{A\}^{t+\Delta t} = [C]^{(e)} \{A\}^t - \Delta t[B]^{(e)} \{(1-\omega)Q^t + \omega Q^{t+\Delta t} \} \]

\[ +\Delta t\{f\}^{(e)} \{(1-\omega)q^t + \omega q^{t+\Delta t} \} \]  \hspace{1cm} (14)

As in the case of overland flow, Equation (14) is applied to all elements and assembled to form a system of equations, which are solved after application of boundary conditions.
2.4. Interflow Model

Interflow model has been formulated based on the through flow equations given by Jayawardena and White [7]. The continuity equation for the interflow is as follows:

\[
\frac{\partial q_i}{\partial x} + \frac{\partial h_i}{\partial t} = I_i
\]

where \( q_i \) is the interflow, \( h_i \) is the saturated layer thickness in which interflow passes, \( \eta \) is the porosity of the interflow zone same as in the infiltration model and \( I_i \) is the lateral recharge rate due to infiltration per unit area. The interflow \( q_i \) can be expressed by Darcy’s law for flow in porous media, in a direction parallel to the overland flow plane slope \( (S_0) \) and is given as \( q_i = K S_0 h_i \). Here \( K \) is the hydraulic conductivity of the interflow zone, same as in infiltration model. Substituting \( q_i \) in Equation (15) gives the interflow as:

2.4.1. Finite Element Formulation

By applying Galerkin finite element formulation to Equation (15), the final form of equation is as follows:

\[
(\eta \{C^{(e)} \})^{(e)} + \{B^{(e)} \} \Delta t \omega KS_0 \} \{h_i\}^{(e)+}\Delta t
= (\eta \{C^{(e)} \} -(1-\omega)\{B^{(e)} \} \Delta t KS_0 \} \{h_i\}^{(e)}
+ \{f_i\}^{(e)} \Delta t (\omega \{I_i\}^{(e)+}\Delta t + (1-\omega)\{I_i\}^{(e)}
\]

As in the case of overland flow, Equation (16) is applied to all elements of overland flow plane and assembled to form a system of equations and solved.

3. Model Development and Evaluation

Based on the above formulation, computer models were developed for interception model, Philip infiltration model, surface runoff routing model based on diffusion wave formulation and interflow model, using C programming language. Further these models are integrated together for the prediction of runoff at any location for the given rainfall conditions. Infiltration from the Philip model is the input to the interflow model assuming that some part of infiltration flows towards the stream through a saturated zone and contributes to the runoff. Finite element mesh and input data required for the integrated model has been prepared using the GIS.

3.1. Model Evaluation

The developed integrated model has been evaluated on the Peacheater Creek watershed in USA. The data required for Peacheater Creek watershed is available in Distributed Model Intercomparison Project (DMIP) website (http://www.nws.noaa.gov/oh/hrl/dmip/index.html). Some data like watershed boundary, stream flow data and gauging station information are obtained through personal communication (Michael B. Smith and Seann M. Reed, DMIP, National Weather Service Office of Hydrologic Development, USA, 2005).

3.2. Study Area and Preparation of Database

The Peacheater creek watershed is one of the study watersheds of DMIP. It has an area of 63.58 km². It is a gauged sub-watershed of Baron Fork catchment, Oklahoma, USA. Gauging station for this watershed is located at Christie, Oklahoma. The watershed consists primarily of silt loam soils with forested (42%), grassy (57%) and urban (1%) areas [24]. ASCII files of DEM (1 arc-second), soil and land use map of the Baron Fork basin with 30 m grid resolution were converted into map format with appropriate projection information. Required DEM, soil and land use maps for Peacheater creek watershed are clipped, based on the boundary of the watershed. DEM of the watershed is shown in Figure 1. Drainage map of the watershed is generated based on DEM using Hydrology tools of ArcMap. Percentage slope map is prepared from DEM by using the slope option of Spatial Analyst in ArcMap. The percentage slope is then converted to slope values by Raster Calculator option in GIS. Hourly rainfall data in binary format were obtained from DMIP website (Stage 3 Next-Generation Weather Radar (NEXRAD) observations at 4x4 km resolution). Based on the procedure explained in the DMIP website, the hourly rainfall maps are prepared and clipped for the watershed.
Figure 2. Peacheater creek watershed with FEM grid.

Table 1. Calibrated parameters for rainfall events.

| Event          | Saturated hydraulic conductivity $K_s$ (cm/hour) | Initial degree of soil saturation ($S_{ini}$) | Overland flow Manning's roughness coefficient ($n_o$) | Channel flow Manning's roughness coefficient ($n_c$) |
|----------------|-----------------------------------------------|---------------------------------------------|-------------------------------------------------|-------------------------------------------------|
| 06 February 1999 | 0.609                                         | 0.2                                         | 0.592                                           | 0.085                                           |
| 17 May 1999     | 0.525                                         | 0.25                                        | 0.19                                            | 0.05                                            |
| 28 June 2000    | 0.55                                          | 0.2                                        | 0.52                                            | 0.18                                            |
| 23 February 2001| 0.25                                          | 0.28                                        | 0.25                                            | 0.05                                            |

FEM grid map of the watershed is prepared with overland flow strips with element size of 500x500m, as shown in Figure 2. The grid map has been overlaid on slope map. By using the Zonal Statistics option in the Spatial Analyst module of ArcMap, the mean value of slope has been calculated for each element of the grid. The attribute table of the grid containing the element number and mean value of slope has been exported as database file. The present model needs input data of slope at the nodal level. It is obtained by taking average of adjacent element values.

3.3. Model Simulation

The developed model has been applied to simulate the runoff in Peacheater Creek watershed for six rainfall events. A constant channel width of 45 m, slope 0.006, time step 30 s, $\eta$ of 0.416 and $\lambda$ of 0.28 have been used in the simulation of rainfall events. In view of the non availability of data for the interception model, $LAI$ of 1.5 and $c_p$ of 0.5 have been assumed.

The model has been calibrated for four rainfall events
Table 2. Model results for the calibration storms.

| Date of rainfall event | Volume of runoff (mm) | Peak runoff (m³/sec) | Time to peak runoff (min) |
|------------------------|-----------------------|----------------------|--------------------------|
|                        | Observed | Simulated | Observed | Simulated | Observed | Simulated | Observed | Simulated |
| 06 February 1999       | 2.025    | 1.590     | 2.668    | 2.679     | 480      | 395.5     |
| 17 May 1999            | 1.001    | 0.764     | 1.071    | 1.054     | 540      | 652       |
| 28 June 2000           | 28.738   | 17.577    | 38.844   | 39.987    | 600      | 608       |
| 23 February 2001       | 35.033   | 15.299    | 27.431   | 25.982    | 1680     | 1723      |

Table 3. Model results for the validation storms.

| Date of rainfall event | Volume of runoff (mm) | Peak runoff (m³/sec) | Time to peak runoff (min) |
|------------------------|-----------------------|----------------------|--------------------------|
|                        | Observed | Simulated | Observed | Simulated | Observed | Simulated | Observed | Simulated |
| 4 May 1999             | 6.477    | 3.300     | 10.731   | 8.330     | 900      | 940.5     |
| 21 June 2000           | 57.673   | 29.035    | 135.159  | 61.971    | 720      | 577.5     |

Figure 3. Observed and simulated hydrographs for calibration events; (a) 06 February 1999; (b) 17 May 1999; (c) 28 June 2000; (d) 23 February 2001.

of Peacheater Creek watershed. Validation of model has been carried out with two rainfall events. Calibration has been performed by altering the values of $K_s$ and $S_{sat}$ by trial and error based on the best visual fit of the hydrographs. The parameters $n_a$ and $n_r$ have been calibrated in the absence of data. Calibrated parameters for Peacheater Creek watershed are given in Table 1. The model fit was checked based on the difference between observed and computed volume of runoff, peak runoff and time to peak. Validation of two rainfall events of Peacheater watershed is carried out by taking average parameters of four calibrated rainfall events.

4. Results and Discussions

The observed and simulated hydrographs for the four calibrated rainfall events of Peacheater Creek watershed are shown Figure 3. Table 2 shows the observed and
simulated values of volume of runoff, peak runoff and time to peak runoff for the four calibrated events. From the simulation results, it is seen that volume of flow has been simulated within variation of ± 56% and peak flow within variation of ± 6% and time to peak within a variation of ± 21%.

The observed and simulated hydrographs for the two validated events of Peacheater Creek watershed are shown in Figure 4. Table 3 shows the observed and simulated values of volume of runoff, peak runoff and time to peak runoff for the two validated events. From the validation of simulation results, it is seen that volume of flow has been simulated within a variation of ± 50% and peak flow within a variation of ± 54% and time to peak within a variation of ± 20%.

Topographic complexity in the Peacheater Creek controls the catchment response to rainfall via interactions between the active shallow aquifer, stream network and land surface [24]. Even though the present model has been simulating interflow, it is unable to simulate perched return flow and ground water exfiltration which are important hydrological processes of this watershed. In addition, the channel slope and width values are assumed to be constant throughout the length of the channel, because of lack of data, and weighted average rainfall for the whole watershed has been used in the model. These factors might have contributed to the discrepancy between the observed and simulated flow.

Also a sensitivity analysis of the model has been carried out by altering the parameters of $K_s$, $S_{ini}$, $n_s$, and $n_c$ by ± 10%. Effect of change in calibrated parameters on computed values of volume of runoff, peak runoff and time to peak runoff are shown for a rainfall event in Figure 5. From the results, it is observed that the values of computed volume of runoff and peak runoff are most sensitive to $K_s$ followed by $n_c$, $n_s$, and $S_{ini}$. The value of computed time to peak runoff is most sensitive to $n_c$ followed by $n_s$, $K_s$, and $S_{ini}$. The change of ± 10% in the parameter $K_s$ caused a variation of 12% in the volume of runoff and peak runoff and 0.5% variation in the time to peak. The change of ± 10% in the parameter $n_c$ caused a variation of 6% in the volume of runoff and peak runoff and 1.7% variation in the time to peak. The change of ± 10% in the parameter $n_s$ caused a variation of 0.9% in the volume of runoff and 4.4% in the peak runoff and 2.4% variation in the time to peak. It indicates that volume of runoff is more sensitive to the infiltration parameters than flow resistance parameters and time to peak is more sensitive to the flow resistance parameters than infiltration parameters. It is also seen that peak runoff is sensitive to both infiltration and flow resistance parameters. The change of ± 10% in the parameter $S_{ini}$ caused a variation of 1% in the volume of runoff and peak runoff and 0.1% variation in the time to peak. It indicates that volume of runoff, peak runoff and time to peak runoff are least sensitive to $S_{ini}$. This may be true since most of the rainfall events simulated are long duration events where initial saturation conditions may not have much influence on the flow parameters. However, it is observed from the sensitivity analysis that all the parameters considered are important in the simulation of runoff and accuracy of these parameters control the accuracy of the simulation results.

5. Concluding Remarks

An integrated FEM and GIS based rainfall runoff model using diffusion wave equation is presented here. Interception has been calculated by an empirical model based on Leaf Area Index (LAI). Philip two-term infiltration model is used for estimation of infiltration. Interflow model has been developed using FEM. The developed model has been calibrated and validated on Peacheater Creek watershed in USA.

The developed model has fairly simulated the hydrographs at the outlet of watershed. From the simulation results of watershed, it is seen that volume of runoff has been simulated within variation of ± 56%, peak runoff within variation of ± 6% and time to peak runoff within variation of ± 21% for calibrated rainfall events.
Model performed well on some of the validation rainfall events. This may be primarily due to differences in characteristics between the calibration storms and the validation storms. Large variation in simulated volume of runoff for some of the rainfall events may be because of simulated interflow not representing the perched return flow and ground water exfiltration which are important hydrological processes of this watershed. Further, sensitivity analysis for various parameters has been carried out. From the sensitivity analysis results, it is seen that volume of runoff is more sensitive to the infiltration parameters than flow resistance parameters and time to peak is more sensitive to the flow resistance parameters than infiltration parameters. It is also observed that all the infiltration and flow resistance parameters considered in the model are important in the simulation of runoff. The model in its present form is not suitable during hiatus and prolonged interstorm periods because it doesn’t account for moisture balance and evapotranspiration losses. In the present study calibrated values of infiltration and flow resistance parameters based on the observed hydrographs have been used in the simulation. The present study, demonstrated the application of GIS in simulation of runoff by integration of FEM based surface runoff and interflow models and conceptually established interception and infiltration models. The model presented here can be applied to agricultural and forested watersheds since this model will take care of interflow and interception. The model can be applied to small to medium watersheds for the event based runoff simulation. Due to lack of availability of suitable data, the model applicability for larger watersheds could not be carried out.

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A Hydrogeophysical Model of the Relationship between Geoelectric and Hydraulic Parameters, Central Jordan

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Abstract

Geoelectrical soundings using the Schlumberger array were carried out in the vicinity of 23 pumping test sites to determine aquifer parameters, central Jordan. On the basis of aquifer geometry, the area has been divided into two hydraulic units: the northern flood plain and the flood plain to its south. Field resistivity data are interpreted in terms of the true resistivity and thickness of subsurface layers. These parameters are then correlated with the available pumping test data. Significant correlations between the transmissivity and modified transverse resistance as well as between the hydraulic conductivity and formation factor were obtained for the two hydraulic units, in central Jordan are presented here.

Keywords: Aquifer Parameters, Electrical Parameters, Geoelectrical Soundings, Jordan

1. Introduction

The development of groundwater resources and the regime of its activity largely depend on the porosity and permeability of water bearing formations. The porosity of rock is a measure of the amount of interstitial space that is capable of holding fluids and the permeability (hydraulic conductivity) of a rock is a quantitative measure of the case with which it will permit the passage of fluids through it under a hydraulic gradient. The determination of aquifer characteristics such as hydraulic conductivity and transmissivity is best made on the basis of data obtained from test pumping wells. These properties are important in determining the natural flow of water through an aquifer and its response to fluid extraction. This paper examines the influence of aquifer anisotropy on the relationship between hydraulic and geoelectrical parameters of aquifers that are needed to develop a hydrogeophysical model for an anisotropic aquifer in parts of central Jordan.

An alternative approach for estimating aquifer characteristics is the use of surface geoelectrical methods. Many investigators have studied the relationship between electric and hydraulic parameters of aquifers. Jones and Buford [1] measured the formation factor and intrinsic permeability of sand samples and found that as the grain size increase, the formation factor and intrinsic permeability also increases. A relation between the aquifer intrinsic permeability and formation factor was developed for a given porosity range [2]. The resistivity and the formation factor of an aquifer have been correlated with the permeability [3]. Empirical and semi-empirical relations between different aquifer parameters and the parameters obtained by geoelectrical soundings under different geological conditions have also been studied by others [4–13]. The analytical relations between aquifer transmissivity and Dar-Zarrouk parameters have been developed and various data sets tested [14,15]. An inverse relationship between porosity and hydraulic conductivity were used to explain the direct correlations between formation factor and hydraulic conductivity [16,17]. Here we present the Schlumberger sounding results in the area of central Jordan to define the aquifer geometry of the study area.

2. Geology and Hydrogeology

2.1. Geological Setting

Physiographically, the study area lies between latitude 31° 29.54’ N to 31° 45.03’ N and 35° 59.58’ E to 36° 14.56’ E, central Jordan (Figure 1). Jaser [18] has given the detailed geology of the area. Bedrock in the investigated area is of sedimentary origin and of Upper Cretaceous to recent age (Figure 1). The oldest outcrops in the area are the Amman Silicified Limestone (ASL) Formation (of Campanian age) of the Balqa Group. In the mapped area (Figure 1), the rock formation consists of...
Figure 1. Geology map of the area (after Jaser [18]) and positions of geoelectrical soundings and wells.
Table 1. Stratigraphic column for the geology of northern Jordan (after Rimawi et al. [19]).

| Period       | Epoch       | Formation                                      | Symbol | Group      | Aquifer       |
|--------------|-------------|-----------------------------------------------|--------|------------|---------------|
| Tertiary     | Oligocene   | Basalt (BS)                                   | BS     | Jordan Valley | Shallow Aquifer Complex |
|              | Eocene      | Wadi Shallala (WS) (Limestone and Chalky Limestone) | B5     |            |               |
|              | Paleocene   | Umm Rijam Chert-Limestone (URC)               | B4     |            |               |
|              | Maastrichtian | Muwaqqar Chalk-Marl (MCM)                  | B3     | Balqa      | B3 Aquitard   |
|              | Campanian   | Amman Silicified Limestone (ASL)              | B2     |            |               |
|              | Santonian   | Wadi Umm Ghudran (WG) Marl and Marly Limestone | B1     |            |               |
|              | Turonian    | Wadi Es-Sir Limestone (WSL)                   | A7     |            |               |
|              | Cenomanian  | Shua‘yb (Echinoidal limestone)                | A5/6   |            |               |
|              |             | Hummar (Echinoidal Limestone)                 | A4     | Ajlun       |               |
|              |             | Fuheis (Nodular Limestone)                    | A3     |            |               |
|              |             | Na‘ur (Nodular Limestone)                     | A1/2   |            |               |
|              | Lower Cretaceous | Subeihi (Vary Colored Sandstone) | K2     | Kurnub Sandstone | Deep Sandstone Aquifer Complex |
|              | Aptian-Neocomian | Arda’a (White Sandstone)                  | K1     |            |               |

Table 2. Summary of results from computer modeling for all sounding stations.

| VES no. | Resistivity of layers ($\Omega$m) | Thickness of layers (m) |
|---------|----------------------------------|-------------------------|
|         | $\rho_1$ $\rho_2$ $\rho_3$ $\rho_4$ $\rho_5$ $\rho_6$ $h_1$ $h_2$ $h_3$ $h_4$ $h_5$ |
| 1       | 22 14 32 60 80 - 1.9 2.5 141 66 - |
| 02      | 25 3 9 35 65 - 0.9 3.9 141 74 - |
| 02      | 17 11 18 27 72 - 1.4 2.0 173 64 - |
| 04      | 29 14 19 31 54 - 1.3 5.7 165 44 - |
| 05      | 18 12 19 34 112 - 1.8 8.7 133 68 - |
| 06      | 37 18 38 81 190 - 1.1 3.9 133 56 - |
| 07      | 26 17 27 78 133 - 1.1 6.4 181 42 - |
| 08      | 33 19 51 68 90 - 1.3 4.7 105 52 - |
| 19      | 31 16 64 93 168 - 1.5 13 140 56 - |
| 10      | 30 5 21 36 90 - 1.7 3.9 116 69 - |
| 11      | 9 15 23 60 - - 1.1 30 200 - - |
| 12      | 28 5 27 68 23 180 1.9 2.5 64 74 40 |
| 13      | 19 13 26 40 108 - 1.5 5.6 142 71 - |
| 14      | 24 16 32 42 64 - 1.0 10 179 40 - |
| 15      | 34 23 35 47 76 - 1.7 3.5 169 67 - |
| 16      | 13 4 18 51 - - 0.5 151 68 - - |
| 17      | 35 9 46 80 - - 0.4 146 72 - - |
| 18      | 62 11 31 69 34 152 1.7 2.3 71 83 48 |
| 19      | 54 6 36 48 - - 0.4 136 54 - - |
| 20      | 24 12 39 103 - - 0.5 163 37 - - |
| 21      | 30 20 59 156 - - 0.9 159 60 - - |
| 22      | 48 32 48 57 73 - - 0.9 3.1 38 150 - - |
| 23      | 37 19 84 112 - - 1.0 154 65 - - |

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limestone, chalk and chalky limestone. The overlying Al-Hisa Phosphorite (AHP) Formation, which belongs to the Balqa Group (Maestrichtian-Campanian in age) consists of limestone and phosphate with chalk and chalky marl units. These are overlain by sediments consisting of marl, chalk and chalky marl of the Muwaqqar Chalk-Marl (MCM) Formation belonging to the Balqa Group of Maestrichtian to Paleocene age. The overlying chalky limestone and chert of the Umm Rijam Chert-Limestone (URC) Formation (Eocene in age) belongs to Balqa Group. Surficial deposits are: marl, clay, sand and gravels of Holocene to Recent age.

2.2. Hydrogeology

Generally, the groundwater aquifers of Jordan are divided into three hydraulic complexes: Deep Sandstone Aquifer Complex, Upper Cretaceous Aquifer Complex, and Shallow Aquifer Complex (Table 1). Although this study is concentrated on the Upper Cretaceous Aquifers (namely B2/A7 aquifer), it is important to indicate the significant role of the Deep Sandstone Aquifer Complex where they occur in the adjacent highlands. They may contribute to the recharge of the Upper Cretaceous aquifers as upward leakage [19].

Based on the lithology log data of 23 wells from the study area, we divide the area into two distinct hydraulic units: Hydraulic unit-1 towards the northern and central parts comprised of almost sorted material (mainly limestone) with low uniformity coefficient and another unit comprising of unsorted materials (limestone and chert) with relatively high uniformity coefficient occurring towards the southern parts (Hydraulic unit-2) (Figure 1).

3. Field Studies

Surface resistivity methods have been used in groundwater research for many years. Earth resistivities are related to important geologic parameters of the subsurface including types of rocks and soils, porosity, and degree of saturation. The detailed description of this method is available in [20]. In general, the resistivity method involves measuring the electrical resistivity of earth materials by introducing an electrical current into the ground and monitoring the potential field developed by the current. The most commonly used electrode configuration for geoelectrical soundings, and the one used in this field survey, is the Schlumberger array. Four electrodes (two current A and B and two potential M and N) are placed along a straight line on the land surface such that the outside (current) electrode distance (AB) is equal to or greater than five times the inside (potential) electrode distance (MN). Vertical sounding, in Schlumberger array, were performed by keeping the electrode array centered over a field station while increasing the spacing between the current electrodes, thus increasing the depth of investigation.

The potential difference (ΔV) and the electrical current (I) are measured for each electrode spacing and the apparent resistivity (ρa) is calculated by the equation:

$$\rho_a = K \frac{\Delta V}{I} \quad \text{(ohm-m)} \quad (1)$$

where

$$K = \pi \frac{AM \cdot AN}{MN} \quad (2)$$

is the geometrical factor that depends on the electrode arrangement for the Schlumberger array.

An integrated approach of hydrogeological and geoelectrical soundings surveys has been used to study the relationship between the geoelectric and hydraulic parameters in the central part of Jordan. Data from 23 deep wells are available, on which pumping tests have been conducted. The pumping test data were analyzed and the aquifer hydraulic parameters (hydraulic conductivity, transmissivity and water resistivity) have been evaluated by Water Authority of Jordan in 2006. The geophysical field work in this study included recording of 23 vertical electrical soundings (VES) carried out in the fall of 2006. The VES were recorded up to a maximum electrode separation of 2000 m. The VES soundings were conducted with the help of Iris Syscal R2 resistivity instrument in the close vicinity of deep wells as shown in Figure 1.

A preliminary interpretation of the sounding curves using partial curve matching [21] provides the initial estimates of the resistivities and thickness (layer parameters) of the various geoelectric layers. The layer parameters derived from the graphical curve matching were then used to interpret the sounding data in terms of the final layer parameters through a 1-D inversion technique (RESIX-IP, Interpex Limited, Golden, Co., USA). Inversion analyses of the sounding curves have been made with an average fitting error of about 5%. Quantitative interpretation of geoelectrical sounding curves is complicated due to the well known principle of equivalence [22]. Data from the two boreholes (CD1245 and CD1232, Figure 1) was used to minimize the choice of equivalent models, by fixing thicknesses and depths to certain levels and allowing the adjustment of resistivity. Correlation between VES interpretation at stations 8 and 19 and borehole lithology determines the electrical characteristics of the rock units with depth (Figure 2). Table 2 presents the results of interpretation of the VES stations.

Figure 3 is a typical sounding data plot and best-fit four-layer model for one selected sounding data. On the left, Figure 2 shows the Schlumberger apparent resistivity curve with data (points) superimposed on the best match 1-D inversion (solid line). On the right the figures shows the interpreted results in terms of resistivity and

---

**Table 1**

| Hydrogeological Unit | Lithologic Characteristics |
|----------------------|----------------------------|
| Hydraulic unit-1     | Sorted material (mainly limestone) with low uniformity coefficient |
| Hydraulic unit-2     | Unsorted materials (limestone and chert) with relatively high uniformity coefficient |

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Figure 2. Comparison of interpreted VES field curves and nearby test-hole logs for two selected sites.

Figure 3. Typical electrical resistivity sounding data and best-fit four layer model interpretation for VES11.

depth together with the allowable range of equivalence (dashed lines). The result of the 1-D inversion of Figure 2 (right) shows a thin topsoil layer about 1 m thick, below which is a 30 m band of chalk and marl representing the upper of the MCM formation with a resistivity value of about 15 Ωm. The third unit has higher resistivity values (23 Ωm) and considerably thick (about 200 m). This layer represents the lower of the MCM formation. The fourth unit has resistivity values of 60 Ωm, representing the saturated zone.

4. Hydraulic Parameters Versus Geoelectric

Figure 4 shows an analogy between the electrical current flow and groundwater flow in layered media. If the flow of electric current is parallel to the geological layering and hydraulic flow [20], the average horizontal hydraulic conductivity ($k_h$) is given as (3):

$$k_h = \frac{\frac{\rho_i}{h_i}}{\frac{\rho_i}{h_i}}$$

And average longitudinal resistivity ($\rho_l$) is given as (4):

$$\rho_l = \frac{\rho_i}{h_i}$$

where $\rho_i$ and $h_i$ are resistivity and thicknesses of $i^{th}$ layer, respectively. The transverse resistivity ($\rho_t$) of the aquifer is determined from the layer parameters as (5):

$$\rho_t = \frac{\rho_i}{h_i}$$

The use of electrical parameters obtained by multiplying the transverse resistance with the modification factor (ratio of average aquifer water resistivity and resistivity of water at a particular site) has been suggested by [14,15]. This approach has been used for the 23 sites using the value of aquifer water resistivity (measured from collected groundwater samples), an average aquifer water resistivity (6.28 ohm-m) and modified transverse resistance ($R'$) have been calculated. Figure 5 shows a scatter plot of transmissivity ($T$) and modified transverse resistance ($R'$). The following linear relationships are obtained (6):

$$T = 0.00096 \cdot R' + 41.56$$

However, when the values are sorted on the basis of hydraulic units 1 and 2, the plot (Figure 5) shows two lines with lesser scatter. The linear relationship for hydraulic unit-1 takes the form of (7):

$$T = 0.0027 \cdot R' + 21.58$$

And for hydraulic unit-2, the relationship is (8):

$$T = 0.0003 \cdot R' + 46.38$$

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formation factor \((F)\) and hydraulic conductivity \((K)\) has been proposed \([4]\) as (9):

\[
K = A \cdot F^m
\]  

(9)

The plot of empirical relationship (Equation 9) is shown in Figure 6 for the actual field data. The values of the coefficient \((A)\) and the exponent \((m)\) in Equation (9) are found to be 0.04 and 0.75, respectively. Substituting the values of \(A\) and \(m\), Equation (9) can be written as (10):

\[
K = 0.04 \cdot F^{0.75}
\]  

(10)

The values of field data are sorted on the basis of hydraulic units 1 and 2, the plot (Figure 6) shows two lines with lesser scatter. The empirical relationship for hydraulic unit-1 takes the form of (11):

\[
K = 0.02 \cdot F^{0.90}
\]  

(11)

And for hydraulic unit-2, the relationship is (12):

\[
K = 0.24 \cdot F^{0.90}
\]  

(12)

Equations (10)-(12) are used to compute hydraulic conductivity and the calculated values were compared with the observed data in Table 4. It is observed that the values computed from Equations (11) and (12) are generally closer to measured values in comparison to those computed from Equation (10).

