CASE STUDY CS (AR) 175

WATER QUALITY MODELING OF KALI RIVER USING QUAL2E

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PREFACE

River/Stream ,since history of our civilation, has been supporting the main sources of our need and for development of water resources programmes. Besides the Socio-Economic , Socio-Cultural development of a region, river/stream plays a significant role in shaping the environmental and ecological balance of a region through which it is routed. Thus control and management of river/stream water quality which in many region is under threat of pollution or already polluted, is mandatory for our sustainable development.

Any rational formulation of water pollution control and management policies and programmes can not be possible without adequate knowledge of the existing nature, magnitude and identification of sources that cause pollution in the river water. The study of behavior of pollutants in the river water is equally important to assess the degree of pollution over time and space to have the in hand information about the incoming pollutants. Modeling of water quality constituents thus come in picture. Water quality processes being complex phenomena it is difficult to assess the exact pollution status unless sufficient historical data are available. Nevertheless, appropriate modeling of water quality parameters demands large number of data that need to be obtained from continuous monitoring.

As an attempt the Institute under its work programme of Environmental Hydrology Division took up the study of pollution assessment of River Kali near Muzzafarnagar and modeling of some selected water quality parameters using Enhanced Stream Water Quality Model, QUAL2E, developed by US-EPA.

The study titled " **Water Quality Modeling of River Kali Using QUAL2E" has been carried out by Shri. N.C. Ghosh, Scientist "C"** *of Environmental Hydrology Division under the work programme of 1994-95 with the supporting inputs from Dr. K.K.S. Bhatia, Sc.'F', Dr. C.K.Jain, Sc.'C', and Shri Dayaram, Sc. 'B'.*

Any field related study needs involvement of large man power; for water quality studies it is even more. Supports of technical staffs of the Division namely: Shri T.R. Sapra, Shri Rakesh Goyal, Shri. Om Prakesh, Miss Babita, and Snri. Tejpal Singh, fulfilled that need.

(S.M. Seth)
Director

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ABSTRACT

River/Stream, whether or not its water is used for different beneficial purposes , plays a significant role in shaping the socio-economic, socio-cultural, over and above, environmental and ecological balances of a region through which it is routed. Despite of all these versatile contributory roles, knowingly or unknowingly, river/stream, in many areas, are being used as a dumping spot of disposal of municipal and industrial wastewaters with the intention that river/stream will take care of these wastes. Waterbodies have some limiting cleansing capacity of wastes- a known fact- excessive discharge of effluent beyond the self purification capacity of waterbodies would definitely pollute the water. If the situation continues for long time, a fresh water stream may even transform to a sewer or in other words, a water surplus region may transform to a water scarce region.

The study of river kali reported here is an unique example of transformation of a fresh water stream (as it is seen before the mixing of wastewaters) to a sewer by the direct disposal of municipal, industrial and sugar mills wastewaters originating from Muzaffarnagar city and adjoing areas.

Attempts have been made in this study to address the following aspects :

i) present state of pollution over stretches downstream of effluent discharges, and modeling of some water quality parameters using QUAL2E,

ii) determination of assimilative capacity of the river with reference to BOD,

iii) flow augmentation required to arrest the water quality within prescribed limits designated for different uses,

and, iv) determination of suitable WLA (Waste Load Allocation), as an alternative to flow augmentation, to maintain the self purification capacity of the river,

Within the study stretch of 65 Km. starting from Malira bridge (about 10 Km. upstream of Muzzafarnagar city) upto the confluence with Hindon river (stretch affected by the human activities) wastewaters quality of 4 point sources originated from the

municipal, industrial and sugar mills, and river water quality of 15 different locations have been monitored and modeled to get the indicative of 8 water quality constituents. Out of the river flows of about 7.25 m^3/sec (dry weather flow) , 1.37 m^3/sec (19 % of river flow) is contributed by 4 outfalls in which two municipal drains contain respectively; 0.30 m^3/sec and 0.36 m^3/sec while industrial and sugar mills drain share 0.554 m^3/sec and 0.166 m^3/sec respectively. The pollution load interms of BOD as measured for all these 4 outfalls respectively are : 325 mg/l, 328 mg/l, 801 mg/l and 1695 mg/l.

Computation of concentration profiles using QUAL2E shows a good match between the observed and computed values. Two distinct DO sags; one is between the municipal drain and industrial drain at river Km. 45 (from confluence) with critical DO value of 1.74 mg/l, another is downstream of industrial and sugar mills drains for a stretch of 20.0 Km. within river Km.32 to 12.00 (from confluence) with critical DO value of "0" , are obtained.

In order to overcome the critical stage of DO deficit, attempt has also been made to assess the flow augmentation required (considering Headwaters is the source) and the tolerance limit of waste load application i.e., control of pollution at source, as two alternatives. It is observed that 4.21 times (15.5 m^3/sec) of headwaters flow (11.82 m^3/sec additional flow) is required to increase the DO level from 1.74 mg/l to 5.0 mg/l for the first DO sag, while in case of second DO sag i.e., after sugar mills drain, 19.1 times (70.26 m^3/sec) of headwaters flow (66.58 m^3/sec additional flow) is required to increase the DO level from "0" to 5 mg/l. For waste load allocations, in order to get back the river DO level at 5 mg/l, about 80% of effluent BOD for municipal wastewates and 85% effluent BOD of industrial and sugar mills wastewaters are to be removed before being discharged to the river. Waste Load Allocation seems to be the better option than the flow augmentation (as there is no source of augmentation of flow) in order to maintain the self purification capacity and also the health of the river. The report also contains the simulated profiles of Nitrogen cycle, Phosphorous cycle, TDS, and pH.

Results reported in this study are based on limited data sets collected by ourselves thus reflect an indicative of pollution status of the river and is an attempt towards application of QUAL2E to see its effectiveness of the model. Details study based on continuous monitoring is, therefore, recommended to check the effectiveness of findings.

which are usually considered as the main component to represent the water quality indices have been modeled with reasonable accuracy using the enhanced stream water quality model, QUAL2E, developed by US-EPA. A good match between the observed and computed values is obtained that reflects the successful application of the model. In order to overcome the situation of present problem of water pollution and to improve the water quality of the river up to the permissible tolerance limits for which waters can be used; different control and management options have been suggested. It is observed that Waste load allocation is the better option than the flow augmentation to gain the self purification capacity of the river.

2.0 NEEDS AND IMPORTANCE OF THIS STUDY

i) Any rational formulation of water pollution control policies and programmes can not be possible without adequate knowledge of the existing nature, magnitude and sources of the various pollution loads which degrades the quality of river water. The study of behavior of these pollution loads specially the concentration profile of different pollutants in the river water is equally important to asses the degree of pollution that are prevailing, and to identify the stretches which violate the standards and harmful for use. Over and above, to determine the permissible assimilative capacity of the river and to make the waste load allocation to maintain the assimilative capacity of the river, it is necessary to assess the prevailing status of pollution and their probable trends.

ii) River whether seasonal or perennial, in many ways, directly or indirectly , serves an area whether it's water is used for beneficial purposes or not. In the direct benefit, river serves; in many areas, daily need of waters for domestic, industrial, and agricultural purposes, as a recreational spot for man, animal and other organisms, used as bathing place for man and animal, and used for laundry purposes. As per the indirect benefit is concerned, the river maintains an ecological balance in the region at large. Activities of human and other organisms, even the flora and fauna are always concentrate within and around the nuclei of water.

It, thus, becomes the responsibility of the society or an individual or our planners to take necessary preventive measure to control pollution of waters that maintain an ecological and environmental balance in a region. If a water body is found polluted, we should not come backward assuming that these waters are not useable or harmful to use and therefor have no significance value to the society at large rather we should come forward to clean the water body with all possible efforts and should lay down all regulatory measure to get back the cleansing capacity of the water and make use of the water for the possible beneficial uses. Before multiplying the problem in many facets, we have time to overcome the situation if we put efforts to do so.

iii) For preparing a realistic pollution control programme, one should not only have a current inventory of pollution loads, but also have a perspective view of the trends of development in specific areas of human activities during the forthcoming period and the likely impact of these activities on pollution loads and the quantity and quality of water in various water bodies. For doing this, it would be necessary to analyze and develop inter-relationships between various aspects of human activities, stream flows, pollution loads, and water quality.

iv) The river kali - the one of main tributary of Hindon - though have little significance as per as uses of waters are concerned, however, has a great deal of socio-economic development of the region through which the river is routed. The important city " Muzzafarnagar city" is located on the left bank of the river. Besides the muzzafarnagar city there are some small towns and villages located in the vicinity of the river basin. Tangible benefits from the river's water , if estimated, would not show any appreciable figure, however, the intangible benefits that are gained by the habitats living in and around the river would be appreciable and would show importance of the river. Kali river is the only natural flowing sources of water in the vicinity of Saharanpur-Muzzafarnagar and, perhaps, play the dominant role in maintaining the ecological, hydrological and environmental balance of the region. One of the basic needs of industrial development is assurance of water supply and requirement of a water body for easy disposal of wastes, both solid and liquid, that are to be generated from industries without creating any nuisance value to the society or that area. Industrial activities in and around Muzzafarnagar district have perhaps been developed and confined mainly to this area because to have the easy access of disposal of wastes to the river. Direct disposal of municipal and industrial wastewaters in to a river would definitely have negative impact, less or more, depending upon the river flow and the quantum of waste loads and their magnitude of pollutant concentrations. Continuous and uncontrolled disposal of wastes ,municipal and industrial, without any regards to water quality of the river have made us curious to take up the study to address the following objectives:

i) to assess the present state of pollution over the stretches downstream of the point of discharges,

ii) to assess the assimilative capacity of the river,

iii) to make suitable waste load allocations for keeping the river water quality within the prescribed limits designated for different uses.

iv) to assess the flow augmentation required for bringing up the water quality within the prescribed limit, as an alternative of Waste load allocations.

and, v) to make suitable recommendations to control and manage the water quality problems of the river in order to avoid the environmental imbalance, if not being observed now but for the future.

3.0 DESCRIPTION OF THE STUDY AREA

The study has mainly been concentrated in the lower stretch of the river for a distance of about 65 Km. started from Malira Railway Bridge located about 10 Km. upstream of Muzzafarnar city , and continued downstream upto the confluence of the river with Hindon river. This is the stretch actually affected by the human activities eg., direct disposal of municipal and industrial wastes. In the upstream, there is virtually no pollution load entering into the river. The subsequent discussions reflect the geographical, Socio-economic, socio-cultural, hydrologic and environmental features of the river basin.

3.1 Geographical Features of the Basin :

The main stream of river Kali- popularly Known as West Kali river and locally known as Kali nadi (reasons of calling Kali nadi is not known, however, local people expressed that the waters of the river being "kala" i.e, in english it is "black"; that is why the river is called as "Kali nadi"- actual reason may be different) - originates from the plains of Himalaya namely from the Doon Valley located in the Western part of Uttar Pradesh. The river basin covers a drainage area of approximately 750 sq.Km.(conservative estimate) within the latitude of 29^0 13' - 30^0 N and longitude of $77^0$32' - $77^0$45'E. From its origin upto the confluence with Hindon river- one of the tributaries of Yamuna, the river travels a distance of approximately 150 Km. over the Saharanpur district ,and Muzzafarnagar district. Despite of considerable drainage areas, the river does not carry any significant surface flow but behaves like a perennial river with influents that are observed after a considerable distance receives from the ground water sources. As such there is no mentionable tributaries joining the river except some nalas containing either seepage waters or waters from agricultural fields through overflow. The important city namely; Muzzafarnagar (District Headquarters of Muzzafarnagar) population according to 1971 cencus was 2,04,156 (CPCB report no - ADSORBS/2/1980-81) present status (upto 1994) if estimated with 2.5% population growth rate , may be in the order of 3,60,000 (compounded figure of population), is situated on the left bank of the river between the Kilometer point 90 and kilometer point 105 from the origin. Many small towns (population more than 25,000 but below 50,000) and villages (population above 2,000 but below 5,000) are also located in the vicinity of the river basin, and taking the advantages of the river. To mention few important one ; on the right side of the river, Deoband, while on the lift side, Shardana, are among the small towns situated in the vicinity of the river basin. Among the villages, Surju, Jaradaunara, Naula, Chandsina, Ancholi, and Pitlokar are located on the left bank of the river while Mansurpur is located on the right bank of the river. Index map of the basin given in Fig.1 represents locations of various details.

3.2 Land uses Features of the Basin

The river basin area is mainly governed by the agricultural activities except the urban portion of Muzzafarnagar city which is governed mainly by small and medium scale industries besides the urban activities. There are many industries like, sugar mills, Steel rolling mills, Chemical factories, etc.located in and around the city of Muzzafarnagar. Economy of surrounding villages are based on the agricultural productions. Crops that are generated from agricultural fields are; Sugarcane, Wheat, and variety of

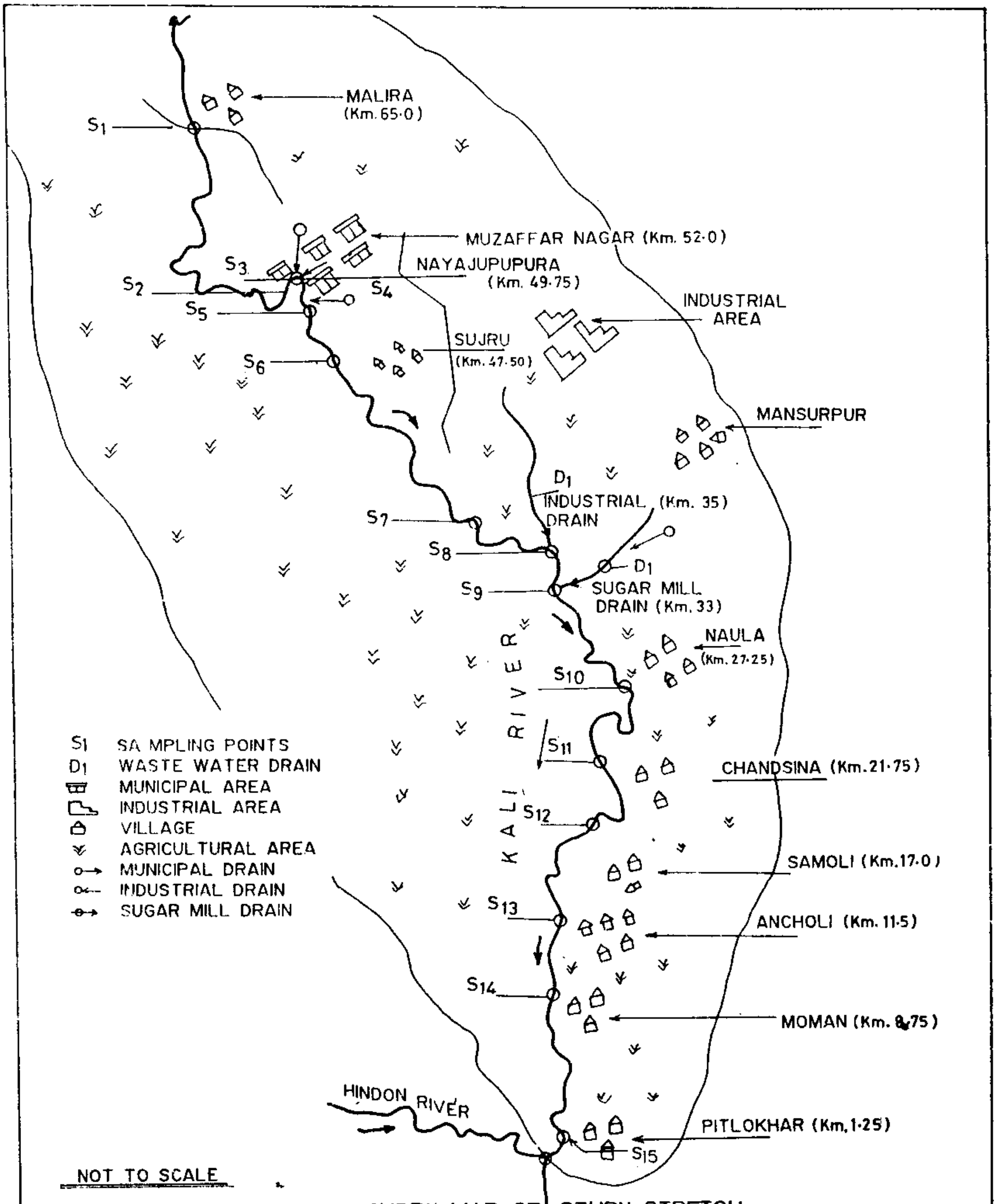


FIG.1 INDEX MAP OF STUDY STRETCH

winter and summer vegetables. Waters for irrigation in the agricultural fields are partly cover by the Upper Ganga Canal and remaining from the ground water sources through tube wells dug by the State Irrigation Department /Agricultural Department or by the farmers themselves. On enquiry from the villagers, it is noticed that river waters were in use for different purposes before the induction of industrial wastes, however, after induction of industrial wastes, river waters were no more in use even for irrigation purposes mainly after downstream of industrial drain, however, river waters, even after the disposal of industrial wastewater, are routinely being used for; washing clothes by the professional laundry men, bathing of domestic animals, used in some stretches as a swimming spot for children, washing vegetables before bringing them to the market; in some places river serves a place for sanctuary of migration of birds, etc..

3.3 Hydrological Features

The flow in the river varies from stretches to stretches considerably because the source of feeding being the ground water. The flow that has been estimated near the malira bridge is observed to vary from 3.60 m^3/sec to 3.76 m^3/sec during dry season while before the confluence of Hindon, dry weather flow varies from 7.25 m^3/sec to 7.40 m^3/sec. Out of the confluence discharge variation of 7.25 m^3/sec to 7.40 m^3/sec, about 19 % (1.37 m^3/sec) of total flow is contributed by the municipal and industrial drains located nearby of Muzzarfarnagar city. Balance flow is entered into the river through the surface and subsurface flow. Upstream of Malira bridge, the river is mainly fed by the subsurface flow with an estimated rate of entry 3648 m^3/day/km. During monsoon period, it is learnt that river carries considerably high flow and gets overflow over banks in many stretches. In the vicinity of the river basin, the Upper Ganga Canal which travels with slight inclination towards east from the flow axis of the river, supports the irrigation facility to the agricultural fields through branch and field canals. It is noticed that in many occasions canal waters are drained out to the river Kali through the drains located near Naula. In such circumstances, it is noticed that river bottoms and river waters get cleaned considerably. This is not done very often, however, very occasional mainly during the time when canal needs cleaning or repairing. Domestic needs in the Urban areas and small towns including drinking water are supported from the deep ground water sources while for villages that are supported through shallow tube wells.

3.4 Socio-Economic and Socio-Cultural Features

Socio-economic and Socio-cultural activities in & around the vicinity of the river basin are largely based on the agricultural activities. Standard of living of people and public health in the urban areas apparently seem to be average as prevailing in other similar cities, however, there are lots of anxiousness among the people about the health hazard problems being created by the unscientific management of wastes. While the standard of living and public health in the rural sectors in general are very poor.

3.5 Pollution Aspects

The city Muzzafarnagar has partly organized sewer system through which municipal wastewaters of about 11520 m^3/day (avg.) and 13850 m^3/day (avg.) enter into the river in two points near Nayazupura (R. Km. 52) and Shamli bridge (R. Km.49.75) respectively. Industries located in and around the city are mostly disposed off their wastewaters without any treatment to sewer canal which joins the river at kilometer point 35.00 (down of municipal drain) from the origin. The daily average flow of raw industrial wastewaters containing high BOD (Bio-chemical Oxygen Demand) & COD(Chemical Oxygen Demand) enter into the river touch upon the figure of 48390 m^3. About 1.75 km. downstream of industrial drain, the sugar mill wastewaters drain with an average flow rate of 14670 m^3/day joins the river. Wastewaters both from municipal and industrial sources are being disposed off to river without any treatment. There is no sewer system for small towns and villages. Wastes, solid and liquid, generated from these areas are disposed off over agricultural lands.

4.0 METHODS AND METHODOLOGIES

In fact, no studies and efforts have been made so far to assess the pollution aspects of Kali river except the efforts made /are being made by the National Institute of Hydrology, Roorkee. However, the Central Pollution Control Board, during year 1978-'79 (ADSORB/2/1980-81) under Yamuna Basin studies had made an assessment of pollution loads generated by different areas in which an indicative of expected pollution from Muzzafarnagar area was touched upon. As such, no baseline information were available before taking up the study. The study was under taken with an aim to conduct detail survey of the river to fulfill the data requirement for water quality modeling, giving special emphasis on collection of flow and river hydraulic data at various locations of the river, monitoring of pollution loads covering flow, BOD, COD, DO, TDS, Solids, pH, Temperature, Hardness, Nitrate, Phosphate, and some other conservative elements etc. for the point sources of pollution and river water as well, at different locations. Sites for collection of samples and monitoring of water quality parameters are initially chosen from the topo-sheet of the basin and finally checked them with local conditions. Locations identified for collection of samples and monitoring of water quality data at sites have shown in Fig. 1. Hurdles faced to fix up the monitoring sites and in collection of samples are a point to make a note. During the time of sampling, efforts have always been made to get samples which would give the representative values what actually prevails in the river. Standard practices and personal efforts as required at the sites have been followed to achieve the reliable data. For example, in places where river is not appreciably deep and there is no scope to collect samples from bridge, samples have been collected from the mid of the river taking a walk inside the river (Plate-1). River cross sections at various locations have also obtained in the similar fashion (Plate -2). Velocities at different locations where cross sections have been measured, are measured by Pigmy Current meter. Total length of 65 Km. starting from Malira Bridge (as a reference point where river waters are not effected by human activities) upto the confluence of the river with Hindon are covered for monitoring the water quality and for detailed study.

Laboratory analysis of samples collected from different locations were in the routine job to assess the concentration of pollutants for BOD_5, DO (either preserved for laboratory analysis or monitored at site), COD, NO_3-N, PO_4, TDS, Solids, and some conservative elements (details reported later on). Parameters monitored at site are; PH, Temperature, river depth, width, and velocity at different locations.

Data collected from sites or obtained through laboratory analysis have been made in representative form to use in the QUAL2E for assessing the impact of pollutants over the downstream and further analysis. The detail descriptions of QUAL2E (enhanced stream water quality model) are given in subsequent sections.

late - 1 : Attempt Towards Collection of Samples from the Mid of the River to Ensure Representative Value

Plate -2 : An Attempt to Measure the River X-Section

5.0 DESCRIPTION OF THE ENHANCED STREAM WATER QUALITY MODEL QUAL2E

QUAL2E, is a comprehensive and versatile stream water quality model, permits simulation up to 15 water quality constituents in any combination desired by the user in a branching stream system using a finite difference solution to the one-dimensional advective-dispersive mass transport and reaction equation. Constituents which can be simulated are :

i) **Dissolved Oxygen**
ii) **Biochemical Oxygen Demand**
iii) **Temperature**
iv) **Algae as Chlorophyll a**
v) **Organic Nitrogen as N**
vi) **Ammonia as N**
vii) **Nitrite as N**
viii) **Nitrate as N**
ix) **Organic Phosphorus as P**
x) **Dissolved Phosphorus as P.**
xi) **Coliforms**
xii) **Arbitrary Nonconservative Constituent**
xiii) **Three Conservative Constituents.**

The model allows for multiple waste discharges, withdrawals, tributary flows, and incremental inflow and outflow. It also has the capability to compute required dilution flows for flow augmentation to meet prespecified dissolved oxygen level. The model can either be used as a steady-state or as dynamic model. When operated as a steady-state model, it can be used to study the impact of waste loads (magnitude, quality and location) on instream water quality and also can be used in conjunction with a field sampling program to identify the magnitude and quality characteristics of nonpoint source waste loads. By operating the model dynamically, it is possible to study the effects of diurnal variations in meteorological data on water quality (primarily dissolved oxygen and temperature) and also can study diurnal dissolved oxygen variations due to algal growth and respiration. However, the effects of dynamic forcing functions, such as headwater flows or point loads, cannot be modeled in QUAL2E.

QUAL2E-Uncas is enhancement to QUAL2E which allows to perform uncertainty analysis on the steady-state water quality simulations. Three uncertainty options are available : Sensitivity analysis, first order error analysis, and monte-carlo simulations. With this analysis, the effect of model sensitivities and of uncertain input data can forecasts. Quantification of the uncertainty in model forecasts allow assessment of the risk (probability) of a water quality variable whether above or below an acceptable level. The uncertainty methodologies provide the means whereby variance estimates and uncertainty prediction can become as much a part of water quality modeling as estimating expected values is today. An evaluation of the input factors that contribute most to the level of uncertainty will lead in the direction of most efficient data gathering and research. In this manner it is possible to assess the risk of imprecise forecasts, and recommend measures for reducing the magnitude of that imprecision.

5.1 Model Representation and Limitations

QUAL2E permits simulation of any branching, one-dimensional stream system. The first step in modeling a system is to sub-divide the stream system into reaches. Each reach is then to be divided into computational elements of equal length to represent an integer number of equal computational elements.

