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Simulation of unsteady flow and solute transport in a tidal river network

Xiaoyong Zhan
Kansas Geological Survey, The University of Kansas, Lawrence, Kansas, USA

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Abstract *A mathematical model and numerical method for water flow and solute transport in a tidal river network is presented. The tidal river network is defined as a system of open channels or rivers with junctions and cross sections. As an example, the Pearl River in China is represented by a network of 104 channels, 62 nodes, and a total of 330 cross sections with 11 boundary sections for one of the applications. The simulations are performed with a supercomputer for seven scenarios of water flow and/or solute transport in the Pearl River, China, with different hydrological and weather conditions. Comparisons with available data are shown. The intention of this study is to summarize previous works and to provide a useful tool for water environmental management in a tidal river network, particularly for the Pearl River, China.*

1. Introduction

Water environment protection and flood defense in river network and estuarine systems require an understanding of the hydrodynamic processes. Qualitative and quantitative evaluation of the hydrodynamic and transport properties can be performed via mathematical or numerical simulation models. To accurately simulate both the temporal and spatial variations of tidal flow, which significantly define the transport processes, the simulation model must be capable of capturing hydrodynamics and accounting for hydraulic and tide-induced fluctuations, water withdrawals, discharges, non-uniform geometric configurations, and other man-made or natural factors in a tidal river network. To facilitate further discussion, a river network is defined as a system of open channels, or river reaches, or river branches, either simply connected in treelike fashion or multiply connected in a configuration that permits more than one flow path to exist between certain locations in the system. The primary subdivision of a channel within a network is referred to as a segment which has two cross sections as terminals. Apparently, computation

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of flow and transport in such a river network is much more involved than that of one direction flow and transport in a single and simple river or stream. During the past several decades, attention has been focused on developing a useful simulator based on the finite difference method. One such model was presented by Schaffranek (1998) and a good review was provided by Sobey (2001).

The purposes of this paper are to bolster the theoretical foundations and supercomputing techniques of a well-developed method for the analysis of tidal flow in a river network and to demonstrate field applications of the method to the Pearl River. The present model is based on the de Saint-Venant equations and implicit finite difference method. While it is implemented for a supercomputer with multi-processors for potentially very large scale applications, the computer simulator is also applicable with desktop computers for individual use to small-scale problems. Applications to the Pearl River network are demonstrated with varying weather conditions. The Pearl River is the largest and the most important river in South China for the social, economic, and cultural development of Hong Kong, Shenzhen, Guangzhou, Zhuhai, Macau, and other cities in the Pearl River Delta region (Chau and Jiang, 2001). The population of the Pearl River Delta is over 20 million. Water in the Pearl River is the source of drinking water and for industrial and agriculture uses, but flood and tidal flow could destroy property and lives if not properly managed. For the application scenario, numerical results are presented comparatively with observed and measured field data.

In Section 2, the governing equations with boundary conditions are described, followed by the presentation of the numerical algorithm in Section 3. Performance of the computer code on a supercomputer is illustrated in Section 4. Applications to seven scenarios with field data are discussed in Section 5. In Section 6, a brief conclusion is presented.

2. Mathematical model

2.1 Governing equations

One-dimensional unsteady, free surface flow in a river network can be described by two partial-differential equations expressing mass and momentum conservations. These well-known equations, often referred to as unsteady flow equations, or de Saint-Venant equations (de Saint-Venant, 1871), can be written as:

$$B\frac{\partial Z}{\partial t}+\frac{\partial Q}{\partial x}=q \tag{1}$$

$$\left(gF-\frac{BQ^2}{F^2}\right)\frac{\partial Z}{\partial x}+\frac{\partial Q}{\partial t}+\frac{2Q}{F}\frac{\partial Q}{\partial x}=\frac{Q^2}{F^2}\frac{\partial F}{\partial x}\bigg|_z-g\frac{n^2Q|Q|}{FR^{\frac{4}{3}}} \tag{2}$$

One-dimensional solute transport in a river network can be described by an advection-dispersion equation for mass balance. It is written as:

$$\frac{\partial(FC)}{\partial t} + \frac{\partial(QC)}{\partial x} = \frac{\partial}{\partial x}\left(EF\frac{\partial C}{\partial x}\right) + S \tag{3}$$

