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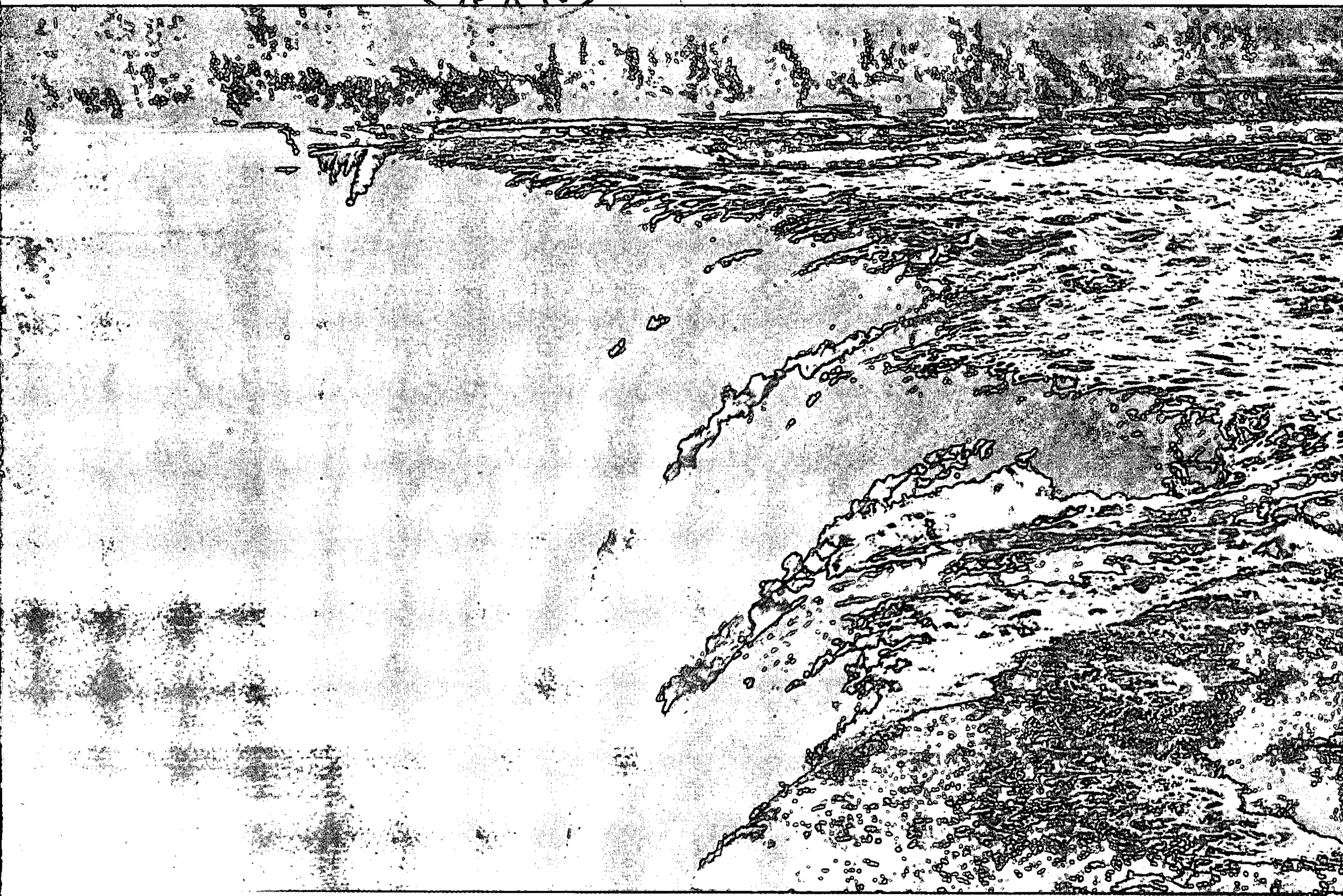

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Dissolved Oxygen Modelling of the St. Croix River — Comparison of Models

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Abstract

A comparative study was carried out to determine the applicability to water quality modelling of the numerical One-Dimensional Hydrodynamic model as opposed to the HEC-2 and WATQUAL models combined. Results of the simulations with the one-dimensional model showed this model to be as good as the HEC-2 and better than the WATQUAL. Comparing the versatility of the alternatives, we find that the One-Dimensional Hydrodynamic model performs to great advantage.

Résumé

On a effectué une étude comparative pour déterminer l'applicabilité du modèle hydrodynamique, numérique et unidimensionnel à la modélisation de la qualité de l'eau, par rapport aux modèles HEC-2 et WATQUAL. Les résultats des simulations faites avec le modèle unidimensionnel ont démontré que ce modèle était tout aussi satisfaisant que le modèle HEC-2 et supérieur au modèle WATQUAL. En comparant les divers modèles du point de vue de leur versatilité, nous avons constaté que le modèle unidimensionnel était le plus avantageux.

Dissolved Oxygen Modelling of the St. Croix River Comparison of Models

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INTRODUCTION

Following the completion of the report entitled "Dissolved Oxygen Modelling of the St. Croix River" (Boutot and Clair, 1981), it was decided that a further investigation of the water quality modelling aspects of the river by means of a more sophisticated model, the One-Dimensional Hydrodynamic model, was desirable. Therefore, a comparison study was carried out in 1984, using the same input data to determine how this model performs as opposed to the simpler HEC-2 and WATQUAL models used before.

Graphs of re-surveyed cross sections, plotted cross sections and a computer printout of the simulation run may be obtained from the authors on request.

MODEL DESCRIPTION

The numerical One-Dimensional Hydrodynamic model was developed at MIT in the early 1970s. Since then, it has been applied to numerous projects in Canada, such as the St. Lawrence River Study, the Lower Fraser River Sediment Study and, most recently, the Peace-Athabasca Delta Study.

The one-dimensional model is capable of simulating a transient flow regime in a network of channels which may consist of embayment storage areas, various hydraulic control structures and tidal boundary conditions. Transient water quality simulation or prediction can be carried out together with the hydraulic aspects, and the entire package represents a comprehensive model for water quantity and quality studies.

The basic equations to describe the process of long wave propagation in open channels are the continuity and the momentum equations, and these are solved by an accurate implicit finite element method in this model. The basic governing equation for water quality simulation is a conservation of mass equation which includes the transport process of advection and dispersion. It also includes reaction processes where mass is added to or removed from the system.

STUDY AREA

The St. Croix River forms the international boundary between Canada and the United States, having an approximate reach of 124 km. In most of the watershed and for the upper 110 km of the St. Croix River the water quality is considered pristine. The region of immediate water quality concern is the lower 14-km reach between Woodland, Maine, and Milltown, New Brunswick. The problem in this region is caused by a major industrial discharge at Woodland and, to a lesser extent, by municipal discharge and combined sewer overflows from the towns of Woodland and Baileyville, Maine. The Georgia Pacific Corporation operates a kraft pulp mill at Woodland, which produces fully bleached kraft pulp and discharges effluent through a secondary treatment facility into the St. Croix River. This effluent has a high organic load that has considerable influence on the river system oxygen demand.

The river bottom is mainly gravel in the fast-flowing sections, with organic detritus accumulations in pools and protected areas. The river system was divided into 31 original cross sections, based on work by the U.S. Army Corps of Engineers (Fig. 1). A more detailed description of the river basin characteristics is presented in Boutot and Clair (1981).

St. Croix Model System Configuration

The river system was divided into four reaches containing 31 original cross sections between 150 and 1200 m long, based on work by the U.S. Army Corps of Engineers (1967). The first reach upstream (Reach 6666) starts at chainage section No. 1000 and terminates at No. 11000. The next reach downstream (Reach 7777) is between chainage station No. 11000 and 27500. Reach 8888 lies between only two original cross sections, i.e., from No. 27500 to 30000. The last reach (Reach 9999) is from 30000 to 46000. In Figure 1, No. 32 represents the inflow from the Georgia-Pacific Dam, and No. 33 stands for the waste loading from the Georgia-Pacific Mill.

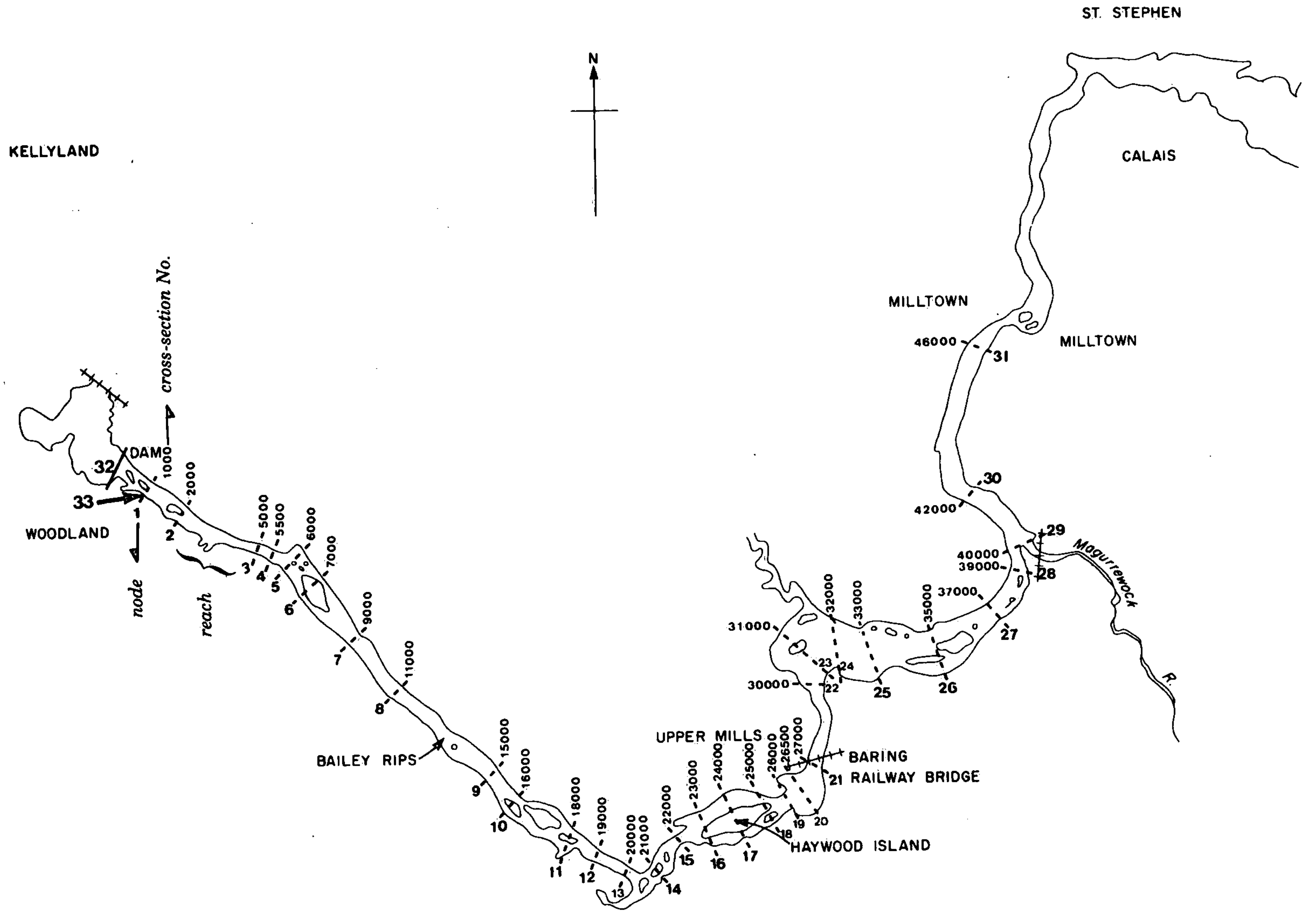


Figure 1. Map of study reach.

INPUT DATA

The same input data sets were used in the One-Dimensional Hydrodynamic modelling as in the WATQUAL modelling, except that the original cross sections for stations 1000, 2000, 11000, 27000 and 30000 were replaced by eight cross sections for stations 1000, 2000, 11000, 27000, 27500 at Baring gauge, 28000, 28500 and 30000, which were surveyed by the Water Survey of Canada in August 1981. A comparison of five of these cross sections with their respective original 1967 surveys by the U.S. Army Corps of Engineers showed that they do not agree well in top width and/or cross-sectional area. The discrepancies noted may be attributed to either the removal of bottom sludge deposits since 1967, or the 1981 cross-sectional measurements not being taken at the exact locations. Since the cross sections could only be located by identification on the large-scale maps, it is possible that positioning errors of the sections along the river could be as great as 60 m.

The One-Dimensional Hydrodynamic model can handle more than ten water quality parameters, including salinity, temperature, biochemical oxygen demand (BOD), dissolved oxygen (DO), fecal coliforms, lignins and various nutrients. In this study, only BOD and DO were simulated to compare with the results by WATQUAL.

Nineteen different flow conditions were chosen for WATQUAL study from Tables 1 and 2.