There are seven different types of computational elements also described in Fig. 2 :

- **i) Headwater element**
- **ii) Standard element**
- **iii) Element just upstream from a junction,**
- **iv) Junction element**
- **v) Last element in system**
- **vi) Input element**
- **vii) Widthdrawal element.**

QUAL2E-UNCAS has been designed to be a relatively general program, however, certain dimensional limitations have been imposed, they are

* Number of reaches	: maximum of 200
* Computational elements	: maximum of 2000
* Headwater elements	: maximum of 7
* Junction elements	: maximum of 6
* Input and withdrawal elements	: maximum of 200
* Monte carlo simulations	: maximum of 5000

5.2 Concepts in Formulation of Model

The primary objective of any stream water quality model development is to produce a tool that has the capability for simulation the behavior of the hydrologic and water quality components of a stream. QUAL2E has also been developed to simulate

EXAMPLE STREAM NETWORK OF COMPUTATIONAL ELEMENTS AND REACHES

Type 1: Headwater
Elements 1, 18, 44

Type 3: Upstream From Junction
Elements 17, 43

Type 4: Junction
Elements 29, 52

Type 5: End of Network
Element 67

Type 6: Point Sources
Elements 7, 24, 60

Type 7: Point Withdrawal
Element 40

Type 2: Standard
All other Elements

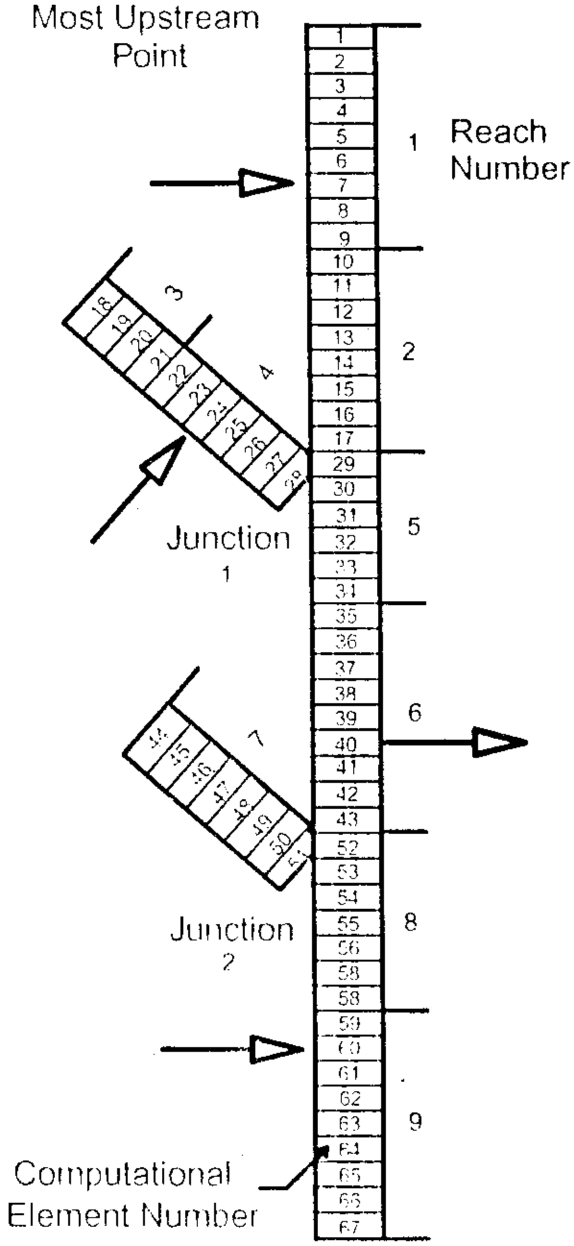


Fig. 2 : Network of Computational Elements & Reaches Described in QUAL2E

prototype behavior by applying sets of mathematical equations as applicable for water quality simulations. Three general phases (Water Resources Engineers, Inc., 1967) have been considered for formulation of the model :

i) Conceptual representation
ii) Functional representation
iii) Computational representation

5.2.1 Conceptual Representation

Conceptual representation involves a graphic idealization of the prototype by description of the geometric properties that are to be modeled and by identification of boundary conditions and interrelations between various parts of prototype. Fig. 3 shows a stream reach (n) that has been divided into a number of subreaches or computational elements, each of length Δx . For each of these computational elements, the hydrologic balance in terms of flows into the upstream face of the element (Q_{i-1}),external sources or withdrawals (Q_{xi}), and the outflow (Q_i) through the downstream face of the element has been written. In the similar fashion, a materials balance for any constituent C is written for the element. In the material balance, both transport (Q.C) and dispersion ($A \frac{D_L}{\Delta x}$ $\partial c/\partial x$) as the movers of mass along the stream axis has been considered. Mass can be added to or removed from the system via external sources and withdrawals $(Q_x C_x)_i$ and added or removed via internal sources or sinks (S_i) such as benthic sources and biological transformation. Each computational element is considered to be completely mixed.

5.2.2 Functional Representation

The basic equation that has been solved in formulation of QUAL2E is the one dimensional advection-dispersion mass transport equation, which has numerically been integrated over time and space for each water quality constituent. This equation includes the effects of advection, dispersion, dilution, constituent reactions and interactions, and sources and sinks. For any constituent, C, this equation can be represented as :

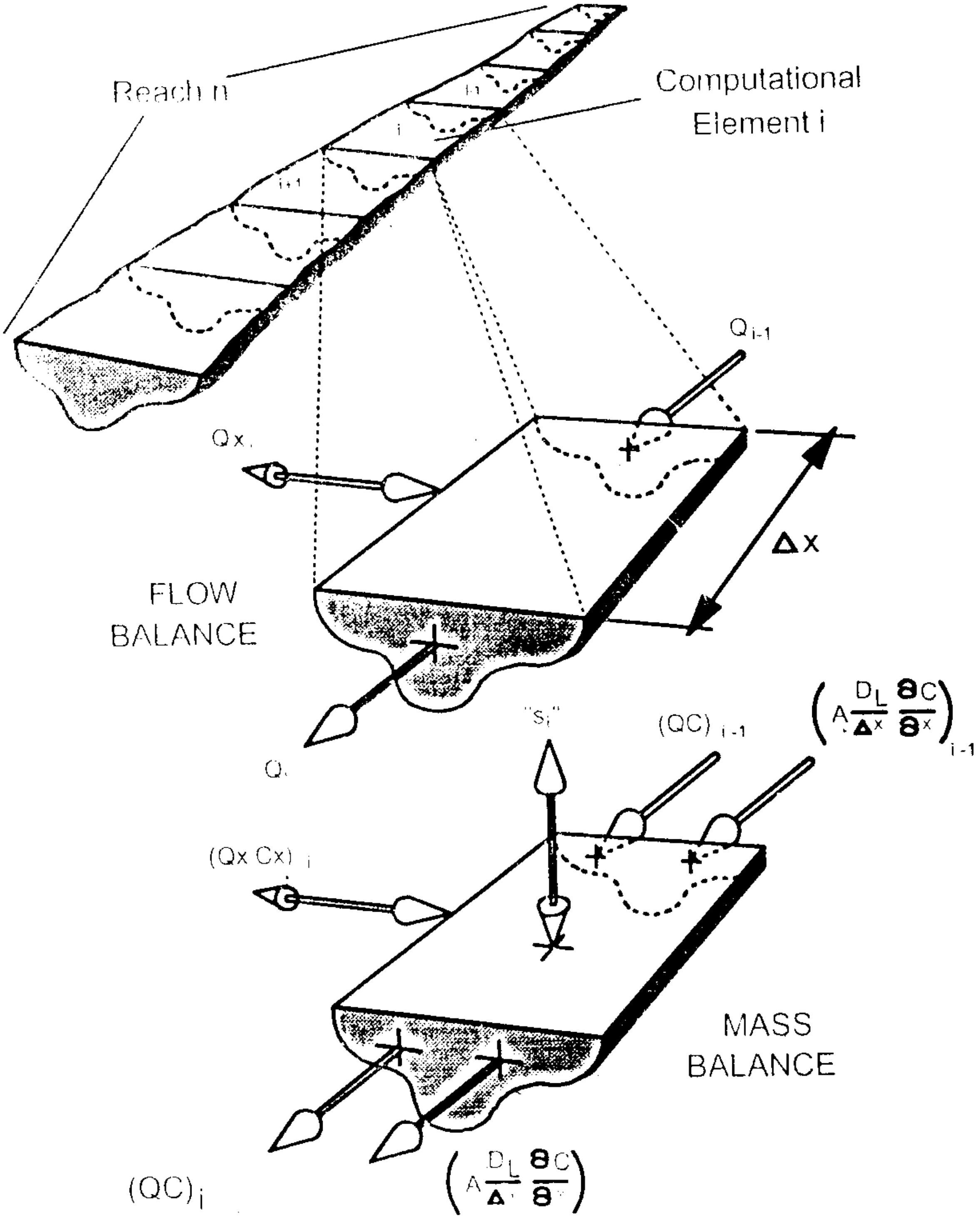


Fig. 3 : CONCEPTUAL REPRESENTATION
Discretized Stream System

$$\frac{\partial M}{\partial t}=\frac{\partial(A_x D_L \frac{\partial C}{\partial x})}{\partial x}dx-\frac{\partial(A_x uC)}{\partial x}+(A_x dx)\frac{dC}{dt}+S\ldots\ldots(1)$$

Where,

M = mass (M)
x = distance (L)
t = time (T)
C = concentration ($M\ L^{-3}$)
A_x= cross sectional area (L^2)
D_L= dispersion co-efficient ($L^2\ T^{-1}$)
u = mean velocity ($L\ T^{-1}$)
S = external source or sinks ($M\ T^{-1}$)

Because, $M = V.C$ and $V = A_x\ dx$. Assuming flow in the stream is steady, i.e. $\partial Q/\partial t = 0$, then

$$\frac{\partial C}{\partial t}=\frac{\partial(A_x D_L \frac{\partial C}{\partial x})}{A_x \partial x}-\frac{\partial(A_x uC)}{A_x \partial x}\frac{dC}{dt}+\frac{S}{V}\ldots\ldots\ldots\ldots\ldots(2)$$

The terms on the right hand side of the equation represent, respectively, dispersion, advection, constituent changes, external sources/sinks, and dilution. The dC/dt term refers only to constituent changes such as growth and decay, $\partial C/\partial t$ on the left hand side is the local concentration gradient. The later term includes the effect of constituent changes as well as dispersion, advection, sources/sinks, and dilutions.

Under steady-state conditions, the local derivative becomes equal to zero; i.e.,

$$\frac{\partial C}{\partial t}=0\ldots\ldots\ldots\ldots(3)$$

Changes that occur to individual constituents or particles independent of advection, dispersion, and waste inputs are defined by the term :

$$\frac{dC}{dt}=0 \ldots\ldots\ldots\ldots (4)$$

These changes include the physical, chemical, and biological reactions and interactions that occur in the stream.

5.2.2.1 Hydraulic Characteristics

QUAL2E assumes that the stream hydraulic regime is steady-state, i.e., $\partial Q/\partial t = 0$, therefore, the hydrologic balance for a computational element can be written as

$$\left(\frac{\partial Q}{\partial x}\right)_i = (Q_x)_i \ldots\ldots (5)$$

where $(Q_x)_i$ is the sum of the external inflows and/or withdrawals to that element.

5.2.2.2 Discharge Coefficients

QUAL2E has an option for computing depth and velocity from streamflow, called "Dispersion Coefficients," is based on empirical observations of the velocity-depth-streamflow relationship. The equations relate velocity, channel width, and depth to streamflow through power functions :

$$u = a\,Q^b \ldots\ldots\ldots\ldots\ldots\ldots (6)$$
$$D = c\,Q^d \ldots\ldots\ldots\ldots\ldots\ldots (7)$$
$$W = e\,Q^f \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

where u is average velocity
D is average depth
W is average width
and a, b, c, d, e, and f are empirical coefficients or exponents.

Given that area is a function of average width (W) and average depth (D); $A = D\,W$ ....(9)

and, $Q = A.V = V.D.W$
$= (aQ^b)\ (cQ^d)\ (eQ^f)$

$$= (a.c.e)\ Q^{b+d+f}$$

therefore, the following relationships hold :

$$a.c.e = 1$$
$$b+d+f = 1$$

QUAL2E only requires specification of the relationships for velocity (equ.6), and depth (equ. 7) but the co-efficients for equation 8 are implicitly specified from the relationships of a,b,...,f as given above.

Alternately, if the cross-sectional properties of the stream segment are available as a function of the depth d, u can be obtained as a function of discharge by trial and error solution of Manning's equation :

$$Q=\frac{1}{n}A_x R_x^{(2/3)} S_e^{(1/2)} \quad \ldots\ldots\ldots (10)$$

where, A_x = cross-sectional area of the channel
R_x = mean effective hydraulic radius
n = Manning roughness co-efficient
S_e = slope of the energy gradient line

5.2.2.3 Longitudinal Dispersion

Dispersion is basically a convective transport mechanism. The term "dispersion" is generally used for transport associated with spatially averaged velocity variation, as opposed to "diffusion", which is reversed for transport that is associated primarily with time-averaged velocity fluctuations.

In QUAL2E, the expression of dispersion co-efficient given by Elder (1959) for vertical velocity gradient has been used:

$$D_L = K\ d\ u_* \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

where, D_L = longitudinal dispersion co-efficient
K = dispersion constant (dimensionless)
u_* = average shear velocity

Shear velocity for steady-state open channel flow has been derived from Chezy's equation :

$$u_*=C\sqrt{R_e S_e}\ldots\ldots\ldots\ldots(12)$$

where, C = Chezy's coefficient
R_e = hydraulic radius

Chezy's coefficient is given by :

$$C=\frac{R_e^{1/6}}{n}\ldots\ldots\ldots\ldots(13)$$

Where, n is the Manning roughness coefficient. The S_e, the energy slope , is given by :

$$S_e=(\frac{un}{R^{2/3}})^2$$

Assuming R_e = d for a wide channel, the expression yields for dispersion coefficient that has been considered for QUAL2E :

$$D_L=3.82K.n.u.d^{5/6}\ldots\ldots\ldots\ldots(14)$$

where, D_L is in ft^2/sec.

5.2.2.4 Flow Augmentation

When the DO (Dissolved Oxygen) concentration in a stream drops below the required target level, such as the water quality standard for DO, it may be desirable to raise this DO concentration by augmenting the flow of the stream. The amount of flow necessary to bring up the DO concentrations up to required standards cannot be calculated by an exact functional relationship. A good approximation of the relationship is used in QUAL2E and has the following quadratic form :

$$DO_R=DO_T-DO_{min}\ldots\ldots\ldots\ldots(15)$$

and,

$$Q_R=Q_C(\frac{DO_R}{DO_T}+0.15(\frac{DO_R}{DO_T})^2)\ldots\ldots\ldots\ldots(16)$$

Where,

DO_R = dissolved oxygen concentration required to meet target conditions, (mg/l)

DO_T = required target level of DO, (mg/l)

DO_{min} = minimum DO concentration (critical level) in the oxygen sag curve, (mg/l).

Q_R = amount of flow augmentation required.
Q_C = flow at the critical point in the oxygen sag curve.

The model augments the stream flow by first comparing, after steady-state conditions have been reached, the simulated DO concentration with the prespecified target level of DO in each reach.If the calculated DO is below the target level, the program finds those upstream sources that has been specified for dilution purposes, and adds water equally from all these sources until the DO target is specified.

5.2.2.5 Constituent reactions and interrelationships

One of the most important considerations in determining the waste-assimilative capacity of a stream is its ability to maintain an adequate dissolved oxygen concentration. Dissolved oxygen concentrations in stream are controlled by atmospheric reaeration, photosynthesis, plant and animal, respiration, benthal demand, BOD(Bio-chemical Oxygen Demand), nitrification, salinity, and temperature, among other factors.

The QUAL2E model includes the major interactions of the nutrient cycles, algae production, benthic oxygen demand, carbonaceous oxygen uptake, atmospheric aeration and their effect on the behavior of dissolved oxygen. Fig. 4, indicates the conceptualization of major constituent interactions considered in QUAL2E.

The mathematical relationships of parameters that have been considered in QUAL2E and are of interest have been outlined below

5.2.2.6 Dissolved oxygen

Considering the interactions and reactions in oxygen balance as indicated in Fig. 4, the differential equation for DO as assumed in QUAL2E is given below :

$$\frac{dO}{dt}=K_2(O^*-O)+(\alpha_3\mu-\alpha_4\rho)A-K_1L-\frac{K_4}{d}-\alpha_5\beta_1N_1-\alpha_6\beta_2N_2 \qquad (16)$$

where,

O = concentration of dissolved oxygen, (mg/l)
O^* = the saturation concentration of dissolved oxygen at the local temperature and pressure, (mg/l).

α_3 = the rate of oxygen production per unit of algal photosynthesis, (mg-O/mg-A)

α_4 = the rate of oxygen uptake per unit of algal respired, (mg-O/mg-A)

α_5 = the rate of oxygen uptake per unit ammonia nitrogen oxidation, (mg-O/mg-N)
α_6 = the rate of oxygen uptake per unit of nitrite nitrogen oxidation, (mg-O/mg-N)

μ = algal growth rate, temperature dependent, (day^{-1})
ρ = algal respiration rate , temperature dependent, (day^{-1})

A = algal biomass concentration, (mg-A/l)

L = concentration of ultimate carbonaceous BOD, (mg/l)

d = mean steam depth,

k_1 = carbonaceous BOD deoxygenation rate , temperature dependent, (day^{-1})

k_2 = the reaeration rate in accordance with the Fickian diffusion analogy,temperature dependent, (day^{-1})

k_4 = sediment oxygen demand rate, temperature dependent, g/ft^2-day)

β_1 = ammonia oxidation rate coefficient, temperature dependent,(day^{-1}).

QUAL2E CONSTITUENT SCHEMATIC

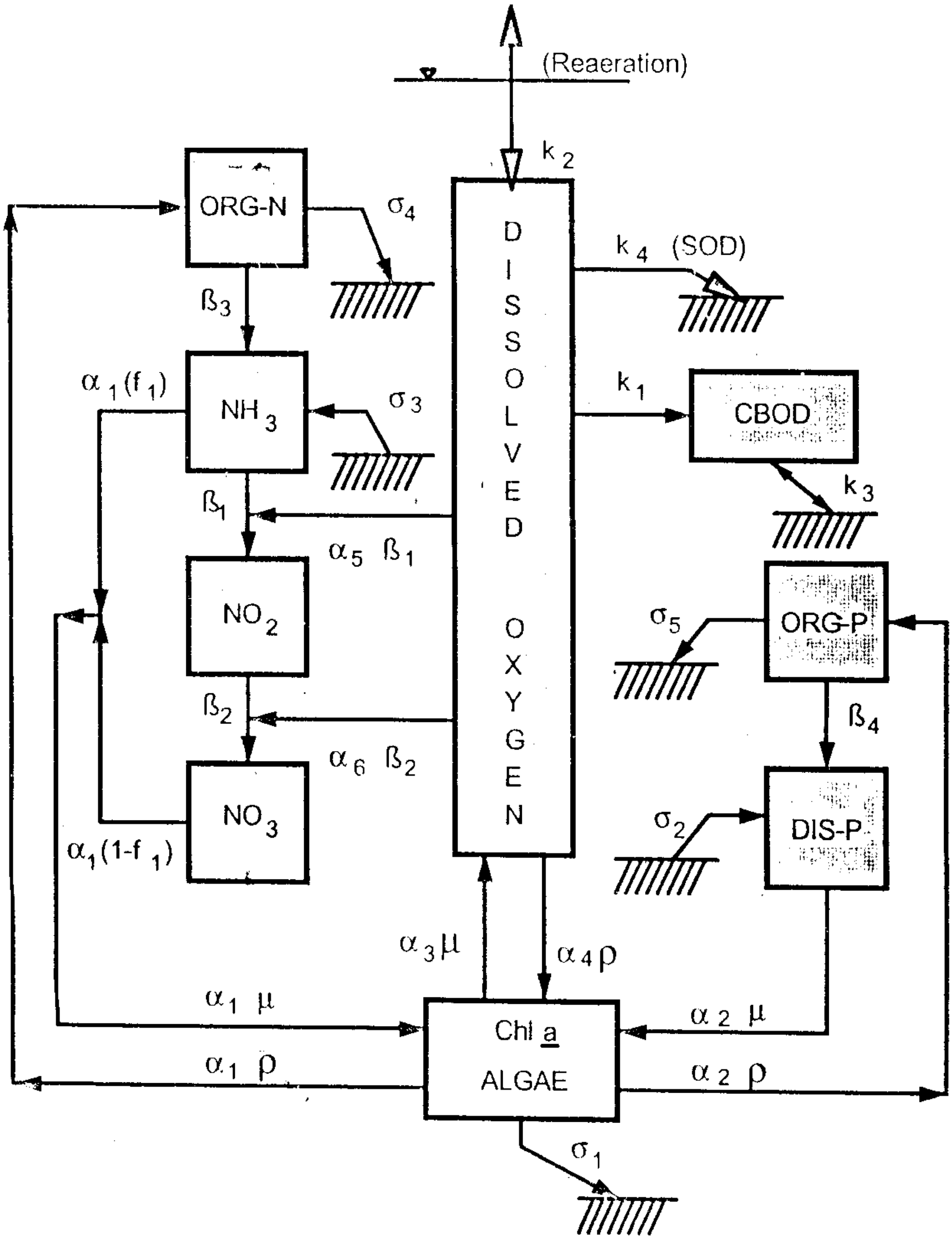


Fig. 4 : QUAL2E Constituent Schematic For DO Simulation

β_2 = nitrite oxidation rate coefficient, temperature dependent, (day^{-1})

N_1 = ammonia nitrogen concentration, (mg-N/l)

N_2 = nitrite nitrogen concentration, (mg-N/l)

5.2.2.7 Dissolved Oxygen Saturation Concentration

The solubility of dissolved oxygen in water decreases with increasing temperature, increasing dissolved solid concentration, and decreasing atmospheric pressure. QUAL2E uses a predictive equation for the saturation (equilibrium) concentration of dissolved oxygen (APHA, 1985) :

$$\ln O^{*} = -139.3441 + (1.575701 . \frac{10^5}{T}) - (6.642308 . \frac{10^7}{T^2}) + (1.2438 . \frac{10^{10}}{T^3}) - (8.621949 . 10^{11}/T^4)$$

where, O^{*} = equilibrium oxygen concentration at 1.00 atm, (mg/l)
T = temperature (^0K) = (^0C + 273.15) and ^0C is within the range 0.0 to 40.0^0C.

5.2.2.8 Atmospheric Reaeration Co-efficient

The atmospheric reaeration co-efficient (K_2) is most often expressed as a function of stream depth and velocity. QUAL2E provides eight options for estimating or reading in K_2 values.

Option 1 : Allows to read in K_2 values that have selected by the modeler. This option is useful in modeling unusual situations such as ice cover.

Option 2 : Based on equation given by Churchill, Elmore, and Buckingham (1962). In this option, the model compute the K_2 values for different reaches.

Option 3 : Based on equation proposed by O'Connor and Dobbins (1958). In this option, the model compute the K^2 values for known depth, dispersion co-efficient and velocity.

Option 4 : Based on equation proposed by Owens et.al (1964).

In this option, the model calculate the K_2 values for known depth and velocity. This option has been found suitable for depths between 0.4 ft. to 11.0 ft. and velocities of 0.1 to 0.5 ft/sec.

Option 5 : Based on equation proposed by Thackston and Krenkel (1966). In this option, model calculate the K_2 values for different stretches.

Option 6 : Based on equation proposed by Langbien and Durum (1967). For known depth and velocity, the model compute the K_2 values for different reaches.

Option 7 : This option compute the reaeration co-efficient from a power function of flow as shown in the case of velocity and depth. This relationship is to be developed from filed data.

Option 8 : Based on the concept of Tsivoglou and Wallace (1972) where it has been derived that K_2 for a reach is proportional to the change in elevation of the water surface and inversely proportional to the flow time through the reach. For given energy slope and known velocity, the model compute the K_2 values in this option.

There are no default K_2 values in QUAL2E. The user has to specify any K_2 options as given above.

5.2.2.9 Carbonaceous BOD

The QUAL2E model assumes a first order reaction to describe deoxygenation of ultimate carbonaceous BOD in the stream. The BOD function as expressed in the model also takes into account additional BOD removal due to sedimentation, scour and flocculation, which do not exert an oxygen demand. Mathematically;

$$\frac{dL}{dt}=-K_1L-K_3L \qquad \text{............} (18)$$

Where,

L = the concentration of ultimate carbonaceous BOD, mg/l

K_1 = deoxygenation rate co-efficient, temperature dependent, day^{-1}.
K_3 = the rate of loss of carbonaceous BOD due to settling, temperature dependent, day^{-1}.

QUAL2E simulates ultimate BOD in the general case; however, the model has the option of 5-day BOD values for input and output. In that case, the model makes the necessary conversions from 5-day to ultimate BOD. The conversion equation is :

$$BOD_5 = BOD_u (1.0 - \exp(5.KBOD)) \quad \ldots\ldots\ldots\ldots\ldots (19)$$

where,

BOD_5 = 5-day BOD, mg/l
BOD_u = ultimate BOD, mg/l
KBOD = BOD conversion rate co-efficient, day^{-1}.

5.2.2.10 Nitrogen cycle

In natural aerobic waters, there is a stepwise transformation from organic nitrogen to ammonia, to nitrite, and finally to nitrate. The nitrogen cycle in QUAL2E contains all four of these components. The differential equations governing transformations of nitrogen from one form to another are given below :

Organic Nitrogen

$$\frac{dN_4}{dt} = \alpha_1 \rho A - \beta_3 N_4 - \alpha_4 N_4 \quad \ldots\ldots\ldots\ldots\ldots (20)$$

where,

N_4 = concentration of organic nitrogen, mg-N/L.
β_3 = rate constant for hydrolysis of organic nitrogen to ammonia nitrogen, temperature dependent, day^{-1}.
α_1 = fraction of algal biomass that is nitrogen, mg-N/mg-A

Ammonia Nitrogen

$$\frac{dN_1}{dt} = \beta_3 N_4 - \beta_1 N_1 + \frac{\sigma_3}{d} - F_1 \alpha_1 \mu A \quad \ldots\ldots\ldots (21)$$

where,

$$F_1=\frac{P_N N_1}{(P_N N_1+(1-P_N)N_3)}\text{...........}(22)$$

N_1 = the concentration of ammonia nitrogen, mg-N/l
N_3 = the concentration of nitrate nitrogen, mg-N/l
F_1 = fraction of algal biomass which is nitrogen, mg-N/mg-A.
P_N = preference factor for ammonia nitrogen (0 to 1.0).