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Table 3. Observed and computed transmissivity values using modified transverse resistance in different equations for all twenty three sites.

| VES no. | Modified transverse resistance (ohm-m) | Observed transmissivity (m²/day) | Calculated transmissivity: \(T = 0.00096R' + 41.56\) (m²/day) | Calculated transmissivity: \(T = 0.0027R' + 21.58\) (m²/day) | Calculated transmissivity: \(T = 0.0003R' + 46.38\) (m²/day) | Hydraulic unit |
|---------|--------------------------------------|-------------------------------|-------------------------------------------------|-------------------------------------------------|-------------------------------------------------|----------------|
| 01      | 11939                               | 51                            | 53.82                                           | 53.82                                           | -                                               | 1              |
| 02      | 8699                                | 42                            | 49.91                                           | 45.07                                           | -                                               | 1              |
| 03      | 16180                               | 57                            | 57.09                                           | 65.27                                           | -                                               | 1              |
| 04      | 6466                                | 39                            | 47.77                                           | 39.04                                           | -                                               | 1              |
| 05      | 13262                               | 59                            | 54.29                                           | 57.39                                           | -                                               | 1              |
| 06      | 20234                               | 77                            | 60.78                                           | 76.21                                           | -                                               | 1              |
| 07      | 19590                               | 75                            | 60.37                                           | 74.47                                           | -                                               | 1              |
| 08      | 14718                               | 66                            | 55.69                                           | 61.32                                           | -                                               | 1              |
| 09      | 30841                               | 100                           | 71.17                                           | 104.85                                          | -                                               | 1              |
| 10      | 13904                               | 65                            | 54.91                                           | 59.12                                           | -                                               | 1              |
| 12      | 16922                               | 77                            | 57.81                                           | 67.27                                           | -                                               | 1              |
| 13      | 12092                               | 48                            | 53.17                                           | 54.23                                           | -                                               | 1              |
| 14      | 11097                               | 49                            | 52.21                                           | 51.54                                           | -                                               | 1              |
| 18      | 15221                               | 60                            | 56.17                                           | 62.68                                           | -                                               | 1              |
| 11      | 29824                               | 55                            | 70.19                                           | -                                               | 55.33                                           | 2              |
| 15      | 18453                               | 52                            | 59.27                                           | -                                               | 51.92                                           | 2              |
| 16      | 13929                               | 51                            | 54.93                                           | -                                               | 50.56                                           | 2              |
| 17      | 18997                               | 51                            | 59.80                                           | -                                               | 52.08                                           | 2              |
| 19      | 10348                               | 50                            | 51.49                                           | -                                               | 49.48                                           | 2              |
| 20      | 22999                               | 52                            | 63.64                                           | -                                               | 53.28                                           | 2              |
| 21      | 35111                               | 58                            | 75.27                                           | -                                               | 56.91                                           | 2              |
| 22      | 16679                               | 52                            | 57.57                                           | -                                               | 51.38                                           | 2              |
| 23      | 21274                               | 53                            | 61.98                                           | -                                               | 52.76                                           | 2              |
Table 4. Hydraulic conductivity from pumping test data, formation factor derived from interpreted resistivity models, along with computed hydraulic conductivity values in different equations for all twenty three sites.

| VES no. | Hydraulic Conductivity (m/day) | Formation Factor | Calculated Hydraulic Conductivity Eq. (10) (m/day) $K = 0.04F^{0.75}$ | Calculated Hydraulic Conductivity Eq. (11) (m/day) $K = 0.02F^{0.99}$ | Calculated Hydraulic Conductivity Eq. (12) (m/day) $K = 0.24F^{0.09}$ | Hydraulic Unit |
|---------|-------------------------------|------------------|--------------------------|--------------------------|--------------------------|----------------|
| 01      | 0.28                          | 10.48            | 0.23                     | 0.21                     | -                        | 1              |
| 02      | 0.13                          | 8.19             | 0.19                     | 0.16                     | -                        | 1              |
| 03      | 0.12                          | 9.55             | 0.22                     | 0.18                     | -                        | 1              |
| 04      | 0.13                          | 7.21             | 0.18                     | 0.14                     | -                        | 1              |
| 05      | 0.42                          | 14.91            | 0.30                     | 0.29                     | -                        | 1              |
| 06      | 0.61                          | 25.57            | 0.45                     | 0.50                     | -                        | 1              |
| 07      | 0.35                          | 22.09            | 0.41                     | 0.42                     | -                        | 1              |
| 08      | 0.24                          | 15.85            | 0.32                     | 0.31                     | -                        | 1              |
| 09      | 0.46                          | 28.67            | 0.50                     | 0.55                     | -                        | 1              |
| 10      | 0.47                          | 16.04            | 0.32                     | 0.31                     | -                        | 1              |
| 11      | 0.45                          | 15.75            | 0.32                     | 0.31                     | -                        | 1              |
| 12      | 0.26                          | 13.17            | 0.28                     | 0.26                     | -                        | 1              |
| 13      | 0.19                          | 11.41            | 0.25                     | 0.22                     | -                        | 1              |
| 14      | 0.56                          | 38.58            | 0.62                     | 0.74                     | -                        | 1              |
| 15      | 0.34                          | 26.77            | 0.47                     | -                        | 0.32                     | 2              |
| 16      | 0.32                          | 15.61            | 0.31                     | -                        | 0.31                     | 2              |
| 17      | 0.35                          | 12.81            | 0.27                     | -                        | 0.31                     | 2              |
| 18      | 0.30                          | 17.86            | 0.35                     | -                        | 0.31                     | 2              |
| 19      | 0.29                          | 9.62             | 0.23                     | -                        | 0.29                     | 2              |
| 20      | 0.28                          | 20.89            | 0.39                     | -                        | 0.31                     | 2              |
| 21      | 0.34                          | 31.52            | 0.53                     | -                        | 0.33                     | 2              |
| 22      | 0.27                          | 14.20            | 0.29                     | -                        | 0.30                     | 2              |
| 23      | 0.29                          | 18.30            | 0.35                     | -                        | 0.31                     | 2              |

Figure 6. Empirical relation between hydraulic conductivity and formation factor. Solid line represents the relationship when both hydraulic units are combined to one unit. Dashed lines represent the relationships for hydraulic unit-1 and unit-2.

5. Conclusions

Geoelectrical surveys, using the Schlumberger array configuration, were carried out in the vicinity of 23 pumping test sites, central Jordan, with an aim to relate geoelectric properties to hydraulic parameters. The present study suggests that aquifer transmissivity and hydraulic conductivity can be estimated more accurately if the values are sorted by hydraulic units. It can be inferred from the study that the geoelectrical sounding method can be successively used not only for exploration of groundwater but also for estimating the hydraulic parameters of the groundwater aquifer.

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Effects of Light and Monosulfuron on Growth and Photosynthetic Pigments of Anabaena Flos-Aquae Breb

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Abstract

The effects of monosulfuron on growth and photosynthetic pigments of the nitrogen-fixing cyanobacterium *Anabaena flos-aquae* grown exposed to 2000-, 3000-, and 4000-lux light intensity were studied. Exposed to three light intensities, the seven concentrations of monosulfuron tested can significantly inhibit algal growth in a dose-dependent manner. The cell numbers and growth rate were decreased with the increase in monosulfuron concentration, and *A. flos-aquae* had different degrees of sensitivity to monosulfuron with the most sensitive light intensity being 4000-lux followed by 3000-lux and 2000-lux. The herbicide monosulfuron appeared to have different effects on the synthesis of photosynthetic pigments. The chlorophyll appeared to tackle monosulfuron concentrations. The carotenoid content of algae treated with 0.008 and 0.08 mg/L monosulfuron exposed to 2000-lux had a different stimulatory effect from that of treatments exposed to 3000-lux and 4000-lux, but an inhibitory effect at concentration above 0.8 mg/L. The effect of monosulfuron on biliprotein in cells of *A. flos-aquae* exposed three light intensities displayed contrary dose dependence.

Keywords: *Anabaena Flos-Aquae*, Growth, Light, Monosulfuron, Photosynthetic Pigments

1. Introduction

Effects of herbicides on non-target organisms in the soil ecosystem such as microorganisms have recently been paid great attention. Soil algae, particularly nitrogen-fixing cyanobacteria, are important photosynthetic microorganisms because they contribute to soil fertility by fixing atmospheric nitrogen. They are sensitive to herbicides because they have many characteristics of higher plants [1]. Many effects of herbicides on non-target algae, such as algal growth, photosynthesis, nitrogen fixation, and metabolic activities have been reported [2]. The previous studies, however, were conducted exposed to standard phototoxicity test condition involving only the results of photoautotrophic growth of algae. In fact, in the fields, the algae grown in soil or inland waters is mixotrophic rather than autotrophic [3], and herbicide toxicity to algae is affected by many environmental factors such as nutrient level, temperature, and light. However, little is known about the specific roles of these environmental factors, particularly, the effect of light on herbicide toxicity to algal growth.

Monosulfuron {\( N\{- (4'\text{-methyl}) \text{ pyrimidin}-2'\text{-yl} \} \text{-2-nitrophenylsulfonyl urea} \) is a relatively new sulfonylurea herbicide that was developed by the National Pesticide Engineering Research Center in Tianjin, China [4]. This herbicide exhibits low mammalian toxicity and is very effective at post-emergence rates of 15 to 30 g ai./ha in a wide range of crops including corn (*Zea mays* L.), wheat (*Triticum aestivum* L.), rice (*Oryza sativa* L.), and millet (*Panicum milacieum* L.) [5]. Currently, over 30 sulfonylurea herbicides are applied worldwide for selective control of weeds in a variety of crops including corn, rice, wheat, and potatoes (*Solanum muricatum* Oit.) [6]. However, the effect of sulfonylurea herbicides on nitrogen-fixing cyanobacteria capable of enhancing the fertility of agricultural soils has not been investigated. *Anabaena flos-aquae* (Lyngb.) Breb., a filamentous nitrogen-fixing cyanobacterium, is one of the most common members of the soil algae community, capable not only of phototrophic growth but also of mixotrophic growth [7]. This work investigated the mixotrophic growth, chlorophyll a, carotenoid, and biliprotein of *A. flos-aquae* responded to the herbicide monosulfuron exposed to three light intensity conditions, attempting to compare and assess the effect of light on this herbicide toxicity.

2. Materials and Methods

Culture of *Anabaena flos-aquae* (FACHB-245) was obtained from the Institute of Hydrobiology of the Chinese
Table 1. Cell numbers of *A. flos-aquae* treated with monosulfuron exposed to different light intensities.

| Light intensity | Concentration | 24h | 48h | 72h | 96h | 120h | 144h |
|----------------|---------------|-----|-----|-----|-----|------|------|
| lux | mg/L | Cell number (x1000/ml) |
|-----|-----|------------------|
| 2000 | Control | 0.99 | 27.99 | 63.70 | 140.82 | 128.24 | 168.02 |
| | 0.008 | 5.05 | 27.17 | 84.61 | 170.86 | 157.06 | 179.38 |
| | 0.08 | 1.79 | 26.36 | 80.75 | 165.99 | 134.73 | 201.91 |
| | 0.8 | 2.62 | 30.22 | 63.50 | 151.98 | 69.79 | 126.62 |
| | 8 | 1.59 | 3.02 | 16.27 | 21.09 | 18.65 | 6.47 |
| | 80 | 1.40 | 3.02 | 15.00 | 20.74 | 15.00 | 3.84 |
| | 800 | 0.99 | 2.23 | 15.00 | 19.90 | 10.81 | 2.21 |
| 3000 | Control | 3.02 | 43.00 | 98.20 | 191.15 | 223.62 | 259.75 |
| | 0.008 | 8.91 | 37.93 | 108.15 | 170.65 | 153.71 | 220.78 |
| | 0.08 | 17.84 | 68.17 | 127.22 | 172.08 | 169.64 | 233.16 |
| | 0.8 | 3.62 | 47.87 | 107.95 | 171.26 | 146.50 | 211.45 |
| | 8 | 3.20 | 2.01 | 13.17 | 36.71 | 22.10 | 14.19 |
| | 80 | 3.09 | 4.44 | 11.90 | 34.79 | 23.32 | 13.17 |
| | 800 | 3.02 | 2.41 | 10.00 | 34.48 | 21.49 | 6.68 |
| 4000 | Control | 0.59 | 19.06 | 71.21 | 188.51 | 132.91 | 244.53 |
| | 0.008 | 0.99 | 16.47 | 73.65 | 117.89 | 92.12 | 152.39 |
| | 0.08 | 3.84 | 21.09 | 72.02 | 131.69 | 130.04 | 186.69 |
| | 0.8 | 1.81 | 25.55 | 70.20 | 137.98 | 96.38 | 155.84 |
| | 8 | 1.69 | 4.44 | 28.39 | 35.29 | 20.27 | 8.71 |
| | 80 | 1.20 | 3.02 | 16.01 | 34.89 | 18.04 | 5.05 |
| | 800 | 0.99 | 3.02 | 10.22 | 25.75 | 15.61 | 3.23 |

Note: The values are arithmetic means (n=3) in each column.

* Sampling time after inoculation

Academy of Sciences in Wuhan, China. Axenic cultures were grown in a liquid sterilized medium as described by Kratz and Myers (1955) [8] at 30 ± 2 °C exposed to fluorescent light at an intensity of 3000-lux (pH 7.2). The experimental cultures were first grown in 250-ml flasks containing 100 ml of medium with 0.5-to-1 million cells per ml exposed to the same conditions as described above. At the exponential growth phase of the algal cultures, monosulfuron from the stock solution was added to the culture medium at 0.008, 0.08, 0.8, 8, 80, and 800 mg/L. Sterilized water was added to some of the culture media rather than the herbicide and these cultures served as controls. Each treatment concentration was replicated four times and all experiments were conducted twice.

Growth of algae was measured by recording light absorbance of the culture at 448 nm using a spectrophotometer. Standard curves relating spectrophotometric absorbance readings (448 nm) with cell numbers were developed for continuous culture of *A. flos-aquae*. These curves were used to determine cell numbers in continuous culture samples used as inoculum for the screening bioassay. Data used to produce the standard curves were obtained from absorbance measurements [9]. Herbicides from stock solution were added to the culture medium to result in the desired concentration and controls (without the herbicide). Each concentration was replicated four times. During the experimental period, samples were withdrawn after herbicide treatment at 24, 48, 72, 96, 120, and 144 hours for measurements of growth. The growth rate (µ) of the algae was calculated by: 

\[ µ = \frac{\ln X_1 - \ln X_0}{T_1 - T_0} \]

where \( X_1 \) represents the number of algal cells in the absorbance at 448 nm at time \( T_1 \), and \( X_0 \) represents the number of algal cells in the absorbance at 448 nm at time \( T_0 \). To determine dry weight of cells, corresponding cultures in triplicate were pelleted; and the pellet washed using distilled water three times before drying to constant weight at 105 °C for 8 h [10].

During the experimental period, samples were withdrawn after herbicide treatment at 48, 96, and 144 hours for measurements of photosynthetic pigments. Chloro-
phyll a was extracted with 90% methanol in dark for 2 h, and centrifuged at 3000g for 3 min; and estimated using absorbance at 665 nm according to Mackinney (1941) [11]. The algal biliproteins were extracted by repeatedly freezing and thawing the pellet in the presence of 0.05 M phosphate buffer (pH 6.7). The solution was centrifuged at 3000g for 15 min, and the absorbance at 618 nm measured [12]. The amount of total carotenoid was calculated from the absorbance at 447 nm according to the method described by Jenssen (1978) [13].

A completely randomized design with four replications was used in all experiments. Experiments were repeated twice. The data represent the average of the two trials because of a non-significant trial by treatment interaction and homogeneity of variance according to Bartlett’s test. Analyses of variance (ANOVA) were performed on non-transformed data. Significant differences were determined using Duncan’s test at the P = 0.05 level of significance (PROC GLM, SAS Institute, 2001).

3. Results and Discussion

3.1. Effects of Light on Herbicide Toxicity to Algae Mixotrophic Growth

The growth of Anabaena flos-aquae treated with different concentrations of monosulfuron exposed to 2000-, 3000- and 4000-lux light intensities are listed in Table 1. It can be seen that the herbicide monosulfuron applied stimulated the algal growth at initially 24 h, but then markedly inhibited the mixotrophic growth of the algal grown exposed to three light intensities in a dose-dependent manner; namely, the inhibitory effect increased with the increase in monosulfuron concentration. The cell number of Anabaena flos-aquae were reduced by 24 to 99 % exposed to 4000-lux light and by 10 to 97% exposed to 3000-lux, at a concentration range of 0.008 to 800 mg/L, after inoculation 144h, in comparison with the control (P<0.05). But the reverse was observed at monosulfuron ranging from 0.008 to 0.08mg/L exposed to 2000-lux, the cell number of Anabaena flos-aquae were increased by 7 to 13% after inoculation 144h, in comparison with the control. Thus it is clear that Anabaena flos-aquae had different degrees of sensitivity to monosulfuron with the most sensitive light intensity being 4000-lux followed by 3000-lux and 2000-lux. However, without herbicide, the growth of Anabaena flos-squae exposed to 3000-lux light was better than those exposed to 2000- and 4000-lux light (Figure 1).

The observed transient stimulation effect of monosulfuron on algal growth at initially 24 h in this study are similar to those reported by Shen et al. (2005) [14] for butachlor and acetochlor on several Anabaena species. Hormesis, the stimulatory effect of sub-toxic concentrations of a toxin, has been documented following the application of other herbicides and allelochemicals [15].

Many previous reports had been demonstrated to be effects of herbicides on nitrogen-fixing cyanobacteria growth. Exposure of several Anabaena species to prometryne even at the relatively low concentration of 4 mg L⁻¹ reduced growth by 79, 71 and 69 % respectively, while no growth was observed at a concentration of 8 mg L⁻¹ [16]. Benthiocarb at concentrations ranging from 6 to 8 mg/L was lethal to some species of Nostoc [17]. The present studies with monosulfuron indicate that Anabaena flos-aquae maintain a dose-dependent manner algal growth, and the algal grown exposed to 4000-lux light exhibited greater sensitivity toward the same cation monosulfuron than exposed to 3000-, and 2000-lux light. It is possible that monosulfuron applied at higher concentrations exposed to 4000-lux result in stronger toxicity to algae due to enhancing algal efficient photosynthesis and special characteristics metabolism exposed to highlight. The results of stimulation effect on algal growth at lower concentrations (0.008 to 0.08 mg/L) exposed to 2000-lux can be explained in two ways: 1) Monosulfuron self-motionly degraded faster at lower concentrations in this experiment and resulted in reducing toxic to algae. Fan et al. (2004) [18] reported that monosulfuron residues have been shown to dissipate rapidly with half-lives of less than 14 d in field tests using HPLC-UV residue analysis. 2) It was demonstrated that Anabaena flos-aquae is capable not only of growing photoheterotrophically, but also of growing chemoheterotrophically like bacteria to a great extent [19]. Alternatively, at lower monosulfuron concentrations, algal cells may have assimilated more organic carbon and nitrogen [20,21], which interfered with herbicide uptake through the formation of an inactive herbicide complex with organic carbon and nitrogen. This process can reverse herbicide toxicity or can convert the herbicide into hydrate of carbon and amino acid metabolites, thus decreasing the toxicity of the herbicide.
3.2. Effects of Herbicide on the Content of Chlorophyll a in Algal Cells Exposed to Different Light Intensities

The results in Figure 2 indicate the impact of monosulfuron on the content of chlorophyll a in *A. flos-aquae*. The effect of monosulfuron on chlorophyll a followed a dose-dependence manner, i.e., the chlorophyll a content decreased gradually, as the concentration of monosulfuron increased from 0.008 to 800 mg/L. The content of chlorophyll a was reduced gradually, as the concentration of monosulfuron increased from 0.008 to 0.08 to 0.8 to 8 mg/L, by 8, 14, 17, and 55% exposed to 4000-lux light, and by 15, 28, 62, and 87% exposed to 3000-lux, as well as by 26, 45, 63, and 95% exposed to 2000-lux, after 6 days, respectively, in comparison with the control (P<0.05). There was a significant difference in chlorophyll a between each concentration (P < 0.05). Reductions in chlorophyll a rose to 92 to 99% as the concentration of monosulfuron increased to 80 and 800 mg/L exposed to three light conditions. The chlorophyll a synthesis of *A. flos-aquae* exposed to 2000-lux light was more sensitive to monosulfuron than those exposed to 4000-lux light.

The dose dependence of photosynthetic pigment affected by the herbicide has been reported to appear several manners. The results in the present study indicate the dose dependence of monosulfuron toxicity on chlorophyll a synthesis is in agreement with that demonstrated by most reports, that the inhibitory effect of herbicide increase with an increase in herbicide concentration, and suggested that the reduction in the growth rate of algae may be due to a decrease in algal photosynthesis caused by the inhibition of synthesis of chlorophyll a, the most important pigment in algal cells for collecting solar energy for photosynthesis [22]. Bhunia *et al*. (1991) [17] reported that the chlorophyll a content of *N. muscorum* was reduced by 56, 74, and 97% at 2, 4, and 6 mg/L benthiocarb, respectively, and suggested that the low pigment content may be caused by photooxidation arising from the inability of chlorophyll a to dissipate its absorbed excitation energy when electron transport is inhibited.

3.3. Effects of Herbicide on the Content of Carotenoid in Algal Cells Exposed to Different Light Intensities

The effect of monosulfuron on carotenoid content in *A. flos-aquae* cells is summarized in Figure 3. The formation of carotenoid was inhibited as the light intensity increased and that the effect of monosulfuron on carotenoid exposed to three light intensities displayed dose dependence. The content of carotenoid in cells of *A. flos-aquae* was reduced by 30 to 95% and 28 to 87% exposed to 3000-lux and 4000-lux after 144 h at the monosulfuron from 0.008 to 800 mg/L, respectively, in comparison with the control (P<0.05). Of particular interest is the observation that the carotenoid content of algae treated with 0.008 and 0.08 mg/L monosulfuron exposed to 2000-lux had a different stimulatory effect from that of treatments exposed to 3000-lux and 4000-lux, but an inhibitory effect at concentration above 0.8 mg/L. The carotenoids synthesis of *A. flos-aquae* exposed to 4000-lux light was more sensitive to monosulfuron than those exposed to 2000-lux light.

Carotenoids are known to be involved in a number of photosynthetic processes, including light harvesting and energy transfer to chlorophyll a, photoprotection by quenching triplet state chlorophyll a molecules, and scavenging singlet oxygen. Regulation of carotenoid biosynthesis is known to be dependent on external stimuli, including light intensity and oxygen availability in a photosynthetic bacterium *Rhodobacter* [23], and is in agreement with our findings. These stimulatory effects of monosulfuron on carotenoid formation in *A. flos-aquae* cells exposed to 2000-lux at lower monosulfuron (0.008 to 0.08 mg/L) are
in accordance with that on algal growth.

3.4. Effects of Herbicide on the Content of Biliprotein in Algal Cells Exposed to Different Light Intensities

The effect of monosulfuron concentration on biliprotein in the algal species tested is shown in Figure 4. Our results showed that the effect of monosulfuron on biliprotein in cells of *A. flos-aquae* exposed to three light intensities displayed contrary dose dependence; it had a stimulatory effect as the concentration increased from 0.008 to 0.08 mg/L, but an inhibitory effect at concentration above 0.8 mg/L. Subjected to lower monosulfuron concentration (i.e. 0.008 to 0.08 mg/L), the content of biliprotein was increased by 11 to 46% exposed to three light intensities, after 6 days, in comparison with the control (P<0.05). But the converse was observed at higher concentrations (i.e. 0.8 to 800 mg/L), the biliprotein content of cells grown exposed to three light intensities was decreased by 33 to 98% after 6 days, relative to the control treatment (P<0.05). The biliprotein formation of *A. flos-aquae* exposed to 2000-lux light was more sensitive to monosulfuron than those exposed to 3000- and 4000-lux light.

The algal biliprotein function as important light-arvesting components for driving for photosynthetic reactions. In this study, exposure of *A. flos-aquae* to monosulfuron at the relatively low concentration (i.e. 0.008 to 0.8 mg/L) had a stimulation effect on the synthesis of biliprotein, it may be related to the herbicide monosulfuron, an organic nitrogen compound, algal cells may have assimilated more organic nitrogen. Harrison et al. (1990) [24] observed the increase in nitrogen concentration in medium may stimulate the formation of biliprotein. The inhibitory effect of monosulfuron on the synthesis of biliprotein at higher concentrations (more than 0.8 mg/L) is similar to the previous report by Marco et al. (1990) [25], who found that the organophosphorus insecticide trichlorfon decreased biliprotein content in Anabaena PCC 7119 at higher concentrations ranging from 20 to 300 mg/L. The herbicide monosulfuron appeared to have different effects on the synthesis of photosynthetic pigments. This may be due to the difference in chemical structures and characteristics of three pigments. For example, biliprotein is not a single compound; it is a complex composed of bili pigments (phycobilins) covalently linked to a specific protein, C-phycocyanin [12].

4. Conclusions

The data obtained in the present study provide information about the inhibitory effect of the sulfonylurea herbicide monosulfuron on mixotrophic growth of *A. flos-aquae* exposed to different light intensities, under which the cyanobacterium exhibits different sensitivity to the herbicide. These findings suggest monosulfuron be applied at low concentrations (i.e. less than 0.8 mg/L) in the field to minimize the negative impact on these beneficial microorganisms, or postemergence application, owing to its relative lower inhibitory effect on nitrogen-fixing cyanobacteria exposed to low light intensities. The physiological mechanism of monosulfuron toxicity to *A. flos-aquae* was found to be related to not only target enzymes ALS activity but also photosynthesis in algae cells. A better understanding of the mechanism requires further study of the effect of monosulfuron on electron transfer, ATP, and fatty acid metabolism in photosystem.

5. Acknowledgements

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Pre- and Post-Urban Wetland Area in Dhaka City, Bangladesh: A Remote Sensing and GIS Analysis

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Abstract

Landscape of Dhaka city - one of the fastest growing mega cities in the world, is undergoing continuous changes and modifications due to progressive urbanization. Pre- and post-urban changes of water bodies in the city were studied using aerial photographs and SPOT images in GIS environment. In 1968, the total area of marshy and peaty inundated low-lying areas was 133 km², which was depicted to be 67 km² in the year 2001. The total area of inland lakes as estimated from the aerial photos of 1968 was 5.1 km² which became 1.8 km² in the year 2001 as seen in SPOT image. More than 50% of the wetland area reduced over the period 1968 to 2001. Changes of the water body mostly occurred in the regions where majority of the urban expansion took place. The urban infrastructures filled and/or compartmentalized the water bodies, causing water loggings problem during wet-season in various part of the city. Development and alteration of the existing water bodies should consider the natural hydrological conditions so that the changes can cope with the artificial intervention.