In these equations, formulated using water-surface elevation Z, tidal-flow discharge Q, and solute concentration C as the dependent variables, distance along the channel thalweg x and elapsed time t are the independent variables. Longitudinal distance x and tidal flow discharge Q are positive in the downstream direction. $F(x, Z(x, t)$ is the area of the cross section, represented by a rectangle or trapezoid; $B(x, Z(x, t)) = \partial F/\partial Z$ is total top width of cross section; $R(x, Z(x, t))$ is the hydraulic radius of the cross section. The commonly used substitution of hydraulic depth F/B is assumed valid for shallow water bodies; that is, channels having a large width-to-depth ratio; n is the Manning's coefficient, expressing flow-resistance; g is the gravitational acceleration; q is the lateral inflow per unit length of channel, negative for outflow; E is the longitudinal dispersion coefficient; and S is source or sink. The above equations are, in general, descriptive of unsteady flow and solute transport in a channel of arbitrary geometric configuration having both conveyance and overflow areas and potentially subject to continuous lateral flow.

The following assumptions are implied for the above equations. The water is homogeneous in density. The hydrostatic pressure prevails everywhere in the channel. The channel bottom slope is mild and uniform. The channel bed is fixed; no scouring or deposition occurs. The reach geometry is sufficiently uniform to permit characterization in one dimension. The frictional resistance is the same as for steady flow, thus permitting approximation by the Chezy or Manning equation.

The governing equations defy an analytical solution. However, they may be numerically solved if some adequate initial and boundary conditions at external and internal junctions are provided.

2.2 Boundary conditions

To solve a group of equations (1)–(3) describing tidal flow and solute transport in a river network, boundary conditions must be specified at internal junctions located at branch confluences within the network as well as at external junctions located at the extremities of branches, for example, where branches physically terminate or are delimited for modelling purposes.

Equations describing the boundary conditions at internal junctions are tidal-flow discharge and water-stage compatibility conditions expressed by neglecting velocity-head differences and turbulent energy losses. At a junction of n branches, discharge continuity requires that

$$\sum_{i=1}^{n} Q_i = W_j \tag{4}$$

where W_j is zero or some user-specified external flow, inflow or outflow, at junction j. The stage compatibility requires that

$$Z_i = Z_{i+1} \quad (i = 1, 2, \ldots, n-1) \tag{5}$$

The concentration compatibility states that

$$C_i = C_{i+1} \quad (i = 1, 2, \ldots, n-1) \tag{6}$$

Boundary condition equations for external junctions are formulated by the model from user-supplied time-series data or functions of water level, flow, or concentration. Various combinations can be specified. For example, a null-discharge condition as at dead-end channel, known stage or discharge as a function of time, or a known, unique stage-discharge relationship can be prescribed.

Together, the internal and external boundary conditions provide a sufficient number of additional equations to satisfy requirements of the solution.

3. Finite difference schemes

The numerical solution of the governing equations are executed in two steps. First, the equations are replaced by algebraic finite difference equations, and second, the solution of the difference equations is obtained through solving linear equations. A variety of numerical methods may be developed on the basis of how the operations in either or both of the steps are performed. In this paper, the finite difference forms for the governing equations may be written for the point $P((x_i + x_{i+1})/2, t^n + \alpha\Delta t)$ within the space-time grid system, which depicts the region in which solution of the governing equations is sought, using the values of various quantities from the four neighboring grid points. The symbols α, β, and γ represent weighting factors employed to specify the location and time, respectively, within the Δx_i, computational cross section delimit, and the Δt_n, computational time increment, at which derivative and functional quantities $f(P)$ are to be evaluated. The time derivatives are approximated by the finite difference expression, centered both in space and time ($\alpha = 0.5$).

$$\frac{\partial f(P)}{\partial t} = \frac{Z_i^{n+1} + Z_{i+1}^{n+1} - Z_i^n - Z_{i+1}^n}{2\Delta t_n} \tag{7}$$

The spatial derivatives are expressed by a finite difference relation weighted in time,

$$\frac{\partial f(P)}{\partial x} = \beta \frac{Z_{i+1}^{n+1} - Z_i^{n+1}}{\Delta x_i} + (1-\beta)\frac{Z_{i+1}^n - Z_i^n}{\Delta x_i} \tag{8}$$

β is usually assigned in the range $0.5 \leq \beta \leq 1.0$ for the unconditional stable situation. A value of 1.0 yields the fully forward scheme used by Baltzer and Lai (1968), whereas a value of 0.5 yields the fully centered scheme employed by Amein (1966).