Nitrite Nitrogen

$$\frac{dN_2}{dt}=\beta_1 N_1-\beta_2 N_2\text{........}(23)$$

Nitrate Nitrogen

$$\frac{dN_3}{dt}=\beta_2 N_2-(1-F)\alpha_1\mu A\text{...........}(24)$$

5.2.2.11 **Phosphorus cycle**

QUAL2E only includes dissolved phosphorus to simulate the interactions between organic and dissolved phosphorus. Organic forms of phosphorus are generated by the death of algae, which then convert to the dissolved inorganic state, where it is available to algae for primary production. The differential equations governing transformations of phosphorus from one form to another are given below :

Organic Phosphorus

$$\frac{dP_1}{dt}=\alpha_2\rho A-\beta_4 P_1-\delta_5 P_1\text{.........}(25)$$

where,

P_1 = the concentration of organic phosphorus, mg-P/L
α_2 = phosphorus content of algae, mg P/mg-A.
β_4 = organic phosphorus decay rate, temperature dependent, day^{-1}.
δ_5 = organic phosphorus settling rate, temperature

dependent, day_{-1}.

Dissolved Phosphorus

$$\frac{dP_2}{dt}=\beta_4 P_1+\frac{\delta_2}{d}-\alpha_2\mu A \quad (26)$$

where,

P_2 = concentration of inorganic or dissolved phosphorus, mg-P/l.

δ_2 = benthos source rate for dissolved phosphorus, temperature dependent, mg-P/L^2-day.

5.2.2.12 Dam reaeration

QUAL2E has the capability of modeling oxygen input to the system from reaeration over dams. The following equation described by Butts and Evans (1983) has been used to estimate oxygen input from dam aeration.

$$D_a-D_b=\left(1-\frac{1}{(1+0.116abH(1.0-0.034H)(1+0.46T))}\right)D_a \quad (27)$$

Where,

D_a = oxygen deficit above dam, mg/l

D_b = oxygen deficit below dam, mg/l

H = height through which water falls

a = empirical water quality factor
= 1.80 in clear water
= 1.60 in slightly polluted water
= 1.0 in moderately polluted water
= 0.65 in grossly polluted water.

b = empirical dam aeration co-efficients
= 0.70 to 0.90 for flat broad crested weir
= 1.05 for sharp crested weir with straight slope face
= 0.80 for crested weir with vertical face
= 0.05 for sluice gates with submerged discharge.

The factors, H, a and b are input for each dam.

5.2.2.13 Arbitrary nonconservative constituent

QUAL2E has the provision for modeling an arbitrary nonconservative constituent. In addition to a first order decay mechanism, there are source and sink terms in the mass balance. The differential equation describing the interactions for any arbitrary nonconservative constituent is :

$$\frac{dR}{dt}=-K_6R-\delta_6R+\frac{\delta_7}{d}\ ....\ (28)$$

where,

R = concentration of the nonconservative constituent,

K_6 = decay rate for the constituent, temperature dependent.

$_6$ = rate co-efficient for constituent settling, temperature dependent, day^{-1}.

$_7$ = benthal source for constituent, temperature dependent.

5.2.2.14 Temperature

Temperature is modeled by performing a heat balance on each computational element in the system. The heat balance accounts for temperature inputs and losses from the forcing functions as well as the heat exchanged between the water surface and the atmosphere. The air-water heat balance term include long and short wave radiation, convection, and evaporation using :

$$H_n=H_{sn}+H_{an}-H_b-H_c-H_e$$

where,

H_n = net heat flux passing the air water surface, Btu/ft^2-day.

H_{sn} = net short wave solar radiation after losses from absorption and scattering in the atmosphere and by reflection at the interface Btu/ft^2-day.

H_{an} = net long wave atmosphere radiation after reflection, Btu/ft^2-day

H_b = outgoing long wave back radiation, Btu/ft^2-day.

H_c = convective heat flux, Btu/ft^2-day.

H_e = heat loss by evaporation, Btu/ft^2-day.

To perform the heat balance computation in QUAL2E, variety of data including the longitude and latitude of the basin, the time of year, evaporation co-efficients, and a dust attenuation co-efficient are necessary as inputs. Local climatological information in the form of time of day, wet and dry bulb air temperature, atmospheric pressure, cloud cover and wind velocity are also required.

5.2.2.15 Temperature dependence of rate coeffic ents

The temperature values computed in QUAL2E are used to correct the rate co-efficients in the source/sink terms for the other water quality variables. These co-efficients are input at

$$K_T = K_{20}\theta^{(T-20)}$$

20^0c and are then corrected to temperature using a Streeter Phelps type formulation.

where: K_T = the value of the co-efficient at the local temperature

K_{20} = the value of the co-efficient at standard temperature, (20^0c)

O = an empirical constant for each reaction co-efficient.

5.3 UNCERTAINTY ANALYSIS IN QUAL2E

One of the first steps in the chain of risk assessment being the quantification of the error in predicting water quality, uncertainty analysis has been the subject of much discussion in the ecosystem modeling aspect. Realizing the needs, systematic approach to uncertainty analysis for the general purpose has been incorporated in the QUAL2E which has come as QUAL2E-UNCAS. The objective was to provide some of the tools for incorporating uncertainty analysis as an integral part of the water quality modeling process.

Three uncertainty analysis techniques has been employed in QUAL2E-UNCAS - sensitivity analysis, first order analysis, and monte carlo simulation. Monte carlo simulation has the advantage of output frequency distributions, but it carries a high computational burden while first order propagation provides a direct estimate of model sensitivity, but that variability is usually more indicative of the variance of model components than of the dynamics of the model structure.

6.0 INPUT DATA FOR QUAL2E

QUAL2E accepts hydrologic and water quality data through 16 input data cards. PC version of QUAL2E compatible with PC/386 and above, has the interface facility for direct accessing of input data on the screen. It is possible, as discussed above, to simulate an individual parameter or multiple parameters by the model. Data cards can be prepared according to interest of simulation of parameter(s). Parameter(s) which is/are of not interest can be bypassed putting the value as "0" or by default the model assume the value as "0", if no value is assigned. Under mentioned sections indicate the preparation of data cards as required for the model.

6.1 Discretization of River Stretch

The first task that involves to fulfil the requirement of QUAL2E model is the discretization of the river stretch into number of reaches, and to choose the length of computational elements. Length of computational element should be such that total number of computational elements in each reach should not exceed 20. However, the length of computational element could be of any magnitude but should be uniform for all reaches. Smaller the computational element length means requirement of more computer memory but better details of computation would be obtained. Once reaches and the computational elements are discretized, the task left is, to identify the functional elements, i.e., the element(s) in which point load(s), withdrawal(s), and tributary(ies) meet.

In this case, since the river has no tributary within the study stretch of 65 Km., the total number of computational elements with element of 0.25 Km/element divided into 15 reaches stand to : 260 (Fig. 5). The details of reaches, number of computational elements in each reach and the location of point loads in the reaches have been tabulated in Table-1 :

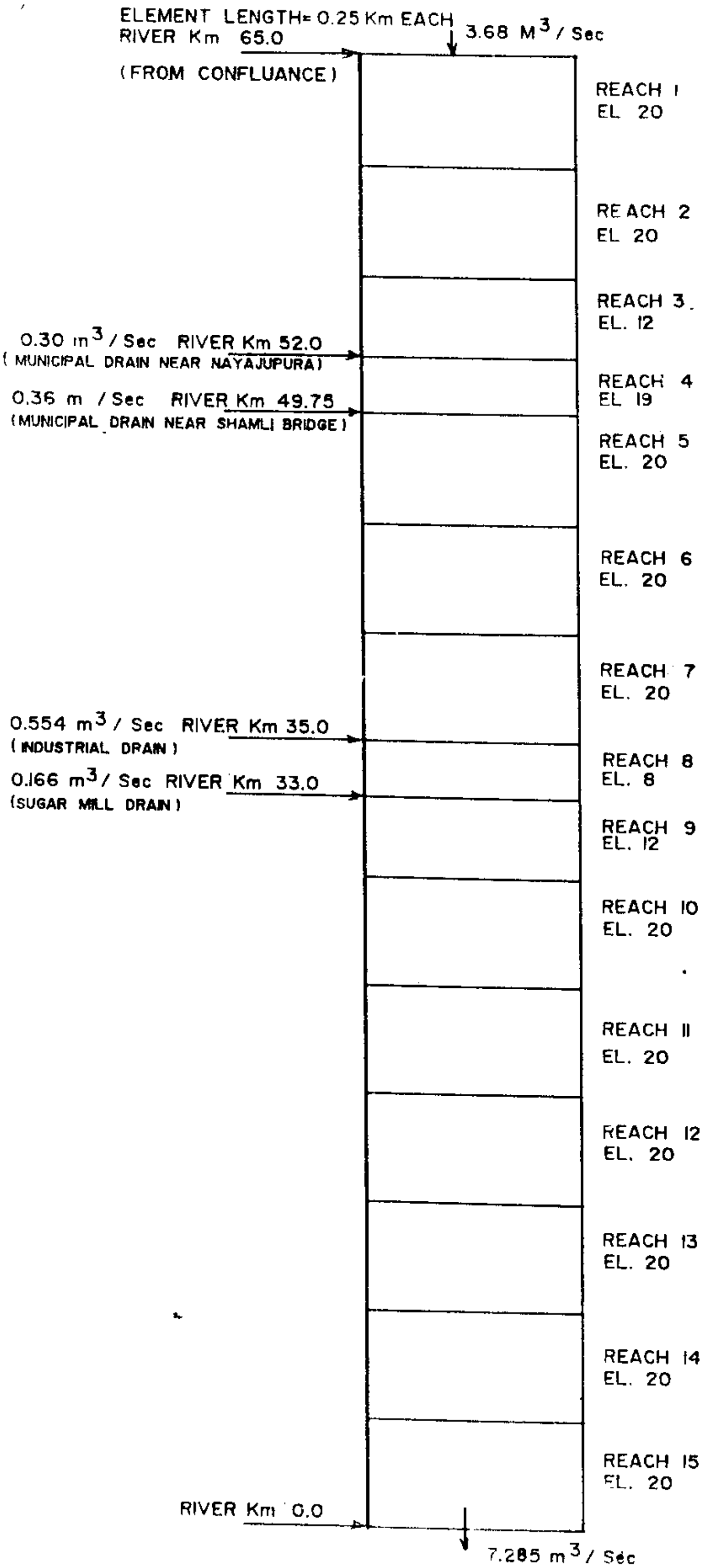


FIG. 5 DISCRETIZATION OF STUDY STRETCH

Table I : Discretization of the River Reach

Reach No	Kilometer point From	To	Number of elements	Location of point loads
1	65	60	20	
2	60	55	20	
3	55	52	12	
4	52	49.75	9	Km. 52.00
5	49.75	45	19	Km. 49.75
6	45	40	20	
7	40	35	20	
8	35	33	8	Km. 35.00
9	33	30	12	Km. 33.00
10	30	25	20	
11	25	20	20	
12	20	15	20	
13	15	10	20	
14	10	05	20	
15	05	00	20	

6.2 Characterization of Hydraulic Parameters and Discharge Co-efficients

As discussed in sections 5.2.2.1 and 5.2.2.2, hydraulic parameters viz.,flow, velocity, depth and Manning's roughness co-efficients are important input parameters for the model to compute the oxygen balance in each segment/reach. However, the model does not accept these parameters directly (except the manning roughness co-efficient) but accept in the form of co-efficient and exponent of velocity, co-efficient and exponent of depth which are to be computed from the field observation of velocity, depth, and width of the river at selected locations over the years. Stage-discharge, velocity-discharge, and width-discharge curves developed from atleast seven consecutive years record would provide the realistic computation.

Since, no baseline information were available, it could not make possible to develop the statistical co-relation between flow-velocity, flow-depth, and flow- width. However, continuous effort over the last one year made it possible to develop representative co-relation between flow-velocity, flow-depth, and flow-width at 5 different locations. Fig. from 6,to 11 and Table:2 indicate co-relations of hydraulic parameters, and the river cross-sections at different locations.

Table II : Discharge Co-efficients at Different Locations

Location	Kilo-meter	Velocity Coeff.	Velocity Expn.	Depth Coeff.	Depth Expn.	Ref.
Malira Bridge	65.0	0.17836	0.333	0.3557	0.48097	6
D/S of Nayazupura drain	51.5	0.1545	0.3001	0.4737	0.3947	7
U/S of Municipal drain at Shamli	50.5	0.13524	0.28716	0.57957	0.3535	8
D/S of Municipal drain at Shamli	49.5	0.25146	0.27064	0.64335	0.3769	9
Before the confluence of Industrial drain	35.0	0.2392	0.2938	0.436	0.3948	10
Near Mansurpur bridge after Sugar mill	32.5	0.17172	0.3532	0.32138	0.4883	11

6.2.1 Methods of Measurement of Hydraulic Parameters

There were neither any permanent Gauge- Discharge site nor any convenient ways to measure the hydraulic data of the river. Bridges over the river located in some places namely ; Malira bridge(km. point 65), Shamli bridge (km. point 50.00) and Mansurpur bridge (km. point 32.0) made it possible to measure the width and depth of

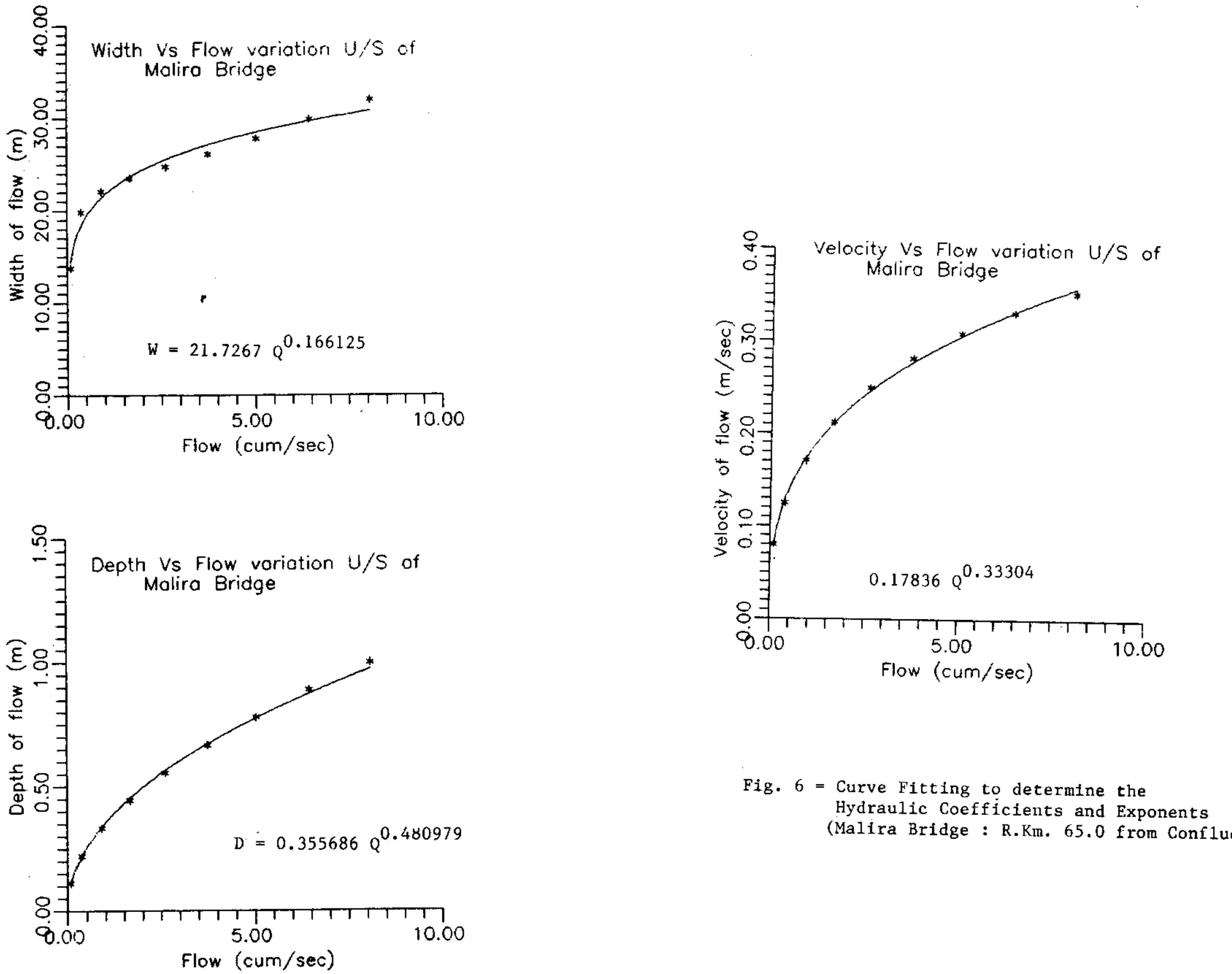
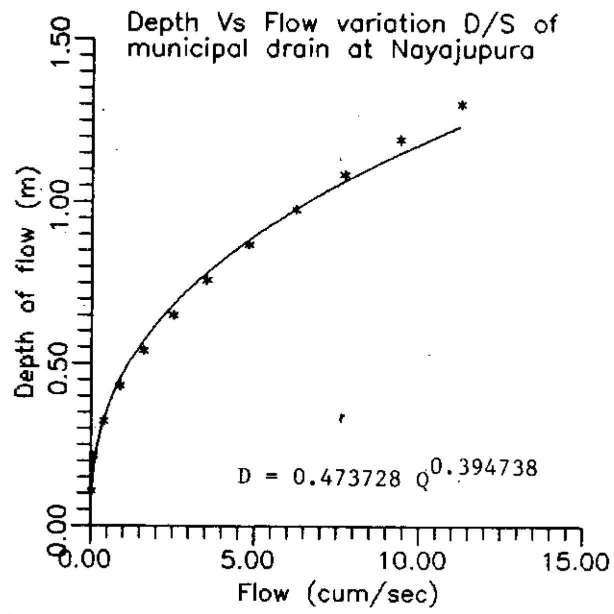
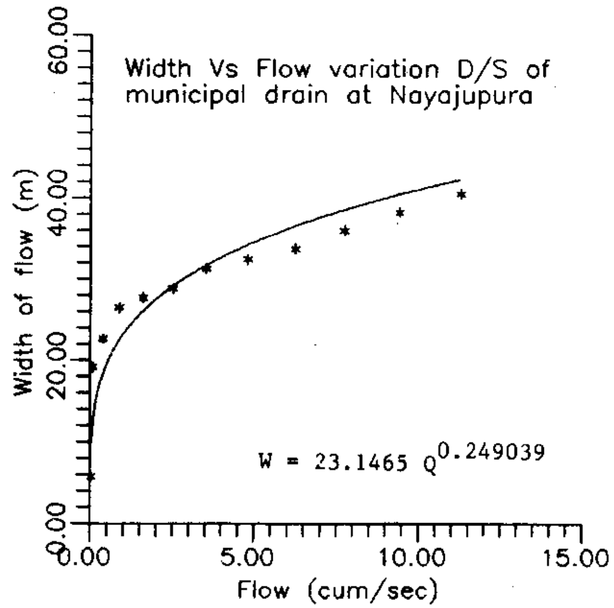


Fig. 6 = Curve Fitting to determine the Hydraulic Coefficients and Exponents (Malira Bridge : R.Km. 65.0 from Confluence)





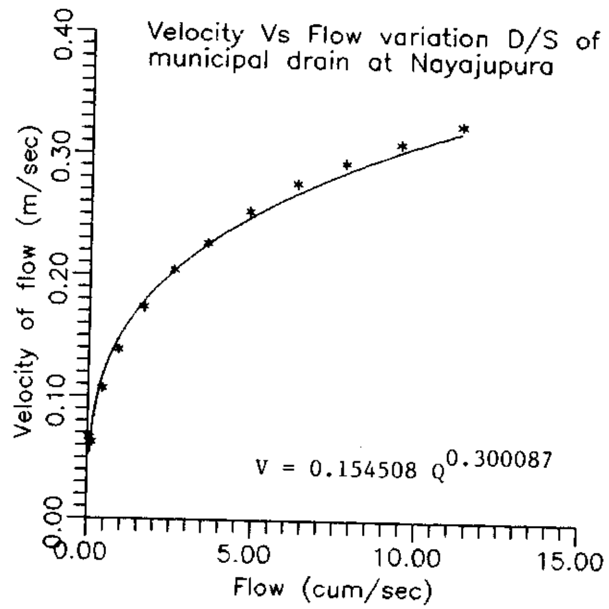
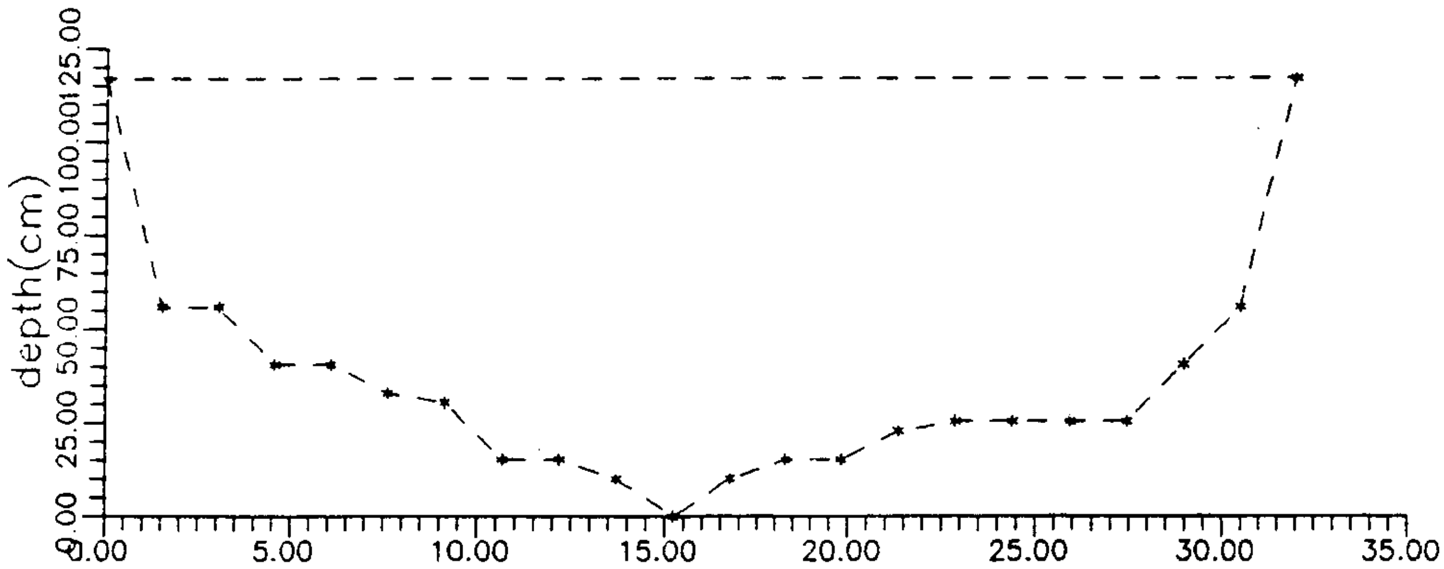


Fig. 7 - Curve Fitting to determine the Hydraulic Coefficients and Exponents (Nayazupur R.Km. 52.00 from Confluence)

Fig. 7A - Cross section of river U/S of municipal drain
(River Km. 50)



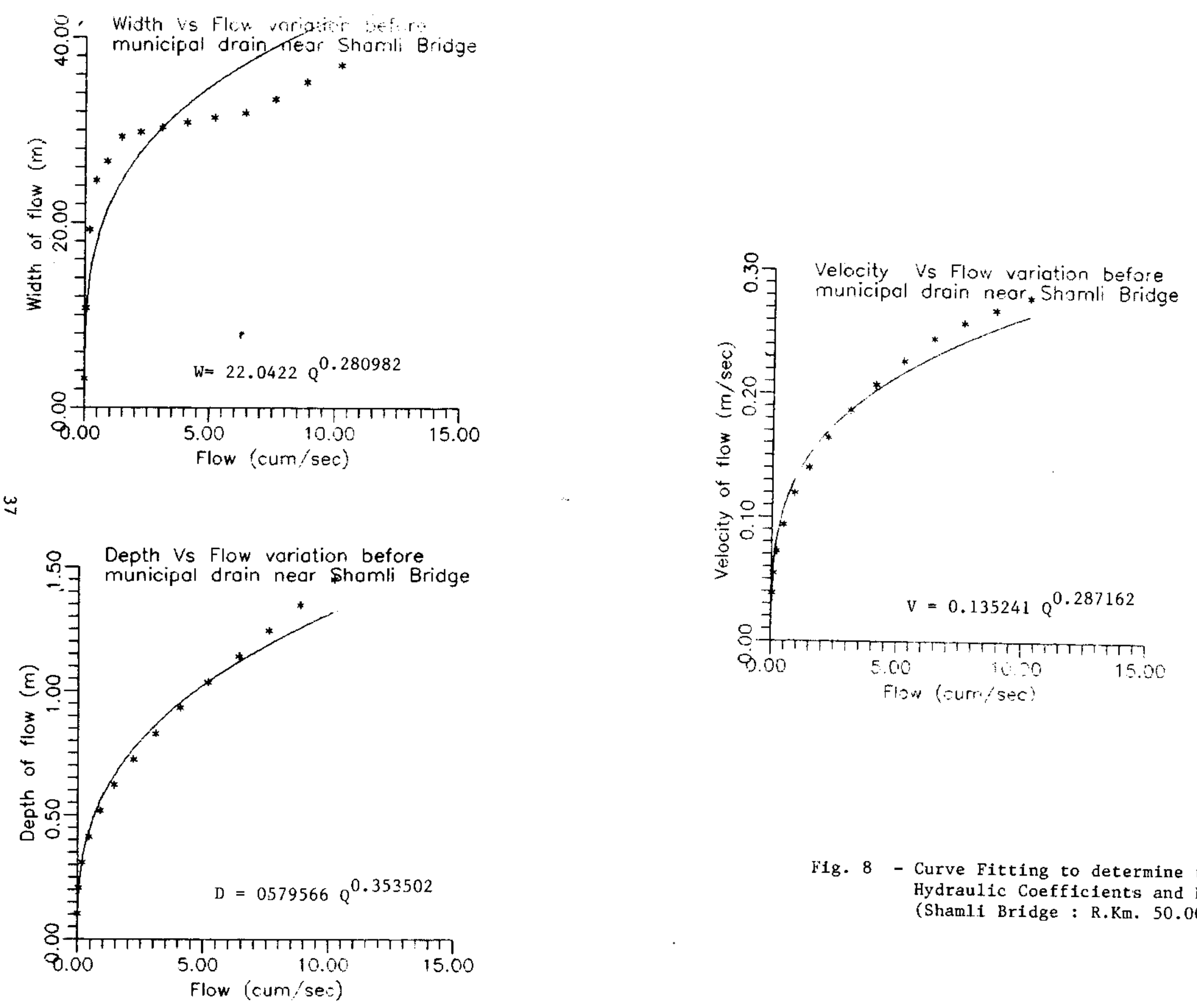
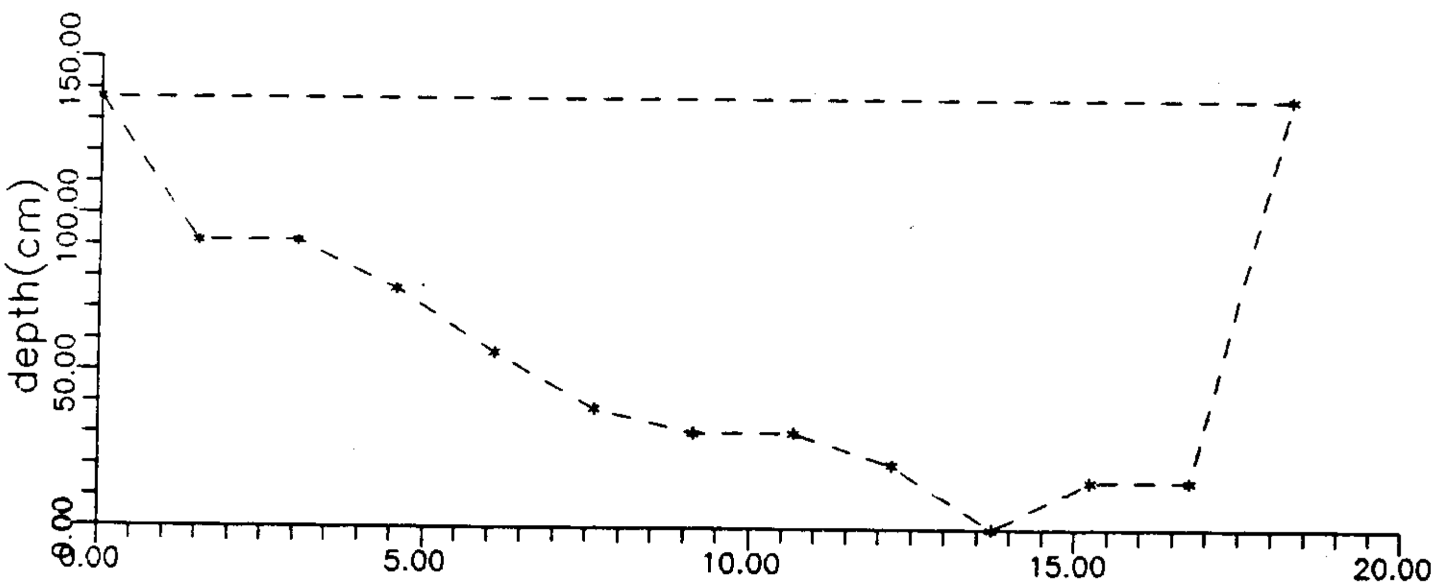