CALIBRATION

Water Quantity

Before inserting water quality parameters with hydraulic input data, the one-dimensional model was first calibrated to match the water surface profile. Values of the Manning "n" roughness coefficient can be seen in Table 3. These values represent calibration at every cross section of the river network. The water

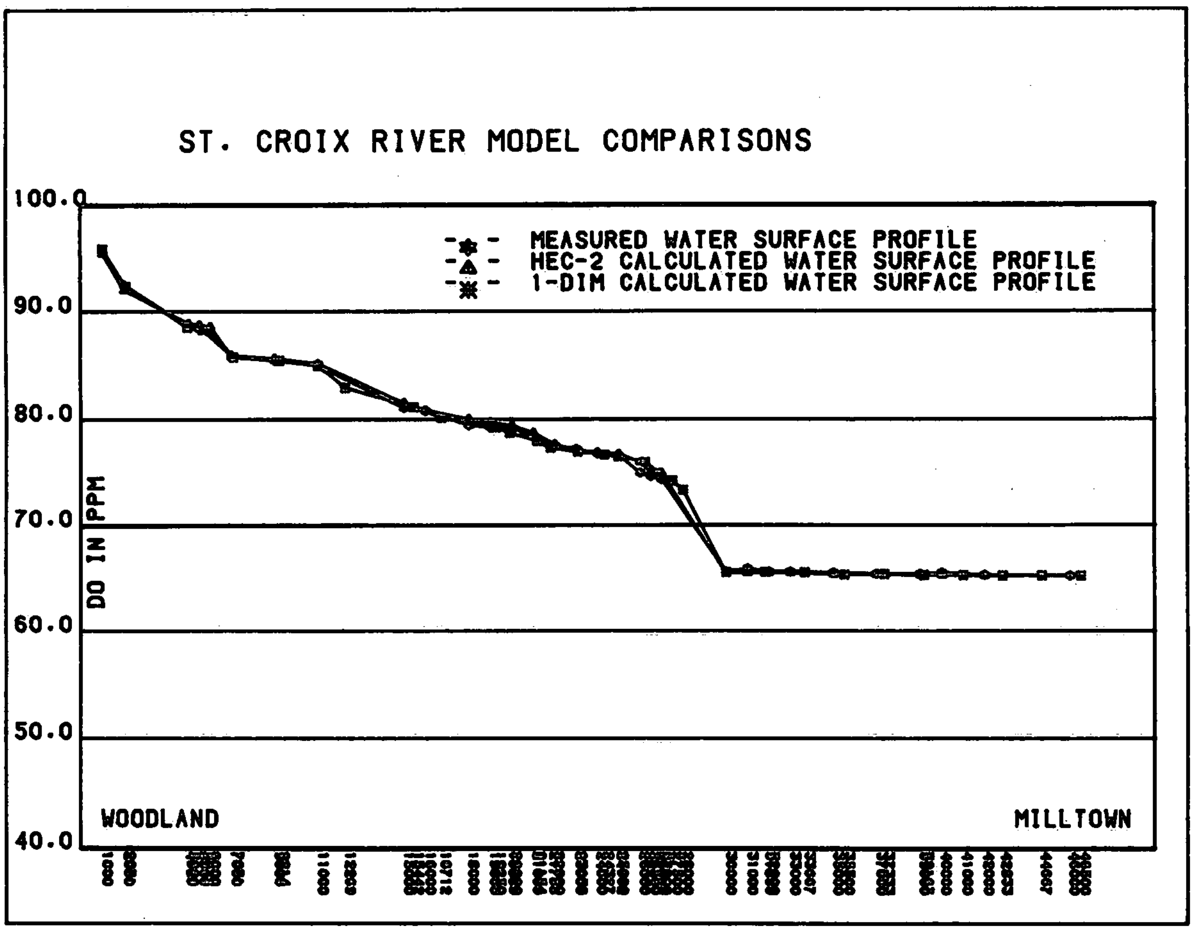


Figure 2. St. Croix River model comparisons.

surface profile calculated by the one-dimensional model versus the measured water surface profile was plotted onto a Calcomp Plotter and is presented in Figure 2.

This figure shows the 1-DIM calculated water surface profile vs. HEC-2 calculated water surface profile against *in situ* measurements. The plot demonstrates that both models can simulate the water line with great accuracy after the refined calibration has been made. It also shows a good mathematical simulation where slopes are greater, i.e., between sections 1000 to 2000 and 27500 to 30000. Where slopes are very small, calibration can be made almost identical with the measured water surface profile.

Water Quality

The water quality models were calibrated by estimating values for the organic decay rate to coincide with measured DO.

Table 1. Water Quantity and Quality Values Used in Calibrating WATQUAL and ONED Models, 1979-1980

Date	River discharge (m^3/s)	Five-day BOD loading (kg/day)	Station	Temperature (°C)	Mean sectional DO (ppm)
80-08-28	15.5	5900	Kellyland	22.5	8.6
			Woodland	22.5	—
			Bailey Rips	22.5	—
			Haywood Is.	22.5	—
			Baring	22.5	—
			Milltown	22.5	0.0
80-06-24	32.6	2287	Kellyland	22.5	8.7
			Woodland	—	—
			Bailey Rips	—	—
			Haywood Is.	—	—
			Baring	23.0	7.6
			Milltown	23.0	5.9
80-02-27	53.6	1555	Kellyland	0.5	13.0
			Woodland	—	—
			Bailey Rips	—	—
			Haywood Is.	—	—
			Baring	—	12.9
			Milltown	—	12.8
79-10-02	36.6	2486	Kellyland	—	10.5
			Woodland	—	—
			Bailey Rips	—	—
			Haywood Is.	—	—
			Baring	8.6	8.8
			Milltown	8.2	8.4
79-08-21	80.0	1552	Kellyland	17.0	9.5
			Woodland	17.0	8.5
			Bailey Rips	17.0	8.5
			Haywood Is.	17.0	8.6
			Baring	17.0	8.0
			Milltown	17.5	7.7
79-07-12	68.9	3081	Kellyland	22.0	8.6
			Woodland	23.0	8.0
			Bailey Rips	24.0	7.2
			Haywood Is.	25.0	6.9
			Baring	23.0	6.4
			Milltown	21.5	6.8
79-06-05	338.2	1980	Kellyland	16.5	12.8
			Woodland	17.0	8.8
			Bailey Rips	—	—
			Haywood Is.	—	—
			Baring	15.1	9.2
			Milltown	15.1	9.2

Simulations by the one-dimensional model were done by using average reaeration coefficient for each reach as opposed to one value per cross section with WATQUAL runs. Values of the sediment oxygen demand (SOD) were not used, as these parameters are not incorporated in the model.

MODEL COMPARISONS (ONE-DIMENSIONAL VS. HEC-2)

HEC-2

The computational procedure used in this model is commonly known as the standard step method and is used to compute the backwater profile in river channels. The following two equations are solved by an iterative procedure (the standard step method) to calculate an unknown water surface elevation at a cross section:

$$WS_2 + \frac{\alpha_2 V_2^2}{2g} = WS_1 + \frac{\alpha_1 V_1^2}{2g} + h_e \quad (1)$$

$$h_e = L\bar{S}_f + C \left| \frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g} \right| \quad (2)$$

Table 2. Water Quantity and Quality Values Used in Calibrating WATQUAL and ONED Models, 1981–1982

Date	River discharge (m^3/s)	Five-day BOD loading (kg/day)	Station	Temperature (°C)	Mean sectional DO (ppm)
81-01-22	61.2	1830	Woodland	6.6	13.6
			Baring	6.6	9.8
			Milltown	6.6	11.7
81-05-12	35.99	1580	Woodland	16.5	9.8
			Baring	16.5	9.7
			Milltown	16.5	8.6
81-06-25	94.59	1141	Woodland	25.5	8.9
			Baring	25.5	7.9
			Milltown	25.5	7.1
81-07-29	65.06	2630	Woodland	28.0	8.0
			Baring	28.0	8.2
			Milltown	28.0	6.9
81-10-01	240.7	386	Woodland	14.0	10.1
			Baring	14.0	10.5
			Milltown	14.0	10.0
81-12-10	243.1	2072	Woodland	4.0	13.3
			Baring	4.0	13.5
			Milltown	4.0	13.1
82-02-12	120.4	2414	Woodland	6.0	14.40
			Baring	6.0	13.90
			Milltown	6.0	13.30
82-03-25	82.5	2023	Woodland	17.0	11.7
			Baring	17.0	12.1
			Milltown	17.0	12.2
82-04-29	220.2	3209	Woodland	10.0	11.8
			Baring	10.0	11.9
			Milltown	10.0	11.7
82-06-03	40.95	6810	Woodland	28.0	8.4
			Baring	28.0	7.8
			Milltown	28.0	6.9
82-07-22	31.0	2741	Woodland	30.0	7.7
			Baring	30.0	6.2
			Milltown	30.0	6.0
82-10-14	55.8	1516	Woodland	18.5	10.5
			Baring	18.5	9.7
			Milltown	18.5	9.7

where WS_1, WS_2 = water surface elevations at ends of reach

V_1, V_2 = mean velocities (total discharge ÷ total flow area) at ends of reach

α_1, α_2 = velocity coefficients for flow at ends of reach

g = acceleration of gravity

h_e = energy head loss

L = discharge-weighted reach length

$\bar{S}_f$ = representative friction slope for reach

C = expansion or contraction loss coefficient.

This method applies Bernoulli's principle for the total energy at each cross section, and Manning's equation for friction head loss between cross sections. In the program, average friction slope for a reach between two cross sections is determined in terms of the average of the conveyances at the two ends of the reach. Other losses are computed using one of several methods. The critical water surface elevation corresponding to the minimum specific energy is computed using an iterative process.

The expression for total energy H may be written as:

$$H = y + z + v^2/2g \qquad (3)$$

where y = vertical distance from the bed to the water surface

z = height of the bed above datum

$v^2/2g$ = velocity head

$y + z$ = WS = water surface elevation.

Table 3. Refined Roughness Coefficients for Each Cross Section

Reach	Original Cross-section No.	Manning's "n" values	Interpolated Cross-section No.	Manning's "n"
6666	1000	0.050	27600	0.050
	2000	0.060	27700	0.060
	5000	0.040	27800	0.065
	5500	0.020	27900	0.060
	6000	0.050	28000	0.065
	7000	0.020	28100	0.065
	9000	0.020	28200	0.085
7777	11000	0.030	28300	0.085
	15000	0.050	28400	0.085
	16000	0.050	28500	0.085
	18000	0.050	28600	0.110
	19000	0.050	28700	0.110
	20000	0.050	28800	0.110
	21000	0.050	28900	0.110
	22000	0.050	29000	0.110
	23000	0.050	29100	0.110
	24000	0.050	29200	0.110
	25000	0.050	29300	0.110
	26000	0.050	29400	0.110
	26500	0.050	29500	0.110
	27000	0.050	29600	0.110
8888	27500	0.050	29700	0.110
	*		29800	0.110
9999	30000	0.035	29900	0.110
	31000	0.035		
	32000	0.035		
	33000	0.035		
	35000	0.035		
	37000	0.035		
	39000	0.035		
	40000	0.035		
	42000	0.035		
	46000	0.035		

*The interpolated cross-section numbers are given in column 4.

The program computes and plots (by printer) the water surface profile for river channels of any cross section for either subcritical or supercritical flow conditions. The effects of various hydraulic structures such as bridges, culverts, weirs, embankments and dams may be considered in the computation. The principal use of the program is to determine profiles for various frequency floods for both natural and modified conditions. The latter may include channel improvements, levees and floodways. Input may be in either imperial or metric units.

One-Dimensional Hydrodynamic Model

The process of propagation of long waves in open channels is described by the Saint Venant equations. These equations represent the conservation of mass and momentum of flow of a fluid in the channel (Harleman and Lee, 1967).

Derivations of the Saint Venant equations by the "material" method formulations were made by Harleman and Lee (1967). The equations in terms of average velocity, v, and water surface elevation, z, are given below (see Figs. 3 and 4 for notation). The *continuity equation* is

$$\underset{1}{B\frac{\partial z}{\partial t}} + \underset{2}{Bv\frac{\partial z}{\partial x}} + \underset{3}{A\frac{\partial v}{\partial x}} + \underset{4}{v\frac{\partial A}{\partial x}} \Bigg|_{z\,=\,\text{const.}} = \underset{5}{q_L} \qquad (4)$$

The *momentum equation* is

$$\underset{6}{\frac{\partial v}{\partial t}} + \underset{7}{v\frac{\partial v}{\partial x}} + \underset{8}{\frac{vq_L}{A}} = \underset{9}{g(S_0} - \underset{10}{\frac{v|v|}{C_z^2R})} - \underset{11}{g\frac{\partial h}{\partial x}} \qquad (5)$$

Continuity Equation

The various terms contained in the continuity equation are defined below:

(1) "Rate of rise" which gives the storage changes owing to water surface elevation changes with time.
(2) "Prism storage" (Fig. 5) owing to variation in velocity with space.
(3) and (4) "wedge storage" owing to aerial variations in velocity with space (Fig. 5).
(5) "Lateral inflow" which gives the net mass change spatially and temporally beyond the storage terms.

Momentum Equation

The terms found in the momentum equation are given here:

(6) Acceleration owing to time variation in flow.
(7) Acceleration owing to spatial variation in velocity.
(8) Acceleration effects owing to lateral inflow.
(9) Gravity body force owing to bed slope.
(10) Frictional force effects.
(11) Pressure force term.

For a wide rectangular channel the continuity equation takes the special form:

$$\frac{\partial h}{\partial t} + \frac{\partial h}{\partial x} + \frac{\partial v}{\partial x} = q_L \qquad (6)$$

Model Limitations

HEC-2

Overview of Program Capabilities

The program is intended for the calculation of water surface profiles for steady, gradually varied flow in natural or man-made channels. Both subcritical and supercritical flow profiles can be calculated. The effects of various obstructions such as bridges, culverts, weirs and structures in the flood plain may be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation with energy loss due to friction evaluated with the Manning equation. The computational procedure is generally known as the standard step method. The program is designed for application in the flood plain management and flood insurance studies to evaluate floodway encroachments and to designate flood hazard zones. Also, capabilities are available for assessing the effects of channel improvements and levees on water surface profiles. Input and output units may be either imperial or metric.

Program Limitations

The following assumptions are implicit in the analytical expressions used in the program:

(1) Flow is steady.
(2) Flow is gradually varied.
(3) Flow is one-dimensional (i.e., velocity components in directions other than the direction of flow are unaccounted for).
(4) River channels have "small" slopes, less than 1:10.

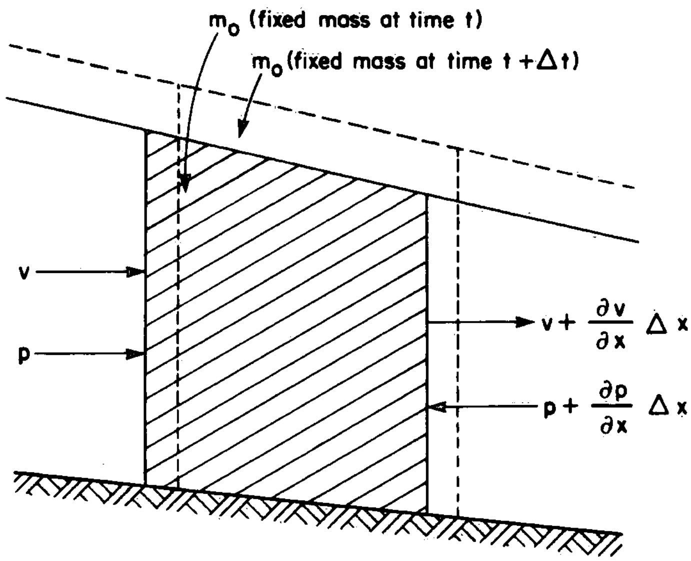


Figure 3. Definition sketch of control volume.