The non-derivative terms are discretized by

$$f(P) = \gamma \frac{Z_{i+1}^{n+1} - Z_i^{n+1}}{2} + (1-\gamma)\frac{Z_{i+1}^{n} - Z_i^{n}}{2} \tag{9}$$

γ may be assigned in the range $0.0 \leq \gamma \leq 1.0$. If γ is not equal to zero, these functional values may be evaluated on the same time level as the spatial derivatives ($\gamma = \beta$) or at any other different level within the time increment, through an iterative procedure.

Substitution of the operators defined earlier into the governing equations (1) and (2) for unsteady flow yields the following finite difference expressions for a segment within a channel,

$$B_p \frac{Z_i^{n+1} + Z_{i+1}^{n+1} - Z_i^n - Z_{i+1}^n}{2\Delta t_n} + \beta \frac{Q_{i+1}^{n+1} - Q_i^{n+1}}{\Delta x_i} + (1-\beta)\frac{Q_{i+1}^{n} - Q_i^{n}}{\Delta x_i} = q_p \tag{10}$$

$$\begin{aligned}
&\left(gF - \frac{BQ^2}{F^2}\right)_p \left[\beta \frac{Z_{i+1}^{n+1} - Z_i^{n+1}}{\Delta x_i} + (1-\beta)\frac{Z_{i+1}^{n} - Z_i^{n}}{\Delta x_i}\right] \\
&\quad + \frac{Q_i^{n+1} + Q_{i+1}^{n+1} - Q_i^n - Q_{i+1}^n}{2\Delta t_n} \\
&\quad + \left(\frac{2Q}{F}\right)_p \left[\beta \frac{Q_{i+1}^{n+1} - Q_i^{n+1}}{\Delta x_i} + (1-\beta)\frac{Q_{i+1}^{n} - Q_i^{n}}{\Delta x_i}\right] \\
&= \gamma \left[\left(\frac{Q^2}{2F^2}\right)_{i+1}^{n+1} + \left(\frac{Q^2}{2F^2}\right)_{i}^{n+1}\right] \\
&\quad \times \frac{F_{i+1}\left(Z_i^{n+1}\right) + F_{i+1}\left(Z_{i+1}^{n+1}\right) - F_i\left(Z_i^{n+1}\right) - F_i\left(Z_{i+1}^{n+1}\right)}{2\Delta x_i} \\
&\quad + (1-\gamma)\left[\left(\frac{Q^2}{2F^2}\right)_{i+1}^{n} + \left(\frac{Q^2}{2F^2}\right)_{i}^{n}\right] \\
&\quad \times \frac{F_{i+1}(Z_i^n) + F_{i+1}(Z_{i+1}^n) - F_i(Z_i^n) - F_i(Z_{i+1}^n)}{2\Delta x_i} - \left(g\frac{n^2 Q|Q|}{FR^{\frac{4}{3}}}\right)_p
\end{aligned} \tag{11}$$

or expressed in the following form,

$$A_{1i}Z_i^{n+1} + B_{1i}Q_i^{n+1} + C_{1i}Z_{i+1}^{n+1} + D_{1i}Q_{i+1}^{n+1} = E_{1i} \tag{12}$$

$$A_{2i}Z_i^{n+1} + B_{2i}Q_i^{n+1} + C_{2i}Z_{i+1}^{n+1} + D_{2i}Q_{i+1}^{n+1} = E_{2i} \tag{13}$$

in which

$$A_{1i} = \Delta x_i B_p = C_{1i} \tag{14}$$

$$B_{1i} = -2\beta\Delta t_n = -D_{1i} \tag{15}$$

$$E_{1i} = \Delta x_i B_p\left(Z_{i+1}^n + Z_i^n\right) - 2(1-\beta)\Delta t_n\left(Q_{i+1}^n - Q_i^n\right) + 2\Delta t_n \Delta x_i q_p \tag{16}$$

$$A_{2i} = -2\beta\Delta t_n\left(gF - \frac{bQ^2}{F^2}\right)_p = -C_{2i} \tag{17}$$

$$B_{2i} = \Delta x_i - 2\beta\Delta t_n\left(\frac{2Q}{F}\right)_p = D_{2i} \tag{18}$$