Fig. 8 - Curve Fitting to determine the Hydraulic Coefficients and Exponents (Shamli Bridge : R.Km. 50.00)

Fig.8A - Cross section of river D/S of municipal drain
(R.Km. 49.50)



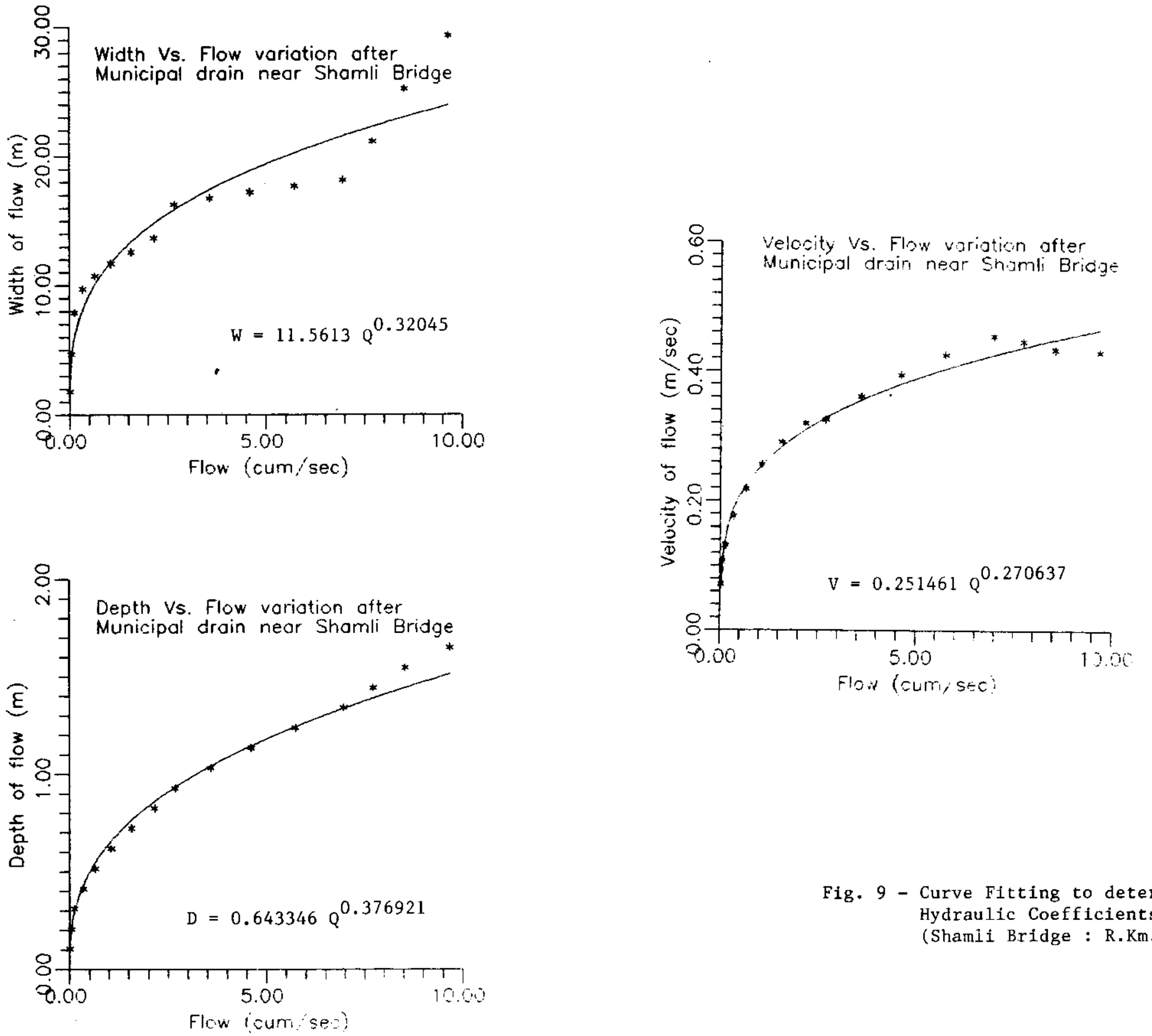
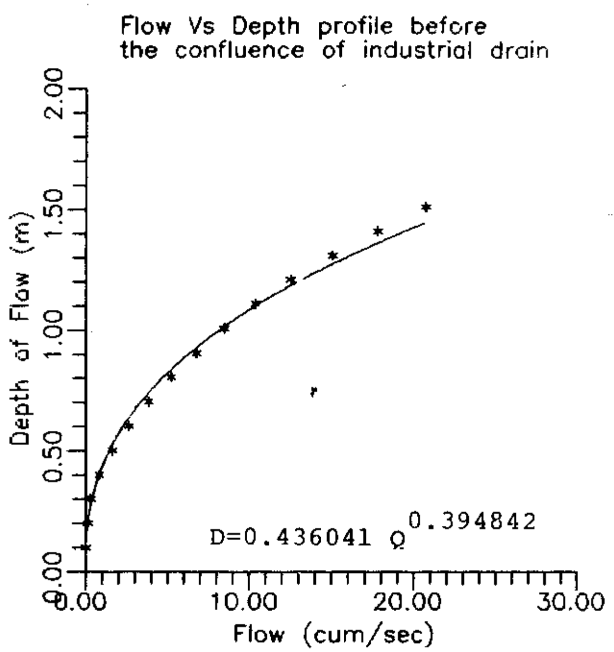
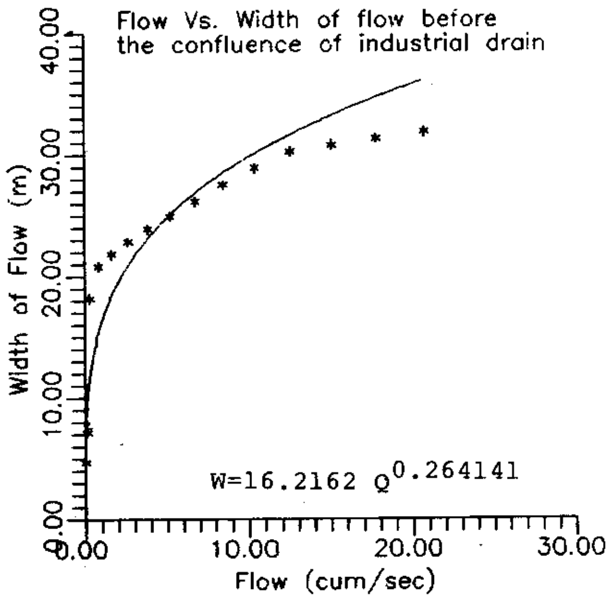


Fig. 9 - Curve Fitting to determine the Hydraulic Coefficients and Exponents (Shamli Bridge : R.Km. 49.50)





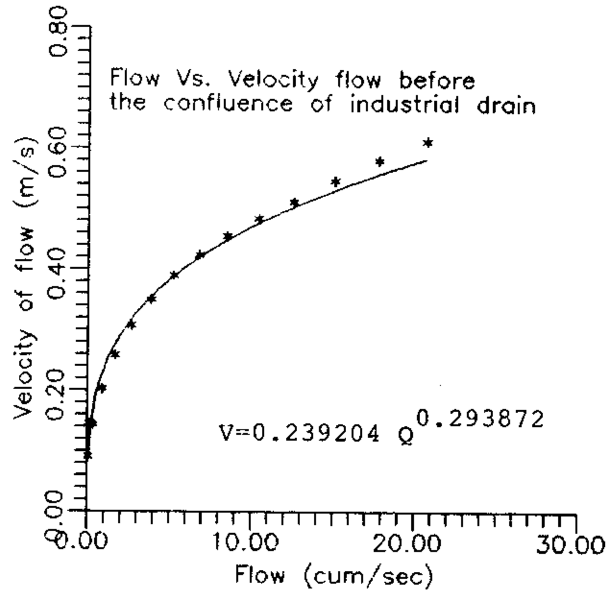
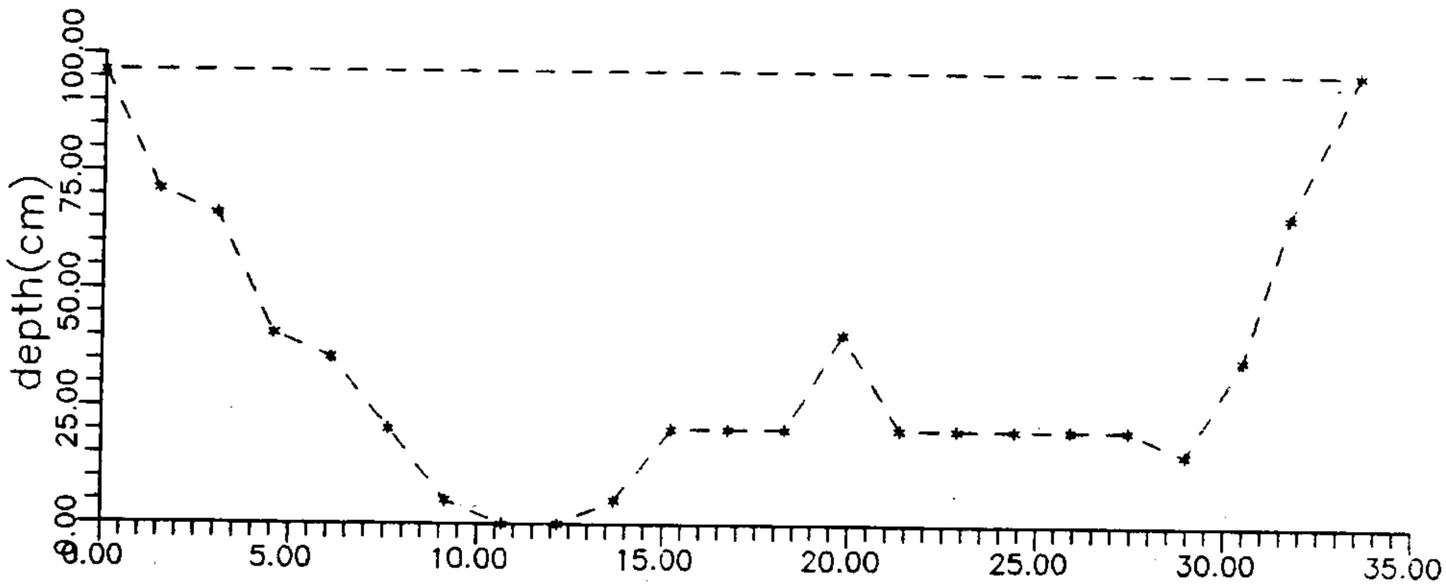
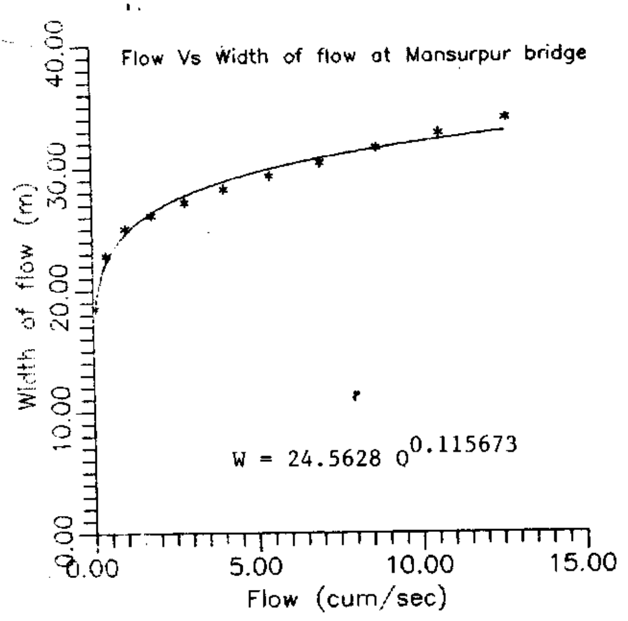
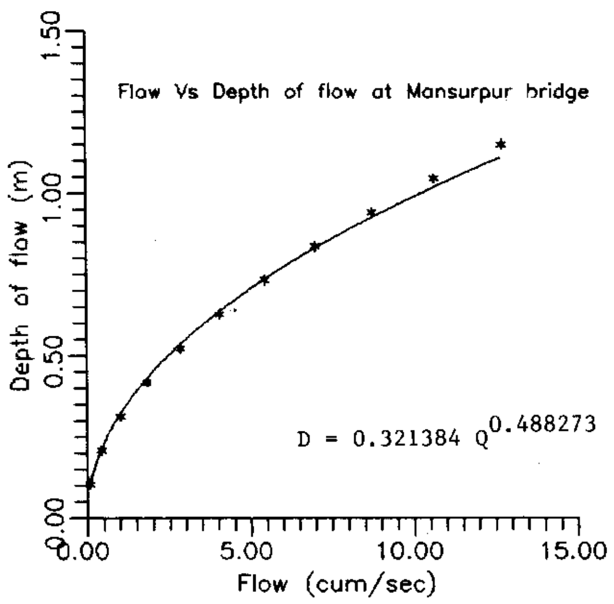


Fig. 10 - Curve Fitting to detrmine the Hydraulic Coefficients and Exponents (Before Industrial drain R.Km. 35.0 from Confluence)

Fig. 10A - Cross section of river D/S of Nayajupura drain
(River Km. 51.50)







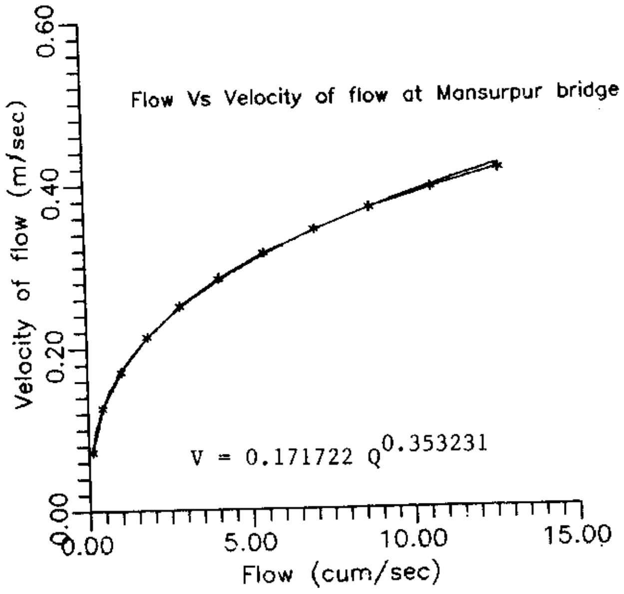
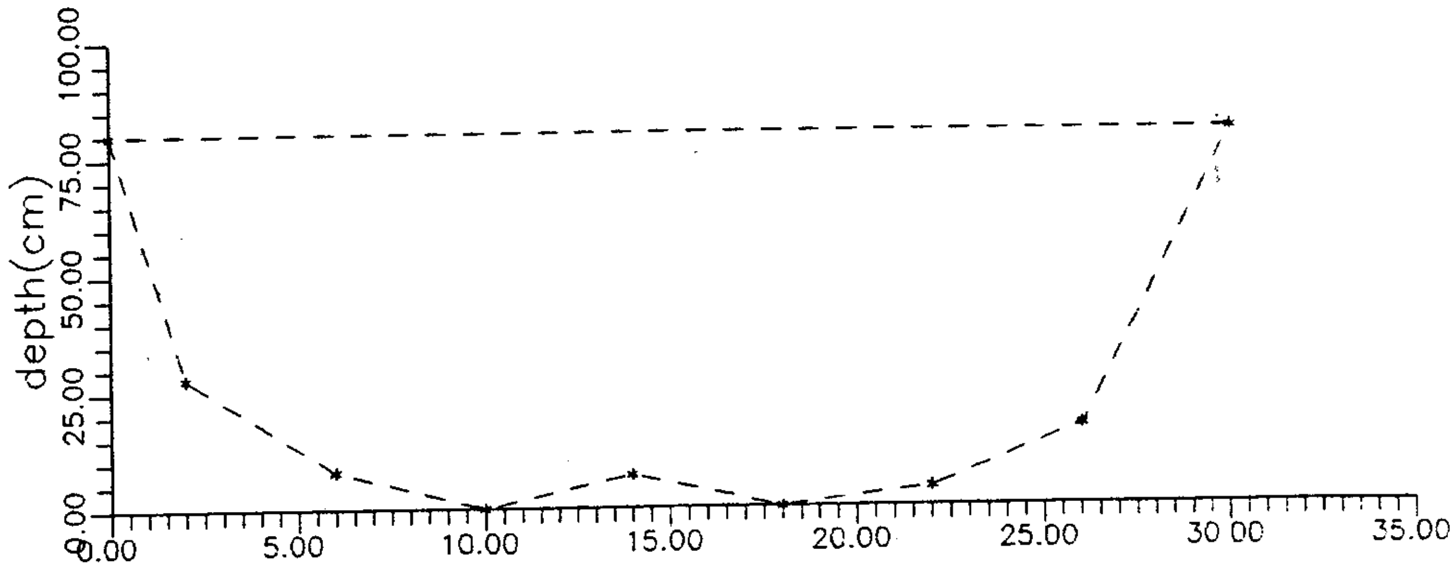


Fig. 11 - Curve Fitting to determine the Hydraulic Coefficients and Exponents (Mansurpur Bridge : R.Km. 33.0)

Fig. 11 A -Cross section of river D/S of suger mill drain
(R.Km. 33.0)



the flow at those locations. Places where no bridges were available eg., Nayazupura, before the confluence of industrial drain, velocity, depth at different sections of the river were measured through personal efforts. Velocities were measured by Pigmy current meter at a depths of 0.2d and 0.8d (d = depth of flow).

Depths along the river cross-section were measured taking difference of top surface elevation and the bottom surface elevation where conveniences of bridges were available. For other places, graduated rod was used for measurement of depths.

Correlation between velocity-discharge, depth-discharge, and width-discharge have been obtained using the Manning's equation:

$$V=\frac{1}{n}R^{2/3}S^{1/2}$$

For given Manning's roughness co-efficient (value chosen from literature keeping similarity with site condition), energy slope has been determined for known velocity , cross-section and wetted perimeter. Using the computed energy slope (variation of energy slope being small as velocity variation is less (0.35 m^2/sec -0.40 m^2/sec)), velocity and the corresponding flow for different depths and widths have been determined.

6.3 Flow Balance

Principle of conservation mass being the basic concept of the model formulation, hydrologic balance in terms of flow in each reach , i.e., flow entered in the reach out of the reach should be equal, is mandatory. Table :3 represents an indicative of measured flow (dry weather flow) at different locations :

In order to balance the flow in each reach, difference of flow in and out of the reach has been distributed uniformly over the reach as incremental flow into the reach. For example, the stretch between Malira bridge (km. point 65) and U/S of Nayazupura drain (km. point 52), i.e before the point flow, the difference of flow for (65 - 49.75) 15.75 km = flow at km. point 65 - flow at km. point 49.75 = 4.55 - 3.68 = 0.87 m^3/sec. Therefore, the incremental flow is equal to 0.057 m^3/sec/km.

Losses due to evaporation and seepage has been taken as nil.

Table III : Measured Flow at Different locations

Location	Kilometer point	Flow (m^3/sec)	Flow from point sources (m^3/sec)	Location of point sources
Malira bridge	65.0	3.68 ± 0.080	--	
U/S of Nayazupura drain	51.50	4.55 ± 0.065	--	
U/S of Shamli bridge	50.50	4.95 ± 0.075	0.30	Nayazupura drain (km. 52.)
D/S of Shamli bridge	49.50	5.25 ± 0.075	0.36	Municipal drain (km. 49.75)
U/S of Industrial drain	35.00	6.35 ± 0.085	--	
Mansurpur bridge	32.50	7.25 ± 0.15	0.554 +0.166	Industrial & Sugar mill drain (km. pts.35.00 & 33.00)
Before confluence with Hindon	0.25	7.25 ± 0.15	---	

Quantitative picture of flow for different stretches has been tabulated in Table -4.

6.4 Assessment of Pollution Loads

Four point sources have been identified as the root cause of creating pollution in the river. There may be some small line sources of pollution or diffuse sources of pollution, as can be seen from the hydrologic balance (Table -4), however, these sources are so small they could be neglected . The difference of flow in each reach has been considered as incremental flow to each reach.

Out of the 4 point sources of pollution, two are municipal drains carry wastewaters of Muzzafarnagar city and join the river

Table IV : Reachwise variation of flow

Reach	Flow (m^3/sec)		incremental flow (m^3/sec)	Flow out m^3/sec)
	River	Point		
1	3.68	--	0.285	3.965
2	3.965	--	0.285	4.25
3	4.25	--	0.171	4.421
4	4.421	0.30	0.12825	4.850
5	4.850	0.36	0.25	5.4600
6	5.4600	--	0.4525	5.9125
7	5.9125	--	0.4525	6.365
8	6.365	0.554	0.12	7.0390
9	7.0390	0.166	0.08	7.285
10	7.285	--	0.0000	7.2850
11	7.2850	--	0.0000	7.2850
12	7.2850	--	0.0000	7.2850
13	7.2850	--	0.0000	7.2850
14	7.2850	--	0.0000	7.2850
15	7.285	--	0.0000	7.2850

at Nayazurapura (km. point 52) and d/s of Shamli bridge (km. point 49.50) (ref. Fig.1). While other two respectively are; industrial and sugar mill drains carries wastewaters from respective areas and joins the river at kilometer point 35 and 33.

As discussed in section , Wastewaters generated from the Muzzafarnagar city mostly, if not fully, drained out from the city through two main sewer canals joining the river at locations described above. Wastewaters are discharged to the river without any treatment. Quantity of sewage being drained out through these two sewer canals has been measured. It is observed that both the drains joining river at Nayazupura and d/s of Shamli bridge during peak hours carry wastewater flow in the order of 0.30 m^3/sec (25920 m^3/day) and 0.36 m^3/sec (31104 m^3/day) respectively.

Since the use of water largely depend upon the supply pattern of waters, thus generation of wastewaters also follow the similar trend as in the of uses with some "time lag" in the occurrence as shown in Fig.12

Clark, et al. (1971) has indicated the domestic

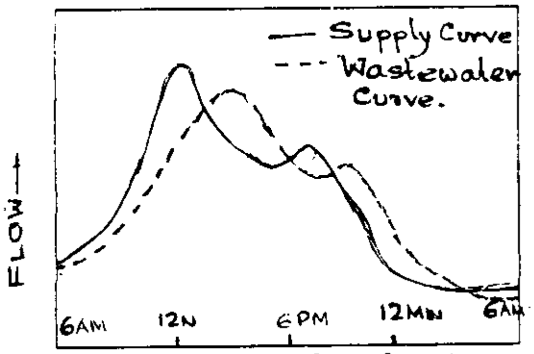


Figure 12: Supply of water and Wastewater Generation Pattern.

wastewater flow to the average ratio as given in Table- 5:

Table V : Domestic Wastewater flow ratios to the Average

Description of flow	Ratio to the Average
Maximum daily	2.25 : 1
Maximum hourly	3.00 : 1
Minimum daily	0.67 : 1
Minimum hourly	0.33 : 1

Using these indicative, the daily average flows from the municipal areas enter into the river through these sewer canal works out as 11520 m^3 and 13824 m^3 respectively. As per the Indian standard for class-I cities (population > 1,00,000), per capita water supply is 147 lpcd. If 75% of the water supply is taken as wastewater, the population equivalent contributing sewage into the river stand on : 2,30,400.