Keywords: Wetland, Dhaka City, Bangladesh, Remote Sensing, GIS

1. Introduction

Urbanization is the major demographic development which is occurring very fast and with larger magnitude in the developing countries. In many cases, specifically, in the poor economic countries urbanization is a demand driven unplanned and bottom up process, which transforming the existing landscape without considering the possible consequences and requirement for environmental sustainability [1]. These urban growths have profound adverse effect on the water resources, particularly in the humid tropical region [2]. In the tropical region where monsoon causes huge rainfall during some part of the year are naturally drains by the gravity drainage through stream-river networks, and wetlands works as natural retention storage. Urbanization, particularly unplanned one, hampered this natural state of drainage, and hence causes sudden inundation and waterlogging. However, hydrological consideration during urban planning can reduce the adverse effects through conservation of wetlands and stream-networks to be used as retention ponds and canals or designing such elements to drain-store-drain the modified landscape [3,4].

Dhaka city, the capital of Bangladesh, is situated in the central part of the country. It has one of the fastest urban growth rates among the developing countries [5,6] and home to more than 10 million people [7]. The landform of the city is characterized by the Madhupur Tract – an elevated Pleistocene terrace [8] that stands higher than the neighbouring floodplain and low-lying marshlands. The land cover of the city is being modified extensively by the progressive urbanization [5]. The rapid rise in urban population is a major drive to the development of infrastructure and services, including road-network, water supply, sanitation, sewerage and drainage services and hence expansion of the city towards the surrounding floodplain and low-lying areas.

The relief controlled landforms of the area were efficiently drained via streams and canals (local term ‘khals’) [9] to the floodplain and low-lying area and ultimately to the downstream via large rivers. These canals, wetlands and depressions have been filled up by new urbanization, both in and around the built-up city area [9-11]. These unplanned urbanizations have been destroying the water-bodies and flow-paths causing rainfall-flooding and drainage congestion in many locations in the city [10,12,
Filling activities, embankments and roads are being compartmentalized the wetlands and water bodies and hence obstructed the natural drainage. There is a general observation that the wetlands and other water bodies in Dhaka city is greatly reduced over the decades due to progressive urbanization [10,14]. There were number of studies mentioned about the land filling activities and their effects on the drainage congestion and water logging [10,11,15–17]. The city at the moment, particularly, during the heavy downfall in the wet-season gets waterlogged [17]. The scenario would be worsening with time as erratic and intense rainfall events postulated [18] to be increasing their frequencies due to climate change [6]. A recent report in a daily newspaper (The Prothom Alo, A national Bengali newspaper, 29th of July 2009) supports the intense and erratic rainfall events occurring in the recent time as well. “It said that on the 28th of July 2009 there was 333 mm rainfall over 9 hours which was similar to 104 mm over 3 hours on the 4th of August 2004. Both the events caused severe flooding which lead to a city-wide disruption, and took several days to recede the water. These events lead the policy maker to think to restore the natural drainage canals and wetlands, which are in many cases filled or modified at the current time.” Rivers around the city gets the storms-drainage and other low grade (mostly households) liquid waste, and also liquid waste from the industries. These liquids and other pollutants have been polluting the river water, see in [19]. Wetlands of the city could play a significant role in reducing the pollutants loads if the wetlands were designed as retention basin [4,20,21] for the urban drainage to the rivers. Plants and soils in wetlands play a significant role in purifying water, removing high levels of nitrogen and phosphorus, and in some cases, removing toxic chemicals through biogeochemical cycling and storage [4]. However, there are very few studies [12,13] have been done on the changes of the wetlands in the city. Chowdhury et al. [12] pointed on the elimination of local water storage and consequences in the mid-central western part of Dhaka city. Reza and Alam [13] showed the changes of wetlands in the western half of the city. Both the studies concentrated on the western part of the city while none look over the whole city specifically, to the eastern part. History of urban growth [17,22] in the region demonstrates that the city is being expanding to the eastern part, low-lying areas, in the recent decades. These urban expansions are mostly occurring on to the wetlands in the eastern part, which were suggested to act as retention basin for urban drained water [9]. It is evident from the literature that the floodplains and low-lying areas were not under any sort of urban infrastructure during 1960s and urban growth in the area boosted during the 1990s. The objective of the present work is to map and distinguish the differences of the water bodies between the pre- and post-urbanization in Dhaka city. In this context an aerial photograph of the year 1968 (as pre-urbanization) and satellite imagery of 2001 (as post urbanization) were used to map and quantify the disparity between water bodies applying remote sensing and GIS as a tool. An aerial photograph for the year 1968 was chosen because of the unavailability (at least with the current source) of a satellite image for that time.

2. Natural Setting and Geomorphology

Dhaka with an aerial extent of 298 km², bounded by the Buriganga River in the south, the Balu River in the east, the Tongi Khal in the north and the Turag River in the west (Figure 1). These rivers are connected to the Ganges-Brahmaputra River system (locally known as the Padma-Meghna-Jamuna River system and also include the Old Brahmaputra river) flowing towards southeast from the all sides of the bigger neighbouring region of the study area. The bigger area is closely dissected by number of rivers and khals which are hydrologically connected to these major rivers.

The area represents mostly flat land with slight undulations and stands few meters higher than the surrounding area. A large part of this city is covered by low-lying depressions. The area represents significant variation in elevation ranges from 1.5 to 15 m with an average of 6 m above PWD (Public Works Datum) (+/- 0.45 meter with respect to mean sea level). The area slopes towards southeast, east and west, but general slope is from the north to southeast where the ground surface merges gently with the floodplains of the Buriganga River. The ele-
vation of the surrounding floodplains of the area is variable. The average elevation of the Buriganga and the Lakhya River floodplains are about 3 m above the PWD.

The metropolis and its surrounding areas are covered with Pleistocene Madhupur clay and Holocene sediments of floodplain origin [24]. Holocene sediments are forming different types of landform in the area like: abandoned channels, depression, floodplain, high floodplain (Figure 1). The western part of the investigated area is the main old city founded on the semi-consolidated Pleistocene sediments (Madhupur clay), slightly elevated than the extreme western edge, which is covered by the Buriganga-Turag floodplain sediments of Holocene to recent time [cf. 23]. The eastern edge is mainly covered by the floodplains of the Balu River. These floodplains are characterized by low-lying depressions and marshy areas which remain inundated during significant period of the year. To the east, the low-lying area (depression and floodplain in Figure 1) is covered with recent floodplain deposits. There are number of lakes, channels and khals in the city, which are connected to the surrounding rivers. The city is crossed by several faults and lineaments which are possibly controlling the orientation of rivers and streams [25]. The downthrown subsiding blocks are hosting marshy and swampy lands i.e. wetlands habitats [5,23].

The storm runoff accumulates in the low-lying areas, flows through khals and local rivers and ultimately discharges to the major rivers. These lowlands and wetlands are performing important drainage function by storing storm water and keep the relatively higher lands free from rainfall flooding [12]. The area has a tropical monsoon (May to October) climate like other part of the country with an average precipitation of 2000 mm/year.

3. Approach and Methodology

3.1. Defining Wetland and Approach

The RAMSER convention has defined wetlands as “areas of marsh, fen, peatland or water, whether natural or artificial, permanent or temporary, with water that is static or flowing, fresh, brackish or salt, including areas of marine water the depth of which at low tide does not exceed six meters” [cf. 3]. According to Khan et al, [26] wetland holds water for a significant duration sufficient to support organism adapted to life in inundated or saturated soil condition and consists of wide variety of types ranging from lakes, rivers and coastal forest to deepwater paddy fields and ponds. The built-up area of the city is traversed and surrounded by wetlands of different types. For the purpose of the study, these wetlands are subdivided into three categories:

1) Open water body comprises marshy and peaty inundated (during significant part of the year) low-lying areas of the Turag-Buriganga and the Balu floodplains.

2) Inland water body includes the lakes and connecting canals of different water bodies in the city area.

3) Fluvial water body denotes the surrounding rivers (Turag, Buriganga, Balu River and Tongi Khal) of the city.

It is however, analyses were done for the open and inland water bodies only. Third category was not included in the analysis as because this needs to be done with high resolution imagery with extensive field verifications. This is also out of the scope of the current work.

3.2. Data and Software Used

In order to compare the water bodies, two digital images: aerial photo of 1968 and SPOT satellite image of 2001 were used. Aerial photo of 1968 is a panchromatic type image with spatial resolution 12 metres (scanned resolution), has been considered for pre-urban state water body analysis. On the contrary, a multispectral SPOT image acquired in 27 December 2001 with 20 metres spectral resolution has been used to map the water-bodies of urbanised state of the city. The exact timing of the aerial photography has not been found with the image. However, it was revealed from the source organization that these images taken during the dry season around December-January time of the year. The temporal variation in the extent of the water-bodies is assumed to be insignificant over 1-2 months, at least for city-wide change analysis. However, this could be a source of uncertainties in the current analysis, in addition to the comparison of the two different spatial resolutions of the images. The data layers (containing the areas of water bodies) of two years were overlaid to reveal the changes.

There are two strips of the aerial photograph to covers the study area. The aerial photographs did not have cartographic standard and three-dimensional geographic distortions were found. The study has been carried out under the framework of Geographic Information System (GIS). The Image Processing tasks have been carried out using Earth Resource Data Analysis System (ERDAS) 8.4 image processing software (Leica Geosystems Geospatial Imaging, LLC, Norcross, USA). Data on wetland features has been extracted using ERDAS imagine software. GIS analysis has been carried out using ArcInfo software (ESRI, Redlands, California, USA) and the outputs have been generated using ArcView 3.3 software (ESRI, Redlands, California, USA).

3.3. Data Processing and Analysis

Data processing and analysis scheme used for the study is illustrated using a flow chart as in Figure 2. The SPOT image of 2001 was collected in ERDAS Imagine (*.img) format. This image was loaded into computer memory
directly from the CD (compact disk). Aerial photographs of 1968 were collected in the bitmap (*.bmp) format. These photographs were imported to *.img format to process the data using ERDAS Imagine software.

Geo-referencing was performed to render real world coordinates to the images to avoid geometric distortion. Aerial photographs of 1968 contain multi-scale geometric distortion. For geo-referencing of this image, fourth order polynomial transformation was used based on 112 GCPs. For better mutual registration of the two data images, GCPs for the aerial photograph were derived from the geo-referenced SPOT image. The SPOT image of 2001 does not contain multi-scale geometric distortion; thus second order polynomial transformation was used for this image. For calculating the transformation matrix, 16 Ground Control Points (GCP) were used. The GCPs were derived from a geo-referenced image. Remarkable features, like road junctions, river bend etc., common to both images (1968 and 2001) were used as GCPs. In case of rivers or lakes, GCPs were chosen in the middle of the channel. Because most of the (spatial) changes to these features occurred at the edges of the channels or lakes (see Section 4). It should point-out here that the lakes in the study area are also channel-like (relict channel) features. The projection system used for the images was Lambert Conformal Conic (LCC). The geo-referenced two strips of aerial-photos were mosaic to obtain the image of the whole study area. Then thematic data layers were generated from the geo-referenced images using on-screen digitization technique. Data layers were generated in ArcInfo vector format using the digitization tools of ERDAS Imagine software. ArcInfo vector data layers generated from the images to render the GIS standard of the data.

The spatial analyses of the thematic data layers were performed to generate composite data layers from the two images. Aerial coverage of individual type of water body for the year 1968 and 2001 was calculated and the disparities between them were estimated. Changes were illustrated through map generation using ArcView software as described in result section.

4. Results

4.1. Open Water Body

Analysis of aerial photo of 1968 revealed (Figure 3) that most of the eastern part of Dhaka city is covered by open water bodies in the form of the marsh-land or peaty areas of the Balu River floodplain. Western edge of the city is also covered by the marshy low lands. These low lands in the west of Muhammadpur and northwest of Pallabi are generated within the floodplain of the Buriganga and the Turag River. Marshy land of the Turag floodplain extended into the built up area (mostly the Madhupur Clay covered area in Figure 1) in the northwest part of the city near Pallabi. The total area is measured as 133.03 km². Satellite image of 2001 (Figures 4 and 5) shows that the coverage of the water bodies in the eastern part of the city has been reduced and became sporadic. The water bodies in the western edge of the city are also reduced and become patchy. The total area is measured 67.79 km² in 2001.

4.2. Inland Water Body

Lakes, channels and khals, which are visible on the images, have been identified. The inland water bodies on the aerial photo of 1968 are more prominent than 2001. Analysis and observation for inland water body on 1968 image show that the Gulshan Lake, Dhanmondi Lake and Ramna Lake are highly visible (Figure 3). Some channels and/or khals are also identified in different parts of the city. Channels are located in the northeastern, eastern, southwestern, southern and north-western corner of the city. The total areas of inland water body are measured 5.1 km². Analysis of satellite image of 2001 (Figure 4 and 6) for inland water body shows that the areas of lakes (Gulshan and Dhanmondi) have shrunken and narrowed down. Some khals and channels are not identifiable or missing in the southwestern (Muhammadpur) and southern (Motijheel) area of the city and the total area is measured 1.8 km² in 2001.

4.3. Water Body Compartmentalization

In most part of the city open water bodies were acting as a single water body and were well in connection with the surrounding river via streams and khals. It is seen that water bodies become more sporadic and patchy in 2001 in compare of 1968 in many part of the city (Figure 5). Water body compartmentalization, specifically, occurred
5. Discussions

5.1. Reduction of Wetland Area

Analysis revealed that area covered by wetlands in the city significantly reduced over the period 1968 to 2001 (Table 1). In 1968, the area of the open water body was 133 km², which became ~68 km² in the year 2001. The amount of the open water body reduction is ~65 km². The wetlands in the south-western corner retreat towards the Turag River in between Mirpur and Muhammadpur area. Minor reduction of the wetland has been occurred in the Pallabi-Cantonment area as well (Figure 5), where...
Figure 7. Water body compartmentalization in the north central part (please see marked black-rectangle in the inset for index) of the city. This sort of partitioning can also be seen in the south central and many others parts of the city through close observation in Figures 3 and 4.

Table 1. Summary of analysis on the changes in water bodies.

| Item/ Class | Open Water Body \((\text{km}^2)\) | Inland Water Body \((\text{km}^2)\) |
|-------------|-------------------------------|----------------------------------|
| Area in 1968 | 133.03                        | 5.1                             |
| Area in 2001 | 67.79                         | 1.8                             |
| Change in area | 66.24                        | -3.3                            |
| (Change in %) | (50.96%)                      | (64.7 %)                        |

'-' sign in change in area row indicate loss in area compared to 1968
Total area of the city is 298.26 \(\text{km}^2\)

low-lying areas were filled and levelled for the urban extension. The reduction of wetland in this area closely matches the ground filling in Dhaka city as described in earlier works, e.g. [10,11].

The area of the inland water body was found to be 1.8 \(\text{km}^2\) in the year 2001 which was 5.1 \(\text{km}^2\) in 1968. The reduction of area in the inland water body was 3.3 \(\text{km}^2\). The shape of the water bodies remains almost same; but the widths of the lakes - Gulshan and Dhanmondi have been reduced (Figure 6). Inland water body has been mostly changed by the land filling and storm- and waste-water laid deposits. There are high-rise buildings constructed by the bank of these lakes encroached part of the lake area in some parts. Some of the channels in southeastern neighboring region of Motijheel area are not identifiable or missing in the 2001. These are narrowed or lost due to either encroachment or acquisition by government for construction of roads, box culverts or underground drains. These require ground truth and/or onsite mapping.

Urban expansion has been encroaching in the low-lying areas to cope with population growth, is the main reason for the reduction of wetland areas in the city. From the history of growth of the city, e.g. [22] demonstrated that the city was remaining almost same up to the late 1980s. Earlier study, Reza and Alam [13] showed that area of water bodies in the western part of the city reduced to 91% of 1963 in 1990, which dropped to 63% of 1963 in 2000. In the present study the change in open water bodies over the entire metropolitan area is calculated to be reduced by 51% in the year 2001 compared to 1968. It can be opined that this reduction happened in the 1990s too. From the earlier work e.g. [14], it is vivid that the city areas in the early 1990s were extending the western low-lying areas while in the late 1990s and as well as in the 2000s the urbanization activities redirected to the eastern wetlands.

5.2. Significance of Wetland

Wetlands associated with river-floodplain system capture and retain water, reducing the duration and severity of floods [3]. Inland wetlands intercept surface flow and slow it down, reducing the potential for floods. In the city development master plan [9], wetlands in the eastern part were proposed as retention basin, which has long been acting so to keep Dhaka free from rainfall- storm flooding. The unplanned urban extension into these wetlands may be causing water-logging problem during wet-season in the eastern part of the city [14]. This unplanned urbanization has also resulted into compartmentalization of the wetland. Due to poor connectivity between the different compartments of the wetland are causing water-logging in the built-up area as well. Water quality of the wetland would have been progressively deteriorating as wetland gets patchy and isolated.

The extent of groundwater recharge by a wetland is dependent upon soil, vegetation, site, perimeter to volume ratio, and water table gradient [27]. The soil under most wetlands in Dhaka city is relatively impermeable; however, perennial presence of these wetlands may signify their potential contribution to groundwater recharge. Moreover, some wetlands particularly around the built-up area have lesser thickness of silty-clay at the bottom; have the higher potential to groundwater recharge. The wetlands around the built-up area and rivers around the city have found to play a significant role in the groundwater recharge [28,29]. Demand driven abstraction of groundwater in the city has been failing to balance with amount of recharge in the recent years leading declining groundwater level and aquifer dewatering in many parts [30]. This may have been further enhanced by the reduction of wetland area.

6. Conclusions and Management Implications

Infilling of natural channels and lowlands for urban infrastructure have been reducing the area, and compartmentalizing the water bodies which causing water loggings and flood hazards in various parts of the city. Among the different types of water bodies marshy and peaty inundated low-lying areas, natural retention basin around the city, are the hardest hit by the unplanned urbanisation. In 1968, the total area of marshy and peaty inundated
low-lying areas was 133 km², which was found to be ~68 km² in the year 2001. Over the last 33 years (1968 to 2001) 50% of the wetlands have been reduced. If the reduction rate (~2 km² per year) remained in place, the total wetland area in the year 2009 would be ~50 km². This means the wetland area may have reduced further from ‘50% of 1968’ in 2001 to ‘37% of 1968’ in 2009.

Most of the wetland losses in the eastern part of the city are associated with an urgent need for shelter by the less-favoured urban population, and growth takes the form of informal settlements. In the long run these informal growths are destroying the wetland ecology, reducing the area, and segmenting the integrity of the wetlands. Wetland loss can be viewed in several ways. The most obvious type of wetland loss is the conversion of a jurisdictional wetland to a non-jurisdictional status [21]. For example, wetlands in the northwest corner of the city (between Kafrul and Mirpur on Figure 1) have been acquired for the development of government offices over the last two decades. These wetland losses could not be stopped if these were jurisdictional wetland instead of non-jurisdictional type. In Dhaka city wetlands should be classified as jurisdictional, and preserved by a regulatory board to preserve and restore the wetlands.

It is true that urbanization in Dhaka city would not be stopped, but these should be based on further specific studies and understanding of the hydrological system of the area, not just demand driven unplanned expansion. In addition, drainage in Dhaka city is strictly controlled by land-relief and hence by gravitational drainage. Special care should be given to the development and alteration of the existing water bodies so that natural hydrological condition can cope to the artificial structural actions. Water management must be the first concern for any development in Dhaka city because of its natural settings, and almost all the time people suffer from water and/or for water in a growing mega city.

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Application of GDAHP on Quality Evaluation of Urban Lake Landscape

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Abstract

The quality evaluation of urban lake landscape (QEULL) is extremely important for the healthy development of lake landscape. In this research, the evaluation model was established with the group decision analytic hierarchy process (GDAHP) method, which consisted of four layers including the target layer, the factor layer, the index layer and the criterion layer, thus forming a model tree based on their subordinate relationships. The GDAHP method was employed to determine the weights of constituting factors of each layer in the evaluation model, and the fuzzy method was used to establish the factors remark sets of the criterion layer, thus the single-layer evaluation and comprehensive evaluation of urban lake landscape quality was carried out. Quality evaluation model of urban lake landscape established based on the GDAHP method can provide grounds for planning, design, and renewal of urban lake landscape. This model has been used to evaluate and analyze the artificial lake in People’s Park of Xinxiang City, Henan Province. The results proved that the overall landscape quality of the artificial lake of Peoples Park in Xinxiang city was good.

Keywords: Quality Evaluation, Urban Lake Landscape, GDAHP

1. Introduction

The urban lake is an important type of wetland, playing a crucial role in maintaining eco-balance, protecting biodiversity, preserving fresh water resources, regulating and storing flood waters, adjusting the climate, replenishing underground water, degrading pollutants, and providing important resources for our life, production and social development [1]. In addition, it also has social functions including relaxation [2], entertainment and economic ones such as tourism development [3]. Modern urban residents show a keen interest in water landscapes, especially lakes [4]. In order to satisfy such needs, a large number of lakes have been constructed in many cities. In spite of the achievements, there have existed many problems in lake landscape construction [5,6], in some, eco-protection was overemphasized while the public needs of traveling and relaxation were ignored; in others, the development was totally centered on relaxation and entertainment while the construction requests of the ecological environment have been neglected. Such problems have seriously undermined the healthy development of lake landscapes. Therefore, how to accurately evaluate the quality of lake landscape to provide grounds for planning and design as well as renewal and construction of such landscape has become a project deserving research. Zhang Fengling and others thereby have established the appraisal standards for urban river and lake ecological health [7].

The quantitative methods have the features of accuracy and easy for comparison, therefore it will be a wide prospect applying it in the QEULL.

The AHP was first introduced by Saaty in 1971 to solve the scarce resources allocation and planning needs for the military [8]. Since its introduction, the AHP has become one of the most widely used multiple-criteria decision-making methods, and has already been applied in many field such as political, economic, social, management sciences, industrial controlling, engineering, medicine and mining industry etc.

For some complicated decision problems, in order to avoid the mistakes and to improve the accuracy, it is needed to rely on the wisdom of a group of experts to make a decision.

2. Methods

In this research, a model was established with the GDAHP method. Details steps are as follows.

Step 1: To establish the pairwise matrix $A$:
where \( w_i \) is the relative importance of the \( i \)-th index of the layer index, as shown in Table 1.

Step 2: To calculate the product of each line \( M_i \):

\[
M_i = \prod_{j=1}^{n} a_{ij} \quad (i = 1, 2, ..., n)
\]

(2)

Step 3: To calculate the \( n \)-th roots (\( \bar{W}_i \)) of \( M_i \):

\[
\bar{W}_i = \sqrt[n]{M_i} \quad (i = 1, 2, ..., n)
\]

(3)

Step 4: To obtain the weight of the \( i \)-th evaluation index (\( W_i \)) by standardizing the \( \bar{W}_i \):

\[
W_i = \frac{\bar{W}_i}{\sum_i \bar{W}_i}
\]

(4)

Step 5: Consistency check:

The maximum eigen value \( \lambda_{\text{max}} \) is:

\[
\lambda_{\text{max}} = \frac{1}{n} \sum_i W_i = \frac{1}{n} \sum_i \frac{\bar{W}_i}{\sum_i \bar{W}_i}
\]

(5)

The consistency index (CI) is:

\[
CI = \frac{\lambda_{\text{max}} - n}{n}
\]

(6)

The consistency ratio (CR) is:

\[
CR = \frac{CI}{RI}
\]

(7)

where \( RI \) is the random index (Table 2). If \( CR < 0.1 \), it means that evaluations tend to be consistent. For multiple levels, \( CRH < 0.1 \) should be satisfied, and

\[
CRH = \frac{CIH}{RIH}
\]

(8)

where \( CIH \) is the consistency index of the hierarchy, \( RIH \) is the random index of the hierarchy, and \( CRH \) is the consistency ratio of the hierarchy.

According to Step 1 to Step 4, the local weights (LW) of each layer and the global weights (GW) are obtained, and the consistency check is tenable based on Step 5.

Step 6: Synthesize the fuzzy comprehensive evaluation result-vector \( B \). Synthesize \( A \) and \( R \) of each evaluated object with the appropriate operator, and obtain the fuzzy comprehensive evaluation result-vector of each evaluated object:

\[
B = A \circ R = (b_1, b_2, ..., b_m)
\]

(9)

Step 7: Calculate the value \( S \), determine the quality ratings of lake landscape, and thus conduct its analysis.

\[
S = B \cdot Med. V_i
\]

(10)

### Table 1. The relative importance scales of AHP.

| Relative importance | Scaled value |
|---------------------|--------------|
| Extremely important | 9            |
| Especially important| 7            |
| Obviously important | 5            |
| Fairly important | 3            |
| Equally important | 1            |
| Fairly not important | 1/3         |
| Not important | 1/5          |
| Less important | 1/7          |
| Minimally important | 1/9         |

### Table 2. Values of random consistency index RI.

| Rank | 1 | 2 | 3 | 4 | 5 |
|------|---|---|---|---|---|
| R. I. | 0 | 0 | 0.52 | 0.89 | 1.12 |
| R. I. | 6 | 7 | 8 | 9 | 10 |
| R. I. | 1.26 | 1.36 | 1.41 | 1.46 | 1.49 |

3. Establish the Evaluation Model—With the Artificial Lake of People’s Park in Xinxiang City

3.1. Establish the Evaluation Factor Set

There are many factors influencing the QEULL. The indices which can reflect essentially the sustainable development of urban lake landscape should be selected and the tree of the QEULL was established (Table 3).

U= \{U1, U2, U3, U4\}={nature, ecology, landscape, traffic}; U1= \{U11, U12\}={waterfront, vegetation}; U2= \{U21, U22\}={aquatic ecology, terrestrial ecology}; U3= \{U31, U32\}={function of use, psychological function}; U4= \{U41, U42\}={internal traffic, external traffic}; U11= \{U111, U112\}={shoreline, embankment}; U12= \{U121, U122\}={community; species}; U21= \{U211, U212, U213\}={water content, water quality, aquatic biology}; U22= \{U221, U222\}={width of vegetation zone, coverage of vegetation zone}; U31= \{U311, U312, U313, U314, U315, U316\}={space, facilities, illumination, hydrophilicity, safety, activities}; U32= \{U321, U322\}={sense of beauty, culture}; U41= \{U411, U412\}={accessibility, public traffic}; U42= \{U421, U422\}={connection, comfort level}.

3.2. Establish the Fuzzy Remark Set

Establish the fuzzy remark set \( V=\{v_1, v_2, v_3, v_4, v_5\} \)={Excellent, Good, Mediocre, Bad, Very bad}, and
respectively assign the value $V_t = \{80 < v_1 \leq 100, 60 < v_2 \leq 80, 40 < v_3 \leq 60, 20 < v_4 \leq 40, 0 < v_5 \leq 20\}$. Med. $V_t = \{90, 70, 50, 30, 10\}$. Evaluation standards of criterion layer see Table 4.