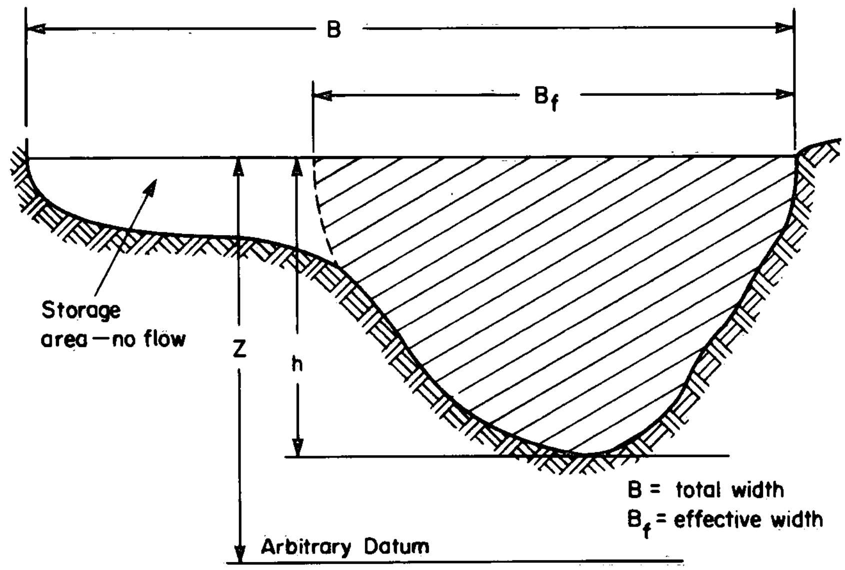


Figure 4. Definition sketch of cross section of arbitrary channels.

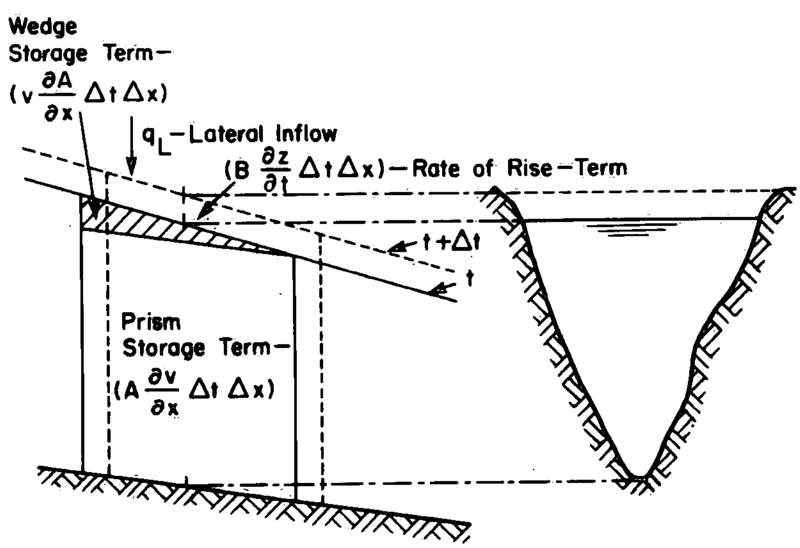


Figure 5. Physical representation of terms in continuity equation.

Flow is assumed to be steady because time-dependent terms are not included in the energy term (3). Flow is assumed to be gradually varied because energy Equation 3 is based on the premise that a hydrostatic pressure distribution exists at each cross section. Flow is assumed to be one-dimensional because Equation 3 is based on the premise that the total energy head is the same for all points in a cross section. Small channel slopes are assumed because the pressure head, which is a component of WS in Equation 3, is represented by the water depth measured vertically.

The program does not have the capability to deal with movable boundaries (i.e. sediment transport) and requires that energy losses be definable with the terms contained in Equation 2 or by using the criteria for bridge, culvert or weir flow described in the HEC-2 manual (U.S. Army Corps of Engineers, 1973).

One-Dimensional Hydrodynamic Model

Overview of Model Capabilities

This model was built as a decision-making hydrologic simulator, for general purpose management problems in water resources planning. It is able to simulate not only the physical parameters (rainfall generation, flood-routing, irrigation) but also decision parameters (reservoir operating rules, power targets) within a framework that is flexible enough to permit the investigation of the operation of one element in the basin (such as a reservoir), or any portion of the river system.

The model is capable of simulating steady and unsteady (hydrodynamic) flow in a river system where structures (dams), weirs, culverts, rapids, ice covers and reservoirs (embankments) form obstructions.

Model Assumptions

The fundamental assumptions made in the derivation of the Saint Venant equations follow:

(1) The flow is assumed to be one-dimensional, i.e., the flow in the channel can be approximated with uniform velocity over each cross section, and the free surface is taken to be a horizontal line across the section. This implies that centrifugal effects due to channel curvature and Coriolis effects are negligible.

(2) The pressure is assumed to be hydrostatic, i.e., the vertical acceleration is disregarded and the density of the fluid is assumed to be homogeneous.

(3) The effects of boundary friction and turbulence can be accounted for through the introduction of a resistance force which is described by the empirical Manning or Darcy-Weisbach friction factor equations.

Having made these assumptions, the conservation equations may be formulated by the "material" method or the "control volume" method. In the material method the flow characteristics are obtained by following the motion of a given mass of fluid through a small increment of time in the vicinity of the fixed section. In the control volume method the equations are derived by considering the fluxes of mass and momentum through a fixed control volume (Figs. 3 and 4). Like HEC-2, this model does not have the capability to deal with sediment transport.

MODEL COMPARISONS (ONE-DIMENSIONAL VS. WATQUAL)

One-Dimensional Hydrodynamic Model

The basic governing equation for water quality simulation is a conservation of mass equation which includes the transport process of advection and dispersion, and reaction processes where mass is added to or removed from the system.

$$\frac{\partial}{\partial t}(Ac) + \frac{\partial}{\partial x}(Qc) = \frac{\partial}{\partial x}\left(AE\frac{\partial c}{\partial x}\right) + \left\{\frac{r_i}{\rho} + \frac{r_e}{\rho}\right\}A \qquad (7)$$

where c = concentration of constituent
A = channel cross-sectional area
Q = tidal discharge
E = longitudinal dispersion coefficient in tidal time
ρ = density of water
r_i = time rate of internal addition of mass per unit volume by generation of substance within the fluid
r_e = time rate of external addition of mass per unit volume by movement across the boundaries of the fluid.

The reaction processes represented by r_i and r_e will vary depending on the water quality constituent being considered. For a conservative substance such as salinity, they will both be zero. Biochemical oxygen demand (BOD), however, is non-conservative, and its internal removal within the water mass is often described in terms of a first-order reaction where $r_i = -Kc$. The first-order reaction coefficient K is expressed typically in units of day^{-1}. A source term of the external type would be replenishment of dissolved oxygen (DO) by transfer across the free surface from the atmosphere.

If a given constituent is part of a set of constituents which interact with each other, a conservation of mass equation is written for each one. For the three constituents considered in the present study, temperature (considered constant), BOD and DO, the conservation of mass equations are:

Temperature:

$$\frac{\partial}{\partial t}(AT) + \frac{\partial}{\partial x}(QT) = \frac{\partial}{\partial x}\left(AE\frac{T}{\partial x}\right) - K_T A\ (T - T_e) \quad (8)$$

BOD:

$$\frac{\partial}{\partial t}(AL) + \frac{\partial}{\partial x}(QL) = \frac{\partial}{\partial x}\left(AE\frac{\partial L}{\partial x}\right) - K_1 AL \quad (9)$$

DO:

$$\frac{\partial}{\partial t}(AC) + \frac{\partial}{\partial x}(QC) = \frac{\partial}{\partial x}\left(AE\frac{\partial C}{\partial x}\right) - K_1 AL + K_2 A\,(C_s - C) \quad (10)$$

where T = temperature
T_e = equilibrium temperature
L = BOD concentration
C = DO concentration
C_s = saturation concentration of DO
K_T = temperature decay coefficient
K_1 = BOD decay coefficient
K_2 = reaeration coefficient.

The BOD and DO are influenced by the temperature distributions in addition to the coupling through the decay term $-K_1AL$.

WATQUAL

The basis of this model is a pollutant-routing algorithm that uses the Streeter-Phelps formulation for the relationship between instream DO and BOD. Other factors included in the model are rate of photosynthesis, sediment oxygen demand and various organic matter decay rates.

The standard Streeter-Phelps dissolved oxygen sag curve equation is arrived at by combining the deoxygenation and reoxygenation process. The method of doing this is described below. The rate of change in the DOD caused by decomposition of biodegradable material is given by:

$$dD/dt = K_1 L \quad (11)$$

where D = dissolved oxygen deficit (DOD) after time t (ppm)
t = time of incubation (or time of flow downstream) (days)
K_1 = coefficient of deoxygenation determined from laboratory tests (base e) (day^{-1})
L = BOD remaining after time t (ppm).

The BOD remaining in the river water after time t is defined by:

$$L = L_{0e}\,K_{rt} \quad (12)$$

where L_0 = ultimate first stage (carbonaceous) BOD (ppm)
K_r = coefficient of deoxygenation for the river (base e) (day^{-1}).

The coefficient, K_r, is defined by:

$$K_r = K_1 + K_3 \quad (13)$$

where K_3 = coefficient of deoxygenation which accounts for oxygen demand changes due to sedimentation, turbulence, biological growth on the river bed, nutrient deficiency and unacclimatized bacteria. The term K_3 can be positive, negative or zero.

The coefficient of deoxygenation is normally based on the results of laboratory tests conducted on samples at a temperature of 20°C. The coefficient for the river is obtained by adjusting the 20°C coefficient by the following equation:

$$(K_r)t = (K_r)20(1.047^{t-20}) \quad (14)$$

where t = temperature of the river water (°C).

The rate of change in the DOD caused by surface reaeration or reoxygenation is described by:

$$dD/dt = K_2 D \quad (15)$$

Atmospheric Reaeration

The atmospheric reaeration coefficient has been the subject of much study and investigation in recent years. The studies have covered a wide range of river situations from the shallow short-run streams of England to the deep, wide and slowly moving rivers of the United States. Owens *et al.* (1964), a British team, combined the British data with those collected on the Tennessee Valley streams to produce the equation:

$$K_2 = 21.6U^{0.67}/H^{1.85} \qquad (16)$$

where K_2 = atmospheric reaeration coefficient
U = mean velocity of the stream
H = average depth.

Photosynthesis

The total change in DOD due to the combined effects of both deoxygenation and reaeration can be represented by a single equation:

$$dD/dt = K_2D + K_1L \qquad (17)$$

Integrated, this gives:

$$D = \frac{K_1L_0}{K_2 - Kr}(e^{K_r t} - e^{K_2 t}) + D_{0e} - K_2t \qquad (18)$$

where D_0 = the DOD at time $t = 0$ (ppm).

Equation 18 is the basic sag curve equation and forms the nucleus around which Program WATQUAL is developed.

There is a provision in Program WATQUAL for including the effect on instream DO of the photosynthetic oxygen production rate, A. Inclusion of this effect requires a term to be added to the basic Streeter-Phelps formulation (Equation 18). The mathematical development of this term is described in the report by Acres Consulting Services (1971). The adjustment to the dissolved oxygen deficit, D, to account for photosynthesis is described by the following equation:

$$D = \text{Equation 18} + (A/K_1)(1-e^{-k2t})$$

where D = dissolved oxygen deficit (DOD) ppm after time t
A = mean cross-sectional area of flow (m^2)
K_1 = coefficient of deoxygenation determined from laboratory tests (base e) (day^{-1})
t = time variable.

Benthic Oxygen Demand

The discharge of settleable waste material often results in the formation of "sludge banks" immediately below a waste outfall (Nolan and Johnson, 1977). These deposits may build up over a period of time if river velocities are too low to permit scouring of the river bottom.

As the depth increases, anaerobic decomposition of the organic material in the deeper layers begins. The products of this decomposition, CO_2, CH_4 and H_2S, proceed up through the sludge layer and into the overlying waters. If gas production is especially high, floating of the bottom sludge may result, leading to a severe aesthetic problem as well as possible transient DO depletion. The surface layer of the bottom deposit in direct contact with the water usually undergoes aerobic decomposition and, in the process, removes oxygen from the supply in the overlying river water. This is the sink of DO, designated S_B.

The oxygen demand of the river bottom may not always be due directly to sewage or industrial sludges. Soluble organic wastes may sometimes result in the growth of attached filamentous bacteria such as Sphaerotilus which can utilize substantial amounts of oxygen. The death of floating and rooted aquatic plants and natural runoff may contribute to bottom organic material which will also require oxygen for stabilization.

The approximate average value of uptake (grams $O_2/m^2/day$) at 20°C for Sphaerotilus (10 g dry wt/m^2) is around 7.

One of the difficulties in properly inputting this DO sink in the mathematical model is the estimation of the aerial distribution of the more pronounced deposits and their rate of oxygen uptake. *In situ* and laboratory measurements are conducted with the DO reduction reported in grams O_2 uptake/m^2/day. It necessitates division by the average depth of the overlying water to obtain the correct representation of S_B in the model, assuming that the river is well mixed in the vertical direction. Thus, if S_B is in grams $O_2/m^2/day$, then:

$$S_B\ (mg/day) = \frac{S_B\ (g/m^2/day)}{H\ (m)}$$

where H = average depth in metres.

Temperature effects of S_B were introduced in the WATQUAL model. This can be approximated by:

$$(S_B)_T = (S_B)_{20}\,(1.250)^{(T-20)}$$

Table 4. Model Comparison

One-Dimensional Hydrodynamic model	WATQUAL model
Limitations	**Advantages**
Cannot use more than one reaeration coefficient per reach.	Can use one reaeration coefficient per cross section.
Sediment oxygen demand is not incorporated.	Can use one value of SOD per cross section.
Advantages	**Limitations**
Unlimited number of point sources of pollutants (waste loadings).	Limit of six point sources of pollutants.
Can simulate up to 13 water quality parameters (S, T, BOD, ON, N, OP, P, CP, CZ, DO, FCOL, CLIG, DLIG).	Can simulate only BOD and DO in a run.
Can use one decay rate for every reach.	Can use only one decay rate for the entire network.
Can simulate water quantity and water quality simultaneously or separately.	Has no feature for water quantity. Another model has to be used to provide hydraulic parameters necessary to run this water quality model.
Can use more than 300 nodes. This number refers to cross sections only. Lateral inflows and waste loadings are almost unlimited.	Maximum number of nodes is 35. This number includes lateral inflows, waste loadings and cross sections.
Longitudinal dispersion coefficient is calculated and included as a separate term in the conservation of mass equation.	Longitudinal dispersion coefficient is not calculated.
Time varying temperature.	Fixed temperature with time.

Table 5. List of Symbols for Water Quality Parameters and Sequence of Identification

Abbreviation	Parameter
S	Salinity
T	Temperature
BOD	Biochemical oxygen demand
ON	Organic nitrogen
N	Inorganic nitrogen
OP	Organic phosphate
P	Inorganic phosphate
CP	Phytoplankton
CZ	Zooplankton
DO	Dissolved oxygen
FCOL	Fecal coliforms
CLIG	Lignins — conservative
DLIG	Lignins — decaying

where T = temperature in °C. Values of $(S_B)_{20}$ for the St.Croix River were obtained by the New England Regional Laboratory, Lexington, Massachusetts, in August 1977.