Industries, in and around the Muzzafarnagar city, disposed off their sewage through the industrial drain located in the outer periphery of the city and joins the river at kilometer point 35.0. Steel rolling mills, Chemical factories, and Diaries are among the major industries contribute sewage through the drain. Wastewaters generated from these industries are directly drained out without even preliminary treatment (Plate -4). Wastewaters coming down the river have been got measured before being entered into the river. Considerable amount of industrial wastewaters in the order of 47866 m^3/day with high pollution load mix with river waters.

Two kilometer downstream of industrial drain, i.e., at kilometer point 33.0, another drain carrying sugar mill wastewaters with flow magnitude in the order of 0.166 m^3/sec (14342 m^3/day) joins the river (Plate -5).

Samples of wastewaters from all these sewer canals have been collected over the years on different seasons and analysed in the laboratory to get the indicative of concentration of following parameters:

Plate - 3 : A View of Measurement of Velocity by Pigmy Current Meter.

Plate -4 : A View of Industrial Drain and Wastewaters Receiving by the River.

6.4.1 Pollution Measurement of the River

Standard Techniques have been followed in collection and monitoring of river water quality parameters. For example, in places where point sources mix with the river water, samples have been collected from both upstream and downstream of the source. In situ measurement of DO, pH, and temperature have also been made at those places. Efforts have always been made to collect samples between the depth of 0.2d to 0.8d (d = depth of flow) of water at various locations of river cross section. Necessary precautions have also been taken for the samples collected for laboratory analysis. Fifteen places have been identified for monitoring water quality parameters of the river.

Using Standard methods (AHPA,1985) concentration of BOD, COD, DO, Solids, TDS, Ca, Mg, etc. have been measured in the laboratory. While NO_3-N, PO_4-P, Na, K, etc. have been measured by photometric methods. Table -7 reflects the measured values of water quality constituents of the river for different locations.

6.5 Determination of Reaction Kinetics

Rate of change of concentration of pollutants, mainly for the non-conservative substances- are governed by the reaction kinetics of the organic substances which again dominated by the hydraulic, hydrologic behavior of the water body and the constituents presence in the wastewaters. Reaction kinetics of most of the non-conservative substances mainly depend on the availability of oxygen and the temperature in the water body. There are some organic substances which take part in reaction even in absence of oxygen i.e., in anaerobic stage. Solar energy in both the cases act as catalyst to the reaction.

For example, BOD (Bio-chemical Oxygen Demand)- one of the primary indicator of water quality - value of 50 mg/l (say) means that organic matters (usually represented by CHONSP) for their decomposition require 50 mg/l oxygen. The time rate of consumption of oxygen depends upon decomposition capacity of wastes and the availability of oxygen. On the other hand, the rate of occurrence of oxygen in the water body is governed by the reaeration co-efficient. Reaction kinetics for conversion of NH_3-N to NO_2-NO_3-N and Dissolved phosphorus to inorganic phosphate are also governed by the availability of oxygen in water.

Table 7 : Measured Values of Water Quality Constituents of the River
(results of samples collected on 7.12.'1994 & 7.2.1995)

Location	R.Km.	DO	BOD	pH	Temp.	TDS	NO	PO	Na	K	Cl
Malira Bridge	65.0	7.6 ±.3	5.2 ±.5	7.1±.1	17.1±.2	265.0±5.	1.2±.15	0.10±.05	55.0±2.	30.0±1.5	105.0±5.
Nayazupura U/S.	52.25	5.7 ±.3	8.8 ±.75	7.3±.1	17.5±.2	281.2±6.	4.0±.2	7.0±.25	74.0±3.	50.0±2.0	167.2±6.
Nayazupura D/S.	51.75	5.28±.2	26.3±1.75	7.5±.2	17.52±.3	285.±6.5	0.45±.2	0.45±.1	23.2±2.	6.0±.75	64.2±2.5
Shamli Bridge U/S.	50.0	4.31±.2	16.7±1.5	7.15±.2	18.0 ±.3	273.7±6.	1.2 ±.2	0.37±.1	24.0±2.	6.8±.75	48.2±3.0
Shamli Bridge D/S.	49.50	3.71±.2	35.6±2.0	7.16±.2	18.0 ±.3	303.7±6.2	4.5 ±.22	5.6±.25	92.0±2.5	8.0±.55	70.0±3.5
Surju Bridge	47.50	1.87±.2	26.0±1.75	7.41±.25	18.5 ±.3	280.0±6.2	0.7±.15	.45±.15	20.7±1.5	6.2±.4	52.2±2.5
U/S Indistrial Drain.	36.0	4.25±.25	20.1±1.50	7.52±.25	19.0 ±.25	251.25±5.7	0.7±.12	.23±.1	21.6±1.5	5.4±.3	53.2±2.0
D/S Industrial Drain.	34.75	---	83.3±5.0	7.16±.25	20.0 ±.35	375.0±6.5	5.0±.2	.25±.1	55.0±2.2	18.0±.5	76.2±2.5
D/S Sugar Mill Drain.	32.75	----	110.9±5.0	7.6 ±.26	19.8 ±.4	388.7±7.0	1.6±.1	.35±.1	37.5±2.0	12.0±.6	68.3±3.0
Near Naula	27.25	----	76.44±3.0	7.65±.25	19.7±.3	372.5±5.0	0.7±.1	.5±.15	40.0±2.2	13.2±.5	58.2±2.5
Near Chandsina	21.75	----	72.6±3.0	7.95±.25	20.5±.25	366.25±6.5	1.0±.2	.45±.2	40.5±1.7	12.4±.5	54.2±1.75
Near Samoli	17.00	----	45.8±2.5	7.80±.2	20.0±.3	398.75±5.5	0.9±.15	.65±.2	39.5±2.0	13.0±.5	70.2±3.0
Near Ancholi	11.5	0.35±.1	32.2±2.0	7.85±.15	19.8±.25	356.25±5.0	0.9±.17	.55±.2	30.0±1.5	10.0±.4	62.2±2.5
Near Mohan	8.75	1.05±.15	24.5±1.5	7.80±.15	19.4±.2	340.0±3.5	0.8±.18	.45±.2	31.0±2.0	11.0±.3	64.2±2.0
Near Pitlokar	1.25	1.72±.2	19.75±1.7	7.82±.15	20.1±.25	356.25±4.0	1.0±.20	.50±.2	36.0±1.5	12.0±.2	70.2±2.5

All values are in mg/l except Temperature (°C) and pH

Plate - 5 : A View of Sugar Mills Drain And Wastewaters Receiving by the River Near Mansurpur Bridge.

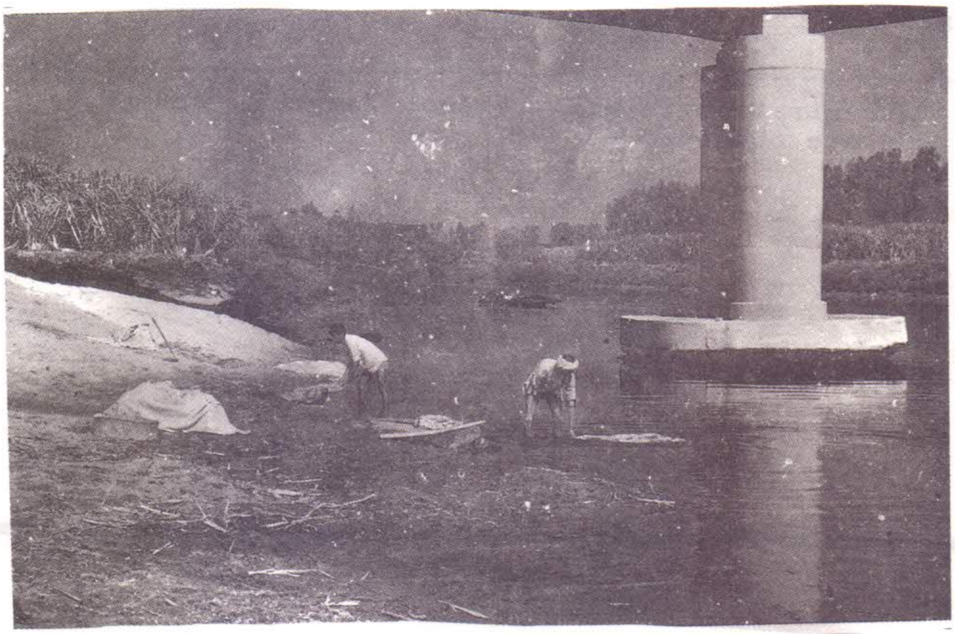


Plate -6 : Socio-Economic And Socio-Cultural Aspects of the River (Uses of River Water)

BOD_5, COD, DO, pH, Temperature, TDS, Solids, NO_3-N, Organic Phosphate, Dissolved Phosphate, Chloride, Na, Ca, Mg etc.

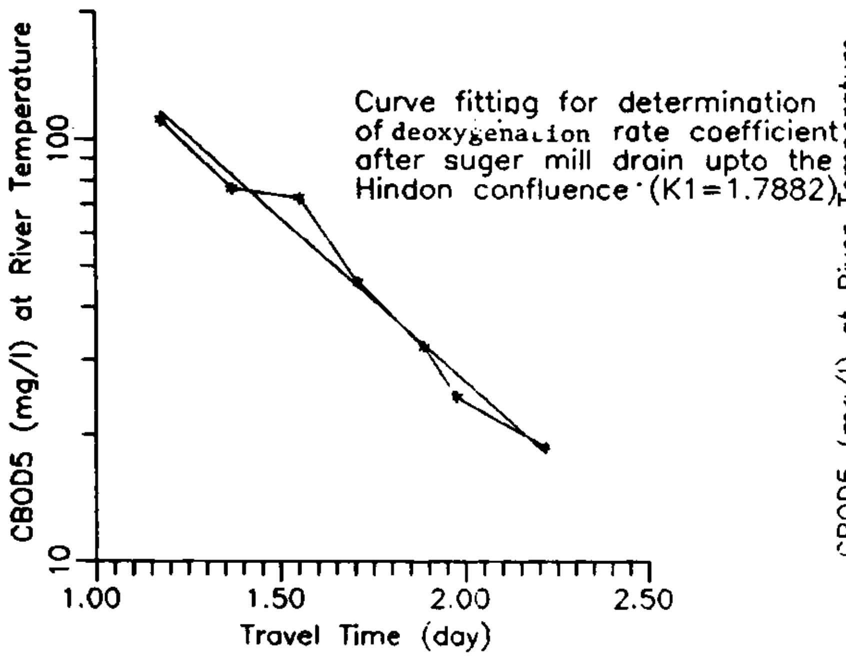
Table-6 represents measured values of pollutant concentration for different wastewaters :

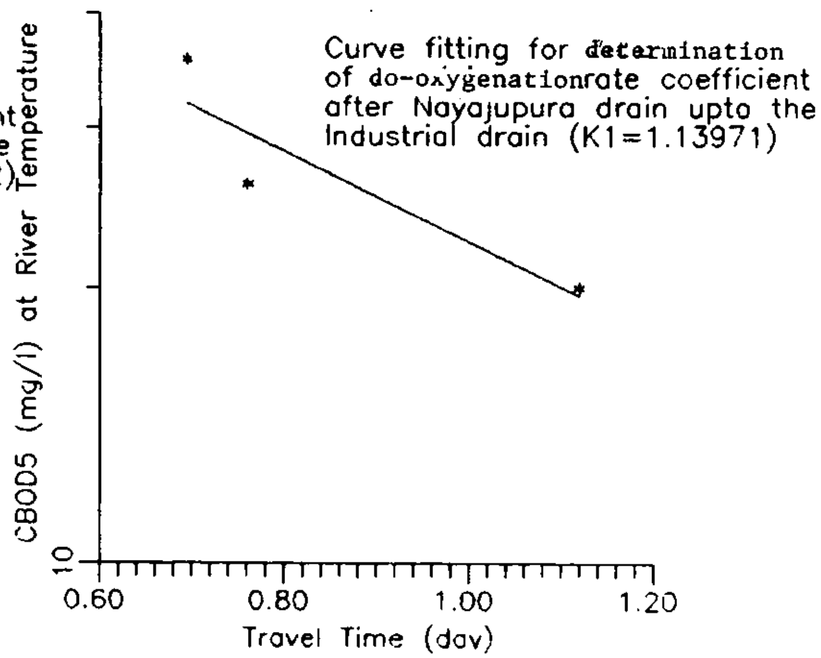
Table VI : Measured values of some selected Wastewater constituents

Location	Source	BOD_5	COD	pH	Temp	TDS	NO_3	PO_4
Nayazupura drain (km.52.)	Municipal	325.0	950.0	8.05	18	712.5	.45	.18
Near Shamli bridge (km.49.75)	Municipal	318.0	927.0	7.52	18.5	717.5	.65	.43
Industrial drain (Km. 35.)	Industries	801.0	5800.0	7.27	21.5	1006.	17	.8
Mansurpur bridge (Km. 33.)	Sugar mill	1695.0	7540.0	7.28	26.5	2467.	-	.35

Concentration in mg/l, expect Temp (^0C) and pH.
DOs are observed as "0"

Fig. 13 - Semilog plots to determine the De-Oxygenation rate coefficients of the river.





incubated temperature, O = constant (temperature dependent and generally taken as 1.047), T = field temperature). Bottle rate co-efficients obtained from laboratory analysis are different than the de-oxygenation rate co-efficient of river water. Because river deoxygenation co-efficient is governed by many factors. Bottle rate co-efficient is an indicative of maximum oxygen demand of organic matters over a specific time. Bottle rate co-efficients for different wastewater samples as obtained from laboratory analysis are : 0.33 day^{-1} (municipal wastewaters), 0.50 day^{-1} (industrial wastewaters), 0.65 day^{-1} (sugar mill wastewaters).

De-oxygenation rate co-efficients for CBOD of the river have been determined from the values of BOD_5 measured at monitoring location of the river. The values of CBOD rate co-efficients are found different for different stretches. Semilog plot of BOD_5 (on log scale) and travel time (ordinary scale) as shown in Fig. 13 provides the CBOD rate co-efficients of the river for different reaches.

6.5.2 Atmospheric Reaeration Rate Coefficient

As discussed in section 5.2.2.8 , QUAL2E has seven options to input the re-aeration rate co-efficient. For given co-efficient and exponent values of velocity and depth, the model calculates the hydraulic parameters for assigned flow in each reach. Any option expect options 6 & 7, there is no need of any input value for determination of re-aeration co-efficient except the selection of option. It has been reported (Brown and Barnwell, 1985) that option 3 (O'Connor & Dobbin, 1985) and option 4 (Owens et al., 1987) provide most closure value of re-aeration co-efficient than any other options (expect measured values), hence options 3 and 4 have been chosen suitably for computation.

6.5.3 Determination of Settling Rate Co-efficient

Settling of organic matters usually more predominant at the vicinity of outfalls and decreases gradually as the pollutants move towards downstream. Settling rate of organic matters has been determined at the vicinity of the four wastewater outfalls by measuring the transmissivity of water between two specified points. Difference of transmissivity has been taken as amount settled between the two specified locations. Knowing the distance between two locations and the velocity of water, the "travel time" which is considered as settling time of particle, is measured. Settling rate

For different level of modeling (as given below) following reaction co-efficients are needed as input to QUAL2E.

For DO Modeling:

	Co-efficient	Desirement
a)	Deoxygenation rate co-efficient (K_1)	essential
b)	Reaeration rate co-efficient (K_2)	essential
c)	Sediment Oxygen Demand (K_4)	essential
d)	Benthic Oxygen Demand (K_3)	essential
e)	Growth rate of Algae (μ)	optional/essential
f)	Respiration rate of Algae (ρ)	optional/essential
g)	Settling rate of Algae (σ_1)	optional/essential
h)	Rate of oxygen production by (α_3) Algae	optional/essential
i)	Rate of oxygen uptake by Algae (α_4)	optional/essential
j)	Rate of oxygen uptake per unit of ammonia nitrogen oxidation (α_5)	optional/essential
k)	Rate of oxygen uptake per unit of nitrite nitrogen oxidation (α_6)	optional/essential
l)	Ammonia oxidation rate co-efficient (β_1)	optional/essential
m)	Nitrite oxidation rate co-efficient (β_2)	optional/essential

For BOD Modeling

	Co-efficient	Desirement
a)	Deoxygenation rate co-efficient (K_1)	essential
b)	Settling rate of BOD (K_3)	essential

For Nitrogen Cycle Modeling

	Co-efficient	Desirement
a)	Algal respiration rate (ρ)	optional/essential
b)	Fraction of algal biomass that is nitrogen (α_1)	optional/essential
c)	Rate constant for hydrolysis of organic nitrogen to NH_4-N (β_3)	essential
d)	Rate co-efficient for organic nitrogen settling (σ_4)	essential
e)	Rate constant for biological	

oxidation of NH_4-N (β_1)

f) Benthic rate constant for ammonia (σ_3) — essential

For Phosphorus Modeling

Co-efficient	Desirement
a) Phosphorus contain in algae (α_2)	optional/essential
b) Organic phosphorus decay rate (β_4)	essential
c) organic phosphorus settling rate (σ_5)	essential
d) Benthic rate constant for dissolved phosphorus (σ_2)	essential

For Algal Modeling

Co-efficient	Desirement
a) Specific growth rate of algae (μ)	essential
b) Respiration rate of algae (ρ)	essential
c) Settling rate of algae (σ_1)	essential

Significance of essential/optional is; if concentration of respective parameter(s) is/are given as input and is/are required to model, the reaction kinetic(s) is/are to be input the model, When the concentration of respective constituent(s) is/are assigned as nil, rate co-efficient become optional input.

6.5.1 Determination of CBOD and De-oxygenation rate constants

The CBOD (Carbonaceous Bio-Chemical Oxygen Demand) of wastewater and river water samples has been obtained by Standard procedure (APHA, 1985) of incubation of samples (diluted in different percentages) over a period of time to describe full course of oxygen utilization. Samples collected from municipal, industrial, and sugar mill drains have been analysed for different days (3,5,7,10,12,15,20 days) upto 20 days at 20^0C to determine the bottle rate constants of the wastewaters. Plots between DO consumption i.e.,oxygen demand and incubation time give the laboratory rate co-efficient at incubated temperature. Conversion of rate co-efficient from incubated temperature to the actual water temperature is done by the well known formula, $K_t = K_{20}\, \Theta^{(T-20)}$ (where K_t= rate constant at field temperature, K_{20} = rate constant at of particles is found to vary between 2.0 - 6.0 day^{-1} . Settling rate co-efficient is found higher in the vicinity of industrial and sugar mill drain (4.0-6.0 day^{-1}) than in the vicinity of municipal drain (2.0-3.0 day^{-1})

6.5.4 Determination of Sediment Oxygen Demand

The discharge of settleable waste components may result in the formation of "sludge banks" or deposits of organic material immediately below the waste outfall. These deposits may build up over a period of time if velocities are too low to prevent scouring of the river. As the depth of the deposited solids increases, anaerobic decomposition of the organic matter in the deeper layer begins. The products of this decomposition, CO_2, CH_4, and H_2S proceed up through the sludge layer and into the overlying waters.Floating of the bottom sludge may lead to severe aesthetic problem as well as possible transient DO depletion. Problem of DO depletion because of settleable organic matters which is called benthic oxygen demand, is relatively less in fast flowing water than the slow moving water like lakes and estuaries.

With this idea in view, SOD (Sediment Oxygen Demand) of river bottom sludges has been measured in four places immediately below the outfalls. Standard Method in which a chamber is submerged on the bottom sludge and then O_2 uptake is measured over time could not apply, however, SOD is measured by collecting undisturbed river bottom samples and placing the sample in oxygenated water to measure the DO reduction over time. Measurement of O_2 uptake over time gives an indicative of SOD of bottom mud. It is visualized during collection of samples that river bottom immediately below the municipal drain is muddy and contains mostly detritus of organic matters while river bottom after the outfalls of industrial and sugar mill drain is mostly sandy and river bottom sands are being lifted up for constructional activities.

Sediment Oxygen Demand of river bottom sludges as obtained in four different places are given in Table : 8

Table - 8 : Measured Values of Sediment Oxygen Demand

Location	SOD rate ($gm/m^2/day$
Below the Nayazupura	4.0

outfall.	
Below the Shamli Bridge outfall.	6.0
Below the Industrial wastewater outfall.	2.5
Below the Sugar mill waste water outfall	3.50

6.5.5 Nitrogenous and Phosphorous Rate Constants

Rate constants for biological oxidation of NH_3 to NO_2 to NO_3 and hydrolysis of organic nitrogen to ammonia and the rate constant for decay of organic phosphorous to dissolved phosphorous have not been determined separately but assumed suitable values from the range given in the user manual. Assumed values have been adjusted accordingly while the model is calibrated.

Variable Description	Assumed values (day^{-1})
Rate constant for the biological oxidation of NH_3 to NO_2 . (β_1)	0.50
Rate constant for the biological oxidation of NO_2 to NO_3. (β_2)	2.0
Rate constant for the hydrolysis of organic N to Ammonia. (β_3)	0.2
Rate constant for the decay of organic-P to dissolved-P (β_4)	1.5

6.5.6 Temperature Dependence of Rate Co-efficients

Temperature values computed in QUAL2E are used to correct the rate co-efficients in the source/sink terms for the other variables. These co-efficients are input at 20^0C and are then corrected to temperature using Streeter and Phelps type formulation:

$$K_t = K_{20}\ \theta^{(t-20)}$$

where,

K_t and K_{20} = the co-efficients at local temperature (t) and standard temperature (20 ^{0}C) respectively.

θ = an empirical constant for each reaction co-efficient.

The values of the temperature correction factors θ are input to the model. If these values are not specified, model assumes default values as employed in the model.

7.0 CALIBRATION OF THE MODEL

Once the data cards are ready to execute the model, the very next task is the calibration and validation of the model. Before doing this exercise, one has to be very thorough about the inter-linkages and inter-actions of parameters that are to be modeled. For example, for DO-BOD modeling, the first task would be to see the matching of observed and computed BOD rather than DO. Concentration of DO is mainly governed by many factors eg., conversion of NH_3-N to NO_3-N, reaeration co-efficient, river hydraulic parameters, algal concentration and respiration etc. Once the BOD is got matched, the second task would be to match the DO concentration in each reach where nitrogen cycle and the carbonaceous cycle are to be adjusted accordingly. Since the reaeration co-efficient varies with river hydraulic data and climatological data data, efforts are to be made to calibrate those data rather than adjusting the measured values. Questions than come which parameters are to be adjusted and what should be the level of adjustment. Option of sensitivity analysis of each/multiple parameters given in the model provides the appropriate tool to determine the response of the parameters on any desire location.

Number of trials would provide pictures as to which parameters are to be adjusted and upto what level. The trial run which represents the best matching between observed and computed values is to be considered as the calibrated values of the model. The model is said to be best calibrated, when the response of the model for any data set of the river is remained within the range of observed values or does not deviate more than 5% of the mean values.

Water quality processes being a complex phenomena, there may be lot of uncertainty and errors in the measurement, analysis , and in the collection of samples. For example, samples collected from a place which actually represent the concentration of pollutants for the whole cross section, may not contain representative organic matters. Thus selection of site is very important. More the samples from a specific site errors would be less. Analysis of samples in the laboratory is another important factor may contribute lot of errors in the data. Results of titration of samples where individual skills are necessary may be different if same samples are titrated by two different persons. There are so many factors where errors could be observed. Thus one can not make sure that data obtained from field do not contain any errors. However, while calibrating the model, efforts are always to be on adjustment of reaction kinetic data rather than the pollution load data in order to a get good matching between computed and observed values. Uncertainty analysis of the parameters give the degree of errors involve in the parameter in any location which could be used as the guideline of adjustment.

Moreover, one/two sets of data can never give the representative calibrated values. Series of data sets, atleast 7 years data, would be necessary for reliable calibration and validation of the model. Model calibrated based on a particular set of data would always fit the observed value on the basis of which the model is calibrated.

Based on the two sets of observed data (obtained from river sampling on 7.12.1994 & 7.2.1995) the model has been calibrated by the first set of data and validated by the second set of data. Calibrated model has further been used for determination of management alternatives.

In this study, the objectives being to get an indicative of pollution that are prevailing in the river and to see the applicability of the model for studying and modeling of water quality, the results reported can be considered as an attempt towards successful application of the model.

8.0 ANALYSIS OF RESULTS AND DISCUSSIONS

As discussed earlier, water quality processes being complex in nature there remains lots of uncertainty in the occurrence, monitoring and in analysis of water quality parameters which can be minimized through continuous monitoring and analysis. On the otherhand, the responses of the model depend upon the accuracy of input data, i.e., better the input data better responses in computation would be obtained. QUAL2E being a steady state one dimensional model it has got its limitation of data acceptability. Keeping all these aspects in view, data collected from field observations and obtained from laboratory analysis have been made in representative form as acceptable to the model (discussed in section 6) and calibrated the model to match the observed values.

Because of the limitations on data, the study has mainly been concentrated on modeling of BOD, DO, NH_3-N, NO_2-N, NO_3-N, PO_4, TDS,and Temperature. Though efforts have also been made to model Cl_2, Na, K, and Mg, however, because of lack of adequate data these parameters could not be simulated with reliable accuracy.

The simulated profile of DO (as shown in Fig. 14) reflects a good match with observed values. In between the Malira Bridge (65 Km. upstream of confluence), - the reference point- and Nayazupura drain (52 Km. upstream of confluence) though there is no point source , however, the DO values have been found deteriorated from 7.6 mg/l (+ 0.3 mg/l) to 5.70mg/l (+ 0.3 mg/l). These variation may be because of diffuse sources of pollution or excessive demand of oxygen by benthic deposits. In the model, these has been accounted for considering organic loads (100 mg/l in terms of BOD) with incremental flows and assuming benthic demands of oxygen as 4.0 mg/sq.m/day between the reach . On assumption of these values a good match between computed profile and observed values of BOD is obtained, Fig. 15.