3.3. Questionnaire

10 sheets of questionnaire were handed out to the experts of landscape planning from Henan Agriculture University, Henan Institute of Science & Technology and Zhengzhou University, etc, to determine the relative importance of each criterion. These experts between 40-60 years old have rich experience because they were engaged in the teaching, research and practice about lake landscape planning and design for a long time, all of them have managed the large-scale lake landscape planning and design directly.

3.4. Weight and the Expert Evaluation

The artificial lake of the People’s Park in Xinxiang City has an area of 7.3 hectares, accounting for 15% of the whole area of the park, and playing the important roles of purifying water quality, regulating partial climate and providing entertainment and sightseeing, etc.

The weights of indices of each layer and the expert ratings are shown in Table 5.

4. Evaluation Results

4.1. Single Evaluation Results

The results (Table 6) were obtained in accordance with the step 4 and step 5. A computational process illustrates as follows:

$$B_{11} = \begin{pmatrix} 0.225 \\ 0.775 \end{pmatrix} \cdot \begin{pmatrix} 0.3 & 0.2 & 0.4 & 0.1 & 0 \\ 0.5 & 0.3 & 0.2 & 0 & 0 \end{pmatrix} = \begin{pmatrix} 0.455 \\ 0.2775 \end{pmatrix}$$

$$S(B_{11}) = 90 \times 0.455 + 70 \times 0.2775 + 50 \times 0.245 + 30 \times 0.0225 + 10 \times 0 = 73.3$$

$$B_{12} = \begin{pmatrix} 0.829 \\ 0.171 \end{pmatrix} \cdot \begin{pmatrix} 0.3 & 0.4 & 0.1 & 0.2 & 0 \\ 0.2 & 0.3 & 0.3 & 0.2 & 0 \end{pmatrix} = \begin{pmatrix} 0.2829 \\ 0.3829 \end{pmatrix}$$

Table 3. Tree of the QEULL.

| U | U_i | U_ij | U_ijk |
|---|-----|------|-------|
| Nature (N) | Waterfront (W) | Shoreline (SH) |
| Ecology (E) | Vegetation (V) | Embankment (EM) |
| Landscape quality (LQ) | Aquatic species (SP) | Community (CO) |
| Ecology (E) | Aquatic environment (AE) | Water content (WC) |
| Vegetation (V) | Species (SP) | Water quality (WQ) |
| Terrestrial ecology (TE) | Aquatic biology (AB) | Coverage of vegetation zone (WVZ) |
| Water content (WC) | Water quality (WQ) | Width of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water content (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |
| Water quality (WC) | Water quality (WQ) | Coverage of vegetation zone (WVZ) |

Table 4. Evaluation standards of criterion layer.

| criterion layer (21 indices) | Evaluation standard |
|-----------------------------|---------------------|
| Shoreline | Winding and zigzagging; bending; straight-line form |
| Embankment | Close-to-natural grass slope embankment; stone embankment; concrete embankment |
| Community | Close-to-natural community with a combination of trees, shrubs and grasses and abundant facades; community |
| Plant species | More than 150; between 70 and 150; less than 70 |
| Water content | Enough water; moderate; not enough |
| Water quality | Clear and tasteless; a little turbid and odorous; serious pollution |
| Aquatic biology | Rich variety; moderate; lacking |
| Width of vegetation zone | More than 50 m; between 20 m and 50 m; less than 20 m |
| Coverage of vegetation zone | More than 70%; more than 40%; less than 40% |
| Space | Reasonable spatial organization; moderate; disorderly |
| Facilities | Enough facilities; moderate; lacking |
| Illumination | Meets safety and landscape requirements; only meets safety requirements; not safe |
| Hydrophilicity | Experiences sufficient hydrophilicity; moderate; lacking |
| Safety | No hidden danger; a little hidden danger; unsafe |
| Activities | Abundant; moderate; poor |
| Sense of beauty | Very beautiful; moderate; unattractive |
| Culture | Sufficient cultural features; moderate; none |
| Accessibility | Connection with urban trunk road; connection with urban branch road; without connection |
| Public traffic | Very convenient; moderate; not convenient |
| Connection | No disconnection; occasional disconnection; frequent disconnection |
| Comfort level | Fine; general; bad |
It can be concluded from the above single evaluation results (Table 6) that among the landscape quality of the artificial lake of People’s Park in Xinxiang, the ecological factor was excellent, and the ecological, landscape and communication factors were good.

### 4.2. Comprehensive Evaluation Results

$$B_i = (0.332 \ 0.432 \ 0.179 \ 0.057)^T$$

$$S(B_i) = 0.5113 + 0.2493 + 0.1314 + 0.0968 + 0.0118$$

$$S_u = 0.8126, 0.3661, 0.0477, 0.0159, 0.0179$$

$$S(B_i) = 0.3112, 0.3661, 0.1524, 0.1713, 0$$

$$S = 0.455, 0.2775, 0.245, 0.2025, 0$$

The results proved that the overall landscape quality of the artificial lake of Peoples Park in Xinxiang city was good.

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**Table 5. Comprehensive evaluation results and Specialist comments.**

| Comprehensive evaluation results | Specialist comments |
|---------------------------------|---------------------|
| **U** | **U\(_i\)** | **LW** | **U\(_j\)** | **LW** | **GW** | **v\(_1\)** | **v\(_2\)** | **v\(_3\)** | **v\(_4\)** | **v\(_5\)** |
|---------------------------------|---------------------|
| W 0.163                         | SH 0.225 0.0121 3/10 | 2/10 | 4/10 | 1/10 | 0/10 |
| N 0.332                         | EM 0.775 0.0419 5/10 | 3/10 | 2/10 | 0/10 | 0/10 |
| V 0.838                         | CO 0.829 0.2372 3/10 | 4/10 | 1/10 | 2/10 | 0/10 |
| SP 0.171                        | WC 0.237 0.0883 7/10 | 2/10 | 1/10 | 0/10 | 0/10 |
| AE 0.866                        | WQ 0.683 0.2586 9/10 | 1/10 | 0/10 | 0/10 | 0/10 |
| E 0.432                         | AB 0.080 0.0302 4/10 | 2/10 | 3/10 | 0/10 | 1/10 |
| TE 0.134                        | WV 0.183 0.0107 1/10 | 2/10 | 5/10 | 2/10 | 0/10 |
| CV 0.817                        | S 0.303 0.0437 5/10 | 1/10 | 0/10 | 1/10 | 0/10 |
| FA 0.356                        | IL 0.032 0.0046 6/10 | 2/10 | 1/10 | 0/10 | 0/10 |
| H 0.179                         | AB 0.080 0.0302 4/10 | 2/10 | 1/10 | 0/10 | 0/10 |
| SB 0.838                        | HY 0.138 0.0199 6/10 | 4/10 | 0/10 | 0/10 | 0/10 |
| AC 0.303                        | SD 0.304 0.0204 3/10 | 1/10 | 0/10 | 0/10 | 0/10 |
| SC 0.817                        | SA 0.072 0.0104 4/10 | 3/10 | 1/10 | 0/10 | 0/10 |
| PF 0.146                        | AC 0.775 0.0256 2/10 | 5/10 | 1/10 | 0/10 | 1/10 |
| PT 0.225                        | CT 0.225 0.0074 5/10 | 4/10 | 0/10 | 0/10 | 0/10 |
| ET 0.417                        | COM 0.183 0.0043 3/10 | 4/10 | 2/10 | 1/10 | 0/10 |

---

**Table 6. Index weight value of modification.**

| B\(_i\) | S | B\(_j\) | S |
|---------|---|---------|---|
| 0.3112  | 0.3661, 0.1524, 0.1713, 0 | 0.245, 0.2025, 0 | 66.4 |
| 0.455, 0.2775, 0.245, 0.2025, 0 | 0.2829, 0.3829, 0.1342, 0.2, 0 | 65.0 |
| 0.8126, 0.1317, 0.0477, 0.0159, 0.0179 | 0.1817, 0.2, 0.4183, 0.1183, 0.0817 | 84.8 |
| 0.4228, 0.2757, 0.1066, 0.0159, 0.0179 | 0.4002, 0.2643, 0.1644, 0.1635, 0.0085 | 164.8 |
| 0.2677, 0.1979, 0.5028, 0.0163, 0.0163 | 0.3833, 0.3430, 0.1627, 0.0659, 0.0452 | 65.2 |
| 0.5451, 0.1549, 0.2817, 0.0183, 0 | 0.4002, 0.2643, 0.1644, 0.1635, 0.0085 | 74.5 |
5. Conclusions

1) In this research, the evaluation model is established with the GDAHP method, which consists of four layers including the target, the factor, the index and the criterion, thus forming a model tree based on their subordinate relationships. The GDAHP method is employed to determine the weights of constituting factors of each layer in the evaluation model, and the Fuzzy method to establish the remark sets of factors of the criterion layer, thus the single-layer evaluation and comprehensive evaluation of urban lake landscape quality is carried out.

2) Application of quantitative methods in the quality evaluation of urban lake landscape in this research has remedied the disadvantages of subjective evaluation, improving efficiency and accuracy. This model can be employed to compare the landscape quality of different lakes as well as for the optimal selection of different plans for the same lake landscape.

3) This model has been used to evaluate the landscape quality of a lake in Xinxiang city, Henan province and analyze the quality of indexes of each layer as well as the overall quality, thus providing grounds for landscape renewal and reconstruction.

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Evaluation of Rainwater Harvesting Methods and Structures Using Analytical Hierarchy Process for a Large Scale Industrial Area

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Abstract

In India, with ever increasing population and stress on natural resources, especially water, rejuvenation of rainwater harvesting (RWH) technique which was forgotten over the days is becoming very essential. Large number of RWH methods that are available in the literature are demand specific and site specific, since RWH system depends on the topography, land use, land cover, rainfall and demand pattern. Thus for each and every case, a detailed evaluation of RWH structures is required for implementation, including the analysis of hydrology, topography and other aspects like site availability and economics, however a common methodology could be evolved. The present study was aimed at evaluation of various RWH techniques in order to identify the most appropriate technique suitable for a large scale industrial area to meet its daily water demand. An attempt is made to determine the volume of water to be stored using mass balance method, Ripple diagram method, analytical method, and sequent peak algorithm method. Based on various satisfying criteria, analytical hierarchy process (AHP) is employed to determine the most appropriate type of RWH method and required number of RWH structures in the study area. If economy alone is considered along with hydrological and site specific parameters, recharging the aquifer has resulted as a better choice. However other criteria namely risk, satisfaction in obtaining required volume of water for immediate utilization etc. has resulted in opting for concrete storage structures method. From the results it is found that AHP, if used with all possible criteria can result in a better tool for evaluation of RWH methods and structures. This RWH structures not only meets the demand but saves transportation cost of water and reduces the dependability of the industry on irrigation reservoir. Besides monetary benefits it is hoped that the micro environment inside the industry will improve due to the cooling effect of the stored water.

Keywords: Rain Water Harvesting, Analytical Hierarchy Process, Large Scale Industrial Area, Aquifer Recharge, Surface Storage Structures, Concrete Storage Structures

1. Introduction

The increasing growth in population, industrialization and urbanization is causing severe impact over the water resources. The overexploitation of natural water resources has already created environmental problems all over the world. In India, conflicts on river water sharing between the states have already started. One of the major solutions to meet ever increasing water demands would be storing the available rainwater through rainwater harvesting techniques (RWH) [1]. The term RWH implies conservation of rainwater where it falls [2] which was also an age old tradition in India [1]. The recorded evidence of water harvesting is found in Harappan and pre Harappan civilizations dating back to 4000 to 6000 years [3]. However with the changing world and modernization, with construction of large scale reservoirs and water supply schemes, concept of RWH has lost its presence in middle era. Recently the increasing water demand, non-availability of space for large reservoirs, and its subsequent problems have forced to revive the concept of RWH.

An overall review on RWH can be seen in Boers and Ben-Asher [4]. Throughout the world many Govern-
mental and Non-Governmental organizations have prepared and issued guidelines regarding RWH [5–8]. Fairly a good number of site specific and case studies based RWH literature is available [9–23]. Most of the above studies concluded that RWH is one of the best methods to solve the serious problem (situation) of catering the increasing water demand, also for Governments it is the drought relief programs. Various types of RWH methods and structures are available in India (Table 1), however the choice of any RWH structure is very site specific and depends on topography, rainfall, runoff, demand, land use pattern and land availability. Almost all RWH studies are aimed at providing good source of water or augmenting irrigation supply or improving the watershed development. Until now in most of the studies, selection of particular type was purely based on hydrological and economic criteria rather than other satisfying criteria. In India, RWH has high potential in large scale industrial sector, where large area is available for RWH. Besides various advantages, the major benefits of RWH in an industrial area are: the end use of harvested water is located close to the source, eliminating the need for complex and costly distribution systems. Rainwater has zero hardness eliminating the need for a sophisticated water treatment process. It can reduce the dependency of the industry on irrigation reservoirs, it is also hoped that this RWH will improve the micro environment inside the industry and contribute to self sufficiency of the industry in its BLUE ENERGY (water power) leading to sustainable development.

In the present study, the main aim is to identify an appropriate RWH structure for a large scale automobile industry in India. The first step in designing any RWH structure is to determine the volume of water to be stored. In this case it was achieved using four methods namely: mass balance, Ripple diagram, analytical and sequent peak algorithm methods. The most appropriate RWH method and number of RWH structures for the given volume of water from various alternatives has been determined using analytical hierarchy process (AHP). The alternatives are evaluated against 16 (quantitative and qualitative) attributes to select an appropriate method and number of RWH structures.

2. Materials and Methods

2.1. Study Area

A large scale automobile industry situated near Nasik (Igatpuri), Maharashtra, India is considered for the study. Presently the industry is purchasing water from an irrigation reservoir (Talegaon dam, situated approximately 2 km on the south of the factory) owned by Maharashtra Jeevan Pradhikaran, Government of Maharashtra. The factory is situated in a tropical wet climatic region and receives an average annual rainfall of 2983 mm. Hence there is scope for reducing the expenditure on water through RWH, and also chances of reducing the dependency on the irrigation reservoir. The industry has a total plot area of 253,000 m² (25.3 ha) with a total built up area of 46,500 m² (the main factory alone). The area is moderately undulating with hard rock sub-surface overlaid by a soil cover ranging from 2 to 3 m. The present water consumption of the industry is 6,616 m³/month leading to an annual demand of 79,392 m³. The present annual expenditure (based on slab rates) on water is Rs. 3,652,032 (1USD = Rs. 45).

2.2. Rainfall Analysis over the Study Area

For the present study, 34 years (1971-2004) of daily rainfall data pertaining to Igatpuri rain-gauge station has been obtained from India Meteorological Department (IMD), Pune. The summary of the rainfall analysis is depicted in Table 2. The region receives an average annual rainfall of 2,983 mm occurring over 103 rainy days. The highest observed rainfall over 34 years is 4,205 mm during the year 1994 and minimum is 2,083 mm during the year 2000. 95% of the annual rainfall occurs during the South West monsoon (June to September). Since the rainy days are more during the monsoon months they show high spread and low peak. From Table 2 it can be seen that the month of July receives highest rainfall in a year, 1061 mm and with no rainfall during March. The rainfall has very low spread and high peakedness during the low rainfall months, (November to May) the reason being less number of rainy days. All the rainfall in the low rainfall month occurs in just 2 to 3 day leading to low spread and high peakedness.

The average daily rainfall at the study area for the past 34 years is shown in Figure 1, indicating the variation of the daily rainfall within a year. Figure 2 shows annual rainfall over Igatpuri region along with number of rainy days in a year. Over the past 34 years the area has seen a maximum of 124 rainy days in 1993 and minimum of 78 rainy days in the year 1972. This daily and monthly rainfall data has been used in estimating the volume of water to be stored in order to meet the daily water demand throughout the year.

2.3. Volume of Water to Be Stored through RWH

With basic calculations, the volume of average annual rainwater available from the roof top area of 46,500 m² with a runoff coefficient of 0.9 (average rainfall of 2,983 mm) is 124,838 m³, whereas the annual demand is 79,392 m³ only. This shows that the runoff available from the single roof top of the industry alone is sufficient to meet the annual water demand. In this case, the supply is more than the demand, thus it is necessary to find the
Table 1. Classification of RWH structures.

| Sr No | Topography                          | Main feature                  | Rainwater harvesting structure |
|-------|-------------------------------------|-------------------------------|-------------------------------|
| 1     | Forest and hilly areas              | Undulating surface, vegetative cover | 1. contour trenching  
2. vegetative barriers  
3. gulley control structures  
4. catch pits  
5. percolation ponds  
6. water spreading |
| 2     | Plain areas                         | Gentle slopes, very low undulating surfaces | 1. percolation ponds  
2. injection wells  
3. furrow ditches  
4. infiltration galleries  
5. ducts  
6. anicuts across streams  
7. minor irrigation tanks  
8. farm ponds |
| 3     | Coastal and desert areas            | Sandy soil. High infiltration | 1. infiltration galleries  
2. sub surface check dams  
3. percolation ponds  
4. canals |
| 4     | Built up areas                      | Higher percentage of impervious surface | 1. temple tanks  
2. rooftop harvesting  
3. wells and radiator wells  
4. parking lot storage  
5. recreational park ponds |

Table 2. Statistical properties of the rainfall data.

| Month   | Average rainfall (mm) | Std Deviation (mm) | Skewness | Kurtosis |
|---------|-----------------------|--------------------|----------|----------|
| January | 2.27                  | 6.31               | 2.85     | 7.44     |
| February| 0.02                  | 0.11               | 5.48     | 30.00    |
| March   | 0.00                  | 0.00               | 0.00     | 0.00     |
| April   | 2.71                  | 10.76              | 4.14     | 17.34    |
| May     | 21.41                 | 65.06              | 4.78     | 24.53    |
| June    | 496.78                | 247.92             | 0.39     | -0.25    |
| July    | 1061.21               | 280.87             | 0.43     | -1.00    |
| August  | 926.07                | 323.98             | 0.55     | 0.48     |
| September| 368.94              | 238.94             | 0.27     | 0.73     |
| October | 78.66                 | 72.94              | 0.80     | -0.54    |
| November| 22.86                 | 41.38              | 2.04     | 3.43     |
| December| 2.51                  | 6.08               | 2.32     | 4.17     |
| Annual  | 2983                  | 385.2              | 2.38     | 6.29     |

2.4. Choice of RWH Structure

Once the volume of water to be stored is determined, the next step is to select the appropriate RWH structure. Analytical Hierarchy Process (AHP) is used for this purpose. The selected RWH structure should have two important characteristics: first one is assured quantity of water at any given time, second is good quality of water. Based on the topography and economics of the study area, three broad RWH structures are considered for detailed AHP analysis, they are:

- RCC water tanks: these are the closed structures with no seepage and less evaporation losses, least interference with atmosphere. They can provide reliable water
supply with good quality with appropriate amount of treatment. But usually the construction costs are very high.

- Surface storage: these can be useful to store surface runoff effectively. They have lower construction cost but prone to seepage and evaporation losses. Also as they are open to surrounding environment and prone to various contaminations and biological activities.
- Ground water recharging: these are effective if sufficient good aquifers available. These have least cost, but the storage capacity depends on many external factors.

The above alternatives have their own advantages and disadvantages over others. The other points to be considered are reliable supply, water quality etc, instead of just going with cost benefit analysis, for this purpose one of the multi criteria decision making processes AHP is used.

2.5. Analytical Hierarchy Process (AHP)

AHP is a general theory of measurement used to derive ratio scales from both discrete and continuous paired comparisons [24]. It is used to determine the relative importance of a set of activities or criteria. The novel aspect and major distinction of this approach is that it structures any complex, multi-person, multi-criterion and multi-period problem hierarchically. Using a method for scaling the weights of the element in each level of the hierarchy with respect to an element (e.g., criterion) of the next higher level, a matrix of pair wise comparisons of the activities can be constructed where the entries indicate the strength with which one element dominates another with respect to a given criterion. This scaling formulation is translated into a principal eigen value problem which results in a normalized and unique vector of weights for each level of the hierarchy (always with respect to the criterion in the next level), which in turn results in a single composite vector of weights for the entire hierarchy. This vector measures the relative priority of all entities at the lowest level that enables the accomplishment of the highest objective of the hierarchy.

3. Results and Discussions

As indicated earlier the primary objective of the study is to select an appropriate RWH method and number of RWH structures for the industry which satisfies the hydrological, technical, economical and satisfaction criteria along with the implementable or amenable solution by the industry. For this purpose first the volume of water to be stored is assessed based on the prevailing hydrologic (rainfall-runoff) condition and demand in the industrial area. Then the appropriate method is selected using AHP based on satisfying criteria, the results are as follows:

\[
Q = C_i A
\]

(1)

where, \(Q\) – runoff, \(C\) - runoff coefficient, \(i\) - rainfall intensity and \(A\) - rooftop area

The shaded portion in Figure 3 is the deficit volume in meeting the demand, this much of volume needs to be stored in the water rich period. Table 3 elaborates the monthly mass balance method. Since 95% of runoff occurs in four months (June, July, August, and September) the demand in these four months is met by the rainfall in these months. However the demand of remaining eight months should be met by the stored water in these four months. From Table 3 it is seen that the demand for eight months is 52,928 m³, hence the size of the reservoir should be 52,928 m³ or atleast 50,761 m³ (as the expected runoff during dry months is 2167 m³).

3.2. Ripple Diagram Method

This method considers the difference between the demand and supply over the period of time. To find out this difference, cumulative runoff is plotted against time. Cumulative demand is plotted and then superimposed on this graph starting from the peak of the dry period. If more peaks are available, the cumulative demand line may be started from each peak. Maximum difference
Table 3. Result of mass balance method.

| Month | Rainfall (mm) | Runoff (m³) | Demand (m³) | % of total runoff | cumulative % runoff | Total Demand (m³) |
|-------|---------------|-------------|-------------|------------------|---------------------|------------------|
| 1     | June          | 496.78      | 20790.2     | 6616             | 16.651              | 16.651           |
| 2     | July          | 1061.21     | 44411.5     | 6616             | 35.57               | 52.221           |
| 3     | August        | 926.07      | 38756       | 6616             | 31.04               | 83.261           |
| 4     | September     | 368.94      | 15440.1     | 6616             | 12.366              | 95.627           |
| 5     | October       | 78.66       | 3292.06     | 6616             | 2.637               | 98.264           |
| 6     | November      | 22.86       | 956.83      | 6616             | 0.766               | 99.03            |
| 7     | December      | 2.51        | 104.9       | 6616             | 0.084               | 99.114           |
| 8     | January       | 2.27        | 94.86       | 6616             | 0.076               | 99.19            |
| 9     | February      | 0.02        | 0.84        | 6616             | 0.001               | 99.191           |
| 10    | March         | 0           | 0           | 6616             | 0                   | 99.191           |
| 11    | April         | 2.71        | 113.41      | 6616             | 0.091               | 99.282           |
| 12    | May           | 21.41       | 896.01      | 6616             | 0.718               | 100              |

between the supply and demand over the period of time is the capacity of RWH structure. This method considers two main assumptions:

1) if N years of data is available, the inflow and demands are assumed to repeat in cyclic progression of N year cycles;
2) the reservoir is assumed to be full at the beginning of dry season.

Figure 4 elaborates the procedure of Ripple diagram method. The maximum deficit works out to be 53,409 m³, and is the volume of water to be stored.

3.3. Analytical Method

In this method, the surplus or deficit for each time period is estimated and the cumulative is calculated. If there is a shift from surplus to deficit or vice versa in a time period, the cumulative is started afresh. Sample calculation for a year (1971) is given in Table 4. The same procedure has been followed for all the years (34 years) individually as well as continuously to take care of carry over storage. The cumulative deficits are listed to find the maximum deficit, the maximum deficit works out to be 53,409 m³ which is same as that of Ripple diagram method.

3.4. Sequent Peak Algorithm Method

This is a variation of the basic mass curve method to facilitate graphical plotting and handling of large data. In the sequent peak algorithm a mass curve of cumulative net flow volume against time (or residual mass curve) is used. This net flow is estimated using Equation (2).

\[
NF_t = R_t - D_t
\]  

(2)

where, \(NF_t\) - Net flow volume during the period \(t\), \(R_t\) - Runoff volume during the period \(t\), and \(D_t\) - Demand volume during the period \(t\).

Cumulative net flow for 34 years is plotted against time as shown in Figure 5. For any peak \(P_i\), the next following peak \((P_{i+1})\) of magnitude greater than \(P_i\), is called a sequent peak. The lowest point between \(P_i\) and \(P_{i+1}\) is called trough \(T_i\). Likewise the sequent peaks \(P_i\) and troughs \(T_i\) can be found and the required RWH structure capacity \(S_i\) is estimated as:

\[
S_i = \text{Max of } (P_i - T_i) \quad i = 1,2,\ldots,n
\]  

(3)

It is evident from Figure 5 that the demand is less than the supply, thus the cumulative net flow is getting accumulating over the years. Values of peaks and troughs were found out from the graph and the differences were calculated. The maximum difference is 53,408.79 m³ and therefore the minimum volume of water to be stored should be 53,409 m³.

3.5. Appropriate Volume of Water to Be Stored

The comparative volume of rain water to be stored re-
Table 4. Analytical method – sample calculations for the year 1971.

| Month   | Rainfall (mm) | Runoff (m³) | Demand (m³) | Surplus (m³) | Deficit (m³) | Cumulative surplus (m³) | Cumulative deficit (m³) |
|---------|---------------|-------------|-------------|--------------|--------------|------------------------|------------------------|
| January | 0             | 0           | 6616        | -6616        | 0            | -6616                  | -6616                  |
| February| 0             | 0           | 6616        | -6616        | 0            | -13232                 | -19848                 |
| March   | 0             | 0           | 6616        | -6616        | 0            | -26464                 | -31364.15              |
| April   | 0             | 0           | 6616        | -6616        | 0            | -31364.15              | -31364.15              |
| May     | 41            | 1715.85     | 6616        | -4900.15     | 0            | -31364.15              | -31364.15              |
| June    | 769           | 32182.65    | 6616        | 25566.65     | 0            | -31364.15              | -31364.15              |
| July    | 922           | 38585.70    | 6616        | 31969.70     | 0            | -31364.15              | -31364.15              |
| August  | 1118          | 46788.30    | 6616        | 40172.30     | 0            | -31364.15              | -31364.15              |
| September| 912          | 38167.20    | 6616        | 31551.20     | 0            | -31364.15              | -31364.15              |
| October | 22            | 920.70      | 6616        | -5695.30     | 0            | -31364.15              | -31364.15              |
| November| 0             | 0           | 6616        | -6616        | 0            | -12311.30              | -12311.30              |
| December| 0             | 0           | 6616        | -6616        | 0            | -18927.30              | -18927.30              |

Table 5: Summary on volume of water to be stored

| Method               | Reservoir capacity (m³) |
|----------------------|-------------------------|
| Mass balance method  | 52928                   |
| Ripple diagram method| 53409                   |
| Analytical method    | 53409                   |
| Sequent peak algorithm| 53408.79               |

Figure 5. Results of Sequent peak algorithm method.

Table 5: Summary on volume of water to be stored

RWH structure are evaluated against 16 attributes listed below using AHP. The initial AHP values for all the 16 attributes against the three alternative RWH techniques are shown in Table 6. Description of these attributes is given below. Attribute A1 to A7 are qualitative attributes while A8 to A16 are quantitative attributes.