$$\begin{aligned}
E_{2i} = {} & -2(1-\beta)\Delta t_n\left(gF - \frac{bQ^2}{F^2}\right)_p\left(Z_{i+1}^n - Z_i^n\right) \\
& + \Delta x_i\left(Q_{i+1}^n + Q_i^n\right) - 2(1-\beta)\Delta t_n\left(\frac{2Q}{F}\right)_p\left(Q_{i+1}^n - Q_i^n\right) \\
& + \gamma\Delta t_n\left[\left(\frac{Q^2}{2F^2}\right)_{i+1}^{n+1} + \left(\frac{Q^2}{2F^2}\right)_i^{n+1}\right] \\
& \times \left(F_{i+1}\left(Z_i^{n+1}\right) + F_{i+1}\left(Z_{i+1}^{n+1}\right) - F_i\left(Z_i^{n+1}\right) - F_i\left(Z_{i+1}^{n+1}\right)\right) \\
& + (1-\gamma)\Delta t_n\left[\left(\frac{Q^2}{2F^2}\right)_{i+1}^{n} + \left(\frac{Q^2}{2F^2}\right)_i^{n}\right] \\
& \times \left(F_{i+1}\left(Z_i^n\right) + F_{i+1}\left(Z_{i+1}^n\right) - F_i\left(Z_i^n\right) - F_i\left(Z_{i+1}^n\right)\right) \\
& - 2\Delta t_n \Delta x_i\left(g\frac{n^2 Q|Q|}{FR^{\frac{4}{3}}}\right)_p
\end{aligned} \tag{19}$$

where the notation f_p denotes the function value of f^{n+1} given by equation (9) and the f term is updated through iteration in the computation process.

Substitution of the operators defined earlier into the governing equation (3) for solute transport yields the following finite difference expression for a segment within a channel. For the time derivative term,

$$\frac{\partial(FC)}{\partial t}=\frac{(FC)_i^{n+1}-(FC)_i^n}{\Delta t_n} \tag{20}$$

For the advection term,

$$\begin{aligned} Q_i<0: &\quad \frac{\partial(QC)}{\partial x}=\frac{(QC)_i^n-(QC)_{i-1}^n}{\Delta x_{i-1}} \\ Q_i=0: &\quad \frac{\partial(QC)}{\partial x}=0 \\ Q_i>0: &\quad \frac{\partial(QC)}{\partial x}=\frac{(QC)_{i+1}^n-(QC)_i^n}{\Delta x_i} \end{aligned} \tag{21}$$

For the dispersion term,

$$\begin{aligned} \frac{\partial}{\partial x}\left(EF\frac{\partial C}{\partial x}\right) &= \frac{1}{\Delta x_i+\Delta x_{i-1}}\frac{\left[(EF)_{i+1}^n+(EF)_i^n\right]\left(C_{i+1}^n-C_i^n\right)}{\Delta x_i} \\ &\quad -\frac{1}{\Delta x_i+\Delta x_{i-1}}\frac{\left[(EF)_i^n+(EF)_{i-1}^n\right]\left(C_i^n-C_{i-1}^n\right)}{\Delta x_{i-1}} \end{aligned} \tag{22}$$

The solute transport equation becomes,

$$C_i^{n+1}=M_iC_{i+1}^n+N_iC_i^n+O_iC_{i-1}^n+P_i^n \tag{23}$$

The equations developed earlier can be applied to all Δx_i segments within the river network and the resultant equation set can be solved directly using appropriate boundary conditions and initial values. The solution process begins at time t_0 by use of specified initial conditions and proceeds in Δt_n time increments to the end of simulation at time t_e. Gauss elimination using maximum pivot strategy is employed to solve simultaneously the system of equations, including the segments and boundary equations. Iteration within a time step is performed to provide results within tolerances specified, such as 0.01 m for tidal level. The primary effect of iteration is to improve on the quantities taken as local constants within the time step, which in turn increases the accuracy of the computed unknowns. Accuracy requirements are typically achieved in several iterations per time step.