Computed profile of DO (Fig. 14) shows two distinct DO sags, one is in between the outfalls of municipal drain and industrial drain with a minimum DO value of 1.74 mg/l at river Km. 45 (from confluence), another is after the industrial drain with "0" DO concentration continues from R.Km.33 upto R.Km. 12.0 and after that DO replenishes to a maximum value of 1.90 mg/l before mixing with Hindon river. These dictate that the stretch between Km. point 52 and Km. 35 is aerobic, and between Km. 33 and Km. 12 is anaerobic. The stretches in terms of DO can be categorized as;

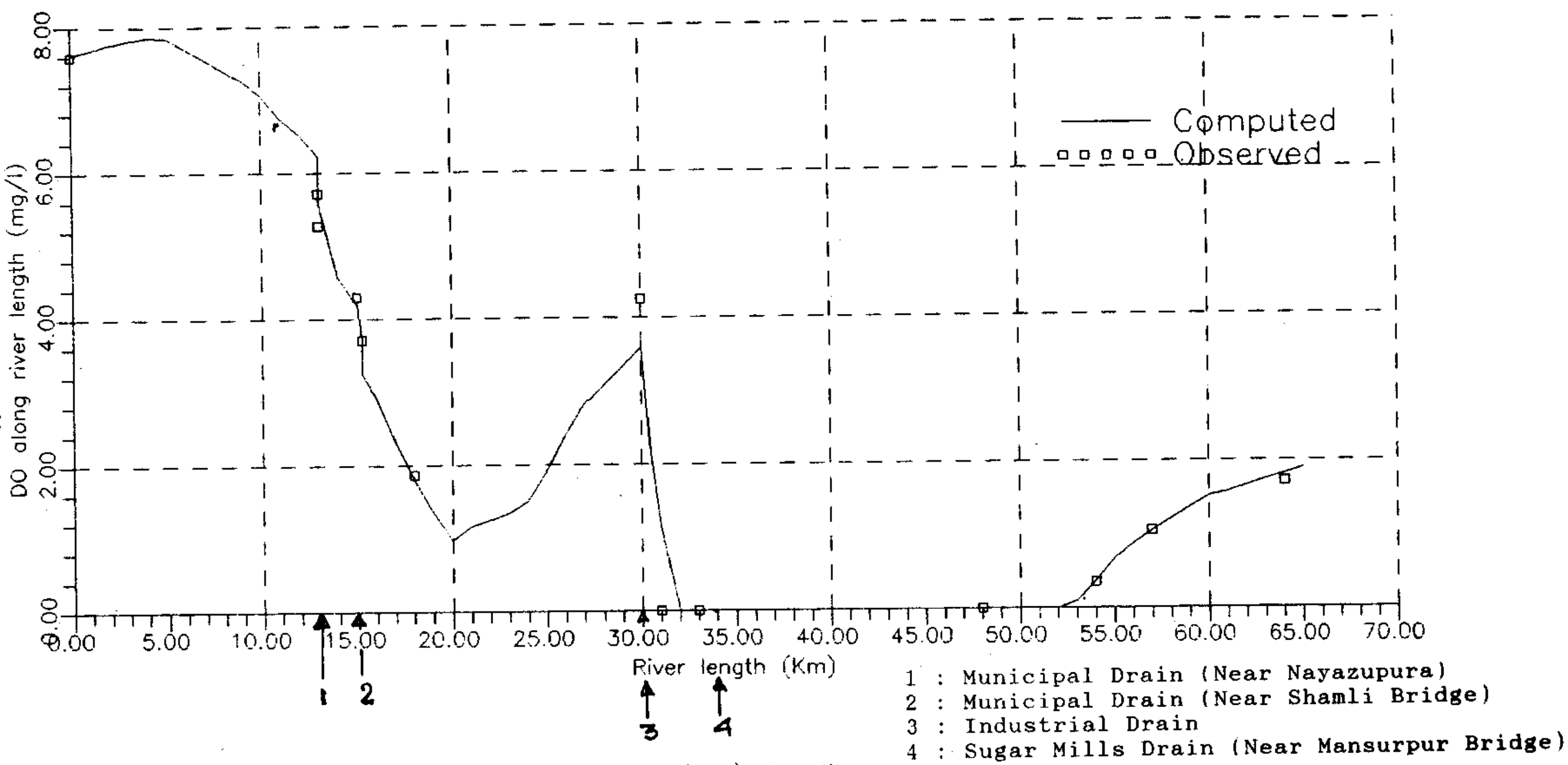


Fig. 14 - Dissolved Oxygen (DO) Profile Along the River Reach

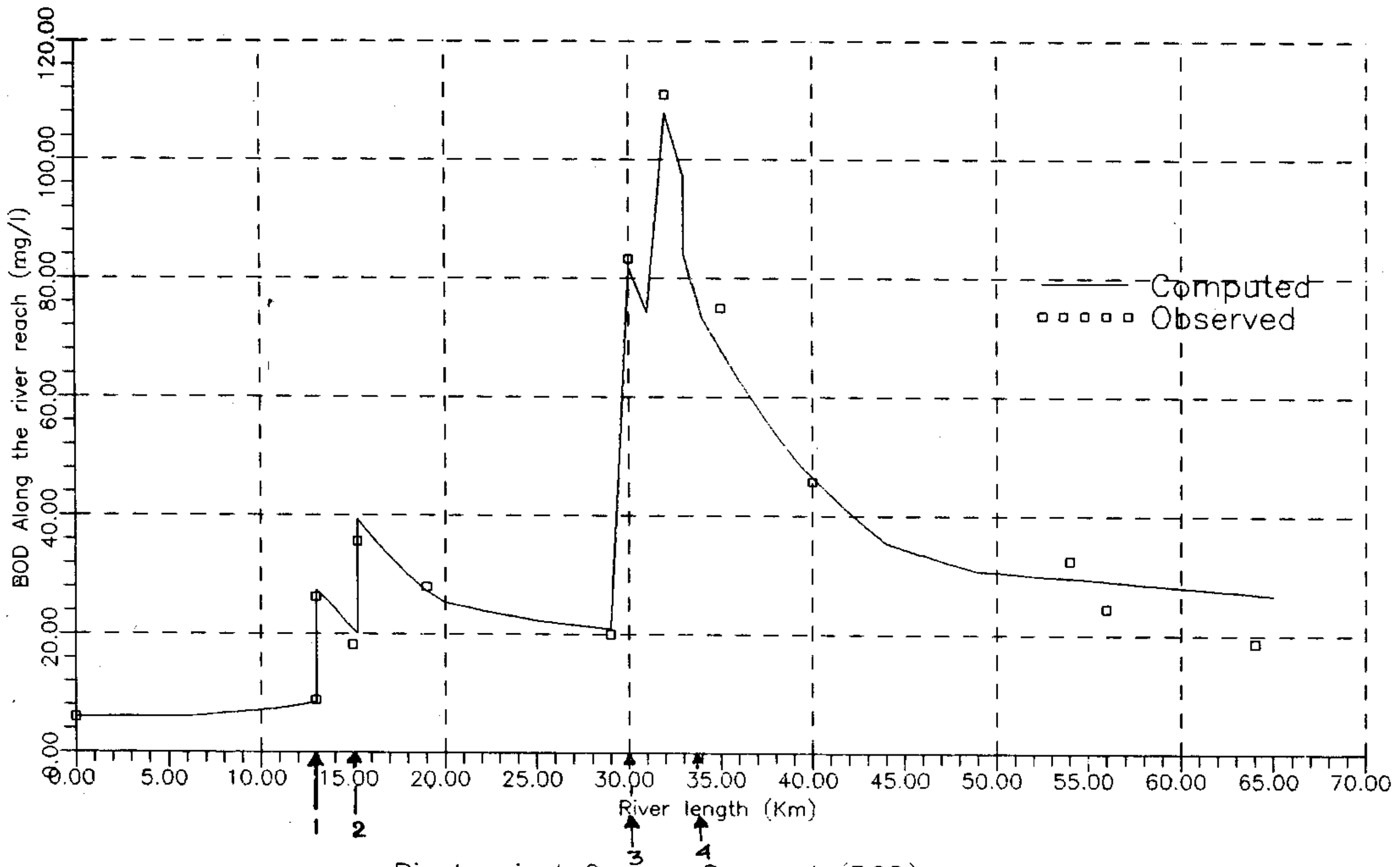


Fig. 15 - Biochemical Oxygen Demand (BOD) Profile Along the River Reach

1 : Municipal Drain (Near Nayazupura)
2 : Municipal Drain (Near Shamli Bridge)
3 : Industrial Drain
4 : Sugar Mills Drain (Near Mansurpur Bridge)

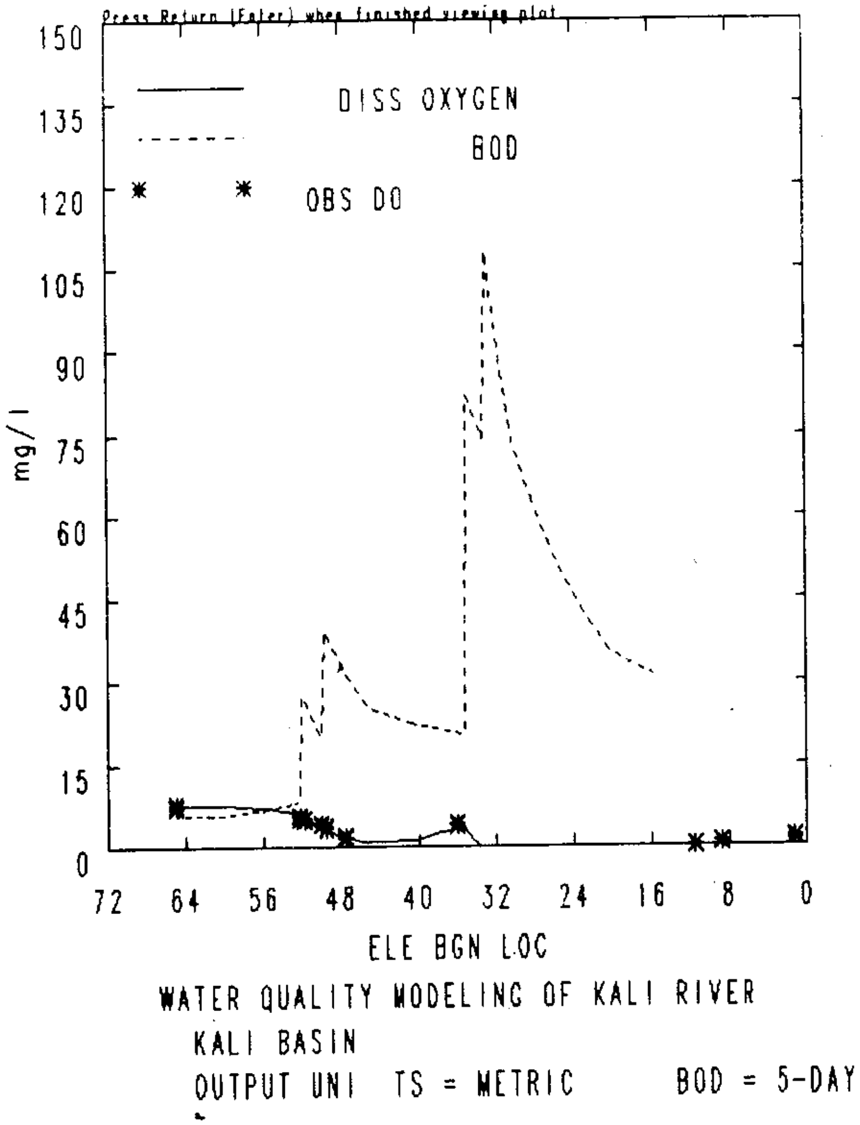


Fig. 16 - Variation of DO & BOD Along the river
Reach (Computed)

Km. point		Classification
From	To	
65.0	52.0	Class B
52.0	45.0	DO Depletion zone
45.0	35.0	Eutrophied zone
35.0	12.0	Anaerobic zone
12.0	0.0	DO replenishing zone

From Fig. 15 and 16 it can be seen that BOD values increases considerably where oufalls join the river, eg., at River Km. 52, Km. 49.75, Km. 35, and Km. 33 from confluence. The maximum concentration of BOD at those locations respectively are 26.5 mg/l, 39.00 mg/l, 81.5 mg/l, and 105.5 mg/l against the observed values of 26.25 mg/l (± 1.75 mg/l) , 35.5 mg/l (± 2.0 mg/l), 83.3 mg/l (± 3.5mg/l), and 110.95 mg/l (± 5.0 mg/l). Appreciable improvements on DO and BOD (Fig. 15 & 16) as observed in between the river Km. 45 and Km.35, may be because of high concentration of nutrients in terms Organic Nitrogen (Fig. 17), Ammonia Nitrogen (Fig. 18) and Dissolved Phosphorous (Fig. 19) which usually activate the growth of algae, reveals that water of this reach can be used for irrigation and wild lives and fisheries as well as for industries ,and can be classified as Category E (based on CPCB classification).

Concentration profiles of Temperature, TDS, and pH as observed and simulated by the model are given in Fig.20, 21, and 22. Computed temperature profile shows (Fig. 20) a increase of temperature by 6.5 ^{0}C from 17^{0}C to 23.5^{0}C, for the total length of 65 Km. against variation of observed values from 17^{0}C to 21.5^{0}C for that total length. These may be because of the in accurate consideration of wet and dry bulb temperature. In the model, similar climatological conditions as prevailed in Roorkee which is about 75 Km. away form the study area, on that dates have been considered for analysis. However, concentration profile of TDS shows good match with observed data. Range of TDS variation being within 200 mg/l to below 450 mg/l seems to be below the prescribed limit as designated for class E category water (1000 mg/l). Concentration profile of pH of river water (Fig. 22) being within the range 7.0 - 8.0, it can be said that water is neither acidic nor alkaline but almost neutral. Based on the existing water quality status of the river, attempt has also been made to zoning the river stretches according to the pollution status, Fig. 31.

As strategies toward management of pollution in order to

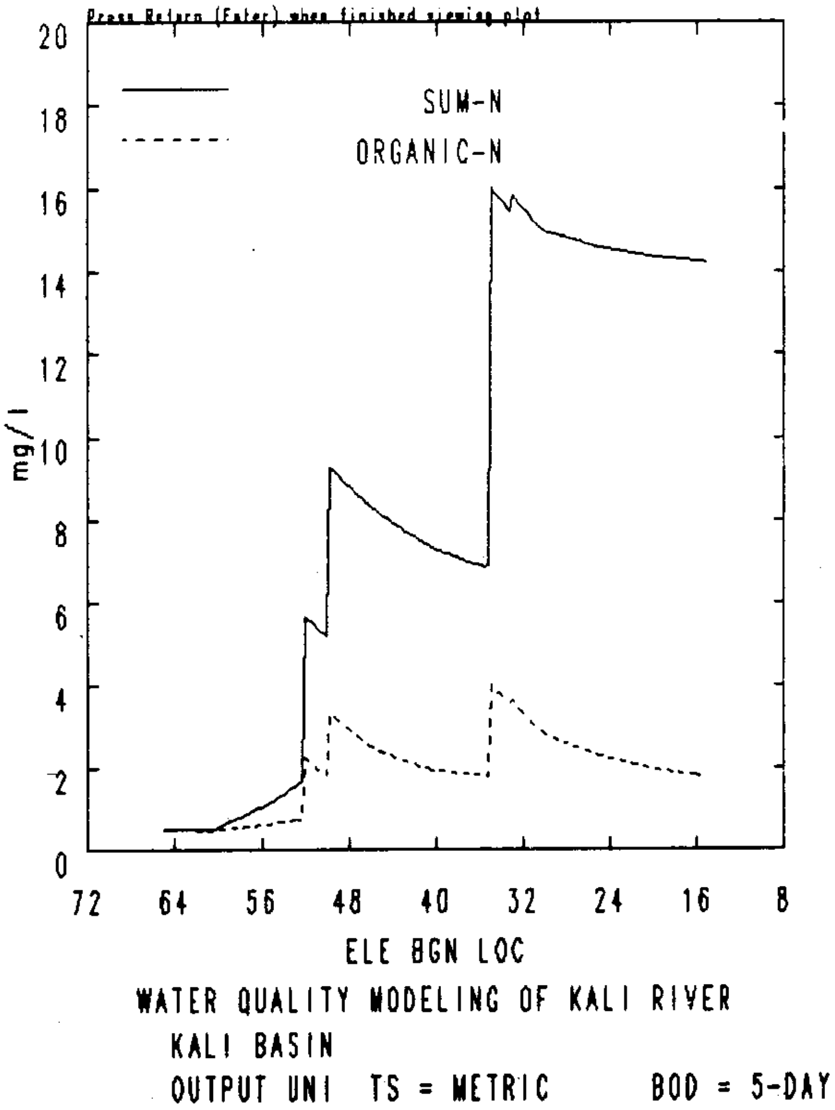


Fig. 17 - Vàriation of Total Nitrogen and Organic Nitrogen Along the river reach (Computed)

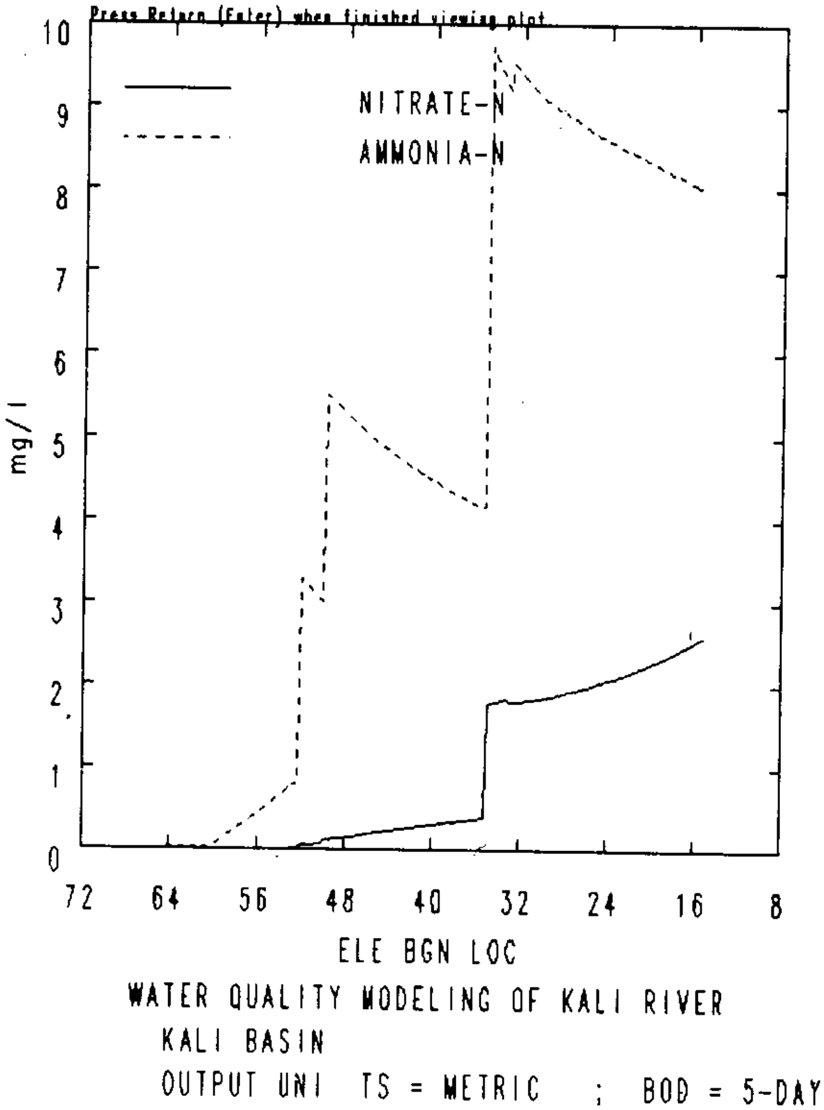


Fig. 18 - Variation of Ammonia-N and Nitrate-N
Along the river reach (Computed)

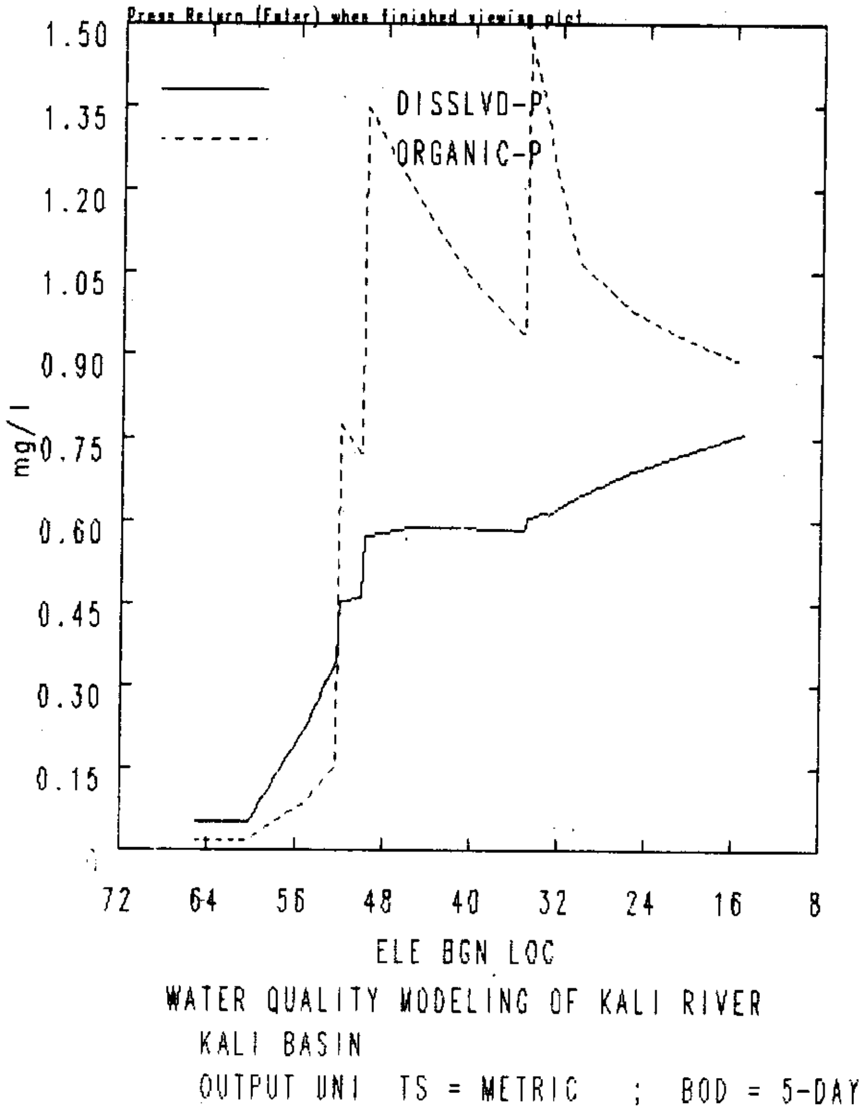


Fig. 19 - Variation of Organic Phosphorous and Dissolved Phosphorous Along the River Reach (Computed)