Table 4 shows the limitations and advantages of using either model in a river study. Table 5 lists the symbols for identifying water quality parameters.

RESULTS AND CONCLUSIONS

Prior to the establishment of secondary waste treatment at Woodland, high organic loads resulted in oxygen concentrations in the St. Croix River during low flow conditions (Figs. 6 and 7). These low oxygen concentrations, coupled with the establishment of control structures, impeded the ordinary pattern of migration of Atlantic salmon (*Salmo salar*). Establishment of a secondary effluent treatment facility in 1978 has improved the oxygen regime considerably (Fig. 8), so that the river is now capable of supporting an anadromous run of Atlantic salmon. This improvement in water quality, in conjunction with the removal of physical barriers through construction of fish ladders, has resulted in a return of Atlantic salmon to the St. Croix River.

With the objective of maintaining the fish run, the Advisory Board on Pollution Control in the St. Croix River has developed a use-specific water quality objective for dissolved oxygen. This objective sets a minimum dissolved oxygen limit for fish passage at a concentration of 5.0 mg/L.

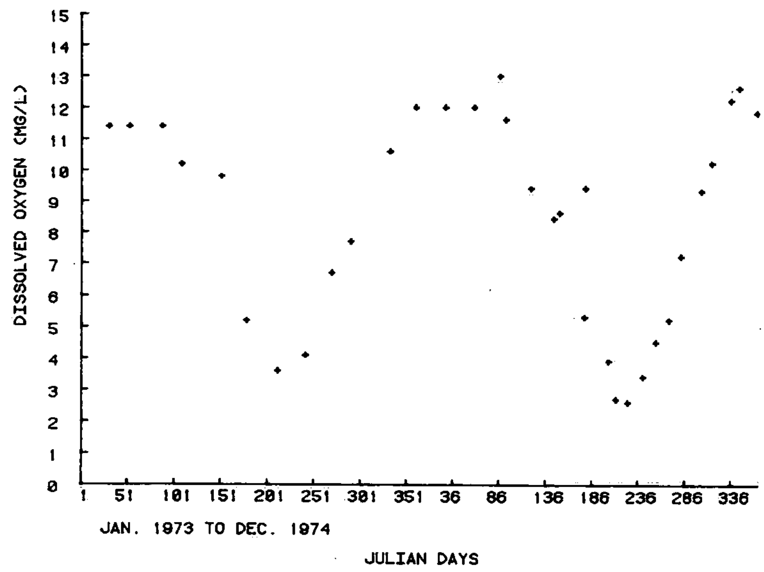


Figure 6. Oxygen concentrations in the St. Croix River during low flow conditions, January 1973 to December 1974.

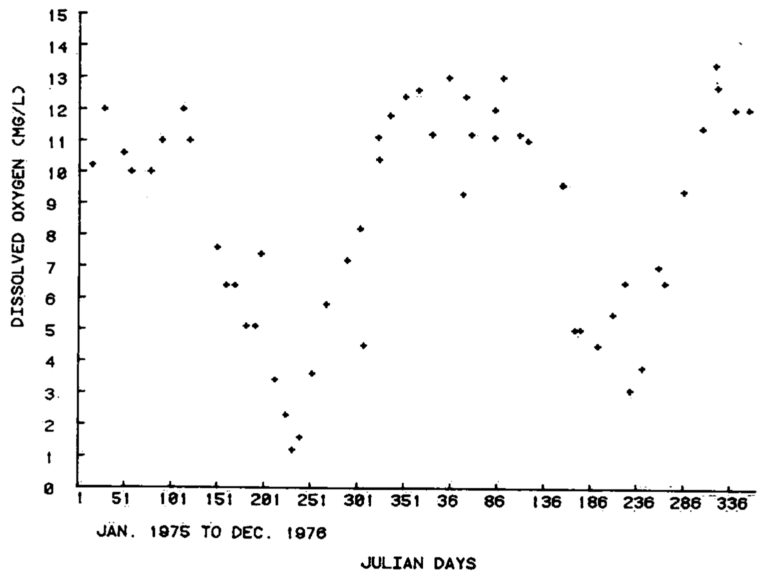


Figure 7. Oxygen concentrations in the St. Croix River during low flow conditions, January 1975 to December 1976.

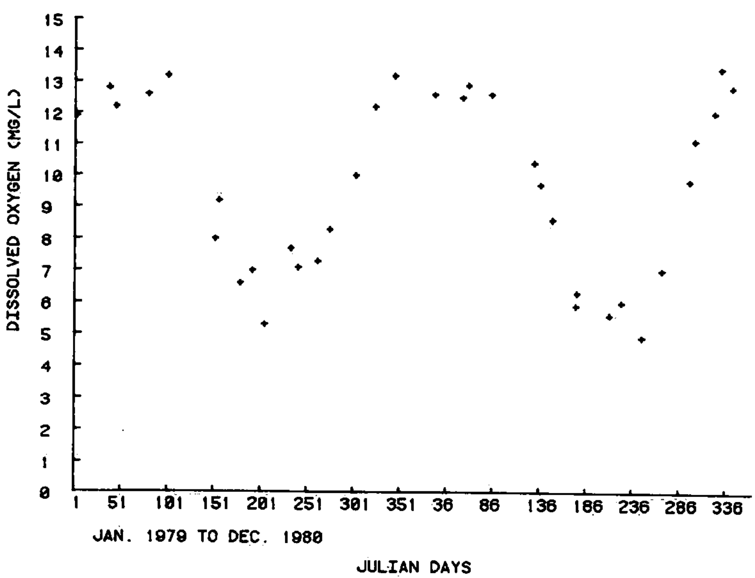


Figure 8. Oxygen regime after installation of treatment facility in 1978.

Since the installation of the secondary treatment facility at Woodland, dissolved oxygen concentrations have generally complied with the water quality objective. However, strict control of river and effluent discharge rate must be maintained to avoid episodal oxygen depletion and the possibility of fish kills. One such event occurred on August 28, 1980 (Fig. 15), when an electrical malfunction at the Woodland Mill caused a reduction in river discharge from the Grand Falls Dam. This, coupled with undiminished effluent loading and high water temperature (23.0°C), resulted in low dissolved oxygen concentrations at Milltown. Following this incident a minimum oxygen concentration of 0.5 mg/L was recorded, and levels remained below the 5.0 mg/L objective for six days.

Figures 9 to 27 present simulated dissolved oxygen concentrations from the two water quality models as well as the measured oxygen profile. For sampling dates during autumn, winter and spring both models predict dissolved oxygen concentrations similar to those actually observed. One exception to this was observed for January 22, 1981 (Fig. 16), when both WATQUAL and 1-DIM predicted oxygen concentrations well above the measured values. Extremely cold air temperatures during this period interfered with both Winkler and meter oxygen determinations and thus the measured values must be viewed with caution. In addition, the St. Croix River was partially ice-covered, which would interfere with the rate of oxygen exchange at the air-water interface. Thus, a combination of measurement uncertainty and an unmodelled reduction in dissolved oxygen diffusion rate probably account for the poor model predictions.

The period of greatest dissolved oxygen concern occurs during the summer and early fall, when the Atlantic salmon are in the river, and water temperature is sufficiently high to result in oxygen depletion. Comparison of model predictions for the summer months indicates overall good prediction by both models, with the 1-DIM profile generally being closer to the measured profile. On June 25, 1981 (Fig. 18), both WATQUAL and 1-DIM predicted dissolved oxygen concentrations much higher than those actually observed. Scrutiny of field observations indicated that inaccuracy of dissolved oxygen measurement was not implicated. In fact, data from the USGS Automatic Water Quality Monitor at Milltown corroborated the observed field values. This suggested that the poor model agreement could be attributed to inappropriate model input data. There is a lack of confidence in both the accuracy and representation of the Georgia Pacific BOD loadings parameter. The BOD values used in the two models are those measured for the sampling date. This does not account for the approximately 16-hour lag or travel time between Woodland and Milltown, or the retention time of the secondary treatment facility. The BOD load on the day before water quality sampling was 4039 kg/day, a value considerably higher than the June 25 value of 2511 kg/day. However, use of this BOD value does not significantly improve the predictability of the two models, suggesting that other input data may be involved in the poor agreement between modelled and measured dissolved oxygen profiles.

In general, the WATQUAL simulation tended to predict values higher than those observed, while the 1-DIM model predicted lower values. This was particularly noticeable at the Milltown node where oxygen depletion is the most severe. With respect to the intention of maintaining the Atlantic salmon in the St. Croix River, the most suitable water quality model is the 1-DIM, as it predicts values lower than those actually observed. This tendency for low prediction should provide managers with sufficient time to adjust river and effluent discharge rates during periods of predicted oxygen depletion.

RECOMMENDATIONS

Modelling

Based on the water quality simulation produced by the One-Dimensional Hydrodynamic model on the St. Croix River, a number of recommendations can be made to improve the use of this model:

(1) The one-dimensional model should be formatted so that one reaeration coefficient value can be read at every cross section, as in Manning's "n" roughness coefficient.
(2) The sediment oxygen demand (SOD) should be mathematically incorporated in the one-dimensional model, for better simulation in case of high deposition areas.
(3) The one-dimensional water quality model user's manual should be updated.

It is also recommended that the St. Croix River be re-surveyed for more accurate soundings and that a complete new set of water quality data be collected, if future modelling is to be done on this river. The present simulation by the one-dimensional model was carried out with soundings collected in 1967 and 1981, surveyed by Boothbay Engineering Services Inc. and Water Survey of Canada respectively, whereas the water quality data were collected in 1979–1982.

FIGURE 9

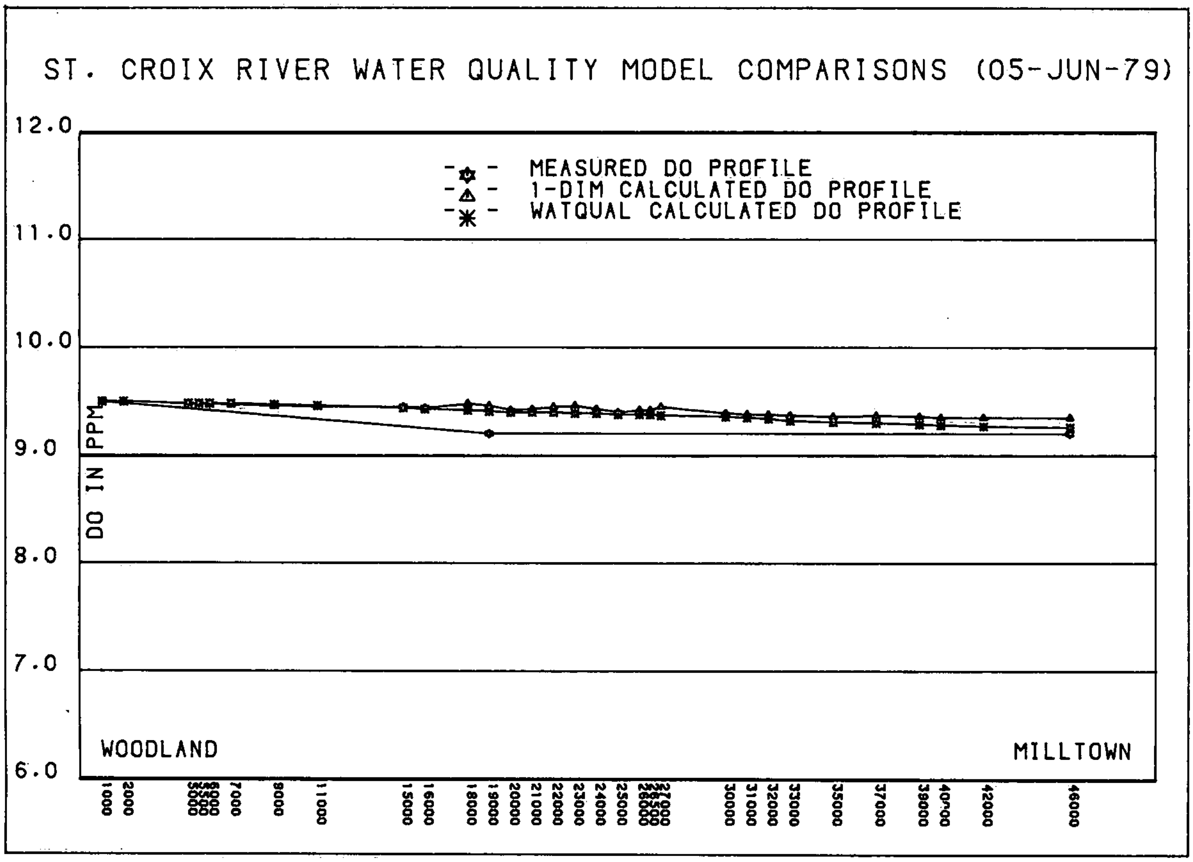
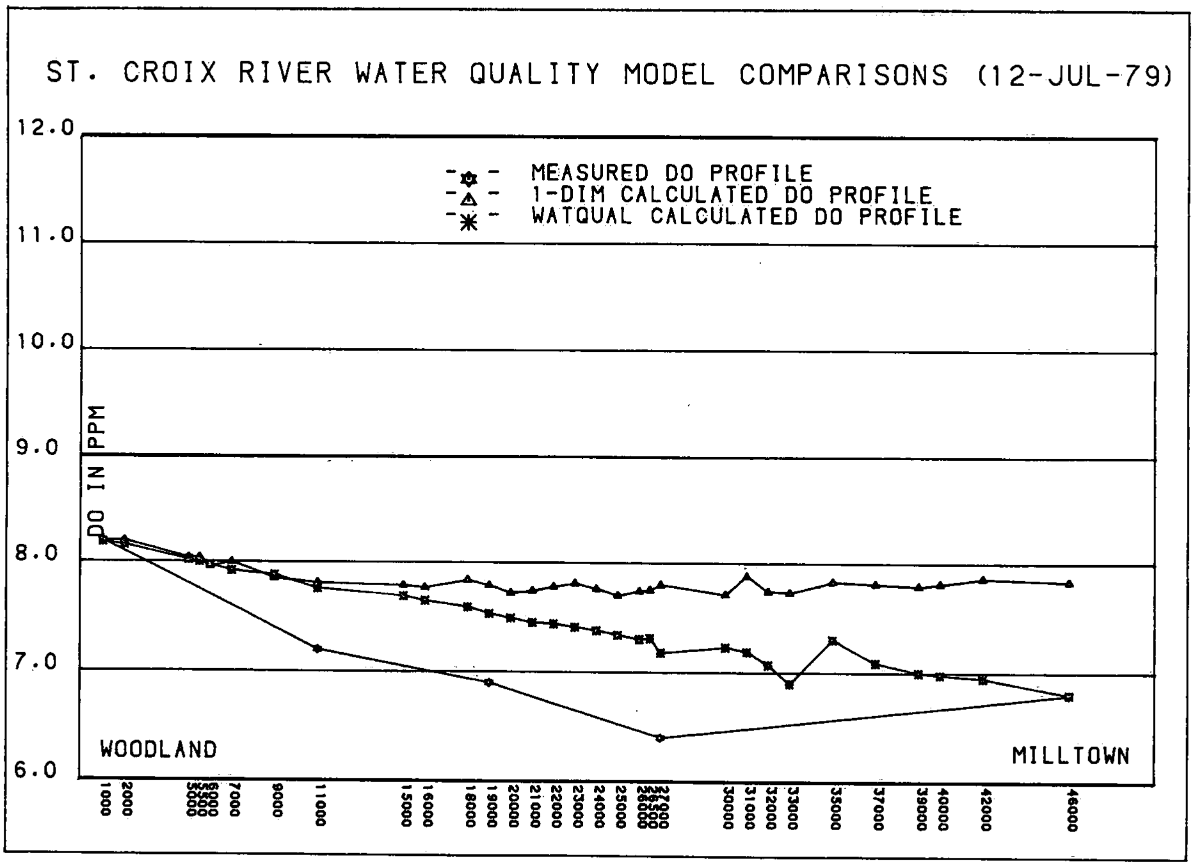


FIGURE 10



Figures 9 to 27 illustrate simulated dissolved oxygen concentrations from the two water quality models, 1979-1982.