4. Computation

Computer code for the present model was written in FORTRAN 77/90 and has been tested on a variety of computer platforms. The computations are mainly performed on a Silicon graphics (SGI) original 2000 supercomputer located at the University of Kansas. Origin is a scalable, shared memory processor (SSMP) system, and it departs significantly from previous SGI large-system architectures. The system at the University of Kansas was upgraded in 2000 when 32 R12000 processors, running at 400 MHz were added. All of the new 400 MHz processors were configured with 8 MB L2 cache, and the total main memory was expanded to 16 GB. Processors and memories are arranged in nodes, where each node includes two processors, 512 MB of memory, and an intelligent hub that connects the processors to the memory and to other nodes in the system. Memory is distributed among all nodes in the system, but it functions as a single, shared address space. Access to a node's memory is provided to both local and remote CPUs by the node hub. The SSMP architecture allows users to enjoy many of the benefits of both distributed memory and shared memory multiprocessor (SMP) systems. Users may design programs as if they are to run on a single, shared memory, as they do in single-processor computer systems and as most programming languages are designed to accommodate. Individual programs execute with considerable isolation from other programs, thereby minimizing competition for system resources that can lower overall system performance. The system is highly "scalable", allowing processors to be added to the system without requiring program modifications to utilize the additional capacity and without compromising overall performance. The operating system may migrate process threads and data among nodes to minimize internode's traffic.

The code for this model is partially rewritten for parallelization and is compiled with automatic parallelization. A comparison between performances of sequential version and parallel version of the code for one scenario of applications to the Pearl River is shown in Table I, where about two to three times difference between using the sequential version and the parallel version can be observed.

5. Applications

Water quantity and water quality have been major issues for the Pearl River in the last several decades. The Pearl River is a multi-purpose water network

Table I. Comparison between performances of sequential and parallel code

Timing in second	Sequential with one CPU	Parallel with eight CPUs
Real	345	111
User	289	52
System	41	39

(Figure 1), which serves for navigation, water supply for industrial and residential users, fishing, recreation, and unfortunately as a receiving body for wastewater discharge for some areas. For economical and environmental reasons, the Pearl River Delta is the most important region in South China. Continued monitoring and investigations for both hydrology and water quality have been undertaken intensively in the last several decades, along with studies on similar rivers for comparison (D'Alpaos and Zhan, 1991; Wang *et al.*, 1986a, b, 1987; Yan *et al.*, 1987; Zhan, 1981, 1983, 1984a, 1988; Zhan and Chen, 1989; Zhan and Feng, 1984; Zhan and Huang, 2002; Zhan *et al.*, 1988, 1989). Apparently, a well-developed quantitative model will help the planning and management of water resource and water environment of the Pearl River network.

For the applications of the current model to practical problems, several scenarios are reviewed or presented here. These scenarios cover a wide range of hydrological conditions, such as spring tide in flooding season, spring tide in drought season, spring tide with storm, typhoon, or neap tide. Most of the field applications involve, as the system shown in Figure 1, 104 channels, 62 nodes, and a total of 330 cross sections with 11 boundary sections. Each channel of the network is defined by a series of non-uniform schematized cross sections. Because of irregular geometry and bottom elevation of channels, cross sections vary along channel and are represented in terms of hydrological parameters such as width, area, and hydraulic radius as a function of depth or water level, based on field-measured data. Storage areas such as an embayment in each cross section are also considered in the relationships of width-depth or area-depth. The parameters used are based on the calibration and verification results in the previous report (Zhan, 1986). Brief descriptions for the scenarios of applications are summarized in Table II.

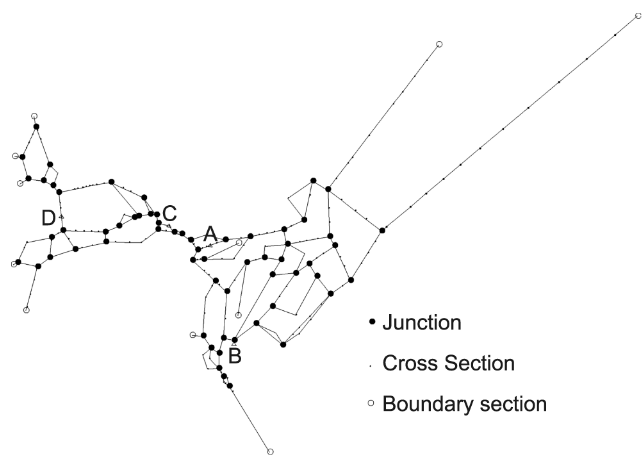

Figure 1.
Computational network of the Pearl River (A, B, C, and D are observation stations for tidal level)

5.1 1968 high tide with flood flow

High tide with flooding was observed on 27 June 1968, and the model was applied to simulate the historical flood. The highest water level of four stations indicated in Figure 1 approaches 2 m, one of the highest in the history and two times that for normal flow season. The lowest water level is less than −1 m, much higher than that for normal flow season. Comparisons between observations and simulations are presented in Figure 2 for the low high tide, high low tide, high high tide, and low low tide for four stations.