1) Certainty of storage estimate is the confidence level by which estimated storage capacity of the structure can be trusted. The highest weightage is given for the structure which has maximum certainty in storage capacity estimation. The tanks are precisely designed to store stipulated volume of water and hence have maximum certainty in the estimate. In case of surface water storage large number of field surveys needs to be carried out to estimate the storage capacity. In case of ground water recharge it is extremely difficult to estimate the capacity of the aquifer and hence it is most uncertain of all. Thus the RCC tank has given maximum priority in calculating the final attribute matrix (Table 7).

2) Location / physical conditions is the location feasibility including the physical conditions like topology, subsurface structure etc. The study area has moderately undulating surface and more than 20 ha of open land is available. There is ample space available to construct a tank. Two locations are available for the construction of surface storage structures. The sub-surface strata mainly comprises of hard rock. This makes it very difficult for water to percolate, if aquifer recharge method is used. Thus for this attribute the tank has highest priority while ground water recharge has lowest priority. Surface storage has slightly less priority than that of tank.

3) Inspection and repairing feasibility is the feasibility to inspect the structure and to repair in case of any damage. The part of the tank above ground level is very easy to inspect and repair if necessary. But the part of the tank below ground level can be inspected only when it is empty. Similarly the wall of surface storage structure is...
### Table 6. Initial AHP attributes to identify most appropriate RWH structure.

| Attributes                      | Technology alternatives                  |
|--------------------------------|-----------------------------------------|
|                                | Tank | Surface storage | Groundwater recharge |
| A1 Certainty of storage estimate | High | Moderate        | Uncertain             |
| A2 Location / physical conditions | Available | Available on different locations | Hard Rock. Difficult to find aquifer |
| A3 Inspection and repairing feasibility | Conventional | Inspection of wall is convenient but under ground inspection is difficult | Very difficult |
| A4 Water assurance               | High | Moderate        | Low                   |
| A5 Area utilization after construction | High | Zero           | Can be used           |
| A6 Quality of water              | No external contamination, low biological activity | External contamination and biological activities bound to happen | Chances of mixing with underground minerals and impurities |
| A7 Danger of catastrophic structure failure | Yes | Yes | Not much |
| A8 Ground Area Required (sq m)   | 13,500 | 13,500 | 100 |
| A9 Minimum storage capacity required (cu m) | 55,000 | 76,500 | 55,000 |
| A10 Construction cost (Rs)       | 35,000,000 | 6,500,000 | 2,060,000 |
| A11 Pumping Cost (Rs / year)     | 114,361 | 114,361 | 171,542 |
| A12 Maintenance cost (Rs)        | 52650 | 15000 | 12400 |
| A13 Unit cost (Rs / cu m)        | 636.36 | 85 | 37.5 |
| A14 Payback period (years)       | 9.7 | 4.8 | 3.03 |
| A15 Evaporation losses (mm / year) | 0 | 1587 | 0 |
| A16 Time required for system to get stabilized (years) | 1 | 5 | 10 |

### Table 7. Final attribute matrix.

| Attribute                                    | Tank | Surface storage | Groundwater recharge | Weight |
|----------------------------------------------|------|-----------------|-----------------------|--------|
| A1 Certainty of storage estimate             | 0.717065 | 0.2171656 | 0.0657693 | 0.1452119 |
| A2 Location / physical conditions            | 0.6153283 | 0.3186614 | 0.0660103 | 0.0273137 |
| A3 Inspection and repairing feasibility      | 0.4666667 | 0.4666667 | 0.0666667 | 0.0217646 |
| A4 Water assurance                            | 0.6716255 | 0.2654333 | 0.0629412 | 0.1423368 |
| A5 Area utilization after construction        | 0.6152534 | 0.0925277 | 0.292219 | 0.045193 |
| A6 Quality of water                           | 0.5714286 | 0.1428571 | 0.2857143 | 0.0466549 |
| A7 Danger of catastrophic structure failure   | 0.2 | 0.2 | 0.6 | 0.0293255 |
| A8 Ground Area Required (sq m)               | 0 | 0 | 1 | 0.0270953 |
| A9 Minimum storage capacity required (cu m)   | 0.5 | 0 | 0.5 | 0.0358576 |
| A10 Construction cost (Rs)                    | 0 | 0.3 | 0.7 | 0.0358576 |
| A11 Pumping Cost (Rs / year)                  | 0.5 | 0.5 | 0 | 0.0768751 |
| A12 Maintenance cost (Rs)                     | 0 | 0.3 | 0.7 | 0.0768751 |
| A13 Unit cost (Rs / cu m)                     | 0 | 0.3 | 0.7 | 0.0358576 |
| A14 Payback period (years)                    | 0 | 0.4 | 0.6 | 0.0768751 |
| A15 Evaporation losses (mm / year)            | 0.5 | 0 | 0.5 | 0.0316942 |
| A16 Time required for system to get stabilized (years) | 0.6 | 0.4 | 0 | 0.1452119 |

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very convenient to inspect but if there are any subsurface cracks or fractures, it is very difficult to inspect and repair. In case of ground water recharge, it is almost impossible to do any inspection. Thus tank and surface storage has same priority, greater than that of ground water recharge.

4) Water assurance is the confidence level by which the user can use the stored water from the RWH structure at any given time. In case of the tank, there is minimum evaporation and seepage loss, and hence the tank can provide assured water. The surface storage structure is prone to water losses in the form of evaporation and seepage; also sedimentation may reduce the storage capacity, hence surface storage is considered to have less assurance than that of the tank. The underground aquifers have least assurance.

5) Area utilization after construction is the possible area of the storage structure that is brought in use after construction. If due care is taken in design and construction of the water tank, the area above the tank may be brought under utilization. On the other hand the area of surface storage structure cannot be brought under any other use. Thus the tank has highest priority and surface storage has least. The ground water recharge structures are small in size and their presence does not affect day to day activities. Thus it has been given priority in between the other two alternatives.

6) Quality of water is the level of contamination to which the water is prone to. As water in tank can be completely isolated from air and sunlight, if filtered properly before its entry into the tank, it can be of good quality. The surface water has more chances of contamination with hazardous substances. Also due to direct contact of water with air and sunlight there is high possibility of plankton growth and other biological activities. Thus the tank has highest priority while surface storage has the least if water quality is considered. Ground water is naturally filtered. It has very less chances of any biological activity. But it may have the problem of dissolved minerals. Hence ground water storage has been given the priority in between the other two.

7) Danger of catastrophic structure failure: Open structures like surface storages and tanks are more prone to this than ground water recharge. Even though due care is taken in the design and construction of water tank and surface storage structure, there is a risk of catastrophic structure failure due to unknown reasons. But in case of ground water recharge only few things like large scale earthquakes or heavy underground blasting may damage the natural aquifers. Thus ground water recharge has been given high priority and the remaining two have same priority but lower than that of the ground water recharge.

8) Ground area required is the total area required above surface for storage of the water. Depth of the water is assumed to be 4 m. Ground area required for tank and surface storage is calculated assuming the average depth of the water storage equal to 4 m. It will change depending on the designed depth of the storage. The area once used for the RWH cannot be used for any other usage except in ground water charge, hence has high priority and other two has no priority at all.

9) Minimum storage capacity required is the capacity of the structure for which the stored water will be sufficient to meet annual demand. Minimum storage capacity of the water storage structure is considered to depend on the water losses that the structure incurs. It is assumed that tank has negligible losses. Hence its minimum capacity should be equal to the minimum volume of water need to be stored to meet daily water requirement of the year. The minimum capacity of surface water storage is equal to the minimum volume of water need to be stored plus the total evaporation and seepage losses throughout the year. It is very difficult to find the water loss in underground aquifers. Since no loss in tank and ground water they have equal priority and higher than surface storage which has more losses.

10) Construction cost is the cost of the structure (in Indian rupees). An estimate of construction cost of the RCC tank, surface storage structure and recharge pits are carried out based on the prevailing cost of materials at the industry. Since the cost of tank is very high its priority is low, the ground water recharge requires less money and has high priority, the surface storage has priority in between these two methods.

11) Pumping cost influences the operation cost of the given alternative. Pump capacity is estimated based on the suction head, then the pumping cost is estimated assuming industrial power rate of Rs. 7 per unit. Since in ground water more number of wells and pumps are required its priority in this case is less than the other two methods.

12) Maintenance cost for tank includes cleaning the filters and the tank. For surface storage, desilting and arresting the plankton growth are the main activities. For recharging structures, removal of sediments is the main maintenance task. Since ground water recharge does not require any maintenance it is given highest priority than other two methods.

13) Unit cost (Rs/ m³) is the cost per cubic meter of water stored. Unit cost of water storage is important in a sense that it gives insight of how much are the charges to store unit volume of water. It is the ratio of total cost (construction, pumping and maintenance cost) to the volume of water stored. Since the unit cost of water stored in tank is very high it has less priority (no priority).

14) Payback period is the time taken to recover the cost of investment made on the RWH structure. To calculate payback period, construction cost and mainte-
nance cost is considered. The data on existing water charges borne by the industry is also used to compare the pay back period.

15) Evaporation and seepage losses (in mm) are prevalent in open structures like surface storages and negligible in tanks, and ground water recharge methods.

16) Time required for the system to get stabilized. Construction of surface storage structure and ground water recharge structures is directly related to the natural hydro-geology of the area, hence it takes some time before the system begins to give consistent results. While the tank has no such conditions, it takes first cycle of rainy and dry periods to show the results and hence has high priority.

The discussion up till now is on the basis of the AHP. The criteria comparison matrix and technology comparison matrices for qualitative attributes are formulated based on above discussion. By processing these matrices, final attribute matrix was determined, and is shown in Table 7 along with the weight of each attribute. Table 7 contains values of quantitative as well as qualitative attributes represented between 0 and 1. Values of qualitative attributes are calculated by pair wise comparison. Values of quantitative attributes are arrived at by simple calculations. In AHP terminology, all the quantitative attributes in discussion are “cost attributes”. This means lower the value better is the alternative. Here 0 being of least desirable alternative and 1 being the most desirable alternative for the given attribute. These are nothing but the relative weightage of each RWH alternative on the scale of 0 to 1.

The results show that each alternative can be a most desirable for one attribute and least desirable for other attribute. For example, in case of construction cost, the tank is least desirable, and the groundwater recharge is most desirable and surface storage stands in between. But in “certainty of storage estimate”, tank is most desirable while the groundwater recharge is least desirable. In case of pay back period ground water recharge is most desirable, but when it comes to water assurance it has the lowest desirable method and also takes more time period to get consistence results. Also one may get wrong idea when considered only pay back period and if time taken to stabilization is not considered, as per the individual priority the ground water has high rank in pay back, but it is not assured because the system itself takes 10 years to get stabilized, thus the priority of individual attributes alone will lead to selection of wrong RWH method. So in order to make a choice between the available technology alternatives, one must know the importance of each attribute over other attributes so that the most desirable alternative with respect to most important attribute may be an ultimate choice.

In order to achieve this, the attribute weights are calculated by pair wise comparison of each attribute with every other attribute. Weights of the attributes (listed in last column of Table 7) are nothing but the importance of each attribute over the other attribute quantified between 0 and 1. The sum of all the weights is necessarily equals to 1. Also the attributes representing assured storage of water (A1, A4 and A16) have higher weightage than other attributes. The weights of construction cost is less than the pumping and maintenance cost, thus giving less weight to one time investment and more weight to recurring cost. The normalized weights with respect to RWH structures are represented by “Pij” and the attribute weights by “Wj”. The final weight of each alternative is calculated by the Equation (4) and resulted final weights and ranking of the alternatives is shown in Table 8.

\[
\text{Final Weight} = \sum_{j=1}^{16} P_{ij} \ast W_j, \quad i = 1, 2, 3
\]

As a result of final weight, the RCC tank has highest weight (0.446) while the surface storage and groundwater recharge has equal weight of 0.277 each. It means that for given perspective of assured storage of water with good quality, the most appropriate alternative is the RCC tank. Though it has highest cost, it can assure the designed storage capacity with good quality of water which has more weightage in this case. By considering the uncertainty in groundwater storage in the study area, surface storage can be given second priority.

3.7. Most Effective Method to Install Tank

As a result of above AHP “RCC Tank” has resulted in as a most appropriate alternative RWH method. RCC tank can be installed in various combinations of size and number of tanks. Each combination has its own advantages and disadvantages over the other in terms of cost, area required, safety and flexibility in operation. Eight different combinations of size and number of tanks listed in Table 9 with respective to their costs and area required are considered for further selections. It is needed to find the most appropriate combination amongst these. It is observed that height of the tank less than 3 m result in very high area on the ground while height more than 4 m cause problems in inspection and repairing. So for comparison purpose, tanks of height 3 and 4 meter are considered. Also it is understood that excessive number of tanks are difficult to manage. So the number of tanks is limited to 4. It can be seen that cylindrical tanks have slightly less area requirement than that of square tanks. But they have higher construction cost. Also square tanks
Table 9. Alternatives of tank installation.

| Storage capacity of each tank (m³) | No of tanks | Height (m) | Total area on the ground (m²) including the wall thickness and other amenities | Total Cost estimate (Rs) |
|-----------------------------------|-------------|-----------|--------------------------------------------------------------------------------|-------------------------|
|                                   |             |           | square tank                                                             | cylindrical tank        |
|                                   |             |           | square tank                                                             | cylindrical tank        |
| 55000                            | 1           | 4         | 13968.76                                                               | 13915.02                |
| 27500                            | 2           | 4         | 14165.27                                                               | 14088.77                |
| 18333                            | 3           | 4         | 14316.98                                                               | 14222.82                |
| 13750                            | 4           | 4         | 14445.52                                                               | 14336.33                |
| 55000                            | 1           | 3         | 18540.66                                                               | 18478.74                |
| 27500                            | 2           | 3         | 18766.95                                                               | 18678.88                |
| 18333                            | 3           | 3         | 18941.52                                                               | 18833.19                |
| 13750                            | 4           | 3         | 19089.31                                                               | 18963.76                |

Table 10. Initial AHP table to identify right combination of tank size and number of tanks.

| Attributes | Alternatives |
|------------|--------------|
| A1 Cost in crores (Rs) | H4#1 | H4#4 | H3#1 | H3#4 |
| A2 Area on ground | 13915.02 | 14336.33 | 18478.74 | 18963.76 |
| A3 Safety | Low | high | Low | High |
| A4 Flexibility | Low | high | Low | High |

H4#1: its full capacity single tank with height 4 m, H4#4: these are 4 tanks with 4 m height. Together they make full capacity, H3#1: full capacity single tank with height 3 m, H3#4: these are 4 tanks with 3 m height. Together they make full capacity, A1: Cost: it is cost of construction in crore Rs.A2: Area on ground: it is area on the ground covered by the structures A3: Safety: it is the risk division in case of any damage A4: Flexibility: it is the flexibility in operation, maintenance and other activities.

Table 11. Final weights and Ranks of the tanks.

| Alternative | H4#1 | H4#4 | H3#1 | H3#4 |
|-------------|------|------|------|------|
| Weight      | 0.2652496 | 0.3938832 | 0.2515686 | 0.3223847 |
| Rank        | 3    | 1    | 4    | 2    |

are easy to construct. Considering this, four extreme alternatives of square tanks were chosen for comparison using AHP, Table 10 is the initial setup for the performance of AHP.

In case of safety, the risk of any damage gets divided on the number of tanks. If there is only one tank and it gets damaged, there is danger of loss of all the water. But in case of 4 tanks; if one is damaged, the water in remaining tanks would be safe. Also more number of tanks gives more flexibility in operation, maintenance, inspection and repairs. It can be seen that cost of combination of four tanks is higher than that of one tank. But four tanks have higher safety and flexibility of operation.

Thus depending on the weightage of attributes the choice of combination will defer. AHP is carried out to identify most appropriate combination out of these four combinations. The result of AHP is shown in Table 11. It can be seen that H4#4 has the highest weight (0.39).

Thus, the installation of 4 square tanks with 4 m height is the most appropriate option considering cost, area utilization, safety and flexibility of usage.

The above results show that RWH is becoming one of the inevitable solutions to cater the increasing water demand and may also solve the dispute of water sharing among the users from a common source. If individual criteria are considered while selecting an appropriate RWH method, they are leading to wrong selection hence the selection of appropriate RWH method should be based on hydrological, demand pattern, storage requirement, economical, certainty of storage water, water assurance, payback period, time of stabilization etc. To have a methodological selection based on the various criteria, AHP is found to be very useful tool, not only for RWH method but also for number of RWH structures. Even though the appropriate RWH method and structures for a large scale industry is evaluated using a
case study industrial area, this methodology can be applied to any other built up area (where less sediments flows into tanks) by using the relevant inputs namely the rainfall, demand and area available.

4. Conclusions

A large number of RWH methods are available in literature. However each method is site specific and demand specific. The RWH system depends on the topography, land use pattern, rainfall, demand pattern and economic status of the stake holder. Each structure requires detailed analysis of hydrology (rainfall and demand), toography and other aspects. The present study is aimed at providing best techno-economic RWH structure so as to minimize or eliminate the dependency of the industry on purchased water. With the available data the first step is to find the volume of water need to be stored. All the four methods employed in the present study resulted in identical volume of 55,000 m$^3$. Then the systematic methodology of AHP was applied to identify most appropriate RWH structure to store the required quantity of water with given conditions. As a result of this process “RCC tank” was identified as the most appropriate RWH structure for given requirement and conditions. Further, AHP was applied one more time to identify right combination of tank size and number of tank. As a result “four RCC cubical tanks with 4 m height” was identified as the most appropriate choice of RWH method for study area under given requirements and conditions.

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Soil Moisture Retrieval Quantitatively with Remotely Sensed Data and Its Crucial Factors Analysis

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Abstract

The Ts/NDVI method was adopted to retrieve soil moisture with multi-temporal and multi-sensor remotely sensed data of ETM+ and ASTER in study area. The retrieved soil moisture maps were consistent with the soil type and vegetation, which were also the two main factors determining the distribution of soil moisture.

Keywords: Soil Moisture, Quantitative Remote Sensing, NDVI

1. Introduction

The Biospheric Aspects of the Hydrological Cycle (BAHC), one of the core projects of the International Geosphere Biosphere Programme (IGBP) coordinated by the International Council for Science (ICSU), was established to study the role of vegetation in the hydrological interactions between the land surfaces and atmosphere. One objective of BAHC is to determine the biospheric controls of the hydrological cycle through field measurements for the purpose of developing models of energy and water fluxes in the soil-vegetation-atmosphere system at temporal and spatial scales ranging from vegetation patches to General Circulation Model (GCM) grid cells [1,2]. This encouraged us to focus not only on the water itself, but also on its correlation factors such as the vegetation, the ground and so on when studying water environments and water resources, especially for water closely related to vegetation [3–9]. Soil moisture observations are one of these studies. Soil moisture is a key component of the hydrological cycle, controlling the partitioning of precipitation between runoff, evapotranspiration and deep infiltration. As a link between the biosphere and the edaphic zone, soil water plays a crucial role in terrestrial ecosystems by determining plant growth. If the soil water level falls below a species-specific threshold, plants experience water stress, and decreased soil moisture under warmer conditions can inhibit photosynthesis [10]. Soil moisture observations over large areas are increasingly required in a range of environmental applications including meteorology, hydrology, water resource management and climatology. Remote sensing can provide considerable cost-and time-savings when applied to mapping soil moisture over large areas. Various approaches have been developed over the past two decades to infer near-surface soil moisture from remote sensing measurements of surface temperature, radar backscatter and microwave brightness temperature [11–15].

In this paper, the soil moisture in study area was retrieved with Landsat Enhance Thematic Mapper Plus (ETM+) data and Advanced Spaceborne Thermal Emission and Reflection Radiometer (ASTER) data first, then, the influence factors of soil moisture was analyzed. The objective of this paper is to provide a new method for soil moisture monitoring.

2. Methods

2.1. Study Area

In this paper, the Maoergai area (from latitude 32°20' to 32°40' north and from longitude 103°00' to 103°30' east, shown as Figure 1), located in the upper Minjiang river, northern Sichuan province, in the southwest of China, was selected as the study area. The upper Minjiang river area has been famous as a “natural barrier” and the “green lake” of Sichuan province for a long time, where the forest area reached 12 thousand km², almost 50% of the whole area, 600 years ago. However, with the destruction of the forest, it had been reduced to only 7.4 thousand km², or about 30% of the whole area, by 1950. Meanwhile, the upper Minjiang is transforming from a pristine region to one with an increasing level of human activity, which leads to a degraded ecosystem in the area. The study area is a specific area in the upper Minjiang River with regard to its vegetation, soil, and mountainous characters, thus, the study result can be popularized in Minjiang river basin.
2.2. Data

In this study, we collected the soil type, groundwater and vegetation distribution data of the study area for the year 2000, which were collected within the gold-eye project subsidized by National High Technology Research and Development Program (HTRDP) [16], 48 ground control points (GCPs) distributed averagely in study area, the moisture of representative bare soils in the study area, the moisture of representative vegetations in the study area, all spectrums of representative soil and representative vegetation in the study area, Landsat ETM+ data of 10 July 2002 and ASTER data of 2 November 2003 of the study area for retrieving soil moisture. The remotely sensed data have been registered to UTM WGS-84 coordinates using the GCPs to allow co-registration of the ground data.

2.3. Retrieval and Analysis Processes

The retrieval and analysis processes were divided into 3 parts in this paper: First, pre-process of remotely sensed data, including geometric register, radiometric correction (radiometric calibration, topographic correction and atmospheric correction); Second, retrieval of biophysical parameter for land cover, including vegetation indices, soil brightness, soil wetness, greenness and surface temperature; Third, soil moisture retrieval with Ts/NDVI model and soil moisture analysis with soil type distribution data, vegetation distribution data and groundwater distribution data.

To better understand the soil moisture retrieval and analysis processes, a flow chart is provided in Figure 2.

3. Results

3.1. Soil Moisture Retrieval from Landsat ETM+ Data

Surface temperature ($T_s$, Kelvin), normalized difference vegetation index (NDVI) and the feature special diagram retrieved from ETM+ data of July 2002 in study area were shown in Figure 3.

To calculate the relative soil moisture, the dry edge linear equation and wet edge linear equation of $T_s$/NDVI feature space should be calculated first. The dry and wet edge linear equations are calculated as follows:

**Dry edge:**

$$T_s - T_{max} = -18.356343 \times NDVI + 307.57766 \quad r = 0.47030258$$  

**Wet edge:**

$$T_s - T_{min} = 0.68168266 \times NDVI + 288.26377 = 0.30658477$$

With the data of soil moisture and experiential formulas [1,2,4,8–10] in study area as a reference, the relative soil moisture of any point in ETM+ data’s $T_s$/NDVI feature space was calculated as Equation 3, where $R_SM$ is the relative soil moisture of any point, $T$ is the surface temperature of the point, $T_D$ is the surface temperature of dry edge which is corresponding to the NDVI of the point and $T_w$ is the one of wet edge.

$$RSM = 100 \times \frac{T - T_w}{T_D - T_w} \times 59.29378862$$  

Then, with the Equation 3, the relative soil moisture of
every pixel can be calculated. With the relative data of soil moisture content data and investigation data in study area as a reference, the practical soil moisture retrieved from ETM+ of July 2002 in study area is shown as Figure 4. According to Figure 4, the soil moisture was relatively high in July 2002 in study area; most points have a soil moisture between 18% and 25%, while few points have a soil moisture less than 15%. Contrasting the soil moisture map with the terrain and false colour maps, we can get that the soil moisture have a relatively high relation with the distribution of vegetation and terrain: the point near to water has a higher soil moisture value, vice versa; the point far from water has a relatively low soil moisture value.

3.2. Soil Moisture Retrieval from ASTER Data

The surface temperature $T_s$ (Kelvin), the normalized difference vegetation index (NDVI) and the feature special diagram retrieved from ASTER data of November 2003 in study area were shown as Figure 5.

As the data process of ETM+ data above, the dry edge linear equation and wet edge linear equation in ASTER $T_s/NDVI$ feature space were calculated first, shown as Equations (4) and (5).

Dry edge:

$$T_{s_{max}} = -35.698122 \times \text{NDVI} + 307.18527$$

$$r = 0.78242239$$  (4)

Wet edge:

$$T_{s_{min}} = 7.9501126 \times \text{NDVI} + 271.93125$$

$$r = 0.64890765$$  (5)

With the data of soil moisture and experiential Formulas [17] in study area being as a reference, the relative soil moisture of any point in ETM+ data’s $T_s/NDVI$ feature space was calculated as Equation 6. Where $RSM$ is the relative soil moisture of a point, $T$ is the surface temperature of the point, and $T_D$ is the surface temperature of
Figure 4. Overlaid map of groundwater distribution map with the soil moisture map retrieved from Remotely Sensed data. (White lines are the boundaries of groundwater, red words are groundwater codes).

Figure 5. NDVI, surface temperature and feature special diagram retrieved from ASTER data.

dry edge.

\[ RSM = 100 - \frac{T - T_{10}}{T_{B} - T_{10}} \times 79.2937862 \]  
(6)

Then, with the Equation 6, the relative soil moisture of every pixel can be calculated. With the relative data of soil moisture content data and investigation data in study area being as a reference, the practical soil moisture retrieved from ETM+ of July 2002 in study area was shown as Figure 6. According to Figure 6, the soil moisture was widely distributed in November 2003 in study area, between 5% and 25%, the reason is mostly like that it was the time when autumn was changing into winter with some bare land and some evergreen woods, leading to a various kinds of soil moisture distribution but most between 10% and 20%, which was consistent with practical situation. Some points have soil moisture of 0, the reason is mostly like that the soil moisture of bare rock and hardened soil in this season was very low. Meanwhile, some points have relatively high soil moisture, the
reason is mostly like that snow and clouds exist in ASTER data.

Contrasting Figure 4 with Figure 6, the soil moisture retrieved from ASTER data of November 2003 in study area has a relatively lower value than that retrieved from ETM+ of July 2002, the reason is that there is less water in winter but more in summer in study area.

4. Discussion

From the results above, we can see that soil moisture was closely related to soil type, vegetation type and water. In order to make clearer the relationship between soil moisture and soil type, vegetation type and water, the distribution map of soil moisture was overlaid on the distribution map of vegetation type, soil type and water system.

Figure 7 is the overlaid map of the soil type distribution map with the soil moisture distributed map retrieved from ETM+ and ASTER. From the contrast of Figure 7(a) and 7(b), we can see that soil moisture distributed map is deeply related to soil type, that is to say, soil moisture is different from one kind soil to another. As a whole, the soil moisture of brown soil (with code of B210) and dark brown soil (with code of B310) is higher, but the soil moisture of terra cinnamon soil (with code of C212) is lower, the soil moisture of other soil types are mid. However, the soil moisture of the same soil is different with the place change, which is similar in the two maps. Also, the soil moistures of different soils in different seasons are different.

Figure 8 is the overlaid map of the groundwater distribution map with the soil moisture distributed map retrieved from ETM+ and ASTER. From the contrast of Figure 8(a) and 8(b), we can see that the distribution of groundwater has some effect on soil moisture distribution, but the influence is weaker than the soil types’. General speaking, the influence of groundwater on soil moisture is stronger in summer than it is in winter; the reason is that the evapotranspiration is stronger in summer.