FIGURE 11

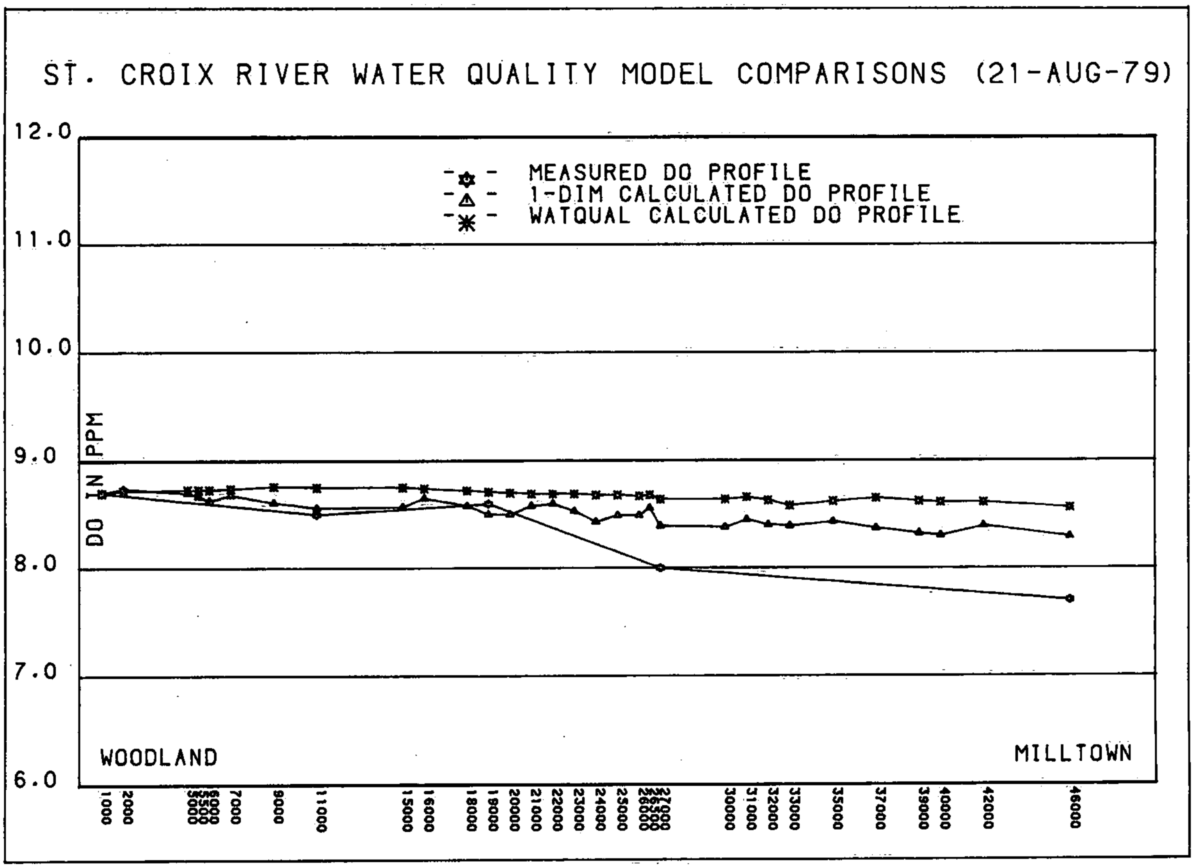


FIGURE 12

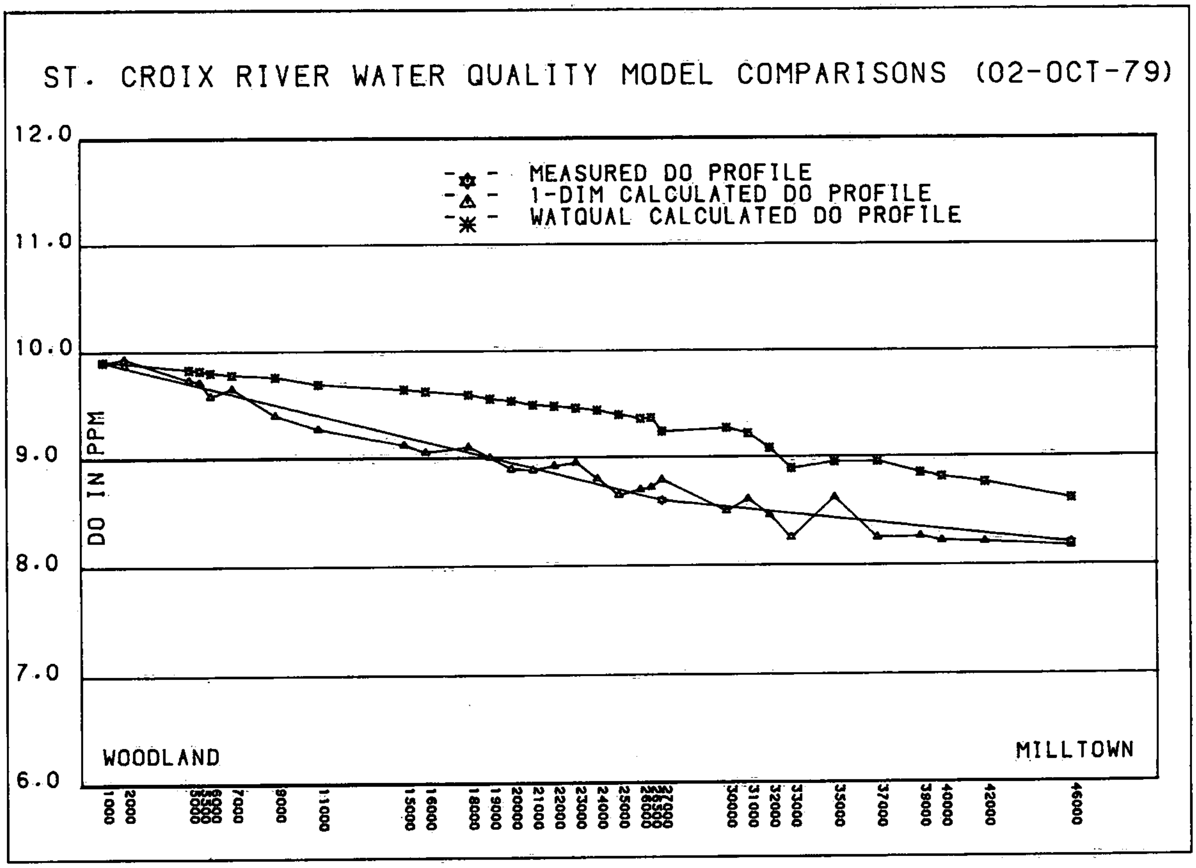


FIGURE 13

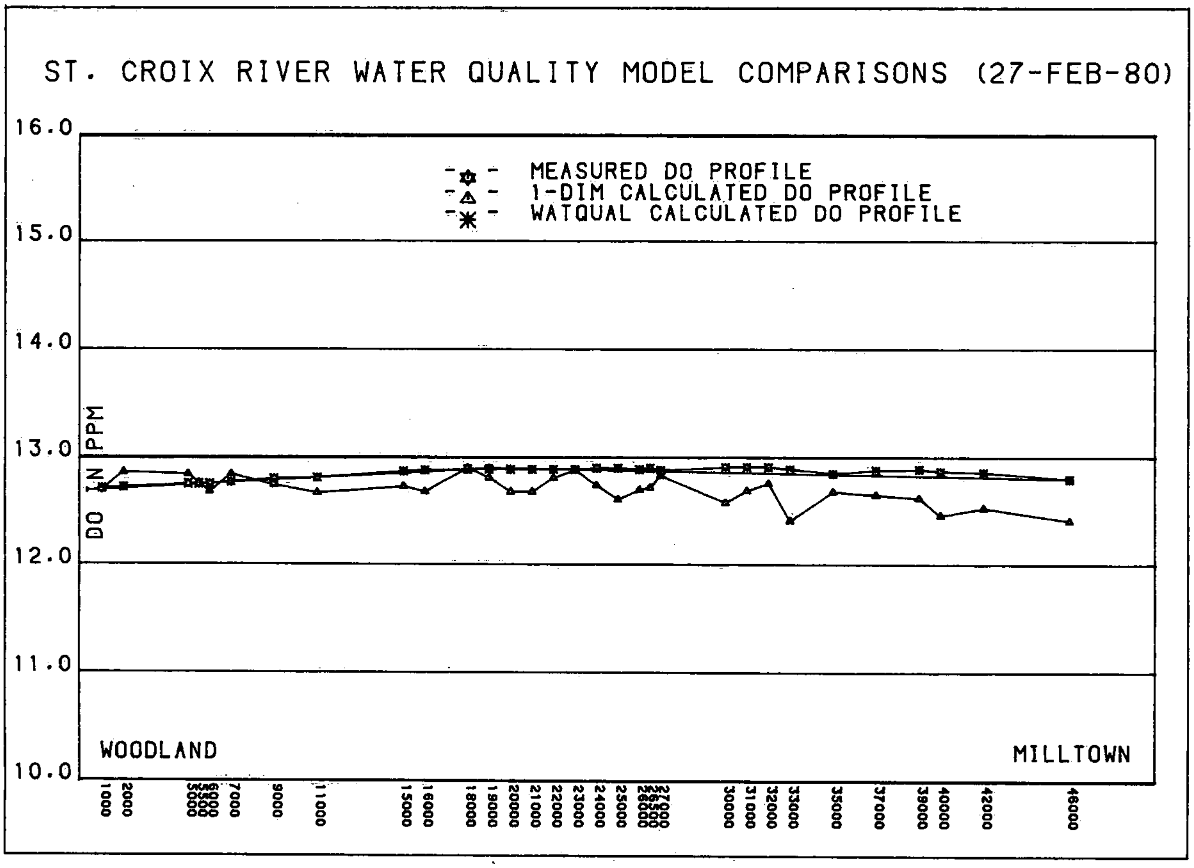
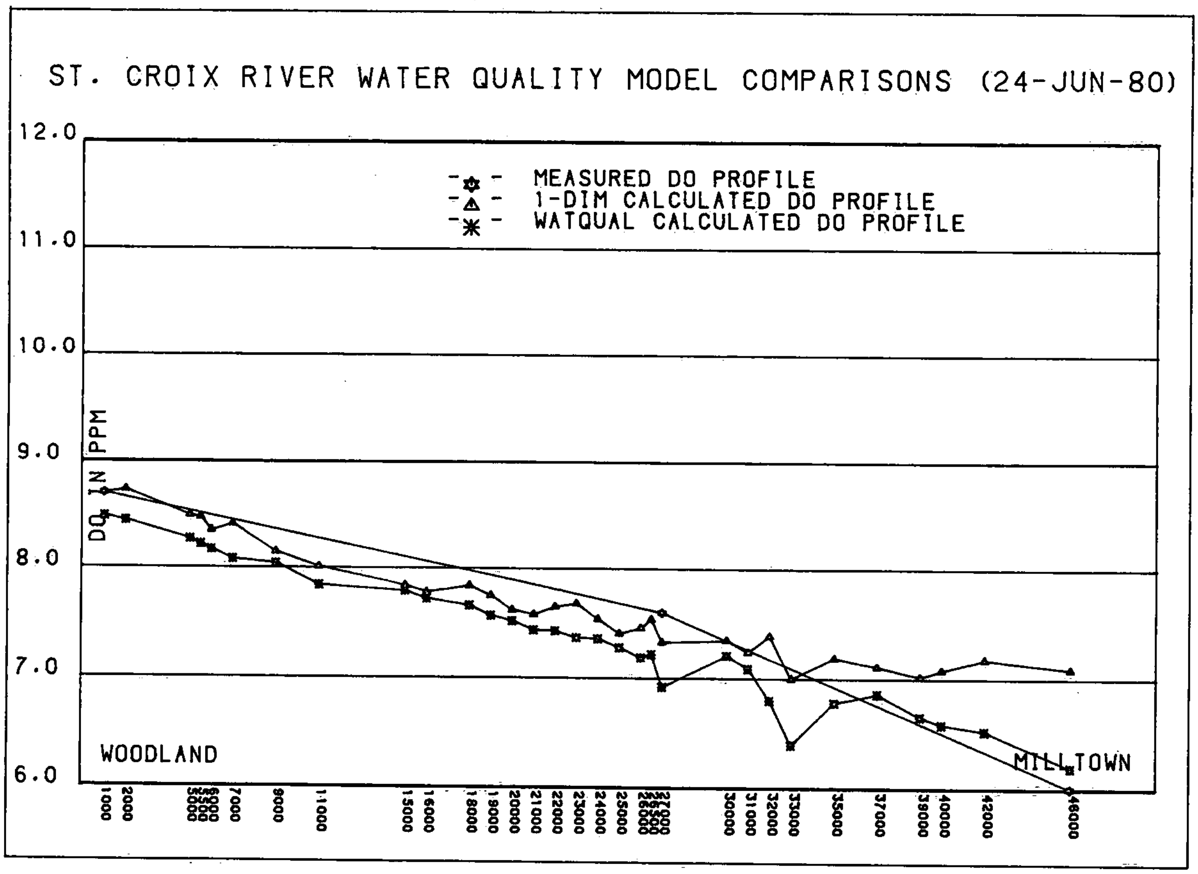