5.2 1974 high tide with typhoon weather condition

Typhoon is a disaster weather condition in South China every year. On 21-23 July 1974, high tide with typhoon weather condition was recorded and one of the real applications of the model involves the simulation of tidal level and flow during the typhoon period. Detailed discussions can be found in the work of Zhan (1992) where the model results, using a 30 min time step for computation,

Network	Time	Channels	Junctions	Sections	Boundaries
Dongjiang	10/1-3/1988	10	4	21	4
Guangzhou	6/19-20/1983	34	20	103	7
Guangzhou	6/25-26/1983	34	20	103	7
Pearl River	12/16-26/1987	98	58	310	11
Pearl River	7/27/1968	104	62	330	11
Pearl River	7/22/1974	104	62	330	11
Pearl River	7/6/1978	104	62	330	11
Pearl River	3/25-27/1982	104	62	330	11

Table II. Scenarios of applications

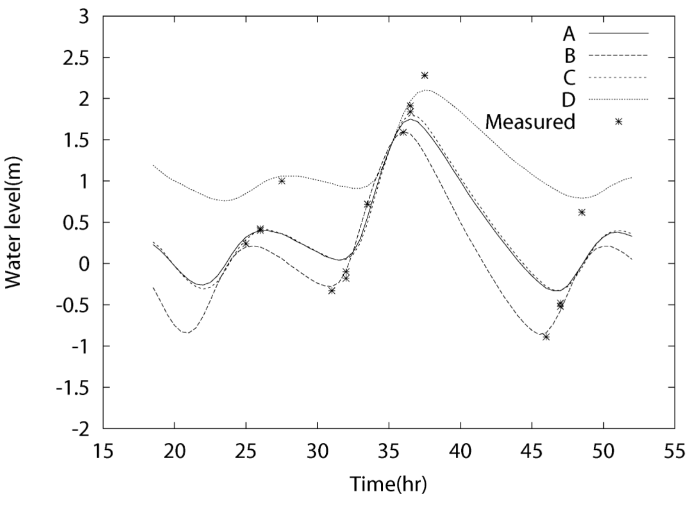

Figure 2. 1968 high tide with flood flow

were shown at four stations for both tidal level and flow for the low high tide, high low tide, high high tide, and low low tide. As shown in Figure 3, the highest water level of four stations indicated in Figure 1 was over 2 m, one of the highest in the history with typhoon weather.

5.3 1978 high tide under normal river flow condition

Spring tide was observed on 6 July 1978, with normal flow conditions. The numerical simulation was conducted, using a 30 min time step for computation, to show at four stations the normal tidal-level variations for spring tide. As in Figure 4, the highest water level is around 1 m, almost half of the water level at flood or typhoon seasons. The lowest water level is around −1.5 m.

5.4 1982 high tide in drought season

Drought can cause trouble to agriculture as well as to water quality due to the saltwater intrusion. From 25 to 26 March 1982, high tide with drought weather condition was recorded and the model was applied to the simulation of tidal level and flow during the drought period. Detailed discussions can be found in the work of Zhan *et al.* (1992) where the comparisons between observed and computed water levels and/or tidal flows at 24 stations were presented. The model results are shown in Figure 5 for tidal level of the low high tide, high low tide, high high tide, and low low tide at four stations. It is clear that highest water level of four stations is around 1 m, but the lowest water levels are around −1 m, indicating the drought condition and little freshwater contribution to the river network.