Figure 9 is the overlaid map of the vegetation distribution map with the soil moisture distributed map retrieved from ETM+ and ASTER. From the contrast of Figure 9(a) and 9(b), we can see that the distribution of soil moisture map is highly related to vegetation. Soil moisture varied with vegetation change. General speaking, the soil moisture on evergreen forest land is relatively higher, while cutover land’s is relatively lower. With the season change, the relativity of vegetation and soil moisture would change accordingly, because most deciduous forests’ evapotranspiration is different in different seasons, lead-
Figure 7. Overlaid map of the soil type distribution map with the soil moisture map retrieved from Remotely Sensed data. (White lines are the boundaries of soil types, red words are soil codes).
(a) Overlaid map of groundwater distribution map with soil moisture map retrieved from ETM.

(b) Overlaid map of groundwater distribution map with soil moisture map retrieved from ASTER.

Figure 8. Overlaid map of groundwater distribution map with the soil moisture map retrieved from Remotely Sensed data. (White lines are the boundaries of groundwater, red words are groundwater codes.)
Figure 9. Overlaid map of vegetation distribution map with the soil moisture map retrieved from remotely sensed data. (Blue lines are the boundaries of groundwater, red words are groundwater codes).
ing to the change of the ratio of soil water-holding.
From the analysis above, the model to retrieve soil moisture from remotely sensed data is viable, and, the potential influence factors of soil moisture are soil type and vegetation type.

5. Conclusions

In this paper, with the ETM+ and ASTER data, the soil moisture maps in study area were retrieved. The retrieved soil moisture maps were in agreement with the soil type and vegetation, which were also the two main factors determining the distribution of soil moisture.
As the remote sensing retrieval model of soil moisture was complex, the study was only at the beginning. In the following work, the authors plan to investigate soil moisture with hyper-spectral remotely sensed data to raise the accuracy and reliability of soil moisture retrieval.

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Stochastic Modelling of Actual Black Gram Evapotranspiration

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Abstract

The study was undertaken to develop and evaluate evapotranspiration model for black gram (Vigna Mungo L.) crop under climatic conditions of Udaipur, India. Pan evaporation data for the duration of twenty three years (1978-2001) and measured black gram evapotranspiration data by electronic lysimeter for duration of kharif season of 2001 were used for analysis. Black gram is an important crop of Udaipur region. No systematic study on modelling of black gram evapotranspiration was conducted in past under above said climatic conditions. Therefore, stochastic model was developed for the estimation of daily black gram evapotranspiration using 24 years data. Validation of the developed models was done by the comparison of the estimated values with the measured values. The developed stochastic model for black gram evapotranspiration was found to predict the daily black gram evapotranspiration very accurately.

Keywords: Black Gram (Vigna Mungo L.) Evapotranspiration, Crop Water Requirement, Stochastic Modelling

1. Introduction

The arid regions of India cover an area of 317,090 km² and lies between 24°-29° N latitude and 70°-76° E longitude. The region is spread over seven states, viz., Rajasthan, Gujarat, Punjab, Harayana, Maharashtra, Karnataka and Andhra Pradesh, the north-western part of the country constituting almost 90% of the total arid zone area. However, Rajasthan state alone accounts for 60% of the arid zone of India. The mean annual rainfall varies from 100 mm in the northwest to 450 mm in the eastern part of Rajasthan. The mean minimum and maximum temperature ranges from 16°C to 40°C. The potential evapotranspiration during summer is 7 to 9 mm/day, monsoon 5.3 to 6.4 mm/day and in winter 1.8 to 2.9 mm/day. Thus, the evapotranspiration far exceeds precipitation throughout most of the year. The present study underlines the importance of black gram as a pulse crop on the backdrop of scarcity rainfall in the arids of Rajasthan.

Mathematical models in hydrology are suitable abstraction of complex physical phenomena. The second order model approach is of major value in the construction of mathematical models of hydrologic time series. Observed data are treated directly to produce efficient estimates of spectral representation (variance spectral) or moment functions (correlograms) of stochastic process. Evapotranspiration process is stochastic in nature. Usually, the deterministic models do not consider the random effects and may not represent the evapotranspiration quite accurately. On the other hand, the stochastic models are based on the time dependent variations and consider random effects involved in the process. Stochastic models explain the extent of dependence of a present observation on the past observations.

A stochastic model is the mathematical abstraction of an empirical process and is governed by probabilistic laws [1].

Stochastic processes deal with continuous or discrete state and time parameters. The analysis of time series is done to understand the mechanism that generate the data and to produce likely future sequences if required. Time series are considered as a part of stochastic process.

The purpose of the stochastic model is to represent important statistical properties of one or more time series. Indeed, different types of stochastic models often studied in turns of statistical time series they generate. Models formulated on the stochastic concept explain the extent of dependence of the present observations on the past observations.

The first step in stochastic model construction is to select suitable classes or families of model from which the most appropriate model to a given time series can be
chosen by following the identification, estimation and diagnostic check stages of model development. For example, when modelling a hydrologic time series one may wish to consider the Auto Regressive Moving Average (ARMA) family of models (Box and Jenkins, 1976), the classes of non Gaussian models suggested by [2] and fractional differencing models [3].

Stochastic processes deal with continuous or discrete state and time parameters. Discrete series occur when the random variate in the time series is continuous but for computation and analysis purpose time is considered discrete. A stationary series may be modelled by short memory models and non-stationary series by long memory models. Short memory models of hydrologic phenomena include moving average (MA) models, autoregressive (AR) models, and autoregressive integrated moving average (ARIMA) models. Autoregressive model (AR) has been used extensively in hydrologic analysis.

Thus,
\[ Z_t - \mu = a_t + \Phi_1 (Z_{t-1} - \mu) + \Phi_2 (Z_{t-2} - \mu) + \ldots + \Phi_p (Z_{t-p} - \mu) \ldots \]  

where,
\[ Z_t = \text{deviation from mean} \]
\[ \mu = \text{sample mean} \]
\[ \Phi = \text{autoregressive model parameters} \]
\[ p = \text{order of moving average} \]

Most of the recent advances in time series analysis are systematically discussed by [4]. Comprehensive discussions on time series modelling of hydrologic variables given by [5]. Stochastic models were used extensively for forecasting of stream flow data [6]. However, the use for modelling of evapotranspiration appears to be very limited. So there is a scope of exploring the possibility of stochastic modelling of evapotranspiration. [7] developed the stochastic series of the weekly evaporation data of the Palanpur, Himachal Pradesh. [8] developed a stochastic model for the weekly evaporation and daily wheat, green gram evapotranspiration values using 20 years data under the climatic conditions of Udaipur. Validation of the developed model was done by comparison of estimated values with measured values. [9] developed a stochastic model for estimation of daily maize evapotranspiration using 23 years data under climatic conditions of Udaipur.

Evapotranspiration is a basic component of hydrological cycle. Modelling of evapotranspiration process is essential for determining of appropriate model in a particular climatic condition. Therefore, stochastic modeling of Black gram evapotranspiration may provide good insight and understanding of the processes for useful applications in water resources development.

2. Material and Methods

2.1. Location of the Study Area

The study was conducted at the College of Technology and Engineering (CTAE), Udaipur. The area comes under the sub humid region of agro-climatic zone IV A of the state of Rajasthan and is situated at 24°35’ N Latitude 73°42’ E Longitude and an altitude of 582.17m above mean sea level. The annual rainfall in the region is 662.5mm and more than 80% of this amount is received as a part of south - west monsoon during the period of 16th June to 15th September.

2.2. Soil of the Study Area

Relative proportion of, sand, silt and clay were found to be about 51.4, 17.5, and 31.1 per cent respectively. As per USDA, soil is classified as sandy loam. Values of bulk density of soil at depth 0-30 cm and below 30 cms were found to be 1.57 and 1.62 g/cc respectively. Average rate of infiltration is about 2.2 cm/hr. Field capacity and permanent wilting point were found to be 21.0 and 6.0 percent respectively on dry weight basis. Average electrical conductivity (EC) of the soil samples was found to be 0.18 dS/m at 25°C. Soil is alkaline in nature having pH 8.2.

2.3. Collection of Evaporation and Metrological Data

The data of pan evaporation, air temperature, relative humidity, wind speed, sunshine hours, for a period of twenty-four years (1978-2001) were collected from Meteorological Observatory of the College of Technology and Engineering, Udaipur. The reference evapotranspiration (ET0) was calculated by the following Penman-Monteith FAO-56 equation [10]:
\[ ET_0 = \frac{0.408 \Delta (R_e - G) + \gamma \frac{900}{T + 273} u_z (e_v - e_s)}{\Delta + \gamma (1 + 0.34 u_z)} \]

2.4. Measurement of Evapotranspiration for Black Gram Crop

An electronic weighing lysimeter consisting of two steel tanks of size 1.17 m x 1.47 m for the inner and 1.23 m x 1.53 m for the outer tanks was used for this study. A pit of 1.5 m x 1.75 m x 1.75 m was made for the installation of outer tank. The bottom of the pit was well compacted. A stone soling was constructed and then a 30.0 cm thick layer of cement concrete was laid. After proper curing for about 15 days the outer tank was rested on the cement platform in such a manner so that the rim of the tank was 10.0 cm above the soil surface. The inner tank was rested on the four load cells fitted on the corner of the outer tank. Data logger was fitted in the data acquisition almirah at 50.0 m away from the lysimeter. The weighing arrangement was made with the help of...
load cells and data logger. When positioned, the gap between the inner and outer tank was kept uniform. A layer of 15 cm gravel and 15 cm sand was laid on the bottom of inner tank. Backfilling of the inner tank was made in layers of 15 cm soil. When filled with soil, all the four load cells were calibrated. A drainpipe was installed at the corner of the inner tank. This pipe was covered with synthetic filter. The weighing lysimeter can read correctly up to 500.0 gm. The least count of the measurement of lysimeter is 0.28 mm.

Black gram crop of variety T-9 was grown from 19th July 2001 to 8th October 2001 for measurement of daily evapotranspiration. The row-to-row spacing was kept 30 cm and plant to plant spacing was kept 15 cm. The seed of black gram was placed 5 cm below the soil surface. The seed rate was taken as 15 kg per hectare. All the agronomical practices were done in accordance with the standard recommendations.

2.5. Formulation of Stochastic Model for Black Gram Evapotranspiration

The evapotranspiration data of 24 years for crop period were generated from the relationship of crop evapotranspiration and pan evaporation.

Stochastic behavior of time series was identified by the estimation of standard statistical parameters such as analysis of Serial Correlation Coefficient at lag one and estimation of coefficient of variation of the historical time series. For the time series with stochastic behavior, the value of lag one Serial Correlation Coefficient should lie outside the range of Upper and Lower limits.

2.6. Stochastic Model

Time series \( X(t) \) was represented by decomposition model of additive type

\[
X(t) = T(t) + P(t) + S(t)
\]

where,

- \( T(t) \) = trend component
- \( P(t) \) = periodic component
- \( S(t) \) = stochastic component

2.6.1. Trend Component

The trend component describes the long smooth movement of variable lasting over the span of the observations, ignoring the short-term fluctuations. The basic idea here is to study only \( T(t) \) while eliminating the effects of other components. For detecting the trend, a hypothesis of no trend will be made and following statistical tests, as suggested by [11], were performed:

1) Turning Point Test

2) Kendall’s Rank Correlation Test

2.6.2. Periodic Component

The time series \( X(t) \) is expressed in Fourier form as follow:

\[
X(t) = A_0 + \sum_{k=1}^{M} \left[ A_k \cos(2\pi kt/p) + B_k \sin(2\pi kt/p) \right]
\]

The periodic component concerns an oscillating movement, which is repetitive over a fixed interval of time. Periodic component was determined by the following:

\[
P(t) = A_0 + \sum_{k=1}^{M} \left[ A_k \cos(2\pi kt/p) + B_k \sin(2\pi kt/p) \right]
\]

It is more convenient to use the alternate form of \( P(t) \) given as under:

\[
P(t) = A_0 + \sum_{k=1}^{M} \left[ D_k \cos(2\pi kt/p) - \theta_k \right]
\]

2.6.3. Stochastic Component

The stochastic component is constituted by various random effects, which cannot be estimated exactly. A stochastic model of the form of autoregressive models (AR) will be used for the presentation of the time series. This model was applied to the \( S(t) \), which was treated as a random variable and calculated by:

\[
S(t) = \sum_{k=1}^{P} \left[ \Phi_{p,k} S(t-k) + a(t) \right]
\]

Where,

- \( \Phi_{p,k} \) = Autoregressive model parameter = 1, 2…P
- \( a(t) \) = independent random number

2.6.4. Estimation of the Autoregressive Parameters

These parameters can be expressed in terms of serial correlation coefficient, as Yule–Walker equations. The general recursive formula for estimating these parameters (\( \Phi_{p,k} \)), where suffix p and k indicate the order in

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AR (p) model, respectively and may be written as:
\[
\phi_{p,p} = \left[ r_p - \frac{1}{p-1} \sum_{k=1}^{p-1} (\phi_{p-1,k}) (r_{p-k}) \right]^{-1} \sum_{k=1}^{p-1} (\phi_{p-1,k}) (r_{k}) \tag{11}
\]
and
\[
\phi_{p,k} = \phi_{p-1,k} - \phi_{p,p} \phi_{p-1,p-k} \tag{12}
\]
where,
\[ r_k = \text{autocorrelation coefficient} \]

For the selection of the order of the AR (p) model residual variance method was used.

2.7. Validation of Stochastic Model

Validation of developed model was performed by comparison of the generated series with the measured historical series. The variation between generated and measured series was presented graphically with respect to time. Linear regression was fitted between generated and measured series.

3. Results and Discussions

The daily black gram evapotranspiration values for twenty-four years (1978-2001) were generated using pan evaporation values. The daily black gram evapotranspiration series was tested for stochastic behavior.

The mean daily series of black gram evapotranspiration has been presented in Figure 1, which confirms the presence of dependent cyclic component and independent part in the series of ETBG.

The findings reveals that there is no large variability among the daily values of black gram evapotranspiration of different years. Mean daily values of evapotranspiration ranged 0.8121 to 4.1261 mm day\(^{-1}\) and standard deviation ranged 0.2217 to 2.0117 mm day\(^{-1}\) during the entire growing season. The variation may be attributed towards the natural change in seasonal climate.

The estimated values of coefficient of variance (CV) of ETBG range from 0.1884 to 0.6888, which signifies the importance of variability ETBG series. Since the values of CV significantly different from zero, it indicates that black gram evapotranspiration is mutually dependent.

The lag one serial correlation coefficient of observed series was found to be 0.686. The values of lag one serial correlation coefficient lie outside the range of confidence limit and are significantly different from zero (Figure 2). This confirms that past and present values of black gram evapotranspiration are highly inter correlated and mutually dependent.

The results of a analysis of coefficient of variance and serial correlation coefficient of different shows that black gram evapotranspiration process is a time variant and not an independent one. Thus, the black gram evapotranspiration time series may be modelled for stochastic process.

![Figure 1. Mean daily black gram evapotranspiration series for 24 years (1978-2001) at Udaipur.](image1)

![Figure 2. Correlogram of the observed series of black gram evapotranspiration for 24 years (1978-2001) at Udaipur.](image2)

3.1. Trend Component

For identification of trend component, annual black gram evapotranspiration series was used [11]. The annual series was obtained by transforming twenty four year seasonal daily evapotranspiration data. For detection, of trend the hypothesis of no trend was made and the values of test statistics (z) was calculated by turning point test and Kendall’s Rank correlation test.

| S.No | Test                        | Values of test statistic (z) |
|------|-----------------------------|------------------------------|
| 1    | Turning Point Test          | -0.837                       |
| 2    | Kendall’s Rank Correlation Test | -0.740                      |

The estimated values of test statistics (z\(_{cal}\)) obtained for turning point test, and Kendall’s rank correlation test were within the 1 per cent levels of significance. Hence, the hypothesis of no trend was accepted. So the observed series may be treated as trend free series.

3.2. Periodic Component

The oscillating shape of the correlogram (Figure 2) having peaks at legs equal to 82 and at other multiples of it confirms the presence of periodic component in the daily black gram evapotranspiration series. Therefore, for the
harmonic analysis of periodic component, time span of periodicity was taken as 82.

3.2.1. Determination of Significant Harmonics
Numbers of significant harmonics were determined by analyzing the periodic mean daily black gram evapotranspiration series using Equations (8) and (9). Selection of number of significant harmonics performed by conducting F distribution test (Table 1) and by plotting cumulative periodogram (Figure 2). The result of analysis of variance indicate that only first three harmonics are highly significant and other harmonics are not significant and, therefore, can be ignored (Table 1).

3.2.2. Parameters of Periodic Component
The first three harmonics explain 92.613 % of the variance. This further confirms that only first three harmonics are significant and may be used to express the periodic component of the daily black gram evapotranspiration series. The values of the Fourier coefficients (A1, A2, A3, B1, B2, B3) were found to be -1.024, -0.42, -0.124, -0.329, 0.057 and 0.018, respectively. Using these six coefficients and Equation (3), the periodic component \( P(t) \) from periodic deterministic process may be mathematically expressed as:

\[
P(t) = 2.914 - 1.024 \cos(2\pi t / p) - 0.329 \sin(2\pi t / p) \\
-0.42 \cos(4\pi t / p) + 0.057 \sin(4\pi t / p) \\
-0.124 \cos(6\pi t / p) + 0.018 \sin(6\pi t / p)
\]

(13)

The deterministic cycle component \( P(t) \) was computed by using Equation (13) for all the values (t=4 to \( t_{\text{MAX}}=1968 \)). A new stationary series \( S(t) \) resulting from deterministic stochastic process was obtained, after removing periodic component from the historical series.

3.3. Stochastic Component
The presence of stochastic component was confirmed by plotting the correlogram (Figure 2), periodogram (Figure 3) of observed series, analysis of serial correlation coefficient (SCC) and the coefficient of variance (CV).

The periodic component was removed from the historical series and rest of the series was analyzed to obtain non-deterministic stochastic component by fitting the autoregressive process of stochastic modelling.

3.3.1. Selection of Model Order
Cumulative periodogram was used to determine the order of the model, which may be significantly representing the non-deterministic stationary stochastic component. The maximum variability of Pi 93.8 per cent observed in the first three orders (Figure 3), so order three was selected to explain the stochastic component of the series.

3.3.2. Mathematical Representation of Stochastic Component
Cumulative periodogram (Figure 2) shows that first three orders explain 92.613 % values while rest explains only 7.38 %. Therefore, autoregressive coefficients (\( \Phi_p \)) for first three order were estimated using Equation (11) and (12) as \( \Phi_{(3,1)}=0.4658, \Phi_{(3,2)}=0.09552, \Phi_{(3,3)}=0.0933 \).

The estimated autoregressive coefficient the stochastic component of daily black gram evapotranspiration series may be expressed as:

\[
S(t) = 0.4568S_{(t-3)} + 0.09552S_{(t-2)} + 0.0933S_{(t-1)} + a(t)
\]

(14)

The non-deterministic stochastic component was estimated by using Equation (14) for all the values (t=4 to 1968).

3.3.3. Residual Series of Stochastic Component
The residual series was obtained after removing the periodic and dependent stochastic parts from the historical series.

The statistical analyses of historical, generated and residual series were presented in Table 2. Residual series shows that the mean is almost equal to zero (0.0003) and standard deviation is equal to one (1.03). The mean, standard deviation and coefficient of variation of the his-
Table 2. Statistical parameters of the historical, generated and residual series of daily black gram evapotranspiration series.

| Series          | Mean (mm day⁻¹) | Standard deviation (mm day⁻¹) | Variance |
|-----------------|-----------------|-----------------------------|----------|
| Historical series | 2.914           | 1.499                       | 2.24     |
| Generated series | 2.917           | 1.094                       | 1.19     |
| Residual series  | 0.0003          | 1.03                        | 1.06     |

Historical and generated series shows closeness between the two.

3.4. Model Structure

Since the observed daily black gram evapotranspiration series was found to be a trend free one, the developed model is a superposition of harmonic deterministic process and third order autoregressive model.

\[
X(t) = 2.914 - 1.024 \cos(2\pi t / p) - 0.329 \sin(2\pi t / p) \\
- 0.42 \cos(4\pi t / p) + 0.057 \sin(4\pi t / p) - 0.124 \cos(6\pi t / p) \\
+ 0.018 \sin(6\pi t / p) + 0.4568S_{(r-1)} + 0.09552S_{(r-2)} \\
+ 0.0933S_{(r-3)} + a(t)
\]

(15)

3.5. Diagnostic Checking of Black Gram Evapotranspiration Model

The sum of square analysis and analysis of serial correlation coefficient of residual series were performed to test the adequacy of the model. The coefficient of determination \(R^2\) was found to be 0.939, which is nearly equal to unity. This leads to the conclusion that the developed model has a fair goodness of fit to generate the daily black gram evapotranspiration series.

The correlation coefficients for residual series were within the prescribed limits of –0.2 to .20. (Figure 4), which confirm that residual series is purely random series and having neither any periodicity nor any stochastic component. Both tests prove the adequacy of the model.

3.6. Validation of Daily Black Gram Evapotranspiration Series

Validation of generated twenty-four years mean daily black gram evapotranspiration series was made with observed twenty-four years mean daily black gram evapotranspiration series. Figure 5 shows there is linear relationship between \(ET_{BGGM}\) and \(ET_{BGM}\) series. The correlation coefficient between \(ET_{BGGM}\) and \(ET_{BGM}\) was found to be 0.96, which is significant at 1 per cent level (Figure 6). The mean values of generated \((ET_{BGGM} = 2.9127\) mm day⁻¹) and observed \((ET_{BGM} = 2.914\) mm day⁻¹) series were very close to each other and standard error \((0.2096\) mm day⁻¹) is quite low. The regression line almost coincide the 1:1 line. Therefore, Equation (15) can be used to predict future values of \(ET_{BG}\) quite accurately.

4. Conclusions

The black gram evapotranspiration is time variant and
mutually dependent and can be modeled on stochastic theory. It was found that the daily time series of black gram evapotranspiration is trend free and periodic - stochastic in nature with periodicity of 82 days. Hence, the developed model superimposes a periodic deterministic process and a stochastic component results from non-deterministic process. The deterministic periodic component of the average daily black gram evapotranspiration was represented by first three harmonics only. Based on findings it is recommended that the developed model can be used for forecasting black gram evapotranspiration in sub-humid regions and could be applied in similar climatic conditions.

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Notation

The following abbreviations and symbols are used in this paper:

| Symbol | Description |
|--------|-------------|
| $A_k, B_k$ | Fourier coefficients at harmonic, $K$ |
| AR | Autoregressive model |
| CTAE | College of Technology and Engineering |
| $ET_0$ | Grass based reference evapotranspiration |
| $ET_{BG}$ | Measured black gram evapotranspiration |
| $ET_{BGGM}$ | Generated evapotranspiration by stochastic model |
| $ET_{BGOM}$ | Observed mean daily black gram evapotranspiration |
| FAO | Food and Agriculture Organization |
| $K$ | Number of significant harmonics |
| $I$ | Lag |
| $M$ | Mean value of stochastic series |
| $p$ | Time span of periodicity |
| $P$ | Order of autoregressive model |
| $P_t$ | Periodic component at time $t$, $t=1,2,\ldots,N$ |
| $S_t$ | Stochastic component at time $t$, $t=1,2,\ldots,N$ |
| $X_t$ | Time series of daily black gram evapotranspiration |
| $Y_t$ | Trend free series |
| $\alpha_k, \beta_k$ | Fourier coefficients of periodic mean series of harmonic, $k$ |
| $\phi_{P,K}$ | $K^{th}$ autoregressive coefficient of model order $p$ |
| $\phi_{P,P}$ | Partial autoregressive coefficient of model order $p$ |
| $a_{(i)}$ | Independent random number |
Source Assessment and Analysis of Polycyclic Aromatic Hydrocarbon (PAH’s) in the Oblogo Waste Disposal Sites and Some Water Bodies in and around the Accra Metropolis of Ghana

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Abstract

The study looked at the levels of polycyclic aromatic hydrocarbons (PAHs) in leachates from a solid waste disposal site and an effluent from an oil refinery in some water bodies around Accra. Sixteen (PAHs) were extracted simultaneously by solid phase and analysis by gas chromatograph. The results of this study generally demonstrated that there were elevated levels of PAHs in the water sample of the Densu River, Chemu, Korle and Kpeshi Lagoons. The average concentration of PAHs in the water ranged from 0.000 of many of the PAHs to 0.552µg/L, for Acenaphththene to 11.399µg/L for Benzo (ghi) perylene of the Chemu Lagoon, 0.00µg/L for Benzo (a) Pyrene to 8.800µg/L for Benzo (ghi) perylene (Korle Lagoon) and 0.052µg/L for Pyrene to 4.703ug/L for Acenaphthylene of the Kpeshi Lagoon and 0.00µg/L for pyrene to Acenaphthylene 2.926µg/L of the Weija Dam. Concentrations ranging from below detection level to 14.587µg/L were also recorded at the Oblogo solid waste dump and it’s environ. The Weija dam supply over two million gallons of portable water daily to the people of Accra and the levels of the PAH determined is worrying, as a result, the Oblogoh disposal site ought to be re-located to avert any possible epidemic.

Keywords: Accra Metropolitan Assembly (AMA), Oblogo Dumping Site, Weija Dam, Densu River, PAHs, Chemu Lagoon, Korle Lagoon, Kpeshi Lagoon

1. Introduction

The disposal of wastes by land filling or land spreading is the ultimate fate of all solid wastes, whether they are residential wastes collected and transported directly to a landfill site, residual materials from materials recovery facilities (MRFs), residue from the combustion of solid waste, compost or other substances from various solid waste processing facilities. Disposing of solid waste in open dumps and burning of such solid waste, is the most common solid waste disposal method in Ghana. Open dump and burning of their content which is a health hazard, is not an acceptable method of solid waste disposal and must henceforth, be discouraged. For example, vinyl chloride and polythene form greater proportion of the solid waste in terms of volume as a result of the packaging industry [1]. Solid waste includes domestic refuse and discarded solid materials such as those from commercial, industrial and agricultural operations. They contain increasing amount of paper, cardboards, plastics, glass, packing materials and toxic substances. Combustion of these wastes completely and incompletely results in the production of toxic and corrosive chemicals such as PAH’s, PCB’s and hydrogen chloride just to mention a few [1].

Leachates from solid waste disposal sites which are chemicals removed from the waste as a result of water passing through is one of the major soil and water pollutants. These leachates are released into water bodies which tend to pollute them and needs to be monitored. A comprehensive waste management program must combine a variety of social, transportation, and treatment technologies. Components, in order of desirability, include prevention of wastes at the source; reuse, recycling, or composting; energy recovery; and putting in a landfill only those materials not amenable to other strategies [2].
The plan should consider impacts on air quality, water quality, traffic, noise, odor, socioeconomic effects, and community acceptance [3]. A modern sanitary landfill is not a dump; it is an engineered facility used for disposing of solid wastes on land without creating nuisances or hazards to public health or safety, such as the breeding of rats and insects and the contamination of ground water [4]. This is not the case in Ghana as open dumping and burning of solid waste is the only way of treating waste in Ghana.