FIGURE 14



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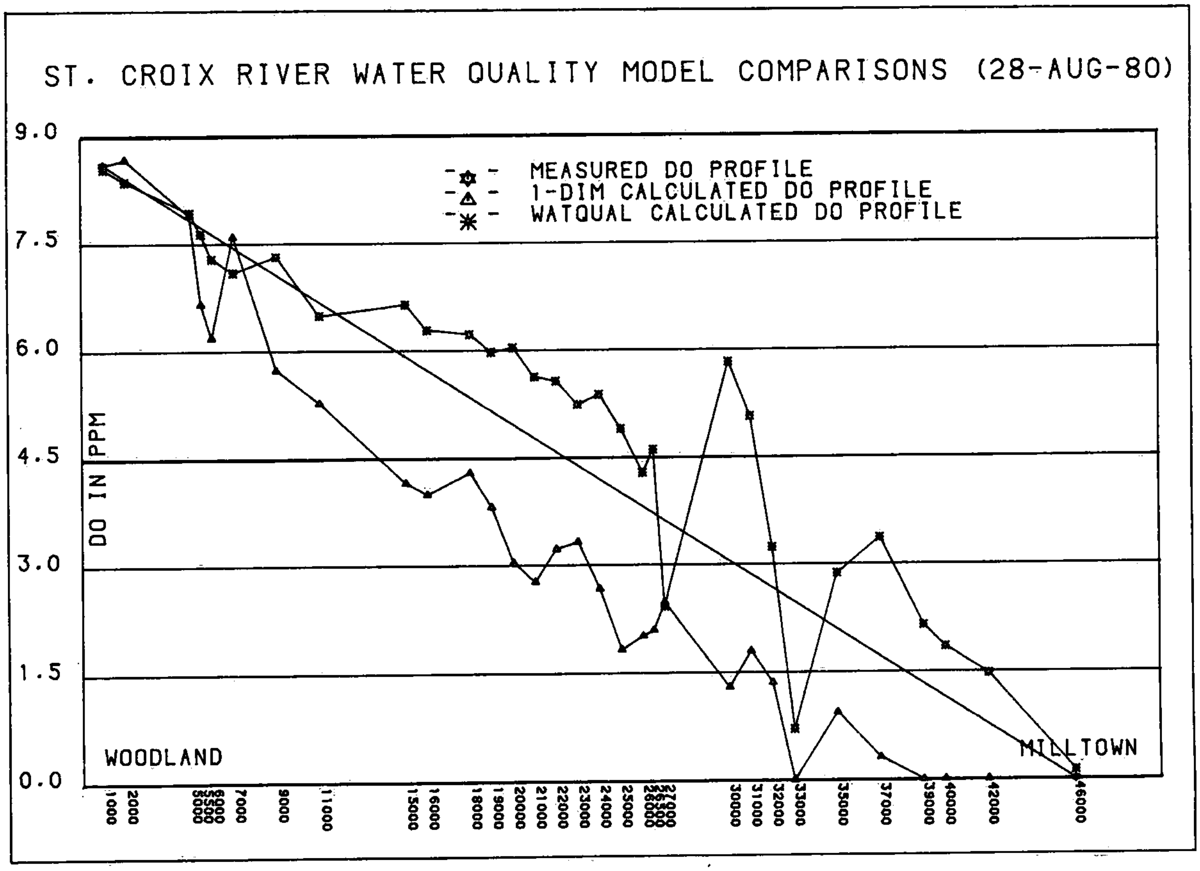