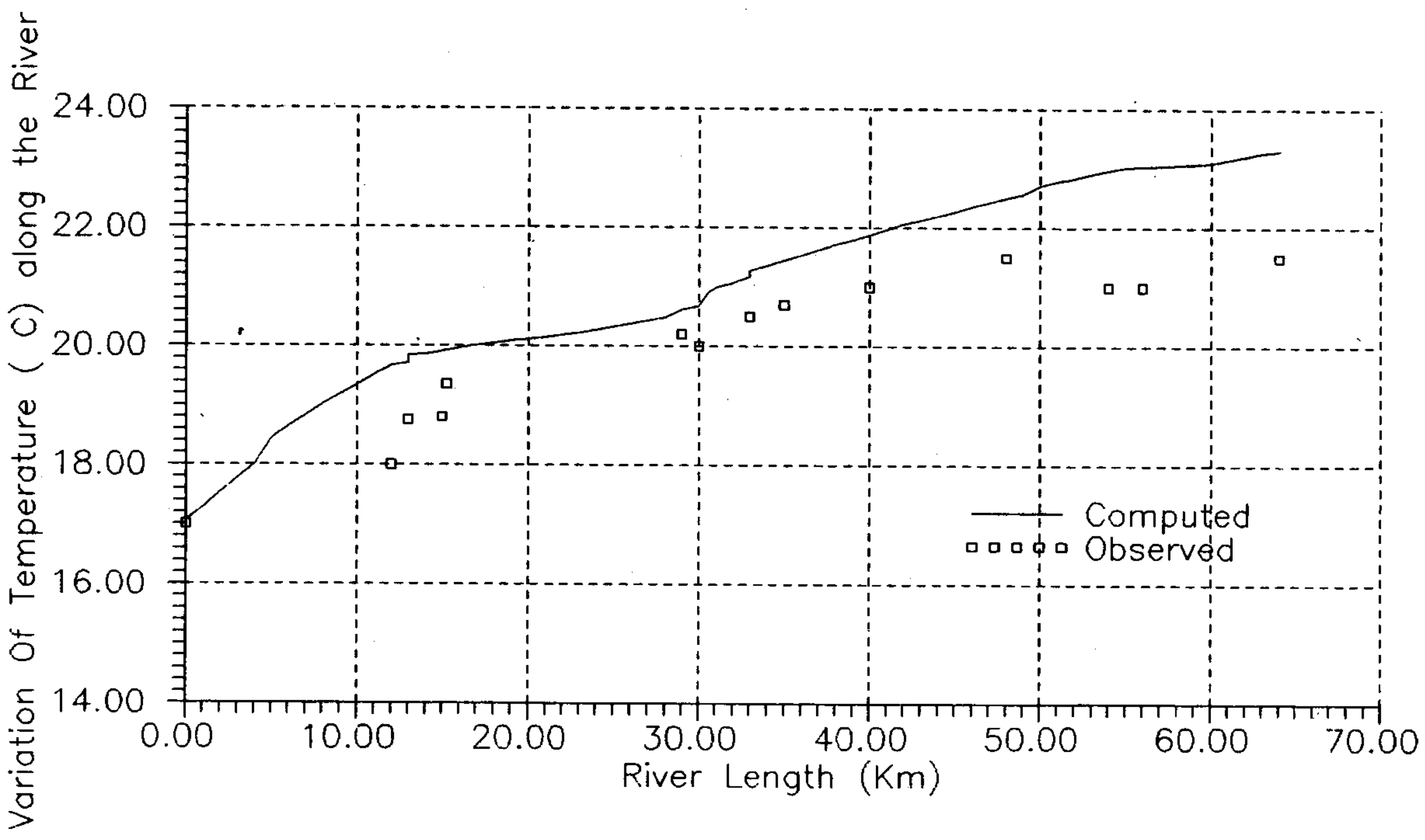


Fig. 20 - Temperature Variation Along the River Reach

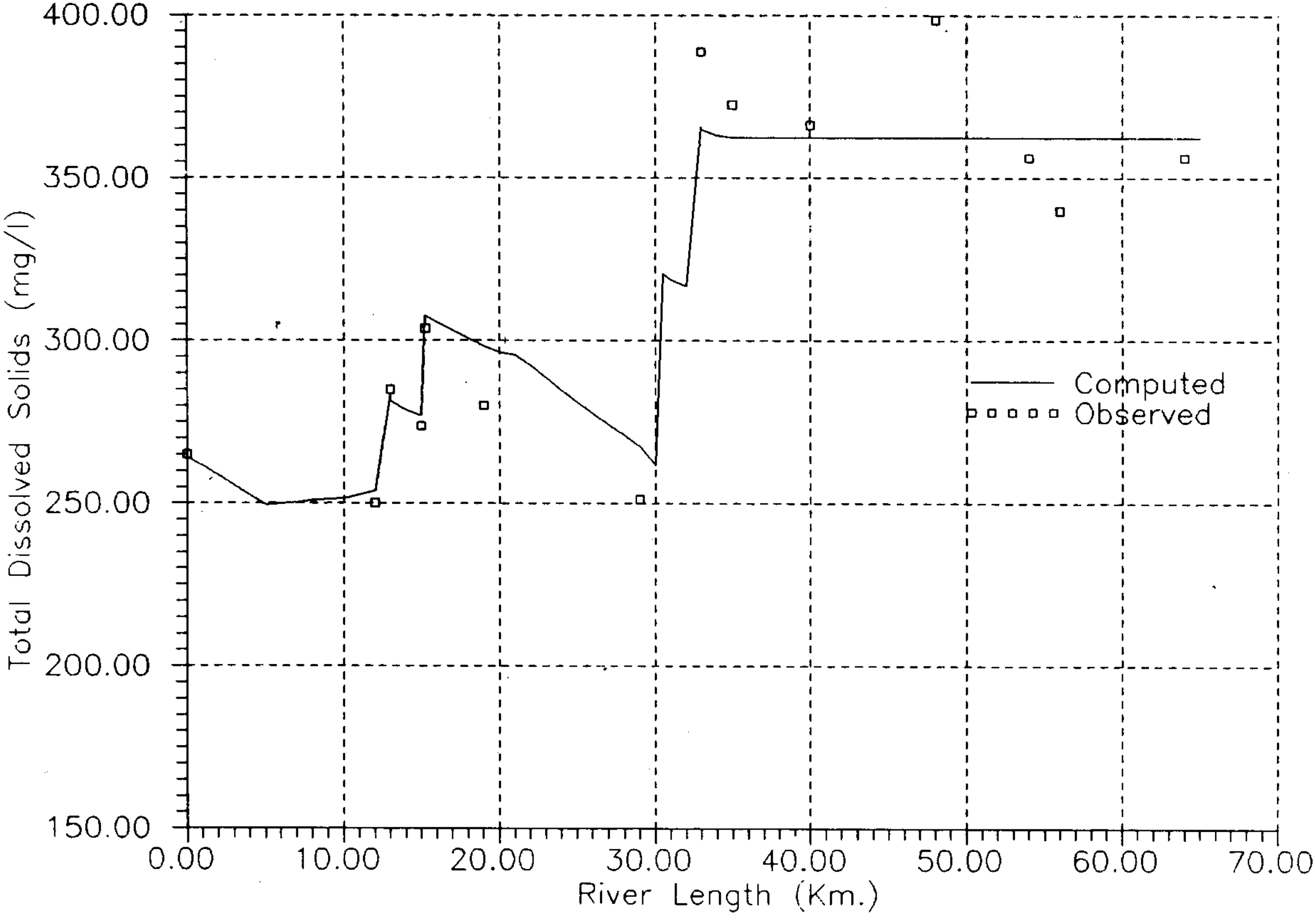


Fig. 21 - Total Dissolved Solids Profile Along the River Reach

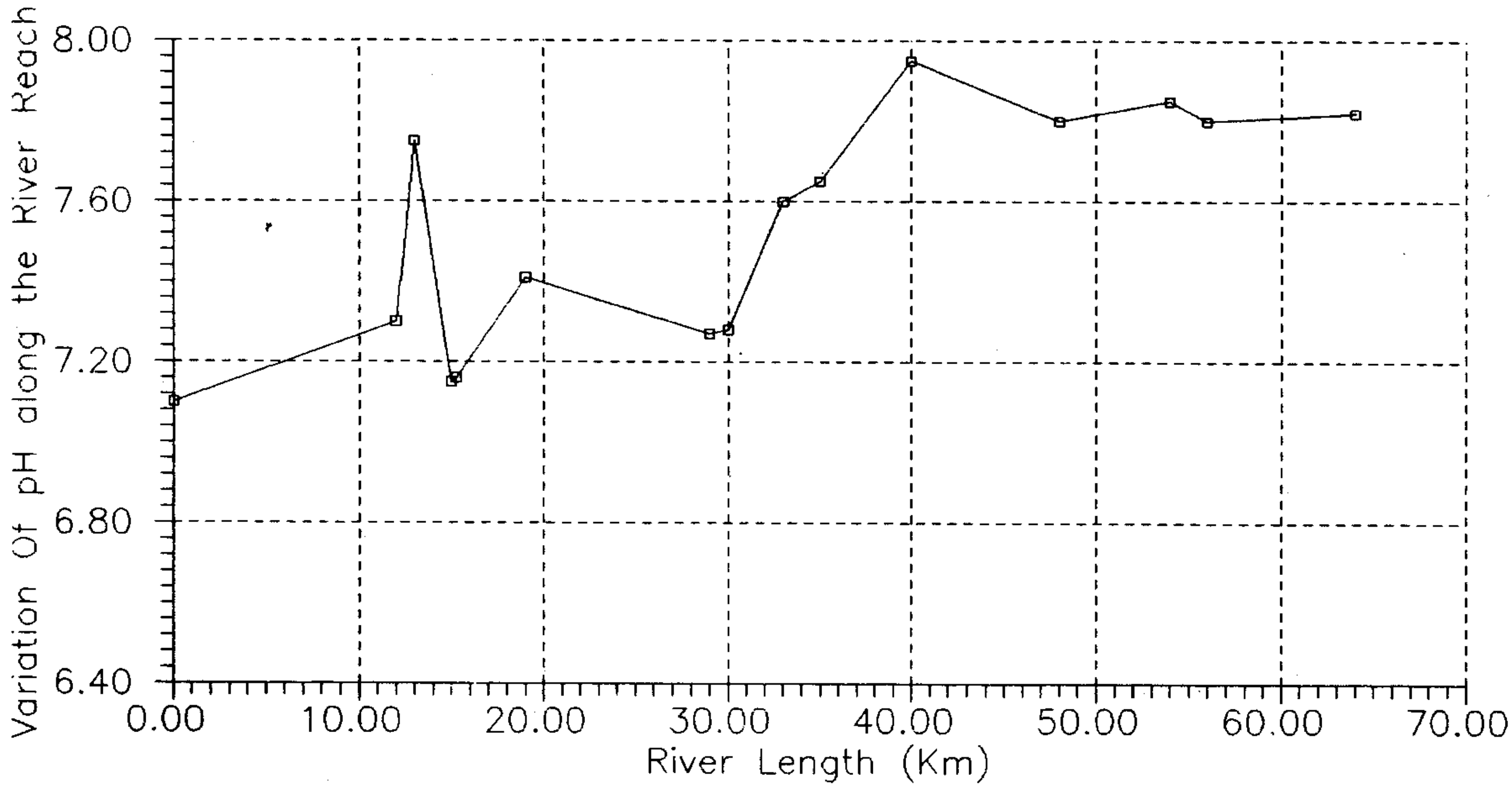


Fig. 22 - Observed pH along the River Reach (Observed Values)

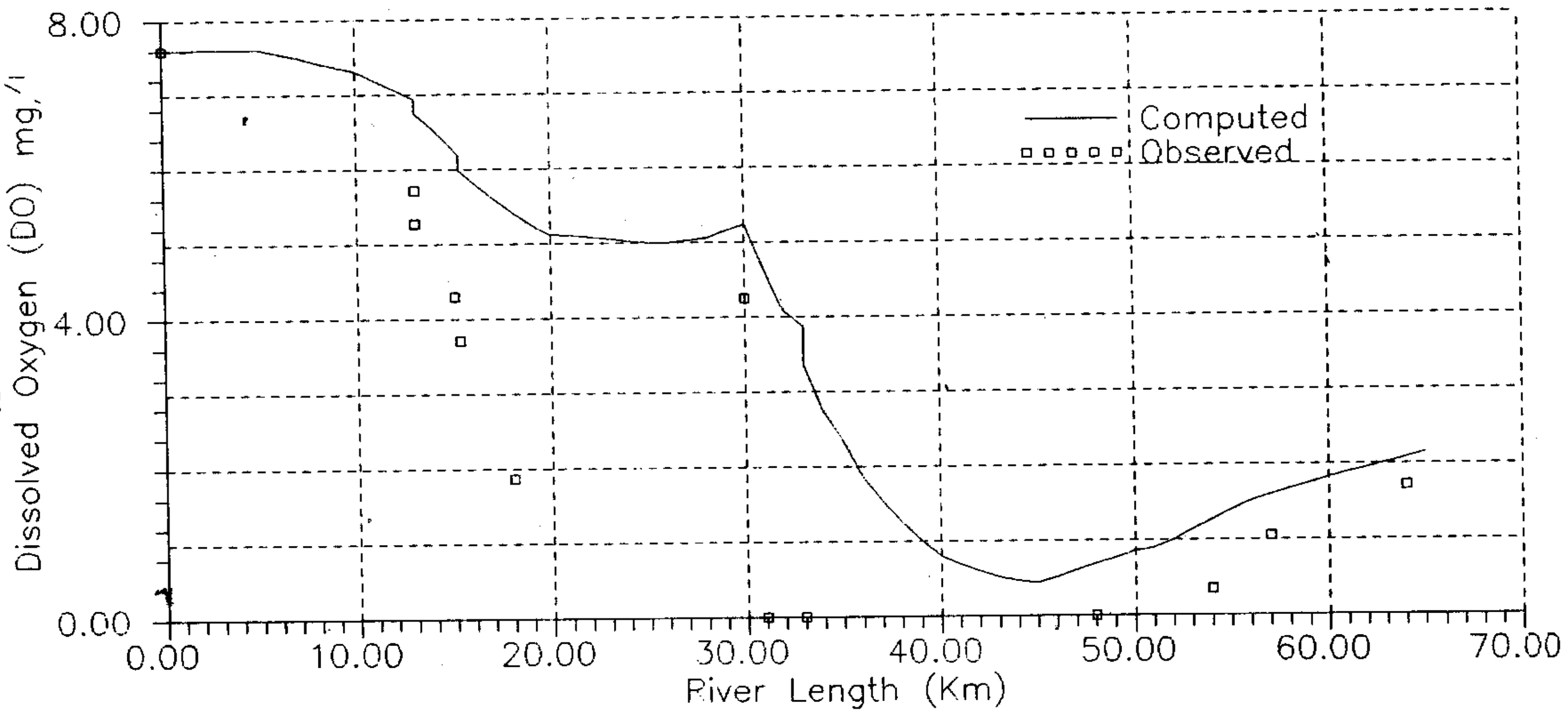


Fig. 23 - Improvement of DO with Flow Augmentation of 4.21 times of Headwaters flow

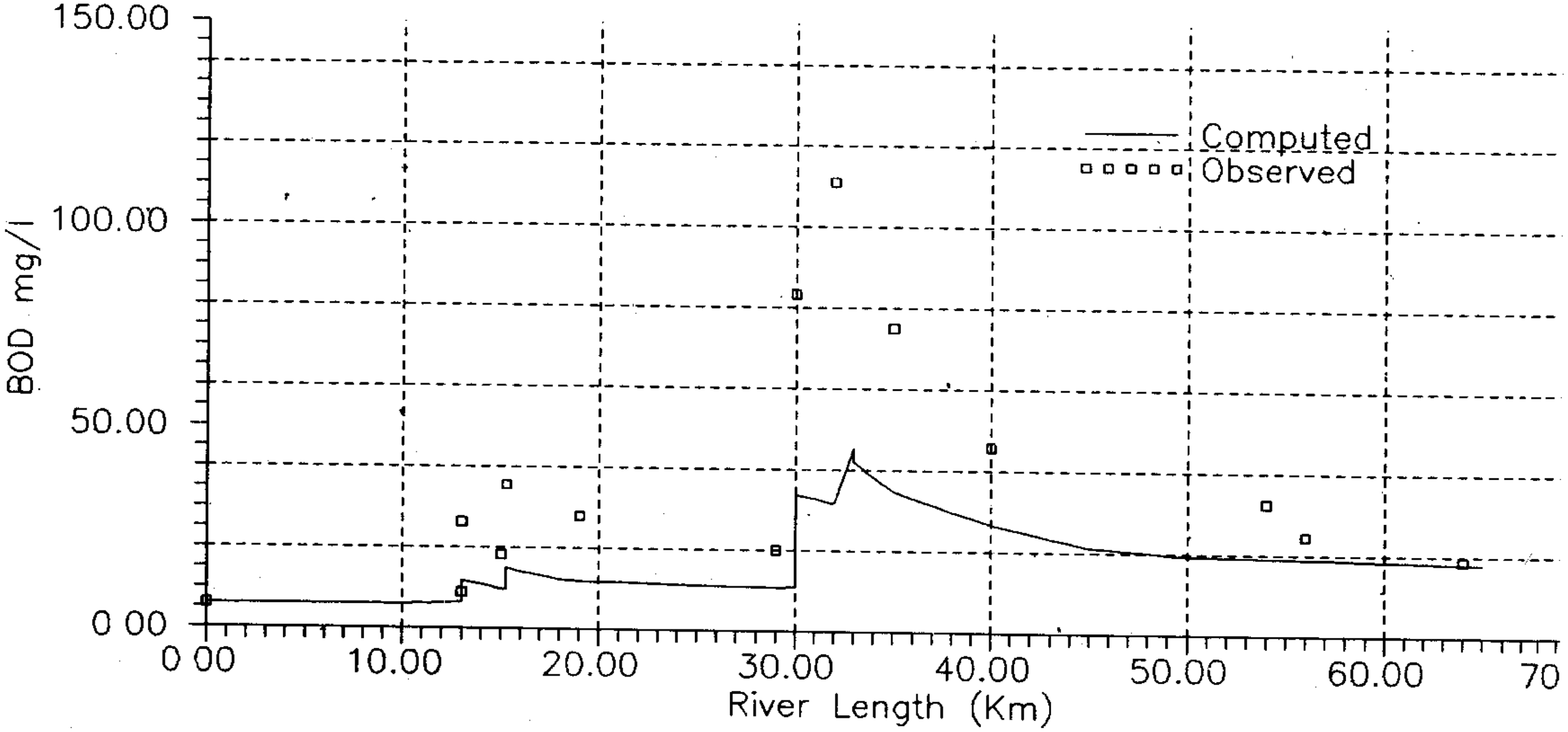


Fig.24 - Improvement of BOD with Flow Augmentation of 4.21 times of Headwaters flow

improve the health of the river, two management options have been attempted; i) augmentation of flow, and ii) waste load allocations. Improvement of Dissolved Oxygen (DO) and reduction of Bio-Chemical Oxygen Demand (BOD) have been considered as the reference parameters of analysis.

For flow augmentation i.e., dilution of polluted river water, headwaters source has been considered as the source of dilution as there is no other source nearby except the source of Upper Ganga Canal waters which is quite distance away. Quality of augmented flow has been considered similar to the quality of headwaters flow. As mentioned above, the river has two distinct DO sags , thus the problem has been addressed in two ways; i) flow augmentation required to improve the DO deficit for the first DO sag where the critical deficit is observed at R.Km. 45 from confluence with DO value of 1.74 mg/l., and ii) flow augmentation required to improve the DO deficit for the second DO sag where critical stretch with "0" DO value has been observed between R.Km.33 to R.Km. 12.0. For the first, it is observed that requirement of flow augmentation follows a exponential increase for increasing value of DO,(Fig. 25) in the following form :

$$Y = 2.51289 \exp(0.353121 * X) \qquad (\text{Fig. } 25)$$

where, Y = Flow augmentation required (m^3/sec) for any desire DO value,
X = Desire DO value (mg/l).

For example, if the DO at critical point (R.Km. 45) is to be improved from 1.74 mg/l to 5.0 mg/l, the requirement of flow augmentation would be 4.21 times of headwaters flow i.e. the headwaters flow must be 15.5 m^3/sec; that indicates an additional flow of 11.82 m^3/sec above the existing headwaters flow of 3.68 m^3/sec would be required for dilution. An indicative of improvement of concentration profiles for DO and BOD against the 4.21 times of augmented flow has been given Fig. 23 and 24 respectively. Though considerable improvement of DO (5.0 mg/l - 1.74mg/l), 3.26 mg/l, is observed in the first DO sag for 4.21 times augmented flow , however, not much improvement is observed for the second DO sag against this augmented flow (Fig. 23). The minimum deficit of DO corresponding to this augmented flow is observed at R.Km. 20 (from confluence) with DO value of 0.45 mg/l.

In the similar way, the requirement of flow augmentation to improve the DO deficit for the second DO sag has been worked out,

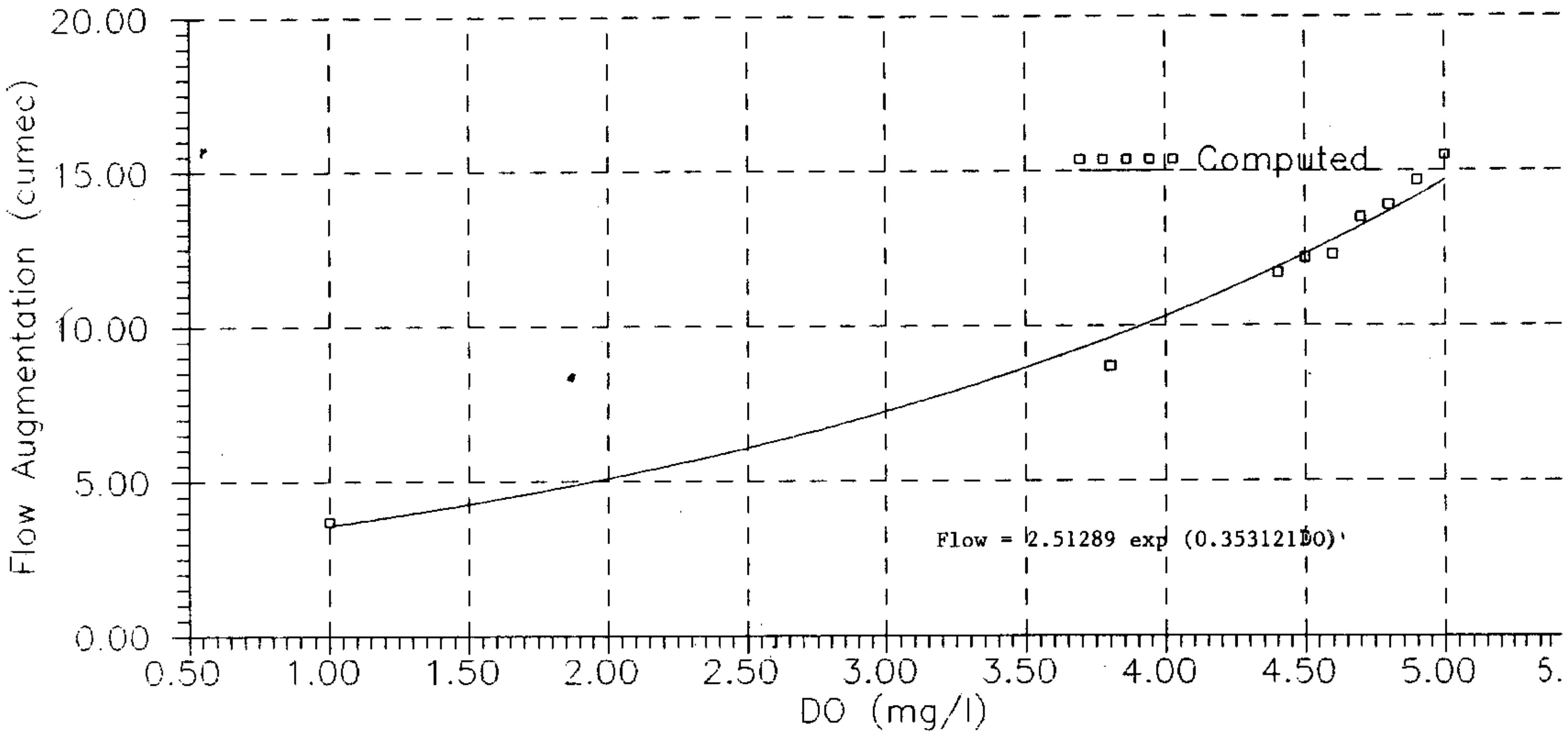
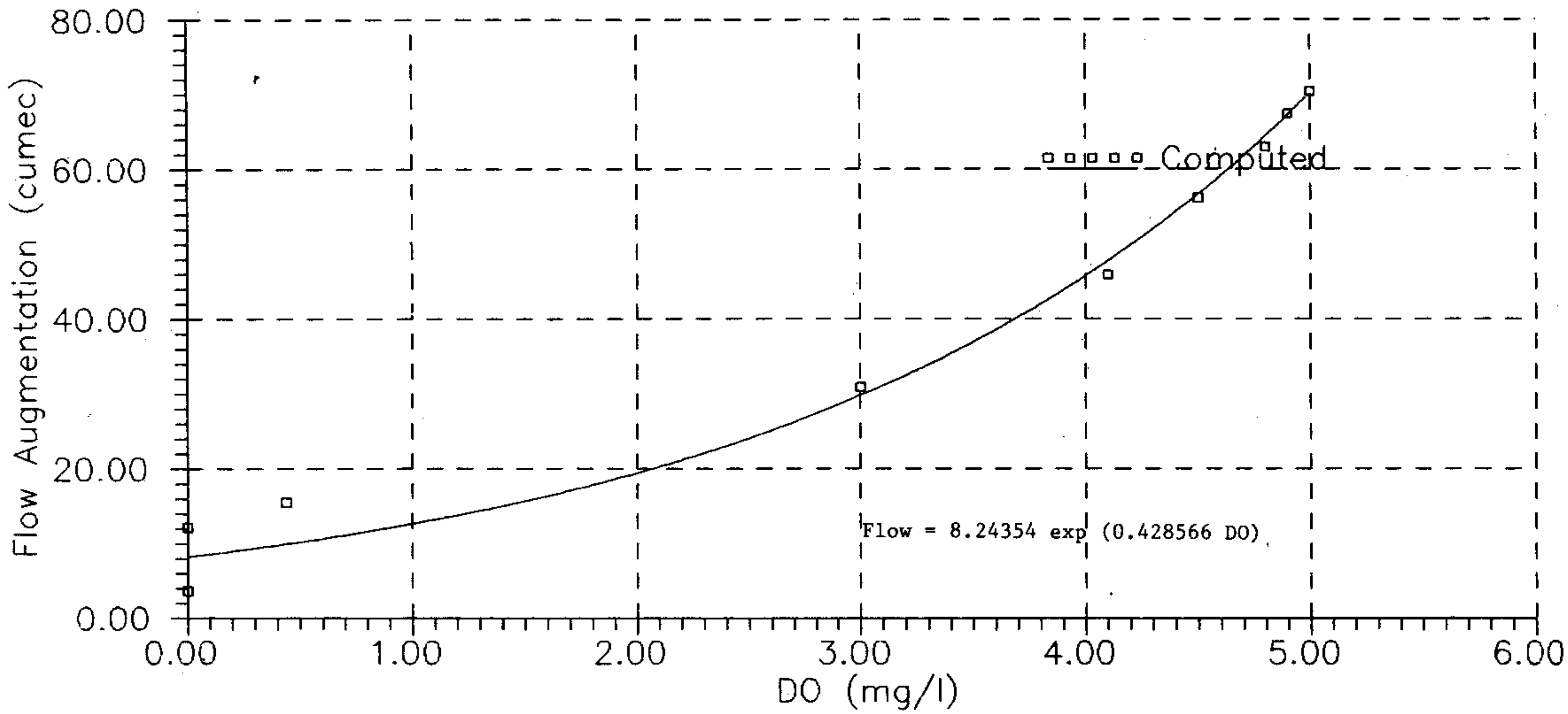


Fig. 25 - Variation of DO (mg/l) for Different
Flow Augmentation at Km point 20 from Malira Bridge

Fig.29 - Reduction of BOD Load at critical Point (km. 20 from Malira) on Reduction of Municipal Effluents BOD Load