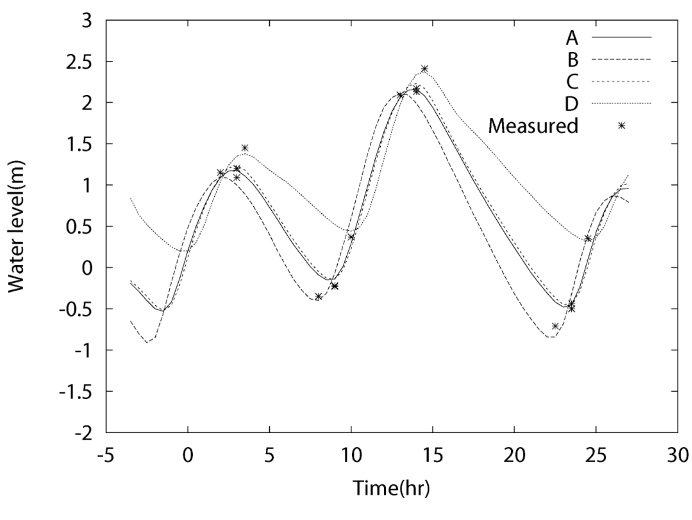

Figure 3.
1974 high tide with typhoon weather conditions

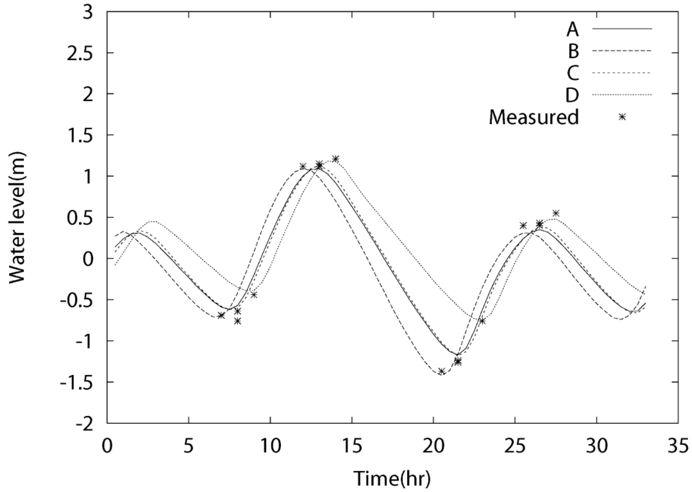

Figure 4.
1978 high tide under normal river flow conditions

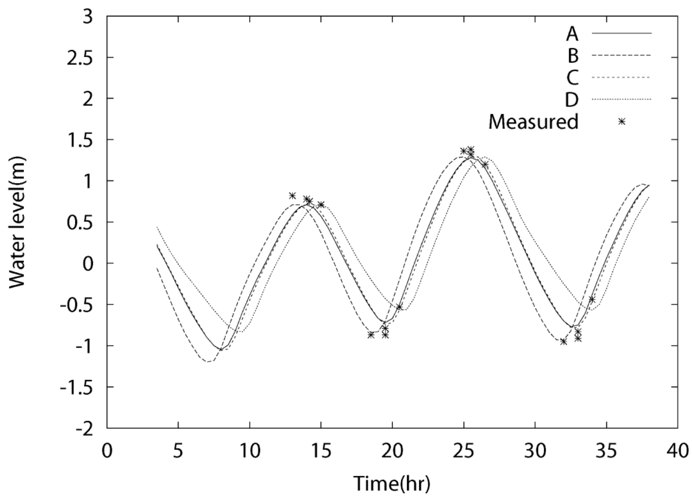

Figure 5.
1982 high tide in drought season

5.5 1983 high tide in flood season

Simulation of tidal flow and solute transport in flood season in the Guangzhou river network were undertaken from 19 to 20 June 1983 for calibration, and from 25 to 26 June 1983 for verification. Detailed discussions can be found in the work of Zhan (1986), which mainly are based on previous investigations by Zhan (1984b). It was the first time ever to deal with solute transport in the Pearl River and reasonable agreement can be found between observed and computed water levels, tidal flow, and chemical oxygen demand (COD) for all stations with all available measured data.

5.6 1987 high tide in flood season

Simulation of tidal flow in flood season in the Pearl River network was undertaken from 16 to 26 December 1987, based on continuous observation data in winter. Detailed discussions can be found in the work of Zhan and Wu (1990) where excellent agreements between observation and computation results for both water level and water flow were shown.

5.7 1988 high tide in flood season in a canal

Simulation of tidal flow in flood season in Dongjiang Canal, part of the Pearl River, was performed from 1 to 3 October 1988, based on continuous observation data in the Fall season. Detailed discussions can be found in the work of Zhan and Wu (1990) where good agreement between observation and computation results was shown.

6. Conclusions

An applicable one-dimensional finite difference model is presented for simulations of tidal and flood flow in a river network of interconnected channels. The benefit of supercomputing of the model is demonstrated. Applications on analysis of tidal level and water flow in the Pearl River indicate that the model covers different hydrological and weather conditions; and therefore, the present model can be used for flood defense, water environmental protection, and water-resource management.

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