FIGURE 16

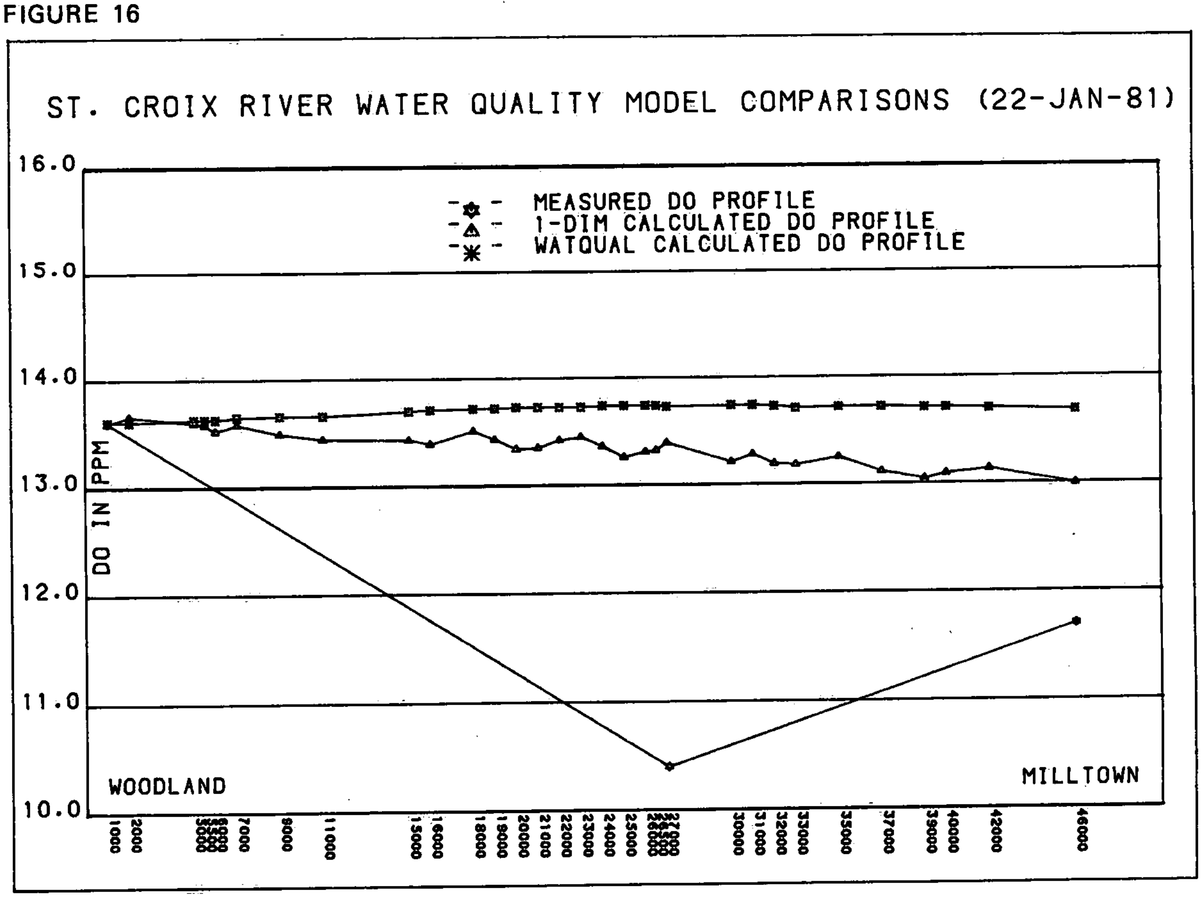


FIGURE 17

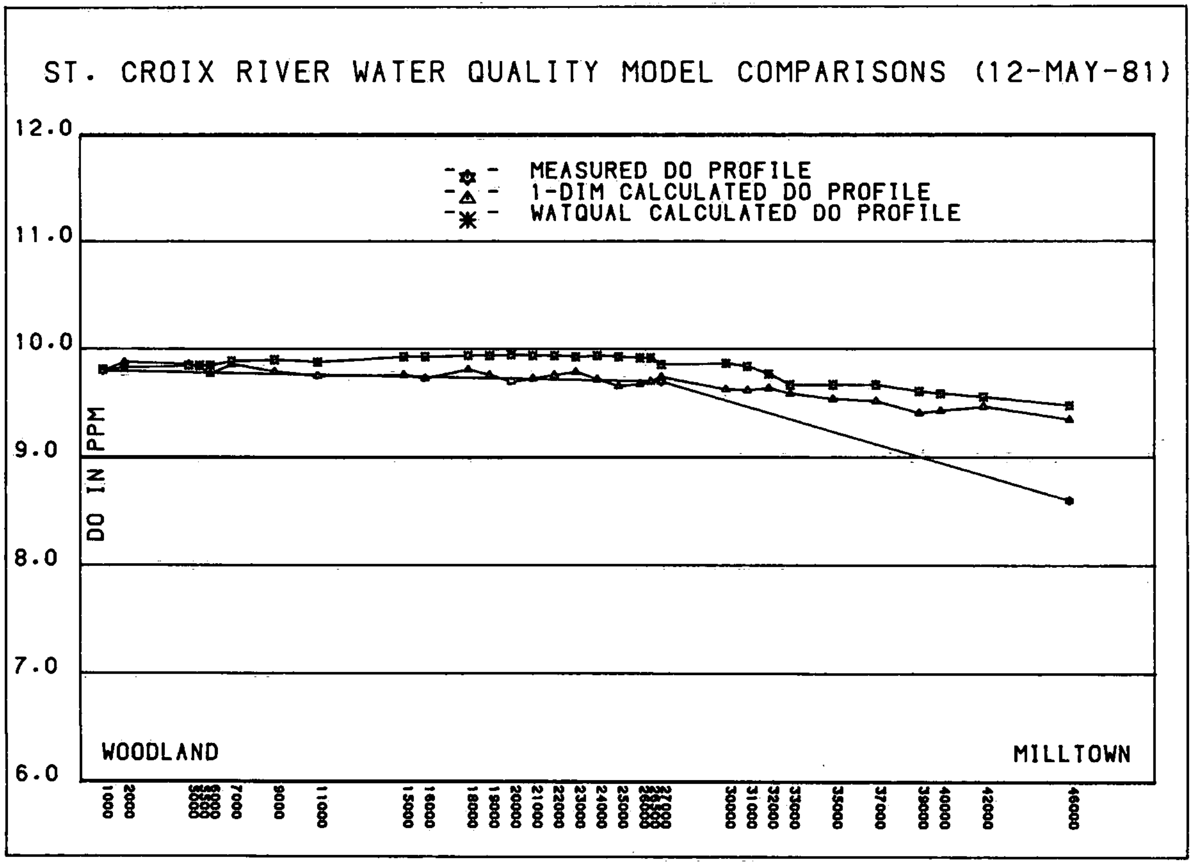


FIGURE 18

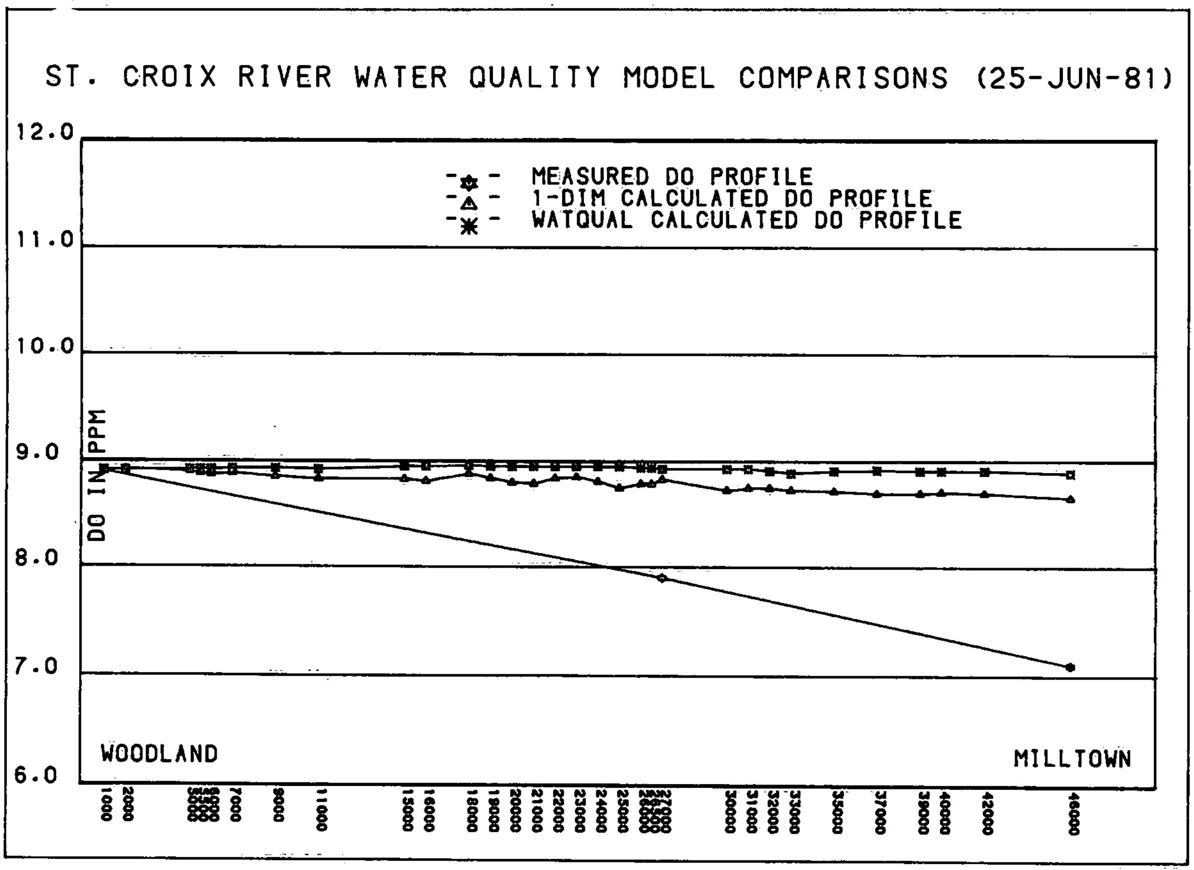


FIGURE 19

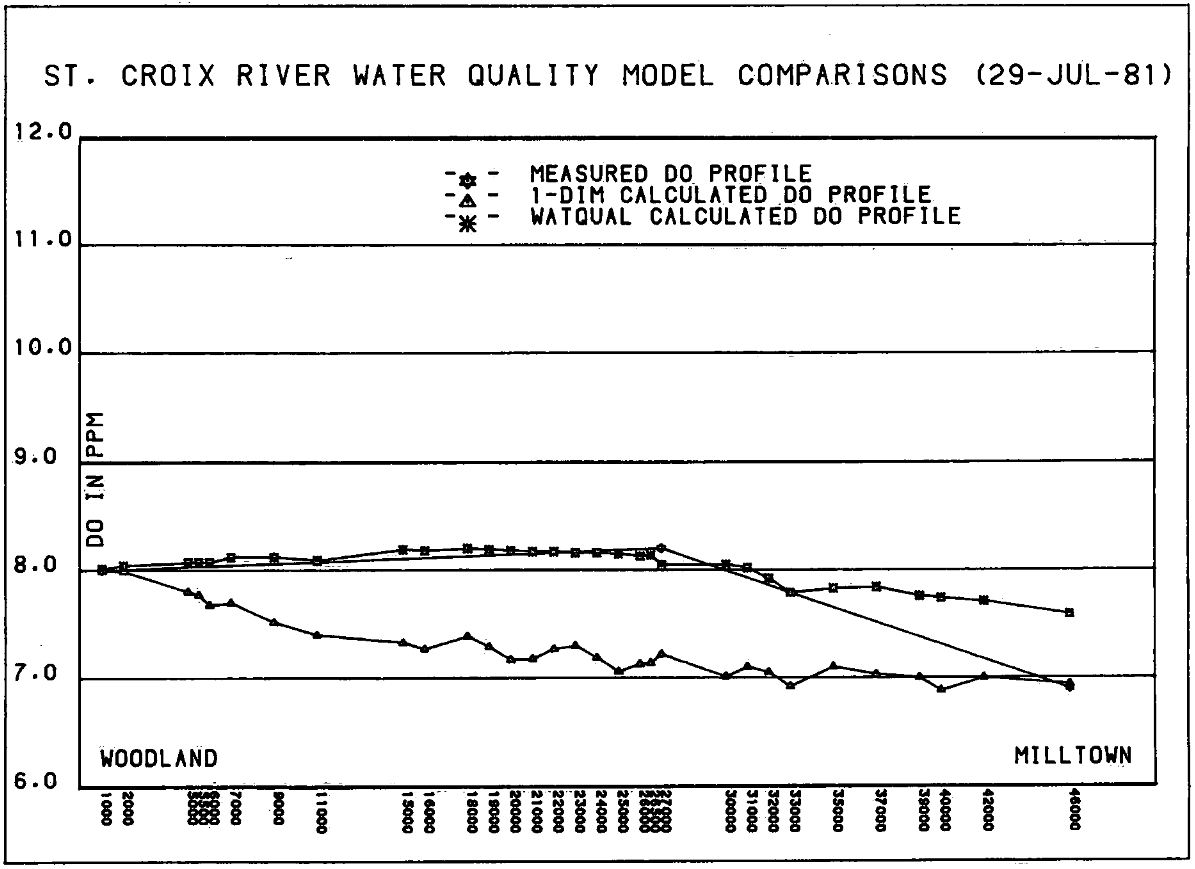


FIGURE 20

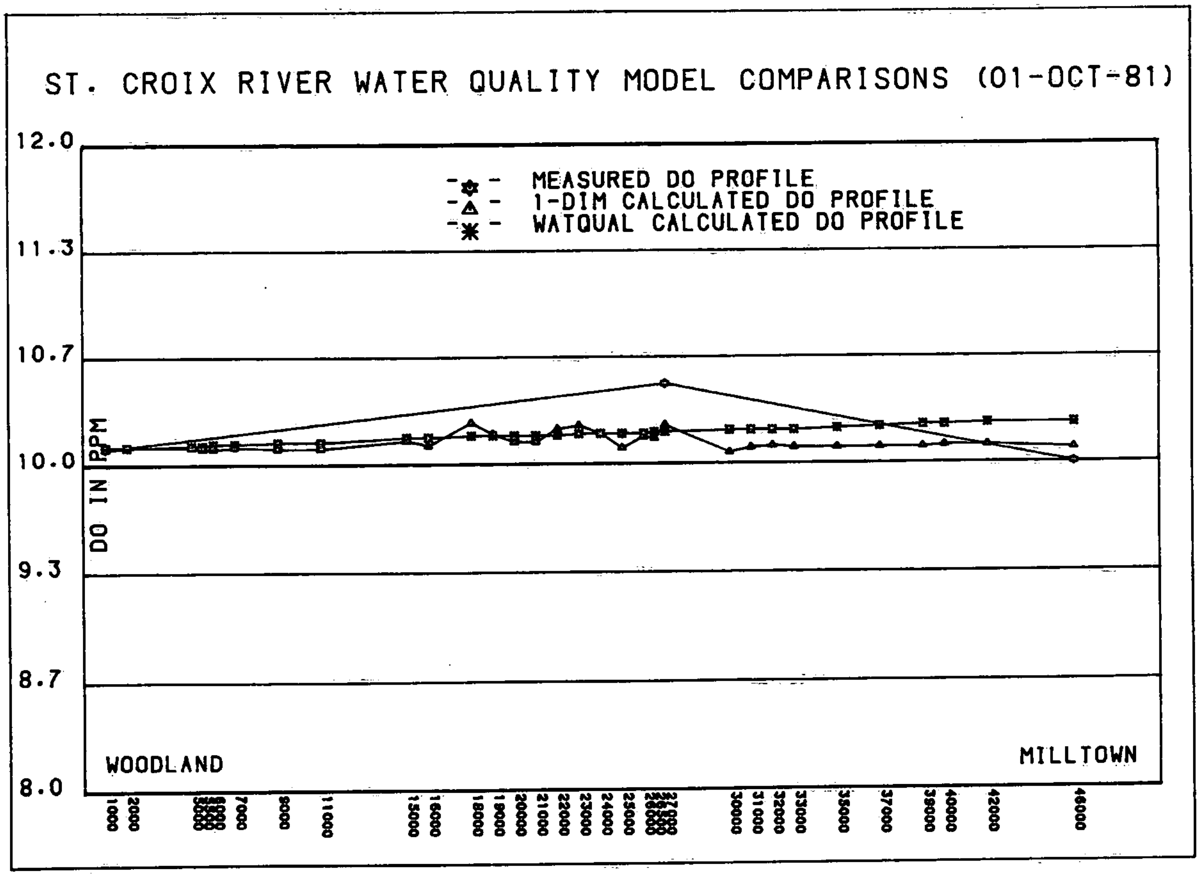


FIGURE 21

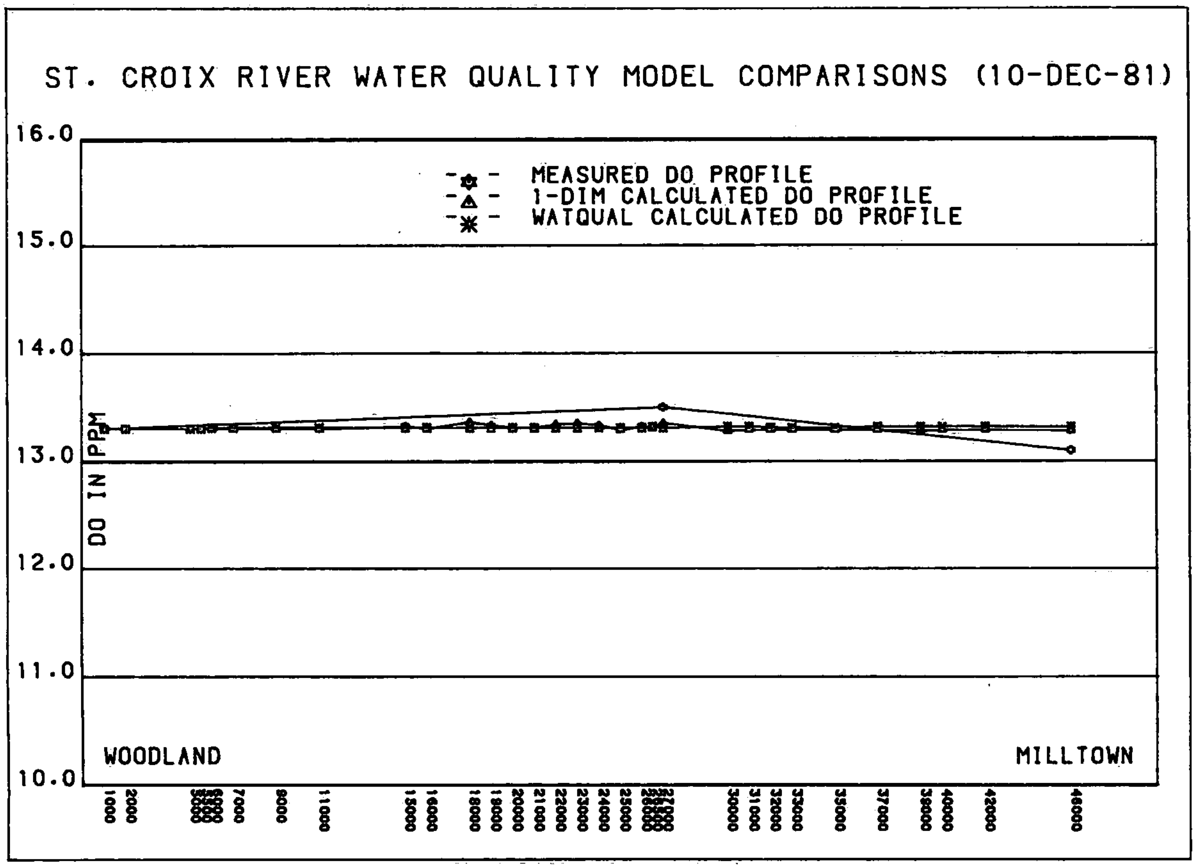


FIGURE 22

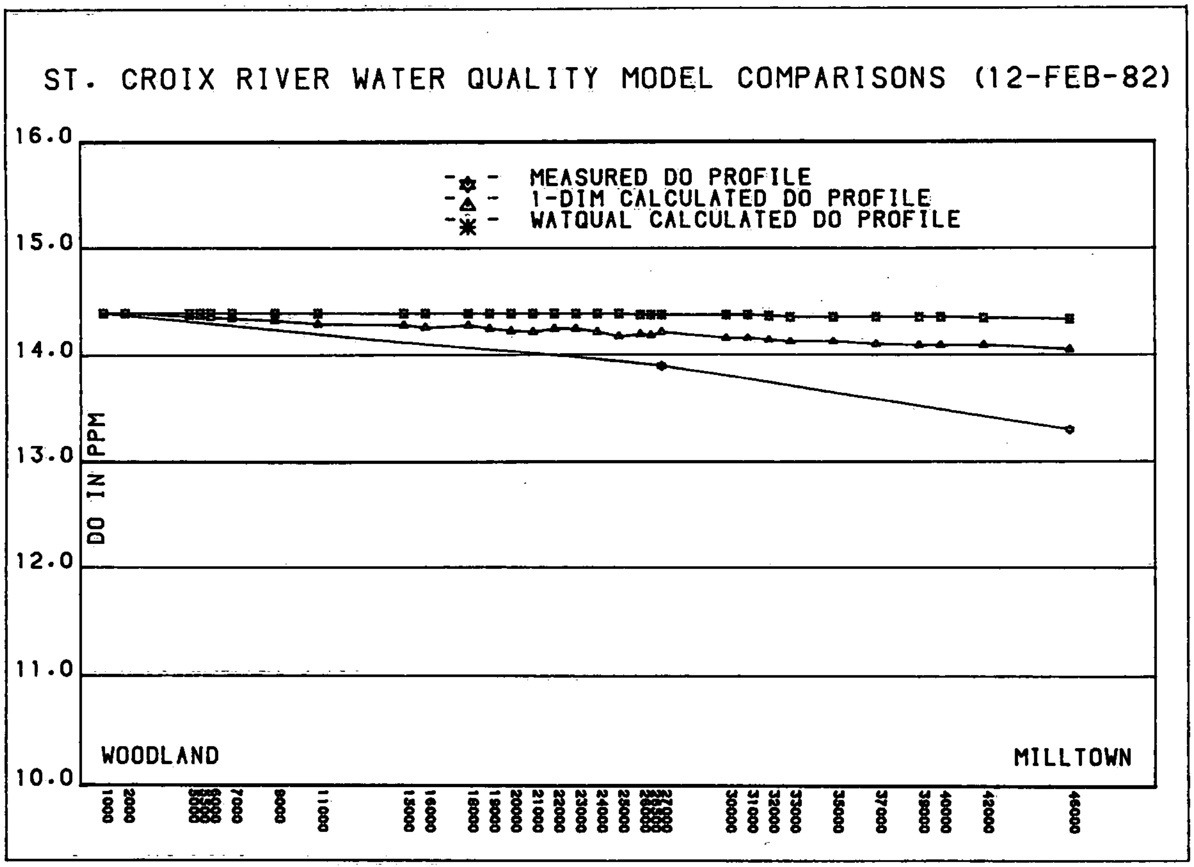


FIGURE 23

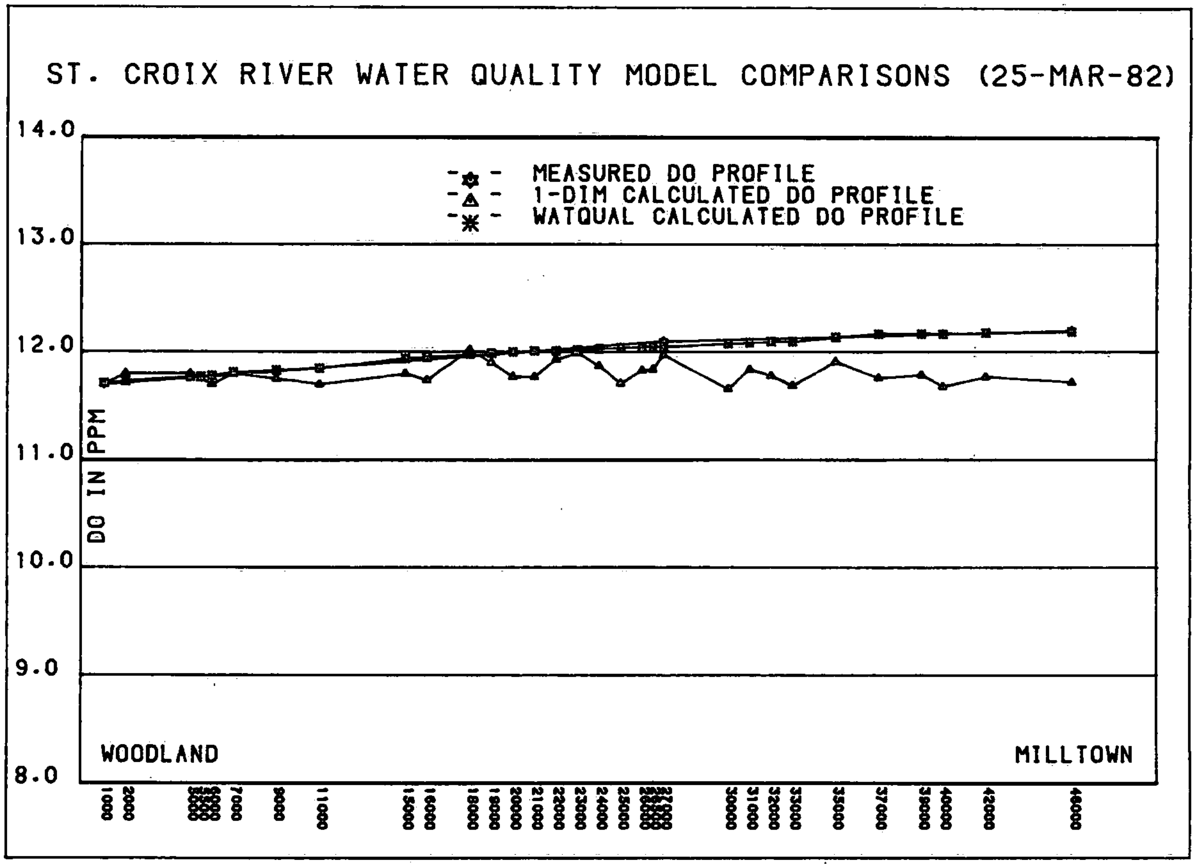


FIGURE 24

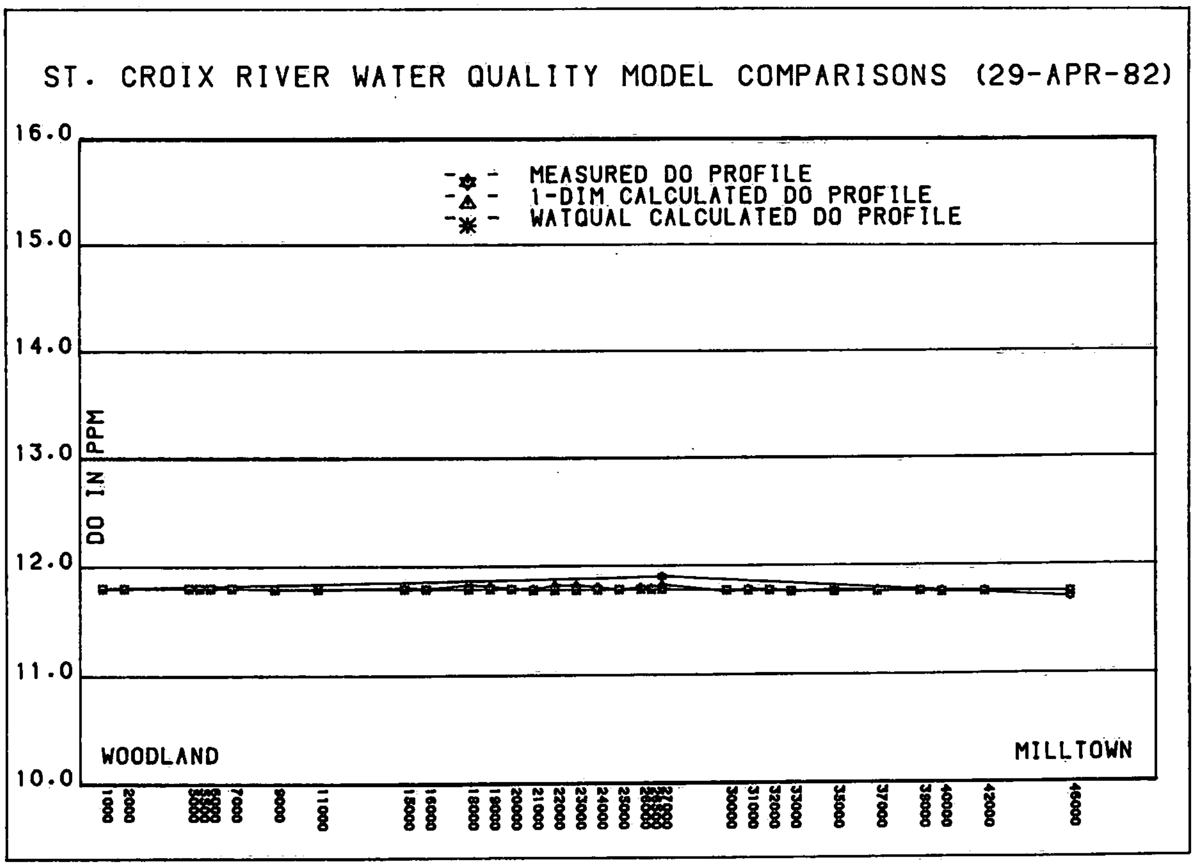


FIGURE 25

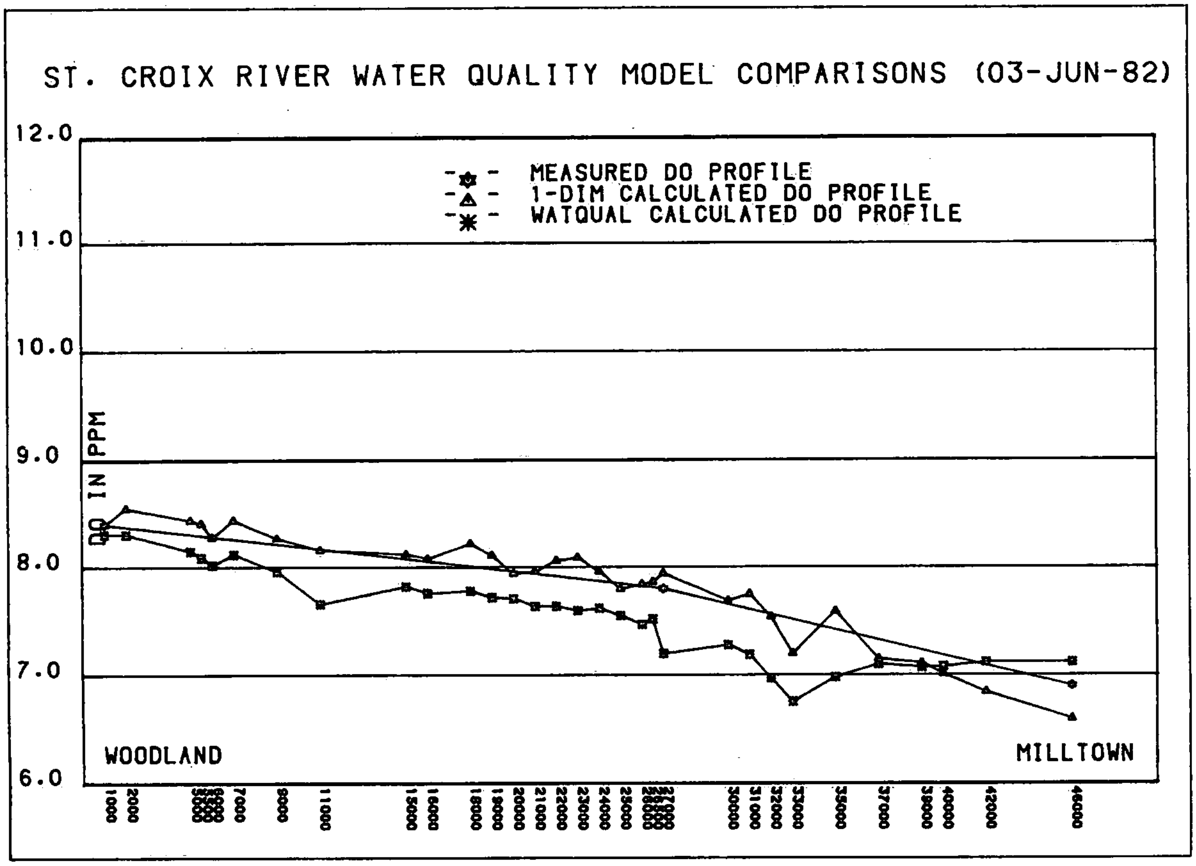


FIGURE 26

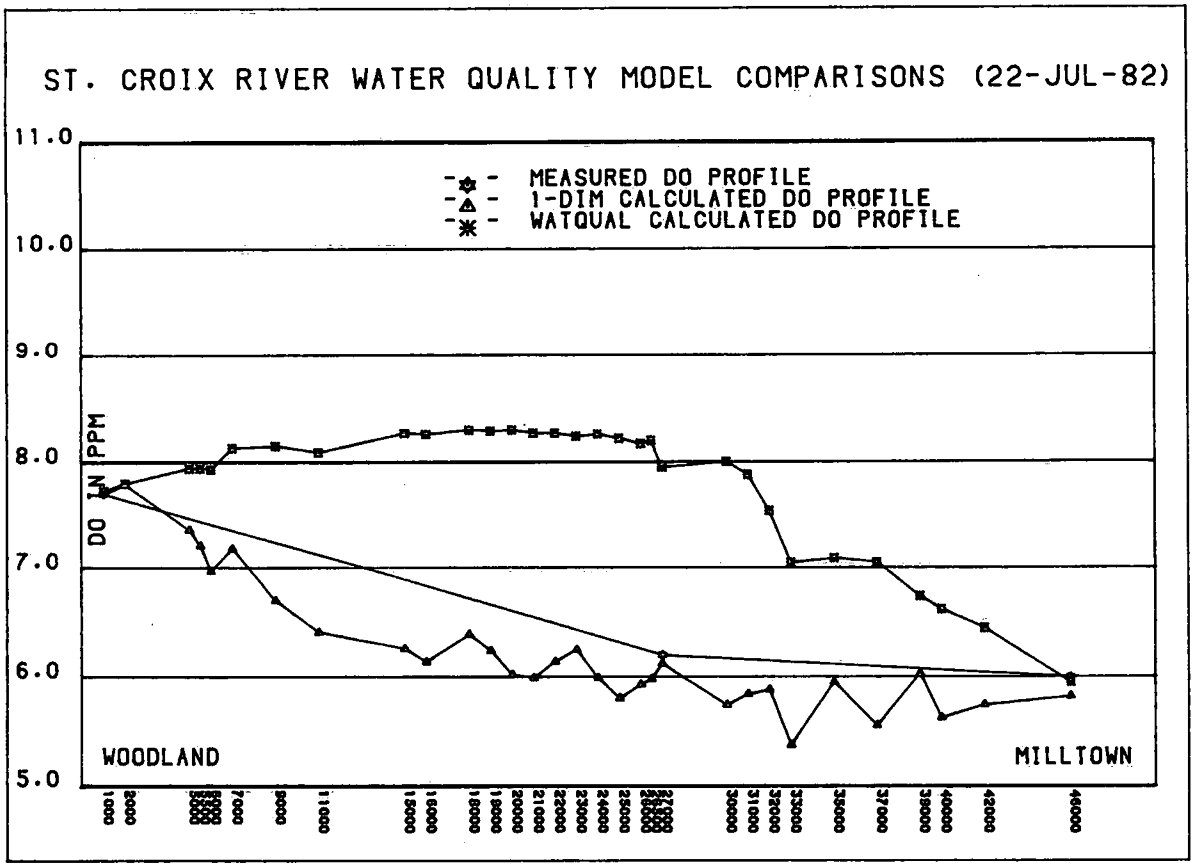
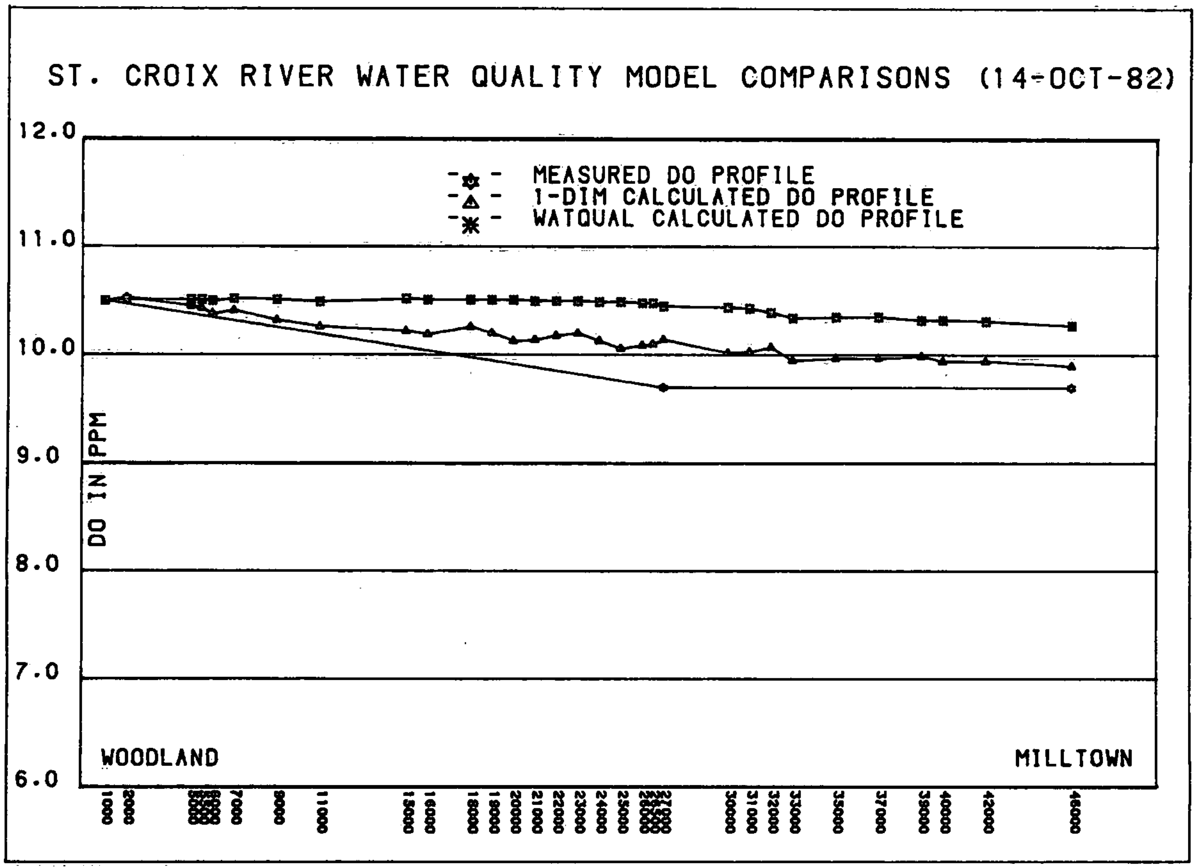


FIGURE 27



Water Quality Management

Two separate modelling studies have been conducted on the St. Croix for water quality. The first was undertaken with the help of two distinct models: the hydraulic model HEC-2 and the water quality model WATQUAL. Simulations had proven to be successful, as seen in Technical Bulletin No. 121 (Boutot and Clair, 1981). The second study was undertaken to investigate the water quality modelling aspects of the river by means of a more sophisticated model, the One-Dimensional Hydrodynamic model.

The simulation with this one-dimensional model was also shown to be successful and is as good as HEC-2 and better than WATQUAL. If we compare the versatility of the one-dimensional model as opposed to WATQUAL and HEC-2, we find that the one-dimensional model has several advantages.

It is recommended that the One-Dimensional Hydrodynamic model be used in future river studies, as this model can simulate water quantity and water quality simultaneously or separately. Also, far more water quality parameters can be analyzed than from the previous study, since the one-dimensional model can accommodate up to 13 such parameters.

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