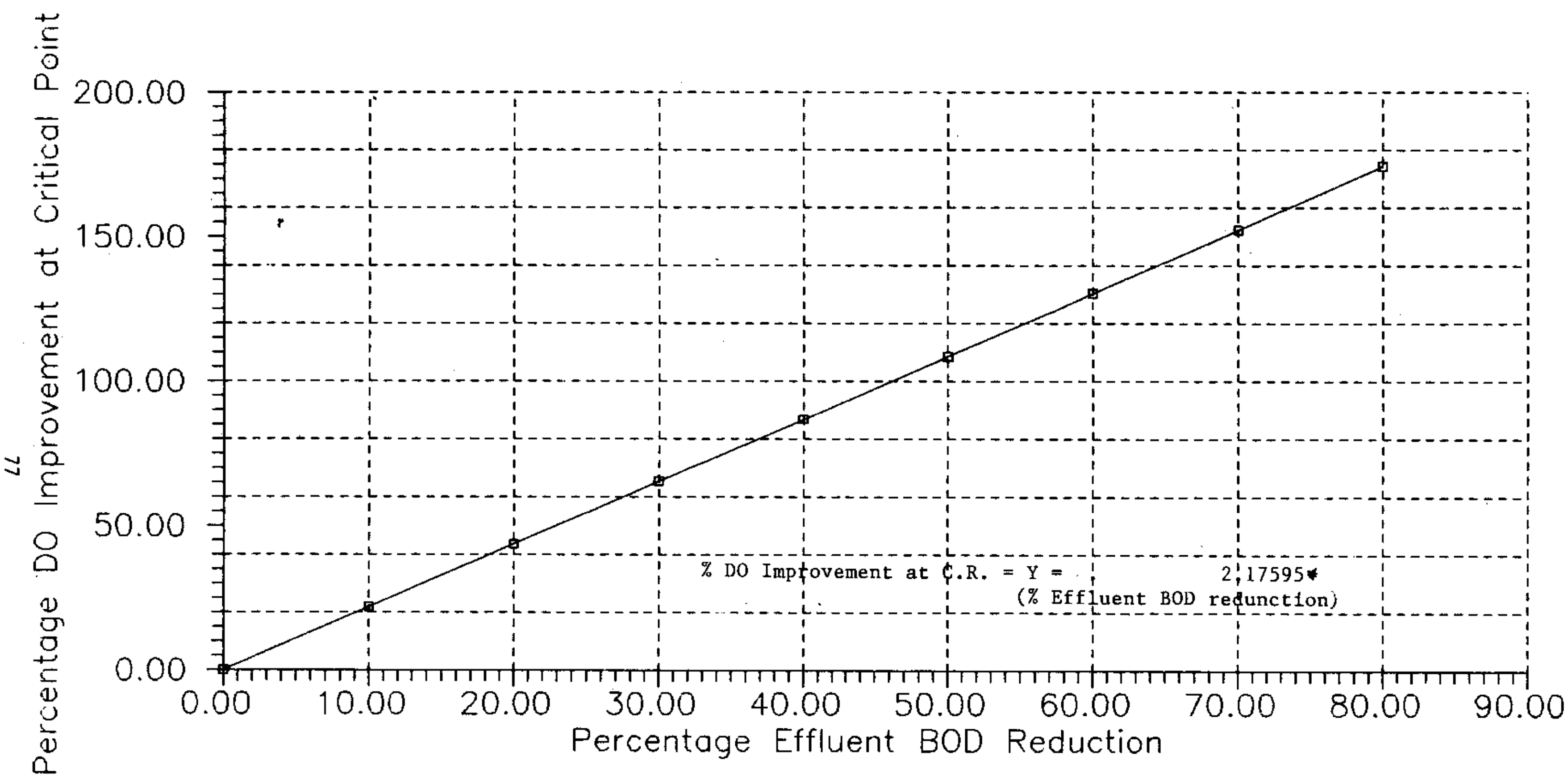


Fig.27 - Improvement of DO at Critical Point (km. 20 from Malira) on Reduction of Municipal Effluents BOD Load

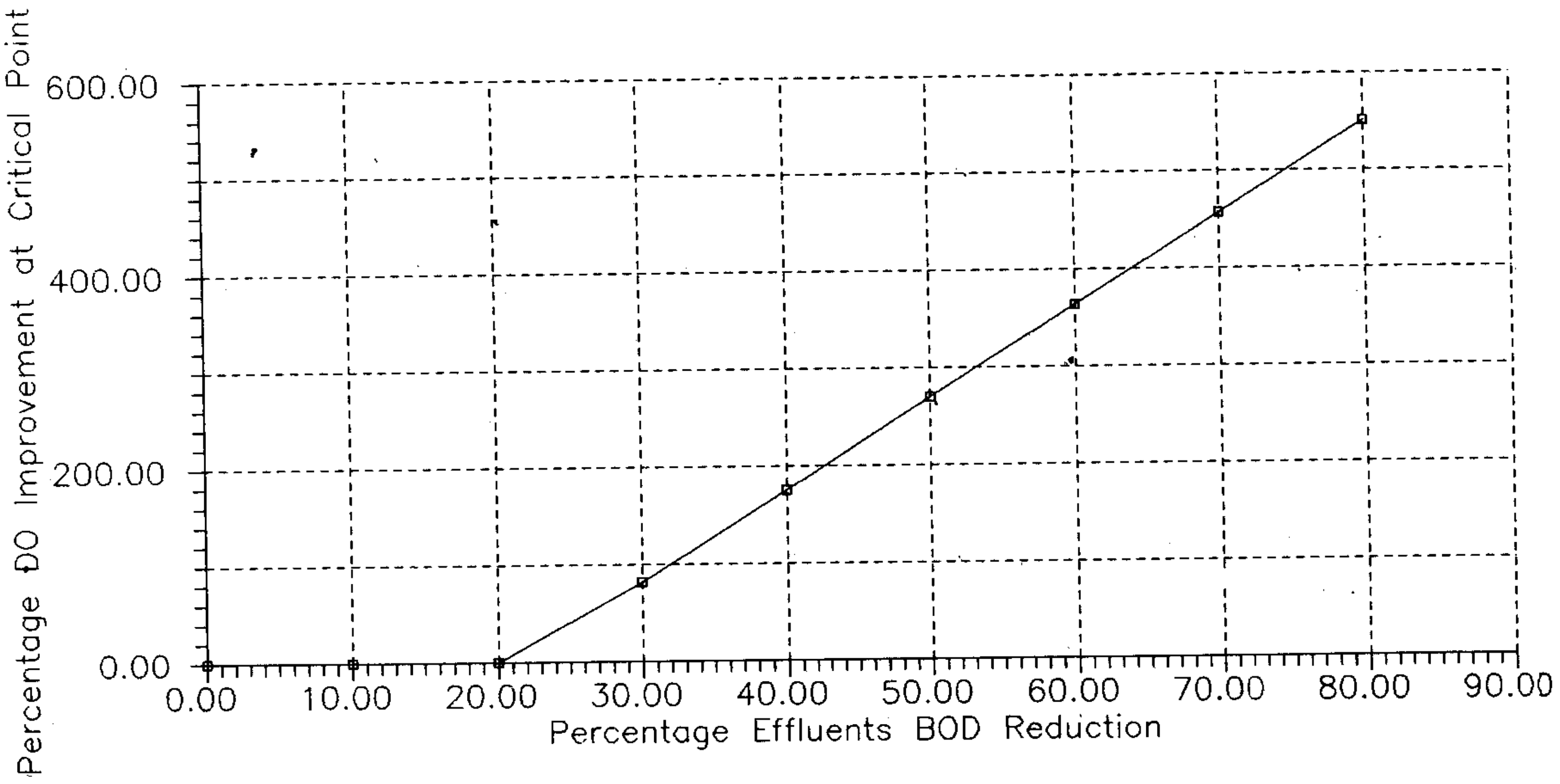


Fig. 28 - Improvement of DO at critical Point (km. 45 from Malira) on Reduction of Industrial & Sugar Mills Effluents BOD Load

and indicates that requirement flow augmentation increases exponentially with increasing DO values in the following form:

$$Y = 8.24354 \exp(0.428566 * X) \qquad \text{(Fig. 26)}$$

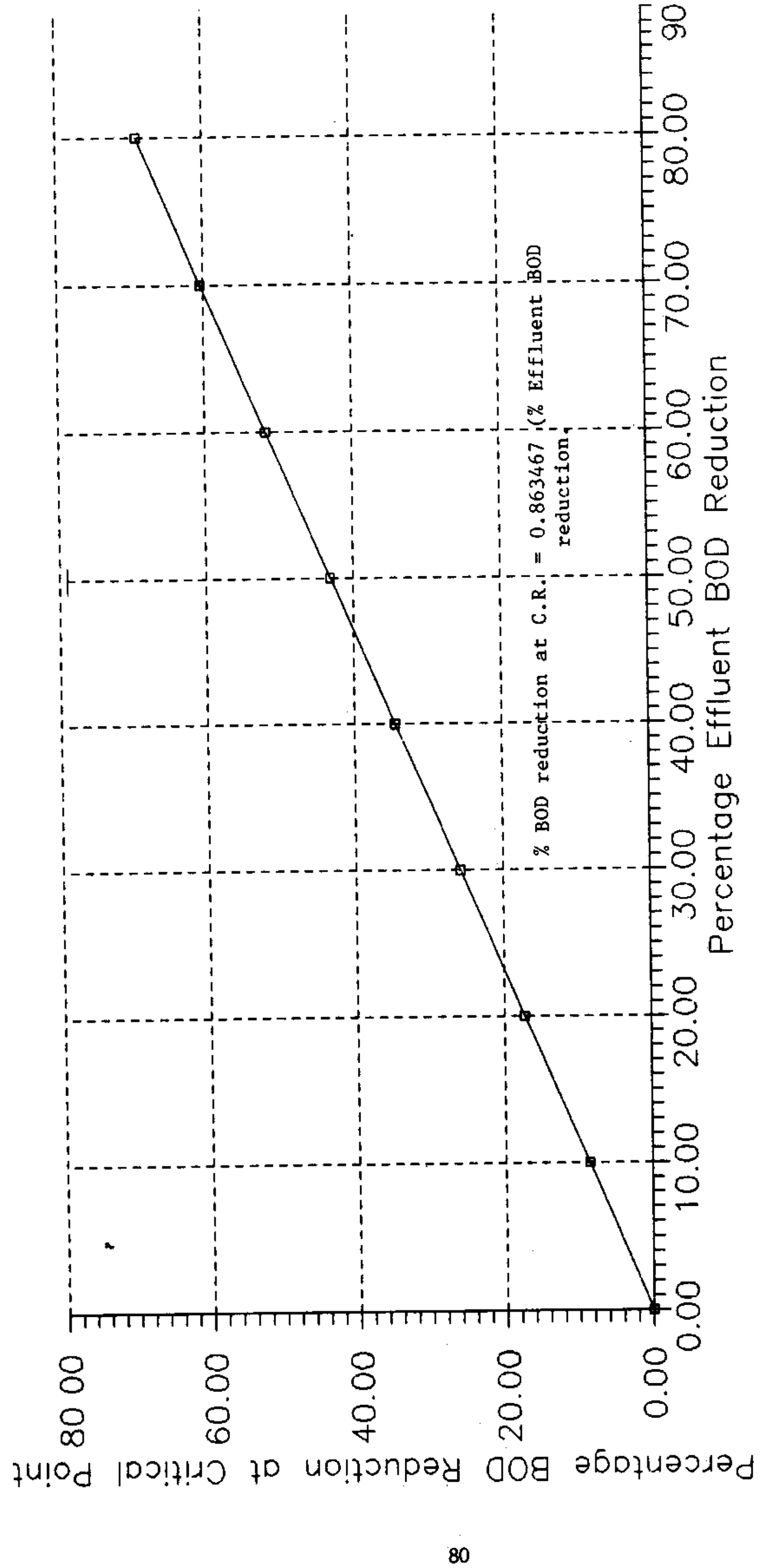
where, Y = flow augmentation required (m^3/sec) for any desire DO value,
X = desire DO value (mg/l).

Which indicates that, for example, if DO for the second sag is to be improved from existing "0" DO values to 5.0 mg/l, the requirement of flow to be augmented would be 70.26 m^3/sec, i.e., 19.1 times of headwaters flow. That is, additional 66.58 m^3/sec over the headwaters flow of 3.68 m^3/sec would be required. Requirement of flow augmentation for different desire DO values has been given in Fig. 26.

Augmentation of flow is,ofcourse, better option of management of pollution where control and regulation of pollution is very difficult and pollution control acts are not that much stringent. And also flow required for augmentation is easily available, However, in cases where quality of wastewaters that are discharged to the river/stream contain high pollution load and constituents are toxic in nature, control of pollution at sources i.e., treatment of wastewaters , would always be preferable. In the present study, attempts have also been made to compute the permissible Waste loads that the stream can carry safely without creating any pollution hazards to the downstream.

The problem has been addressed in two ways; i) allocation of waste loads for the municipal wastewaters entering the river at Km. point 52 and Km. point 49.75 to maintain the river health downstream of discharge, and ii) allocation of wastewaters for the industrial and sugar mill drains joining the river at Km. point 35 and Km. point 33 in order to keep the self purification capacity of the river. Dissolved Oxygen and Bio-Chemical Oxygen Demand has been taken as the main components of analysis.

Fig. 27 and 29 reflect respectively; the percentage improvement of DO and percentage reduction of BOD at the critical point i.e., Km.45 from confluence, of first DO sag on percentage reduction of municipal effluents BOD load. That means, for example, if effluent BOD of municipal drains at Km. point 52 and Km. point 49.75 are treated to an extent of 80% of the observed BOD, i.e., 260 mg/l and 255 mg/l of BOD are removed before being discharged to



% BOD reduction at C.R. = 0.863467 (% Effluent BOD reduction)
Percentage Effluent BOD Reduction
Percentage BOD Reduction at Critical Point
0.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00
80.00
90
0.00
20.00
40.00
60.00
80.00

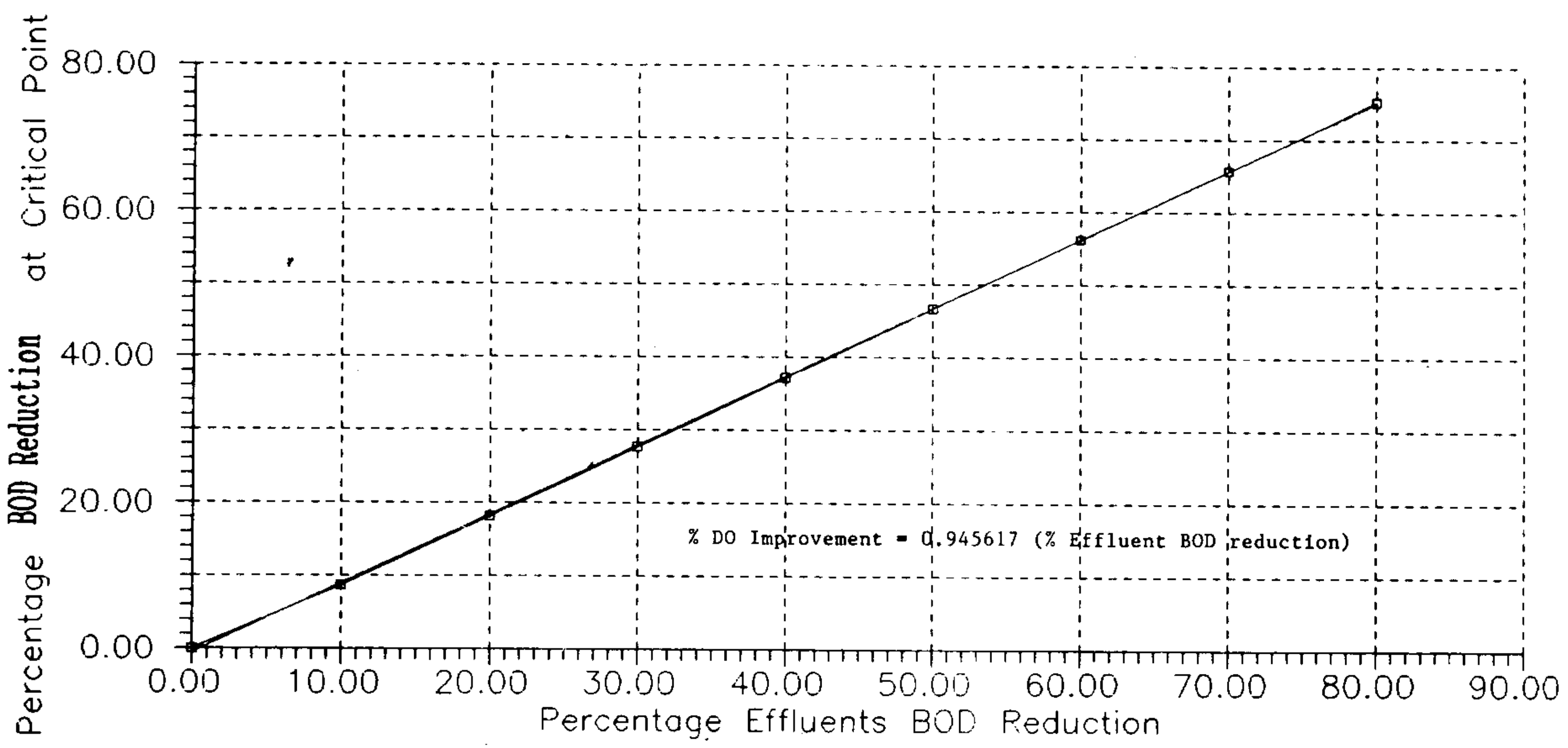


Fig. 30 - Reduction of BOD at critical Point (km. 45 from Malira) on Reduction of Industrial & Sugar Mills Effluents BOD Load

the stream, the percentage improvement of DO and percentage reduction of BOD at the critical point observed at Km. point 45 (from confluence) would respectively be 174% and 69% on the observed DO (1.74 mg/l) and BOD (25.2 mg/l) values. It is observed that 80% of the effluent BOD is to be removed before discharging the wastewaters in to the stream in order to improve the existing DO level from 1.74 mg/l to 5.0 mg/l. Fig 27 also shows the variation of percentage improvement of DOs at critical point on different percentage reduction of Effluents BODs follows a linear equation in the following form :

Y = 2.17595 * X (Fig. 27)

where, Y = % DO improvement at Critical Point

$$= \frac{((\text{Computed DO at C.R} - \text{existing DO at C.R}) * 100)}{\text{existing DO at C.R}}$$

X = % reduction of effluent BOD.

The percentage reduction of BOD at critical point has also found to follow linear relationship with percentage reduction of effluent BOD (Fig. 29) in the following form :

Y = 0.863467 * X (Fig. 29)

where, Y = % reduction of BOD at Critical point

$$= \frac{(\text{Existing BOD at C.R} - \text{Computed BOD at C.R}) * 100}{\text{Existing BOD at C.R}}$$

X = % reduction of effluent BOD.

In the similar way, Computations of waste load allocation for industrial and sugar mill drains have also been performed and given in Fig. 28 and 30. The critical point for the second DO sag being R. Km.20 (from confluence), the improvement on DO and reduction on BOD over the existing DO and BOD have been taken with reference to this point. It is observed that about 85 % of industrial and sugar mill wastewaters BOD is to removed before discharging the wastewaters in to the river to improve the existing DO level from "0" to 5.0 mg/l. Fig. 28 represents that DO deficit at critical point does not have any impact on reduction of effluent BOD till 20% reduction, however, considerable reduction on BOD upon the effluent BOD reduction is observed (Fig. 30). In both the cases, linear relationships have been observed between the reduction of effluent BOD and improvement of DO and reduction of BOD at critical

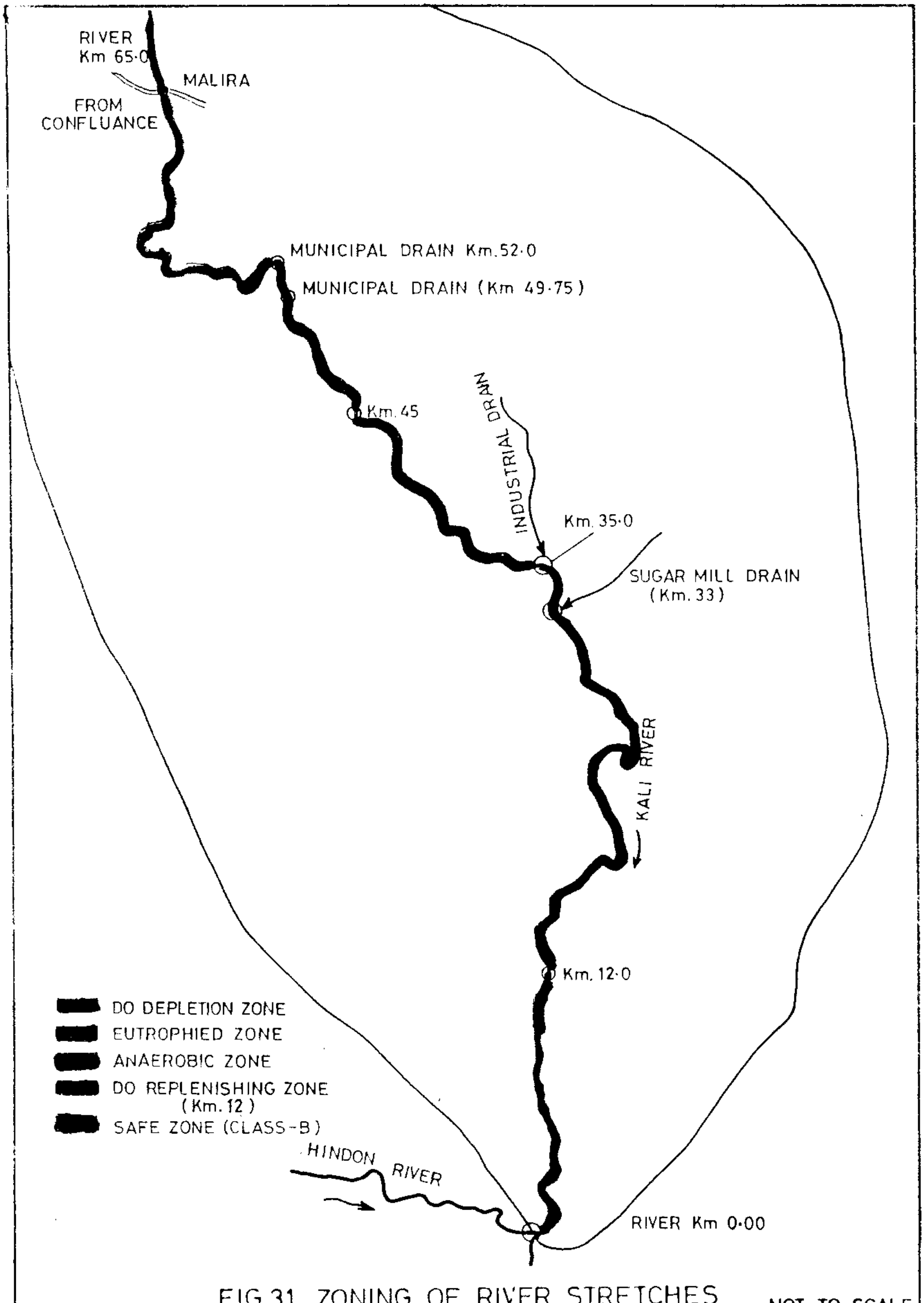


FIG. 31 ZONING OF RIVER STRETCHES

NOT TO SCALE

point Km. 20. From Fig. 30, the percentage BOD reduction at critical point with percentage effluent BOD reduction follows the undermentioned relation:

$$Y = 0.945617 * X \qquad \text{(Fig. 30)}$$

where, Y = percentage BOD reduction at Critical Point

$$= \frac{\text{(existing BOD at C.R - Computed BOD at C.R.)}*100}{\text{existing BOD at C.R}}$$

X = % effluent BOD reduction

In order to maintain the self purification capacity of the stream the effluent quality of wastewaters in terms of BOD could be as follows :

i) Municipal drain near Nayazupura (R.Km. 52) : 65 mg/l
ii) Municipal drain near Shamli Bridge (R. Km. 49.75): 63 mg/l
iii) Industrial drain at R. Km. 35 : 120 mg/l
iv) Sugar mill drain near Mansurpur Bridge (Km. 33): 254.0 mg/l

9.0 CONCLUSIONS

Based on the physical observations of site and analysis of data and results following inferences have been drawn :

i) effectiveness of QUAL2E for modeling of water quality parameters has been tested and 8 water quality constituents, namely; DO, BOD, NH_3-N, NO_2-N, NO_3-N, PO_4, TDS and Temperature have been simulated with reasonable accuracy,

ii) the existing status of the Kali river water quality indicates that the river which contains fresh water before mixing of wastewaters, is being transformed to a sewer by the continuous discharge of municipal, industrial and sugar mills effluents,

iii) to improve the health of the river atleast to an extent upto the self purification capacity of the river, immediate attention is necessary. Between the two alternative management approaches as touched upon in this study, Waste Load Allocation i.e., control of pollution at sources, seems to be the better option than the flow augmentation as because there is no source of flow nearby for augmentation,

iv) as two sets of data collected in same season of a year guided the calibration and validation of the model and considered for further analysis, computed results reported in the study could be considered as an indicative. Details study based on the data of different seasons are therefore recommended to select appropriate management options,

v) within the study stretch of 65 Km. the river faces two distinct DO sags; one -because of the discharge of municipal wastewaters is found depleted to 1.74 mg/l and another because of discharge of industrial and sugar mills wastewaters is found depleted to "0" which continues for a stretch of about 20 Km, and increases to about 1.9 mg/l before mixing with Hindon river,

vi) computation of waste load allocation shows that about 80% of effluent BOD reduction for municipal wastewaters and about 85% of effluent BOD reduction for industrial and sugar mill wastewaters would be necessary for maintaining the self purification capacity of the river. While in case of flow augmentation, about 4.21 times of headwaters flow, i.e., additional 11.82 m^3/sec over the headwaters flow of 3.68 m^3/sec, for municipal wastewaters and about 19.1 times of headwaters flow i.e., additional 66.58 m^3/sec over the headwaters flow of 3.68 m^3/sec, for industrial and sugar mills wastewaters, would be necessary to harness the pollution.

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Title of the Report : Water Quality Modeling of River Kali Using QUAL2E.

Director : Dr. S.M. Seth

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Divisional Head : Dr. K.K.S. Bhatia, Scientist 'F'

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