Solid waste comes from various sources. Other forms of waste that can vary by location include agricultural waste, mining waste, and hazardous waste. Waste streams differ in the following attributes: physical (e.g., compatibility, density); combustion (temperature, residual ash percentage, heat content in BTUs); chemical composition, percentage of nitrogen, carbon, oxygen, chloride; and concentrations of toxic polycyclic aromatic hydrocarbons (PAHs) and metals; potential for recycling various components; and ease of separation [3].

Polycyclic aromatic hydrocarbons (PAHs) are another group of hazardous compounds which man introduces into the environment in large quantities with little or no awareness. These are a suit of organic compounds release into the environment as gas particles during incomplete combustion of organic material. PAHs have a number of sources including: Mobil sources such as cars, buses, trucks, ships, and aircrafts; industrial sources such as power generation, steelworks, and coke ovens, aluminum production, and cement kilns, oil refining as well as waste from incineration. Domestic sources include combustion for heating and cooking especially solid fuel using coal and wood. Fires and smoke resulting from burning of vegetation in agricultural process, bushfires, grilling of food, or tobacco smoke [5].

These compounds (PAHs) are also cumulative and may cause a whole lot of health related complications ranging from mutations in lower animals to cancerous cells in humans [5,6]. Other environmental factors affect the distribution of PAHs. For example it has been proved by Shahunthala 2006 that increases in salinity decreases the exposure of PAHs and also dispersant effectiveness decreased only at the highest salinity. Hence, risks to fish of PAH from dispersed oil will be greatest in coastal waters where salinities are low [7].

The smallest member of the PAH group is naphthalene, a two-ring compound, which is gaseous at room temperature. PAHs are usually found as a mixture containing two or more of these compounds, such as soot. PAHs are highly potent carcinogens that can produce tumors in some organisms at even single doses; but other non-cancer-causing effects are not well understood [8]. PAHs can occur naturally or can be man-made. Manufactured PAHs usually exist as colorless, white, or pale yellow-green solids. PAHs are commonly found in coal tar, crude oil, creosote, and roofing tar. Some are used in medicines or to make dyes, plastics, and pesticides [9]. Man made sources such as automobile exhausts and coal burning contribute far more PAHs to the environment than natural sources.

PAHs are dangerous, thus, increases risk of cancer and creates advance glycogen end product which leads to an increased risk of coronary heart disease and diabetes [10]. Laboratory and field evidence indicates that PAHs induce neoplastic and genotoxic effects in aquatic biota. Data from mammals indicate that these animals may be susceptible to such effects, but no studies were identified documenting such effects in wild mammals. PAHs known for their carcinogenic, mutagenic (gene mutation causing agent) and teratogenic (chemicals that affect the normal development of foetus) properties are Benzo[a]pyrene, Benzo[a]anthracene chrysene, Benzo[b] fluoranthene, Benzo [j]fluoranthene, Benzo[k] fluoranthene, Benzo [ghi]perylene, corone, Dibenz[a,h] anthracene (C20H14), Indeno [1,2,3-cd]pyrene (C22H12) and ovalene. Mice that were fed high levels of one PAH during pregnancy had difficulty reproducing and so did their offspring. These offspring also had higher rates of birth defects and lower body weights. It is not known whether these effects occur in humans. Animal studies have also shown that PAHs can cause harmful effects on the skin, body fluids, and ability to fight disease after both short- and long-term exposure. But these effects have not been seen in human beings. Some people who have breathed or touched mixtures of PAHs and other chemicals for long periods of time have developed cancer [11].

Some PAHs have caused cancer in laboratory animals when they breathed air containing them (lung cancer), ingested them in food (stomach cancer), or had them applied to their skin (skin cancer). A research conducted by the Agency for Toxic Substances and Disease Registry [12] under the Canadian department of Health and Human services in the year 2007, ranked PAHs as the sixth most hazardous substance among a number of 275 compounds on which the research was conducted. According to the research the first six most hazardous compounds were arsenic, lead, mercury, vinyl chloride, polychlorinated biphenyls and PAHs.

Although solid waste can be properly treated before disposal, solid waste problem arise from; rapid increase of human population, aggregation of people in urban areas (rapid advance in technology and social attitudes).

Solid waste materials pose a serious threat because the leach from it remain in place for a relatively longer period of time unless removed, burned or otherwise destroyed [13]. The combustion of solid waste leads to the formation of PAHs and the main problem of this study is to analyze the concentration of PAH in the leach from solid waste disposal site since burning of solid waste is the only way of treating waste in Ghana. The leachates from this waste deposition site run into water bodies eg. Densu River flows to join Weija Dam and other lagoons.
in Accra Metropolitan Assembly. The Ghanaian ecosystem plays host to a number of lagoons which serve various functions. The most important of them is being the home for various species of fish. For example the tilapia which is a delicacy in most Ghanaian communities finds its haven in most of the lagoons. The Kpeshie, Korle, and the Chemu lagoons (all in the greater Accra region), the Fosu lagoon (central region) and others throughout the country until recently had been a good sources of fish (mainly tilapia). The Korle lagoon, owing to its extent of pollution, not much living things were present in it for some years, it has recently been dredged. Its scent wafts back to envelope the adjoining shanty town which is the home of hundreds of families who, because they have no sanitation facilities, have turned the shores of the lagoon into a giant latrine. Large portions of the Kpeshie lagoon and it mangrove at La an Accra suburb are being reclaimed and sold to individuals for residential and business development purposes. As a result the lagoon and it mangrove are disappearing fast. Extensive portion of the lagoon have been filled with sand, construction debris and garbage ready to be sold to prospective buyers.

In addition the Korle, Kpeshie and the Chemu lagoons are close to solid waste disposal site and are near major roads used by various kinds of motorists which emit smoke continuously into the environment. Also near the Korle lagoon is located a slaughter house which produces smoke continuously from the processing of hide using car tire. The Chemu lagoon located in Tema New town is being exposed to smoke from vehicles. It is also close to the Tema oil refinery which continuously emits smoke into the environment. Due to the above mentioned facts it is suspected that the Korle, Kpeshie, and the Chemu lagoons may have considerable amounts of PAHs dissolved in them. It is in this views that this study has been designed to determine the level and distribution of PAHs in leachates from the Oblogo solid waste disposal site, waters of the Korle, Kpeshie and the Chemu lagoons and their interrelationships with physiochemical parameters such as pH, salinity, chloride, turbidity and conductivity in the greater Accra regions of Ghana.

2. Materials and Methods

2.1. Sample Collection

Samples were collected from Oblogo solid disposal waste site, Weija dam and the down stream of River Densu (thus, the mixture of the Weija dam and leach) as well as three lagoons namely, Kpeshie, Korle and the Chemu lagoons. These samples were taken from different points on the lagoons so as to get fairly representative samples of each of the lagoons. Three samples were taken from each of the lagoons and four samples from oblogo solid waste site bringing the total number of samples taken to thirteen. Since all the three lagoons were connected directly to the sea, all the first samples taken were made closer to the sea and the other two taken from different intervals (0.5km) from the bank of the lagoons.

Clean amber glass bottles were used in the collection of the sample to prevent sunrays and it effect on any present bacteria. The amber glass bottles were washed with detergents (liquid soap) and rinsed with lot of water to remove any trace of soap, distilled water is then used to wash the bottles to remove ions present. The water samples were then collected into the bottles, covered in an ice chest with ice and transported to the laboratory for analysis.

2.2. Methodology

The research was carried out at the Centre for Scientific and Industrial Research (CSIR), Environmental Division (ED), Water Research Institute (WRI), Organic Laboratory. The parameters measured includes; conductivity, pH, salinity and Polycyclic Aromatic Hydrocarbons.

2.3. Conductivity

The conductivity was measured by mixing the sample very well and pouring into a clean cup. The conductivity meter was immersed in the water sample and the cup swirled to get the appropriate reading and recorded in microsiemens (µs). The instrument was calibrated using standard KCl (0.01M) which has a conductivity of 141µs/cm at 25°C, each reading was done three times [14].

For theoretical purpose; K = Km × c / (1+0.0191)(T-25)

Where, Km = measured conductivity, µS/cm at 25°C
C = cell constant, cm⁻¹, T = temperature of sample

2.4. pH

The pH was determined by using the pH meter and combination electrode for measurement, the electrode was immersed into the water sample and the cup swirled to get accurate results. The pH was recorded in pH units. Calibrate by; washing the electrode of the meter very well with distilled water, the electrode is first calibrated against a pH buffer 4 then 9 and then 7, a reference solution of known pH was measured to check the sensitivity and accuracy of the electrode.

2.5. Salinity

For salinity, chloride was determined. 50ml of the sample was used for the determination of chloride but due to high conductivity of the Oblogo leachates and diluted Oblogo leachates, 1ml of the sample was used and di-
luted to 50ml with distilled water. When diluted, endpoint was easy to attain. 1ml of potassium chromate was added and titrated against 0.0141M silver nitrate to obtain a pinkish yellow endpoint. The reading on the 50ml graduated burette was recorded.

To calculate for chloride; 

\[ \text{MgCl}^-/L = (A-B) \times M \times 3540 \]

\[ A = \text{ml titration for sample} \]
\[ B = \text{ml titration for blank} \]
\[ M = \text{molarity of AgNO}_3 \]

To ensure accuracy of work, the AgNO\(_3\) was standardized with NaCl; thus

About 10mL of standard NaCl solution was measured (pipetted) into a flask and 2 drops of potassium chromate indicator was added. This was titrated with the AgNO\(_3\) solution to obtain a pinkish yellow endpoint.

To calculate for salinity;

\[ S\% = 0.03 + 1.805 (\text{Cl}^- \times 1.00045)/1000 \]

2.6. Extraction of PAH from Water

For PAHs, 1000ml (1L) of water sample was poured into a separating funnel. 50ml of dichloromethane was added followed by 0.2ml internal standard to correct errors using micro syringe. The content of the separating funnel was shaken well for the dichloromethane to extract as much organic components as possible from the water sample. The separating funnel was left undisturbed on a retort stand for sometime, so that the mixture separates into the organic and water layer. The separating funnel was then opened to drain the water layer. The organic layer was drained through a glass funnel which was plugged with glass wool, filter paper and sodium sulphate into a Zymark tube. The sodium sulphate was used to absorb water that might be present in the organic layer.

A second extraction was carried out using 50mL of dichloromethane and the extract was added to the one in the Zymark tube. One drop of iso-octane was added to the contents in the Zymark tube and was placed into a Turbo Evaporation Unit to reduce the volume to 1ml by evaporation. The iso-octane served as a keeper to prevent evaporation of the needed components. The extract in the Zymark tube was then transferred into test tubes using pasture pipettes. The Zymark tube was washed with 2ml of dichloromethane and added to the content in the test tube. The test tube was heated in a block heater and a gentle steam of nitrogen was used to reduce the volume to 0.5ml. 1ml of cyclohexane was added and the mixture was evaporated to dryness followed by the addition of 0.5ml hexane [16-17].

2.7. Clean-Up

Most of the unwanted components were removed from the extract leaving the components of interest. This was achieved by using solid phase extraction tubes containing 500mg florisil, 3ml by volume. This solid phase was conditioned using 6ml of hexane. 0.5mL of the extract was added and eluted with 6.0mL hexane into a test tube. The PAHs in the extract was held by the florisil column. The column was eluted again using 20% dichloromethane in hexane into another test tube and this fraction contained the PAHs. The volume was reduced to 0.5ml and was transferred into sample vials for gas chromatography run [11].

2.8. Gas Chromatography

Gas chromatography (GC) is a common confirmation test. GC analysis separates all of the components in a sample and provides a representative spectral output. Before the sample was analyzed, the instrument was tuned and calibrated. Tuning was accomplished using specific concentrations of Decafluorotriphenylphosphine and p-Bromofluorobenzene to test the instruments reporting accuracy. The sample vials that contained the extracts were arranged on a plate at the injection point and the injection was done automatically by the machine.

The sample was introduced as a vapor onto the chromatographic column. On the column, the solubility of each component in the gas phase was dependent on it vapor pressure, which was in turn a function of the column temperature and the affinity between the compound and the stationary phase. To ensure proper separation, the sample must enter the column in a discreet, compact pocket. The gas chromatography instrument uses the flame ionization detector with the model 6890N to measure the different compounds as they emerge from the column. The principle behind Gas Chromatography states that the rate of migration of the solute depends upon the rate of interaction of the solute with a two phase, the mobile phase and the stationary phase as the compound travels through the supporting medium.

3. Results and Discussions

Reliability of any analytical results can be verified using certain indicators which include the method and equipment used, accuracy, precision, etc. The precision and suitability of the method to the measuring equipment used, was initially established using the certified reference material. This was done by using the reference material alone and also treated as a sample. The percentage recoveries were then calculated. The method verification and sample results are tabulated in Table 1 below.

3.1. Data Analysis

Estimation of PAHs was done by expression: Concentra-
Table 1. Summary of system suitability and percentage recovery using certified reference material.

| COMPOUND NAME                  | AMOUNT µg/ml | AVE (µg/ml) | TRUE Rel. Value Error (%) µg/ml | Stand. % recovery Dev. | Rel. Stand. Dev. % |
|--------------------------------|--------------|-------------|---------------------------------|------------------------|--------------------|
| Naphthalene                    | 14.2         | 14.3        | 14.2                            | 15.0                   | -5.3               | 0.577              | 94.67              | 0.4063             |
| 2-Methyl-1-naphthalene         | 14.7         | 14.8        | 14.8                            | 15.0                   | -1.3               | 0.577              | 98.67              | 0.3899             |
| Biphenyl                       | 14.8         | 15.2        | 14.9                            | 15.0                   | -0.7               | 0.2309             | 99.33              | 1.5497             |
| 2,6-Dimethylnaphthalene        | 14.8         | 14.9        | 14.8                            | 15.0                   | -1.3               | 0.577              | 98.67              | 0.3899             |
| Acenaphthylene                 | 14.8         | 14.9        | 14.7                            | 15.0                   | -1.3               | 0.1000             | 98.67              | 0.6757             |
| Acenaphthene                   | 14.8         | 14.8        | 14.8                            | 15.0                   | -1.3               | 0.0000             | 98.67              | 0.0000             |
| 2,3,5-Trimethylnaphthalen      | 14.7         | 14.7        | 14.7                            | 15.0                   | -2.0               | 0.0000             | 98.00              | 0.0000             |
| Fluorene                       | 14.7         | 14.9        | 14.8                            | 15.0                   | -1.3               | 0.1000             | 98.67              | 0.6757             |
| Phenanthrene                   | 14.7         | 14.8        | 14.8                            | 15.0                   | -1.3               | 0.577              | 98.67              | 0.3899             |
| Anthracene                     | 14.8         | 14.9        | 14.9                            | 15.0                   | -0.7               | 0.577              | 99.33              | 0.3872             |
| 1-Methylphenanthrene           | 12.8*        | 14.7        | 14.7                            | 15.0                   | -2.0               | 0.0000             | 98.00              | 0.0000             |
| IS 3,6-DMP                     | 10.0         | 10.0        | 10.0                            | 10.0                   | 0.0                | 0.0000             | 100.00             | 0.0000             |
| Fluoranthene                   | 14.8         | 14.8        | 14.8                            | 15.0                   | -1.3               | 0.0000             | 98.67              | 0.0000             |
| Pyrene                         | 14.2         | 14.2        | 14.2                            | 15.0                   | -5.3               | 0.0000             | 94.67              | 0.0000             |
| Benzo(a)anthracene             | 14.5         | 14.5        | 14.5                            | 15.0                   | -3.3               | 0.577              | 96.67              | 0.3979             |
| Chrysenne                      | 14.6         | 14.5        | 14.5                            | 15.0                   | -3.3               | 0.577              | 96.67              | 0.3979             |
| IS BB-Biphenyl                 | 10.0         | 10.0        | 10.0                            | 10.0                   | 0.0                | 0.0000             | 100.00             | 0.0000             |
| Benzo(b)fluoranthene           | 15.3         | 15.0        | 15.3                            | 15.0                   | 1.3                | 0.1732             | 101.33             | 1.1395             |
| Benzo(k)fluoranthene           | 14.1         | 14.0        | 14.1                            | 15.0                   | -6.0               | 0.577              | 94.00              | 0.4092             |
| Benzo(o)pyrene                 | 14.7         | 14.3        | 14.5                            | 15.0                   | -3.3               | 0.2000             | 96.67              | 1.3793             |
| Benzo(a)pyrene                 | 14.8         | 14.4        | 14.5                            | 15.0                   | -3.3               | 0.2646             | 96.67              | 1.8248             |
| Perylene                       | 14.8         | 14.4        | 14.7                            | 14.6                   | -2.7               | 0.2082             | 97.33              | 1.4260             |
| Indenol(1,2,3cd)pyrene         | 15.3         | 15.1        | 14.9                            | 14.8                   | -1.3               | 0.6110             | 98.67              | 4.1284             |
| Dibenzo(a,h)anthracene         | 14.1         | 15.4        | 14.8                            | 15.0                   | -6.0               | 0.7000             | 94.00              | 4.9645             |
| Benzog(h,j)perylene            | 14.8         | 13.6        | 14.6                            | 14.3                   | -4.7               | 0.6429             | 95.33              | 4.4958             |

* Statistically rejected data as an outlier using Q-Test.

Table 2. Summary of physico-chemical parameters (levels) of the water samples.

| Name of sample                  | Average Conductivity (S/m) | Average (pH unit) | Average Salinity (ppm) |
|--------------------------------|----------------------------|-------------------|------------------------|
| Leach from Oblogo solid waste site | 1.333                      | 8.43              | 3.5×10-3               |
| Leach diluted with rain water    | 0.976                      | 8.38              | 2.3×10-3               |
| Down stream of river Densu       | 0.025                      | 7.75              | 7.7×10-5               |
| Weija dam                        | 0.027                      | 7.51              | 7.5×10-5               |
| Chemu Lagoon                     | 5.677                      | 7.687             | 5.72                   |
| Korle Lagoon                     | 40.83                      | 7.52              | 26.53                  |
| Kpeshie Lagoon                   | 40.50                      | 7.52              | 26.20                  |

Table 2 shows the detailed data of the physicochemical properties of the sampling site, the average pH was around neutral with a value ranging 7.51 to 8.43. The Chemu, korle and kpeshie lagoons recorded an average high conductivity of 5.677, 40.83 and 40.50 S/m respectively, while samples from oblogo sampling sites recorded very low conductivity range of 0.027 - 1.333 S/m. The very high value in the Lagoons is expected because the Chemu as well as the other two lagoons flow into the sea and the concentration of dissolved ions is expected to be very high which has been proved by high salinity values (5.72 – 26.53) recorded as shown in Table 3 above.

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Table 3. Summary of mean concentrations of PAHs in the Oblogo dumping sites and its environ.

| PAHs                  | Leach from Oblogo solid waste disposal site | Leach diluted with rain water | Down stream of River Densu | Weija dam site Average |
|-----------------------|--------------------------------------------|------------------------------|-----------------------------|-------------------------|
| Naphthalene           | 2.248                                      | 2.165                        | 2.42                        | 2.171                   |
| Acenaphthylene        | 9.878                                      | 2.495                        | 1.318                       | 2.926                   |
| Acenaphthene          | 2.624                                      | 14.587                       | 5.091                       | 0.051                   |
| Fluorine              | 2.715                                      | 0.648                        | 13.539                      | 0.691                   |
| Phenanthrene          | 0                                          | 1.199                        | 5.188                       | 0.041                   |
| Anthracene            | 5.368                                      | 2.43                         | 6.382                       | 0.486                   |
| Fluoranthene          | 1.912                                      | 2.548                        | 1.412                       | 1.638                   |
| Pyrene                | 0.029                                      | 0                            | 0.105                       | 0                       |
| Benzo[a]anthracene    | 0                                          | 0.323                        | 3.018                       | 0.03                    |
| Chrysene              | 0                                          | 0                            | 1.295                       | 0                       |
| Benzo[b]fluoranthene  | 4.921                                      | 1.26                         | 0.88                        | 1.156                   |
| Benzo[k]fluoranthene  | 0                                          | 0                            | 0.059                       | 0.058                   |
| Benzo[a]pyrene        | 0.199                                      | 0                            | 0.085                       | 0.071                   |
| Indeno[1,2,3-cd]pyrene| 0.779                                      | 0                            | 1.788                       | 0                      |
| Dibenz[a,h]anthracene | 1.731                                      | 0                            | 0.093                       | 0                      |
| Benzo[ghi]perylenel   | 1.905                                      | 1.042                        | 0.647                       | 0.626                   |
| Total                 | 37.906                                     | 28.699                       | 43.235                      | 10.058                  |

 Mean Concentration in µg/L±SD

Expression = Amount (µg/ml) X Final volume of Extract (ml) / Weight taken (g)

From Table 3, the PAHs distribution in the leach from Oblogo solid waste site has been observed with Acenaphthylene recording the highest concentration of 9.878 µg/L and Pyrene with the least concentration of 0.029 µg/L. The carcinogenic PAH detected at this site were Benzo[b]fluoranthene, Benzo[a]pyrene, Indino[1,2,3-cd]perilene and Dibenz[a,h]anthracene with concentration of 4.921µg/L, 0.199 µg/L, 0.779 µg/L, 1.731 µg/L respectively (Table 3). These concentrations may resulted from the combustion of the solid waste with the presence of domestic refuse and discarded solid materials such as those from commercial, industrial and agricultural operations: they contain increasing amount of paper, cardboards, plastics, glass, packing materials and toxic substances.

The PAHs distribution in the leach diluted with rain water has been observed with Acenaphthene recording the highest concentration of 14.587µg/L and Benzo (a) anthracene with the least concentration level of 0.323 µg/L. The carcinogenic PAHs detected at this site were Benzo (a) anthracene and Benzo (b) fluoranthene with concentrations of 0.323µg/L and 1.26µg/L respectively. All the concentrations detected at this site were above the WHO’s limit of 0.05µg/L [13]; this indicates high level of contamination in the leach diluted with rain water. The higher concentration of some PAHs in the diluted leach with rain water shows how the atmosphere has been polluted through anthropogenic source (automobiles, burning of biogas, industrial activities etc). PAH distribution in the downstream of river Densu has been observed with Fluorine recording the highest concentration of 13.539µg/L and Benzo[k] fluoranthene with the least concentration of 0.059µg/L. The carcinogenic PAHs detected at this site were chrysene, Benzo [a] anthracene, Benzo[b] fluoranthene and Benzo [k] fluoranthene Indino[1,2,3-cd]pyrene and Dibenz[a,h] anthracene with concentrations of 1.295µg/ L, 3.018µg/ L, 0.8 8µg/
L, 0.059µg/L and 1.788 µg/L and 0.093 µg/L respectively (Table 3).

All the concentrations detected at this site were also above the WHO’s limit of 0.05µg/L [13]; this indicates high level of contamination in the downstream of river Densu. These concentrations could be due to the leach joining the downstream of river Densu. PAH distribution in the Weija dam has been observed with Acenaphthylene recording the highest concentration of 2.926µg/L and Benzo[a] anthracene with the least concentration of 0.030µg/L (Table 3). The carcinogenic PAHs detected at this site were Benzo[a] anthracene, Benzo[b] fluoranthene and Benzo[k] fluoranthene and Benzo[a]pyrene with concentrations of 0.030µg/ L, 1.156µg/ L and 0.058µg/L and 0.085µg/L respectively.

Table 4 compares the average concentration of the individual PAHs compounds in the three lagoons. In all, the Chemu lagoon recorded the highest PAHs concentration with a total of 61.712µg/L followed by the Korle lagoon and then the Kpeshie lagoon with total PAHs of 38.889µg/L and 34.09µg/L respectively. Both the Chemu and the Korle lagoons had Benzo (ghi) perylene as the compound with the highest concentration, whiles the Kpeshie lagoon on the other hand had Acenaphthylene as the compound with the highest average concentration. Samples from Oblogo damping site and it’s environ recorded appreciably PAH values with 14.587µg/L of Acenaphthylene being the highest compared to the three lagoons. This could be as a result of the low salinity nature of the water which may increase dispersant effectiveness. Hence, risks to fish of PAH from other sources would be greatest in coastal waters where salinities are low and however fish from Densu River and Weija Dam may be at risk [18].

The average concentration of the PAHs in the lagoon water ranged from 0.552µg/L, for Acenaphthene to 11.399µg/ L for Benzo (b) fluoranthene. Other PAHs which recorded extremely high average values in the Chemu lagoon are Acenaphthylene (9.146µg/L), Benzo (k) fluora- nthene (6.644µg/L), Benzo (b) fluoranthene (9.948µg/L). This high PAHs contamination may result from the fact that Chemu lagoon has various refuse dumping sites along the bank where continuous burning of refuse are carried out, also fish smoking homes are located near the banks. In addition smoke emitting vehicles that continuously pry the road near and across some parts of the lagoon coupled with smoke emission from the chimney of the Tema oil refinery may be the major contributing factors to the high levels of contamination in the Chemu lagoon.

The average concentration of PAHs in the water sample of the Korle lagoon ranges from 0.000µg/L for Benzo (a) pyrene to 8.800µg/L for Benzo (ghi) pyrene. Apart from Benzo (a) pyrene which recorded zero micrograms per liter in the Korle lagoon. Other PAHs which recorded extremely high values in the Korle lagoon include Anthracene, Fluoranthene, Chrysene, Dibenz (a, h) anthra-

cene and Acenaphthylene with average concentrations of 8.310µg/L, 7.796µg/L, 3.099µg/ L, 6.198µg/ L, 2.978-

µg/L respectively (Table 4). The Korle lagoon stretches along the Accra Korlebu high-way thus receiving heavy smoke from vehicles that move constantly on the road. Aside this problem at some distance from the lagoon, there is a slaughter house which also produces thick smoke that may have caused PAHs accumulation in the lagoon. It is also suspected that effluent from the Korlebu Teaching Hospital may get into the lagoon which may also contribute to the level of pollution. In general it is gratifying to note that the level of PAHs in Chemu lagoon is relatively higher than that of the Korle lagoon. This may be due to the dredging process that was ongoing during the time of sampling at the Korle lagoon. In spite of this reduction, all PAH compounds analyzed with the exception of Benzo (a) pyrene exceeded the WHO acceptable limits and thus consumption of fish or any other food substances from the lagoon may prove dangerous to the health of the consumers.

Table 4 showed a compilation of PAH concentrations from the Kpeshie lagoon. The average concentrations of PAHs in the waters of the Kpeshie lagoon range from 0.052µg/L for Pyrene to 4.703µg/L for Acenaphthene. Other PAHs which recorded extremely high average values in the Kpeshie lagoon include Benzo (b) fluoranthene, Acenapthethene, Chrysene, Benzo (ghi) pyrene, Anthracene, Fluorine with average concentrations of 4.680µg/L, 4.003µg/L, and 4.374µg/L, 7.847µg/L, 3.362µg/L, 1.112µg/L respectively. Along the banks of the Kpeshie lagoon is stretch of mangroves which is used as hiding places for petty criminals therefore smoking of cigarettes and Indian hemp at these places is routine. Again some parts of the Kpeshie lagoon stretches along the Accra-Tema high way near La. It is interesting to note that just at the portion where the lagoon begins is also the starting point of a very serious traffic jam that has terrorized the inhabitants of Teshie and Nungua for years. Another very important consideration about the location of the lagoon its closeness to the Accra International Trade Fair Center. Waste discharged from the trade fair center to the lagoon might have also increased the level of PAH contaminations. It is therefore not far from right to say that these two major activities may have contributed to the PAH levels in the Kpeshie lagoon.

The concentration levels of PAHs detected were slightly varied from the location. The commonly found PAH compounds in water samples were acenapthene, fluorine, phenanthrene, fluoranthene, pyrene, benzo [b] fluoranthene. Naphthalene, acenaphthylene, benzo [a] pyrene and benzo [ghi] pyrene were found in all samples taken from oblogo dumping site and it’s environ. In the case of the three lagoons all PAHs were detected. There were huge variations between sites for some compounds (e.g., acenaphthene, acenaphthylene and benzo [a]pyrene), whereas the concentrations of the majority of
Table 4. Summary of PAH concentrations (µg/L) ±SD comparison from the Chemu, Korle and Kpeshie Lagoons.

| PAH              | CHEMU (µg/L) ±SD | KORLE (µg/L) ±SD | KPESHIE (µg/L) ±SD |
|------------------|------------------|------------------|--------------------|
| Naphthalene      | 0.78±0.437       | 0.44±0.581       | 0.19±0.000         |
| Acenaphthylene   | 9.14±3.356       | 2.97±0.431       | 4.70±4.742         |
| Acenapththene    | 0.55±1.006       | 1.02±1.254       | 4.00±4.505         |
| Fluorine         | 2.87±4.002       | 0.80±0.651       | 3.12±3.217         |
| Phenanthrene     | 0.93±0.387       | 0.45±0.082       | 1.14±0.609         |
| Anthracene       | 2.02±0.978       | 8.31±7.318       | 3.36±4.329         |
| Fluoranthene     | 4.78±3.367       | 7.79±5.413       | 1.57±0.912         |
| Pyrene           | 2.39±2.483       | 0.40±0.000       | 0.05±0.012         |
| Benzo (a) anthracene | 2.67±2.384   | 0.14±0.270       | 0.57±0.000         |
| Chrysene         | 1.48±0.000       | 3.66±2.923       | 4.37±6.190         |
| Benzo (b) fluoranthene | 9.94±8.668     | 0.49±0.167       | 4.68±4.024         |
| Benzo (k) fluoranthene | 6.64±9.739   | 0.42±0.326       | 2.32±2.547         |
| Benzo (a) pyrene | 1.14±1.357       | 0.00             | 0.37±0.656         |
| Indeno(1,2,3-cd)pyrene | 1.18±0.931   | 0.07±0.000       | 0.83±0.721         |
| Dibenz(a,h)anthracene | 3.74±2.133    | 3.09±1.654       | 1.62±0.708         |
| Benzo (ghi) perylene | 11.39±10.311  | 8.80±6.188       | 3.69±2.492         |
| TOTAL            | 61.712           | 38.889           | 34.091             |

Table 5. Summary of Correlation coefficients for seven different locations sample site for 16 PAHs (n =16).

| OBSW | LDRW | RD | WDS | CMU | KLE | KPSH |
|------|------|----|-----|-----|-----|------|
| OBSW | 1    | 0.228 | 0.208 | 0.708** | 0.305 | 0.495* | 0.208 |
| LDRW | 1    | 0.225 | 0.058 | -0.203 | 0.013 | 0.186 |
| RD   | 1    | -0.017 | 0.343 | 0.312 | -0.034 |
| WDS  | 1    | -0.046 | 0.083 | 1.000 | 1.000 |
| CMU  | 1    | 0.083 | 1.000 | 1.000 | 1.000 |
| KLE  | 1    | 0.046 | 0.083 | 1.000 | 1.000 |
| KPSH | 1    | 1.000 | 1.000 | 1.000 | 1.000 |

** Correlation is significant at the 0.01 level. * Correlation is significant at the 0.05 level.

OBSW = Oblogo solid waste disposal site, LDRW= Leach diluted with rain water, RD=River Densu, WDS= Weija Dam site, CHE= Chemu Lagoon, KLE= Korle Lagoon and KPSH= Kpeshi Lagoon.

compounds were comparable at the various sites which is similar to similar work by Kanchanamayoon and Tathathun (2009) [19]. The highest PAHs were recorded by compounds with molecular weights ranging from 128–154 (i.e., naphthalene, acenaphthene and acenaphthene) and those from 252–276 (i.e. [a] fluoranthene, perylene, benzo[a]pyrene, and benzo[g,h,i] perylene) which has also been reported by [20].

As far as the compositional pattern of PAHs is concern, the lagoon was generally dominated with all the PAHs. This relative abundance of low molecular weight PAHs (LPAHs) indicated that the PAHs were from petrogenic origin such as oil leakages or inadvertent oil spills [21].

Currently, there are no specific standards in Ghana for both inland and coastal waters for PAHs however evaluation of the toxicity that results from measured PAHs in the lagoons may be done by assessing their compliance with known international, national and provincial standards. According to the world health organization (WHO) the concentration of PAHs in water exceeding 0.05µg/L indicates some level of toxicity [22]. From Table 3 and 4 it can be observed that all the individual PAH compounds analyzed exceeded the WHO accepted value of 0.05µg/L and hence can be said that the Chemu, Korley and Kpeshi lagoon as well as Oblogo dumping site and its environ are polluted with PAHs. Therefore consumption of fish or any other edibles from the lagoons, Densu River, and Weija Dam may prove detrimental to the health of consumers.

3.2. Source Assessments

The differences in the type of PAH compounds at the different sites indicate that there are potentially different sources of PAHs in the area; possibly including sewage outfalls, industrial wastewater, thermal combustion processes (e.g., cooking and heating oils, and coal burning) followed by atmospheric fallout, oil residues, vehicular emissions (e.g., automobiles and trucks), and biomass burning (e.g., fire woods, charcoal, etc) [20]. From inspection of the distribution of PAHs in the surface water alone, it is often difficult to differentiate between the sources of inputs. The ratios of specific parent PAH...
Table 6. Summary of Correlation coefficients for PAHs compounds in the water samples at seven different locations (n = 13).

|       | NAP   | ACL   | AC    | F     | PH    | AN    | FL    | PYR   | BZA   | CHR   | BZB   | BZaP  | Ind   | Dib   | BZghi |
|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|-------|
| NAP   | 1     | 0.064 | 0.303 | -0.205| -0.052| -0.255| -0.504*| -0.225| -0.342| -0.059| -0.198| -0.105| -0.207| -0.113| -0.339|
| ACL   | 1     | -0.368| 0.209 | -0.010| -0.057| -0.084| 0.555* | 0.528*| -0.111| 0.762**| 0.546*| 0.570* | 0.170 | 0.504*| 0.046*|
| AC    | 1     | -0.012| 0.202 | -0.098| -0.167| -0.249| -0.188| 0.172 | -0.160| -0.127| -0.093| -0.096| -0.460| -0.353|
| F     | 1     | 0.715**| 0.171 | -0.092| 0.234 | 0.721**| -0.041| 0.335 | 0.313 | -0.093| -0.096| 0.460 | -0.353|
| PH    | 1     | 0.186 | -0.302| -0.232| 0.165 | -0.270| -0.150| -0.252| -0.237| 0.353 | -0.313| -0.358|
| AN    | 1     | 0.624*| 0.043 | 0.018 | -0.189| -0.127| -0.121| -0.190| -0.237| 0.114 | 0.067 |
| FL    | 1     | 0.447 | 0.155 | 0.072 | 0.133 | 0.270 | 0.207 | -0.164| 0.543*| 0.640**|
| PYR   | 1     | 0.769**| 0.090 | 0.805**| 0.857**| 0.881** | 0.432 | 0.746**| 0.833**|
| BZA   | 1     | 0.016 | 0.793**| 0.807**| 0.750**| 0.678** | 0.364 | 0.596* |
| CHR   | 1     | 0.216 | 0.399 | 0.411 | 0.176 | 0.074 | 0.241 |
| BZB   | 1     | 0.927**| 0.923**| 0.514* | 0.495* | 0.671**|
| BZK   | 1     | 0.975**| 0.574* | 0.481 | 0.728**|
| BZaP  | 1     | 0.560*| 0.527*| 0.709**|
| Ind   | 1     | 0.089 | 0.185 |
| Dib   | 1     | 0.877**|
| BZghi | 1     |       |       |

*Pearson Correlation is significant at the 0.05 level (1 tailed).
** Pearson Correlation is significant at the 0.01 level (1 tailed)

Naphthalene (NAP), Acenaphthylene (ACL), Acenapththene (AC), Fluorine (F), Phenanthrene (PH), Anthracene (AN), Fluoranthene (FL), Pyrene (PYR), Benzo (a) anthracene (BZA), Chrysene (CHR), Benzo (b) fluoranthene(BZB), Benzo (k) fluoranthene (BZK), Benzo (a) pyrene (BZaP), Indeno(1,2,3-cd)pyrene (Ind), Dibenz(a,h)anthracene (Dib), Benzo (ghi) perylene (BZghi).

Table 7. Summary of PAH ratios of water samples from sampling sites.

| RATIO | OBSW  | LDRW  | RD   | WDS  | CMU   | KLE   | KPSH  |
|-------|-------|-------|------|------|-------|-------|-------|
| AN/(AN+PH) | 0.599 | 0.330 | 0.552| 0.775| 0.683 | 0.948 | 0.747 |
| AN/(AN+F)  | 0.665 | 0.789 | 0.320| 0.413| 0.414 | 0.414 | 0.912 | 0.481 |
| NAP/(NAP+ACL) | 0.185 | 0.465 | 0.647| 0.426| 0.079 | 0.129 | 0.040 |
| NAP/(NAP+AC) | 0.461 | 0.122 | 0.322| 0.973| 0.586 | 0.301 | 0.046 |
| FL/(FL+PYR) | 0.985 | 1.000 | 0.931| 1.000| 0.667 | 0.951 | 0.912 |
| BZA/(BZA+CHR) | 0.000 | 1.000 | 0.699| 1.000| 0.372 | 0.039 | 0.116 |
| BZaP(BZghi) | 0.104 | 0.000 | 0.000| 0.136| 0.100 | 0.000 | 0.101 |
| BZB/(BZB+BZK) | 1.000 | 1.000 | 0.937| 1.038| 0.599 | 0.543 | 0.659 |
| Ind/(Ind+Dib) | 0.310 | 0.000 | 0.950| 0.000| 0.241 | 0.02  | 0.327 |

Compound abbreviations in Table 6.

Compounds have also been identified to be one approach to distinguish between different sources of PAH in a particular environmental matrix, [20] and this method is used in the present study to characterize the PAH sources to the lagoon. Correlation analyses can also provide information about associations between sites, between the individual PAH compounds, and between some specific PAH compounds and heavy metals to determine com-
mon origins.

3.3. Site Correlations

Correlation analyses between the sites’ individual PAH compound levels (n = 16) Table 5, indicates no significant correlation between leach diluted with rain water, River Densu, Chemu, korle and kpeshi with any other site suggesting the unique origin of PAHs from these sites. Weija Dam Site correlates strongly with Oblogo solid waste site with significant coefficient of 0.708 at 0.01 levels. The correlation between the two sites for the individual PAH compounds is also reflected in the similarities of the compositional patterns at these two sites (see map). However, despite the distance apart of Korle lagoon to Oblogo solid waste disposal site, their correlation is only fair (0.495) at 0.05 level, suggesting that PAH origins from these sites are quite different to overcome the site proximity.

3.4. PAH Interrelationships

In order to assess PAH associations and their possible origins, correlation analyses were conducted among the concentration of the individual PAHs in the water samples. The results are summarized in Table 6. It is known that where two compounds have a common source, there is more likely to be a correlation between their concentrations [20]. Strong positive significant correlation was observed between individual PAHs. Benzo (a) Pyrene and Benzo (k) fluoranthene showed the highest PAH interrelationship with correlation coefficient of 0.975 followed by Benzo (b) fluoranthene/Benzo (k) fluoranthene and Benzo (a) Pyrene/Benzo (b) fluoranthene correlated with 0.927 and 0.923 respectively, significant at 0.01 levels. The following pairs also interrelated strongly at the significant level of 0.01: Benzo (a) Pyrene/Pyrene (0.881), Dibenz (a,h) anthracene/Benzo (ghi) perylene (0.877), Benzo (k) fluoranthene/Pyrene(0.857), Benzo (ghi) perylene/Pyrene(0.833), Benzo (k) fluoranthene/ Benzo (a) anthracene(0.807), Benzo (b) fluoranthene/Pyrene (0.805), Benzo (b) fluoranthene/ Benzo (a) anthracene (0.793), Benzo (a) anthracene/Pyrene (0.769), Benzo (b) fluoranthene/Acenaphthylene (0.762), Benzo (a) Pyrene/ Benzo (a) anthracene (0.750), Pyrene/ Dibenz (a,h) anthracene(0.746±0.00), Benzo (ghi) perylene/ Benzo (k) fluoranthene(0.728), Benzo(a)anthracene/Fluorine (0.721), Fluorine/Phenanthren (0.715±0.03), Benzo (ghi) perylene/ Benzo (a) Pyrene (0.709), Indeno(1,2,3-cd)pyrene/ Benzo (a) anthracene(0.678), Benzo (ghi) perylene/ Benzo (b) fluoranthene(0.671), Benzo (ghi) perylene/ Fluoranthene(0.640).

At 0.05 level, significant positive correlation were also observed between PAHs with Anthracene/ Fluoranthene recording the highest correlation coefficient of 0.624 followed by Benzo (a) anthracene/ Benzo (ghi) perylene (0.596), Fluorine/ Indeno(1,2,3-cd)pyrene (0.588), Benzo (k) fluoranthene/ Indeno(1,2,3-cd)pyrene(0.574), Benzo (a) Pyrene / Acenaphthylene (0.570), Benzo (a) Pyrene/ Indeno(1,2,3-cd)pyrene (0.560), Pyrene/ Acenaphthylene (0.555), Benzo (k) fluoranthene/Acenaphthylene(0.546), Dibenz (a,h) anthracene/Fluoranthene(0.543), Acenaphthylene/ Benzo (a) anthracene (0.528), Dibenz (a,h) anthracene/ Benzo (a) Pyrene (0.527), Benzo (b) fluoranthene/ Indeno(1,2,3-cd)pyrene (0.514), Acenaphthylene/ Dibenz (a,h) anthracene (0.504), Benzo (b) fluoranthene/ Dibenz (a,h) anthracene(0.495), Benzo (ghi) perylene/ Acenaphthylene(0.486) and Benzo (k) fluoranthene/ Dibenz (a,h) anthracene (0.481) in that order. The results reveal that these compounds, and to a lesser extent pyrene, were possibly derived from a common anthropogenic origin. No significant correlation was identified between Acenaphthene and chrysene compound with any of the other PAH compounds measured which indicate other source of these two PAHs. In one case, Fluoranthene (FL) (containing 3 fused aromatic rings) showed inverse correlation with Naphthalene (at 0.01 level) (Table 6) containing 2 fused aromatic rings. It is speculated that some fraction of these compounds could be from the biodegradation of Fluoranthene (FL) by natural occurring population of water microorganisms since Fluoranthene is a polycyclic aromatic hydrocarbon (PAH) consisting of naphthalene and a benzene unit connected by a four-membered ring. It is also known to occur naturally as a product of plant biosynthesis [27]. Further studies are required to verify this speculation.

3.5. PAH Isomer Pair Ratios as Diagnostic Source Indicators

The ratios of specific PAH compounds have been identified to possess the potential to distinguish natural and anthropogenic sources. [24-25]. To minimize confounding factors such as differences in volatility, water solubility, adsorption etc. ratio calculations are usually restricted to PAHs within a given molecular mass [24]. Yunker et al 2002 have summarized the literature on PAH ratios for petroleum, single-source combustion and some environmental samples and made the following conclusions. For mass 178, an anthrancene to anthracene plus phenanthrene (AN/[AN+PH]) ratio of >0.50 usually is an indication of biomass & coal combustion transition point. For mass 202, a fluoranthene to fluoranthene plus pyrene (FL/[FL+PYR]) ratio of >0.50 seems to be the characteristic of grass, wood or coal (biomass) & coal combustion transition point, though not definite. For mass 228, a benzo[a]anthracene to benzo[a] anthracene plus chrysene ratio <0.20 imply petroleum, 1.2–5.0 indicates wood burning and coal burning [26], and >0.35 imply combustion [25]. The ratios of the above-specified PAHs in the Oblogo dumping site, Leach diluted with
rain water, River Densu, Weija and Dam sites as well as Chemu, Korle and Kpeshi lagoons were calculated and are shown in Table 6. The AN/AN+PH ratios are all >0.50, suggesting grass, wood or coal (biomass) & coal combustion sources of PAH from all eight sites. However, the smaller ratios (0.330) obtained for Leach diluted with rain water (LDRW) distinguishes it from the other sites. It appears there is mixed petroleum and combustion sources at this site. The BZA/(BZA+CHR), AN/(AN+F), FL/(FL+PYR), BZB/(BZB+BZK), Ind/(Ind+Dib), mixed ratios of >0.01, 0.4–0.5 and ≥0.50 re-echo the predominance of grass, wood (biomass), coal and petroleum combustion are the main source of PAH from the Oblogo solid waste dumping site down stream to Weija Dam down to Densu River. This confirms the belief that the burning of solid waste at Oblogo solid waste dump site is polluting the environment with PAHs. At the Chemu, Korle and Kpeshi Lagoons NAP/(NAP+ACL) and BZAP/(BZghi) ratio of >0.10 suggests a combustion source which is said to be affluent from Tama oil refinery. The BZA/ (BZA+CHR) ratios whose interpretations are said to be more definitive [24] provided more distinctions between the sites. Based on >0.35 as the transi
tion ratio, the calculated 0.375 suggests a combustion source for Chemu Lagoon, mixed unburned petroleum and combustion sources for Korle and Kpeshi. The mixed petroleum and combustion sources at this site is confirmed from the Ind/(Ind+Dib) fraction, where ratios of 0.241 (Chemu Lagoon) and 0.327 (Kpeshi Lagoon), which falls within the generally observed mixed-source ratio of 0.2–0.35 for mixed petroleum/combustion origin of pollution are observed. At Korle Lagoon ratio of >0.10 implying unburned petroleum source has also been observed.

Despite the lack of consistency in some cases, there seems to be a general consensus by all the ratio indicators that combustion is the dominant source of PAH input into the lagoon. Although not conclusive, there is also an indication of petrogenic source contributions to sites such as chemu, korle, kpeshi as well as Weija Dam (Table 6). Variations in additional input sources (e.g., high or medium temperature combustion processes, different fossil materials) may also account for the differences in the composition pattern of PAHs between sampling sites (Table 7) [27]. Despite the apparent dominance of combustion and wood/coal burning (pyrogenic origin) as the major source of anthropogenic PAH to the Oblogo solid waste dump site, weija Dam, River Densu, Chemu, Korle and Kpeshi Lagoons sites (using the ratio indicators above). The AN/(AN+F) and NAP/(NAP+ACL) ratios suggests petroleum combustion for Waija Dam, Chemu and Kpeshi sites (Table 7). It is therefore possible that combustion of liquid fossil fuel is the major source of PAH to the lagoons and the other sites.

NB: Source Patterns from the literature [24-25]; >0.10: combustion source; <0.10: unburned petroleum source, >0.50: biomass & coal combustion, 0.4–0.5: petroleum combustion, <0.40: unburned petroleum, >0.35: combustion, 0.20–0.35: mixed petroleum/combustion <0.2: unburned petroleum source, 1.2–5.0: wood burning and coal burning [26].

4. Conclusions

Results obtained from the study clearly demonstrated that the leach from Oblogo solid waste disposal site and its environs as well as Chemu, Korle and Kpeshi Lagoons are polluted by Polycyclic Aromatic Hydrocarbons with concentration ranging from below detection level to 14.587µg/L. However, seven carcinogenic PAHs were detected in different concentrations from the various sites. It is important that, those PAHs promulgated by USEPA to be toxic and need to be investigated in developing countries. Acenaphthene, Anthracene, Benzo[a]anthracene, Benzo[b]fluoranthene, Chrysene, Phenanthrene occurred at the various sites above the safety level set by WHO. It can be concluded that people living around the Oblogo solid waste disposal site, who swim and bath in the downstream of River Densu would be exposed to these PAHs and may be at risk of their harmful effects.

The average concentration of PAHs in the water ranged from 0.552µg/L, for Acenaphthene to 11.399µg/L for Benzo (ghi) perylene of the Chemu Lagoon, 0.00µg/L for Benzo (a) Pyrene to 8.800µg/L for Benzo (ghi) perylene (Korle Lagoon) and 0.052µg/L for Pyrene to 4.703ug/L for Acenaphthylene of the Kpeshi Lagoon.

Good site correlations shown by water samples from Oblogo solid waste site and Weija Dam which derive their source mainly from burning of biomass and coal and combustion processes demonstrate how open dumping and burning procedure used in Ghana can pollute the environment. Other site far apart seem to inter-relate in terms of their PAH levels. Close relationships were also found between all individual PAH compounds except Acenaphthene and chrysene which did not show any correlation with other PAHs. Benzo (a) Pyrene and Benzo (k) fluoranthene showed the highest PAH-PAH associations. The correlation and ratios of PAHs results revealed that these compounds were possibly derived from a common anthropogenic origin. There seems to be a general consensus from some three PAH-PAH ratio indicators that combustion and burning of biomass are the dominant source of PAH input into the Oblogo dumping sites down stream and the three lagoons studied. Although not conclusive, there is also an indication of petrogenic source contributions from some sites especially, the Chemu, Korle and Kpeshi Lagoons. Particularly in the vicinity of the Tema oil refinery.
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### Appendix

#### SUMMARY OF PAH CONCENTRATIONS IN CHEMU

| PAHs                      | 1   | 2   | 3   | AVERAGE±SD |
|---------------------------|-----|-----|-----|-------------|
| Naphthalene               | 0.862 | 1.481 | 0.000 | 0.781±0.437 |
| Acenaphthylene            | 12.739 | 6.092 | 8.609 | 9.146±3.356 |
| Acenaphthene              | 0.000 | 0.117 | 1.539 | 0.552±1.006 |
| Fluorine                  | 7.496 | 0.682 | 0.454 | 2.877±4.002 |
| Phenanthrene              | 0.351 | 0.195 | 0.929 | 0.939±0.387 |
| Anthracene                | 3.195 | 1.583 | 3.350 | 2.020±0.978 |
| Fluoranthene              | 7.726 | 1.529 | 1.329 | 4.787±3.367 |
| Pyrene                    | 1.828 | 0.238 | 5.107 | 2.391±2.483 |
| Benzo (a) anthracene      | 5.906 | 1.493 | 0.620 | 2.673±2.384 |
| Chrysene                  | 4.456 | 0.000 | 0.000 | 1.485±0.000 |
| Benzo (b) fluoranthene    | 19.863 | 6.178 | 3.803 | 9.948±6.688 |
| Benzo (k) fluoranthene    | 17.827 | 2.079 | 0.026 | 6.644±9.739 |
| Benzo (a) Pyrene          | 2.704 | 0.252 | 0.469 | 1.142±1.357 |
| Indeno(1,2,3-cd)pyrene    | 0.195 | 1.130 | 2.056 | 1.187±0.931 |
| Dibenz(a,h)anthracene     | 5.357 | 1.323 | 4.544 | 3.741±2.133 |
| Benzo (ghi) perylene      | 23.091 | 3.604 | 7.502 | 11.399±10.311 |

#### SUMMARY OF PAH CONCENTRATIONS IN KORLE

| PAHs                      | 1   | 2   | 3   | AVERAGE±SD |
|---------------------------|-----|-----|-----|-------------|
| Naphthalene               | 1.101 | 0.209 | 0.011 | 0.440±0.581 |
| Acenaphthylene            | 4.952 | 4.343 | 0.000 | 2.978±0.431 |
| Acenaphthene              | 0.283 | 0.310 | 2.469 | 1.021±1.254 |
| Fluorine                  | 1.389 | 0.915 | 0.102 | 0.802±0.651 |
| Phenanthrene              | 0.548 | 0.389 | 0.436 | 0.457±0.082 |
| Anthracene                | 10.186 | 0.236 | 14.508 | 8.310±7.318 |
| Fluoranthene              | 3.998 | 5.397 | 13.995 | 7.796±5.413 |
| Pyrene                    | 0.000 | 0.000 | 1.202 | 0.401±0.000 |
| Benzo (a) anthracene      | 0.403 | 0.021 | 0.000 | 0.143±0.270 |
| Chrysene                  | 4.129 | 5.779 | 1.100 | 3.669±2.923 |
| Benzo (b) fluoranthene    | 0.235 | 0.569 | 0.395 | 0.490±0.167 |
| Benzo (k) fluoranthene    | 0.974 | 0.102 | 0.160 | 0.412±0.326 |
| Benzo (a) Pyrene          | 0.214 | 0.000 | 0.000 | 0.071±0.000 |
| Indeno(1,2,3-cd)pyrene    | 0.856 | 3.735 | 3.707 | 3.099±1.654 |
| Dibenz(a,h)anthracene     | 1.834 | 13.66 | 10.906 | 8.800±6.188 |

#### SUMMARY OF PAH CONCENTRATIONS IN KPESHIE

| PAHs                      | 1   | 2   | 3   | AVERAGE±SD |
|---------------------------|-----|-----|-----|-------------|
| Naphthalene               | 0.058 | 0.000 | 0.000 | 0.195±0.000 |
| Acenaphthylene            | 1.423 | 10.158 | 2.592 | 4.703±4.742 |
| Acenaphthene              | 1.352 | 1.470 | 9.214 | 4.003±4.505 |
| Fluorine                  | 1.120 | 6.832 | 1.409 | 3.120±3.217 |
| Phenanthrene              | 1.174 | 1.732 | 0.516 | 1.140±0.609 |
| Anthracene                | 0.377 | 8.329 | 1.385 | 3.364±4.329 |
| Fluoranthene              | 0.746 | 2.552 | 1.422 | 1.573±0.912 |
| Pyrene                    | 0.000 | 0.070 | 0.087 | 0.052±0.012 |
| Benzo (a) anthracene      | 0.000 | 1.715 | 0.000 | 0.572±0.000 |
| Chrysene                  | 1.531 | 0.116 | 11.475 | 4.374±6.190 |
| Benzo (b) fluoranthene    | 0.062 | 7.435 | 6.543 | 4.680±0.424 |
| Benzo (k) fluoranthene    | 0.000 | 1.690 | 5.292 | 2.327±2.547 |
| Benzo (a) Pyrene          | 0.000 | 0.094 | 1.022 | 0.372±0.656 |
| Indeno(1,2,3-cd)pyrene    | 1.512 | 0.078 | 0.928 | 0.839±0.721 |
| Dibenz(a,h)anthracene     | 1.930 | 2.119 | 0.809 | 1.626±0.708 |
| Benzo (ghi) perylene      | 1.511 | 6.362 | 2.948 | 3.697±2.492 |
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Environmental Pollution and Public Health (EPPH2010)
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