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Metropolitan Water Reclamation District of Greater Chicago

RESEARCH AND DEVELOPMENT DEPARTMENT

REPORT NO. 03-18

HYDRAULIC CALIBRATION OF AN UNSTEADY FLOW MODEL FOR THE CHICAGO WATERWAY SYSTEM

Prepared By

Institute for Urban Environmental Risk Management
Marquette University, Milwaukee Wisconsin

September 2003

Metropolitan Water Reclamation District of Greater Chicago
100 East Erie Street Chicago, IL 60611-2803 (312) 751-5600

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Institute for Urban Environmental Risk Management
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Research and Development Department
Richard Lanyon, Director

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Marquette University, Milwaukee WI 53201-1881

TECHNICAL REPORT # 14

HYDRAULIC CALIBRATION OF AN UNSTEADY FLOW MODEL FOR THE CHICAGO WATERWAY SYSTEM

SUBMITTED TO

The Metropolitan Water Reclamation District of Greater Chicago

Ram L. SHRESTHA, M.S.

Department of Civil and Environmental Engineering

Charles S. MELCHING, Ph.D, P.E.

Department of Civil and Environmental Engineering

Milwaukee, Wisconsin

August 2003

ABSTRACT

The flow and the water-quality processes in the Chicago Waterway System (CWS) are very complex and critical water-quality conditions may result under a wide range of flows. The dominant uses of the CWS are for commercial and recreational navigation and for urban drainage. The water-quality model QUAL2E has been previously developed for water-quality planning and management purposes for the CWS. Due to its applicability only for steady and low flows, QUAL2E cannot solve the problems related to reverse flow of the Chicago River, impact of reduced discretionary diversions from Lake Michigan, changes in runoff and nonpoint source loads resulting from the Tunnel and Reservoir Plan (TARP). The water quality and flow in the CWS change very frequently. So a water-quality model that can simulate water-quality processes under unsteady flow conditions is necessary for water-quality management. The DUFLOW model developed in The Netherlands was selected for simulation of the CWS. The model was run at a 15-min. time step for 8 long periods of complete data during the period August 1, 1998 to July 31, 1999. The main objective of this study is to construct an accurate hydraulic model for unsteady-flow conditions on the CWS so that it could be coupled with water-quality simulation routines and used for water-quality planning and management.

Comparison of measured and simulated stage data is good at four locations on the Chicago Sanitary and Ship Canal. The stage simulation agreed with the measured data nearly always within one percent relative to the depth. In case of discharge, simulated flows at the upstream boundaries are substantially less than the measured values. This results because the measured and estimated flows into the CWS were 1.2 to 7.7% higher than the measured outflow for the CWS. Thus, the flow imbalance was compensated for at the upstream stage boundaries. During the period with only a 1.2% difference between inflows and outflows on the CWS (May 27 - June 12, 1999) the agreement between the simulated and measured flows at the boundaries was quite close. Thus, the developed hydraulic model was considered adequate for water-quality simulation on the CWS.

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CHAPTER ONE: INTRODUCTION

1.1 Introduction

The Chicago Waterway System (CWS) is that part of the Calumet and Chicago River Systems that has significantly changed since the time of European settlement. Perhaps no other system of natural rivers has been so completely transformed as has the CWS. Over time the Calumet and Chicago Rivers and some of their tributaries have been deepened, straightened, and widened, and canals have been dug to aid in reversing the course of both rivers to carry drainage and effluent from the Chicago area away from Lake Michigan. Upstream tributary reaches were first channelized for agricultural drainage, and are now maintained for urban storm drainage. Lower reaches were channelized and armoured with concrete, steel, or timber walls to accommodate commercial navigation. The Calumet and Chicago River Systems are shown in Figure 1.1.

The sewerage system of early Chicago was primitive, with gutters serving as drains in many streets. Improvements were made in the sewerage system using underground pipes, but they discharged either directly into Lake Michigan or into the river, which flowed into the lake. Due to the pollution of drinking water sources, people were plagued by typhoid fever and dysentery. Disease resulting from water polluted by human waste and the nuisance condition of the rivers brought about a demand for action.

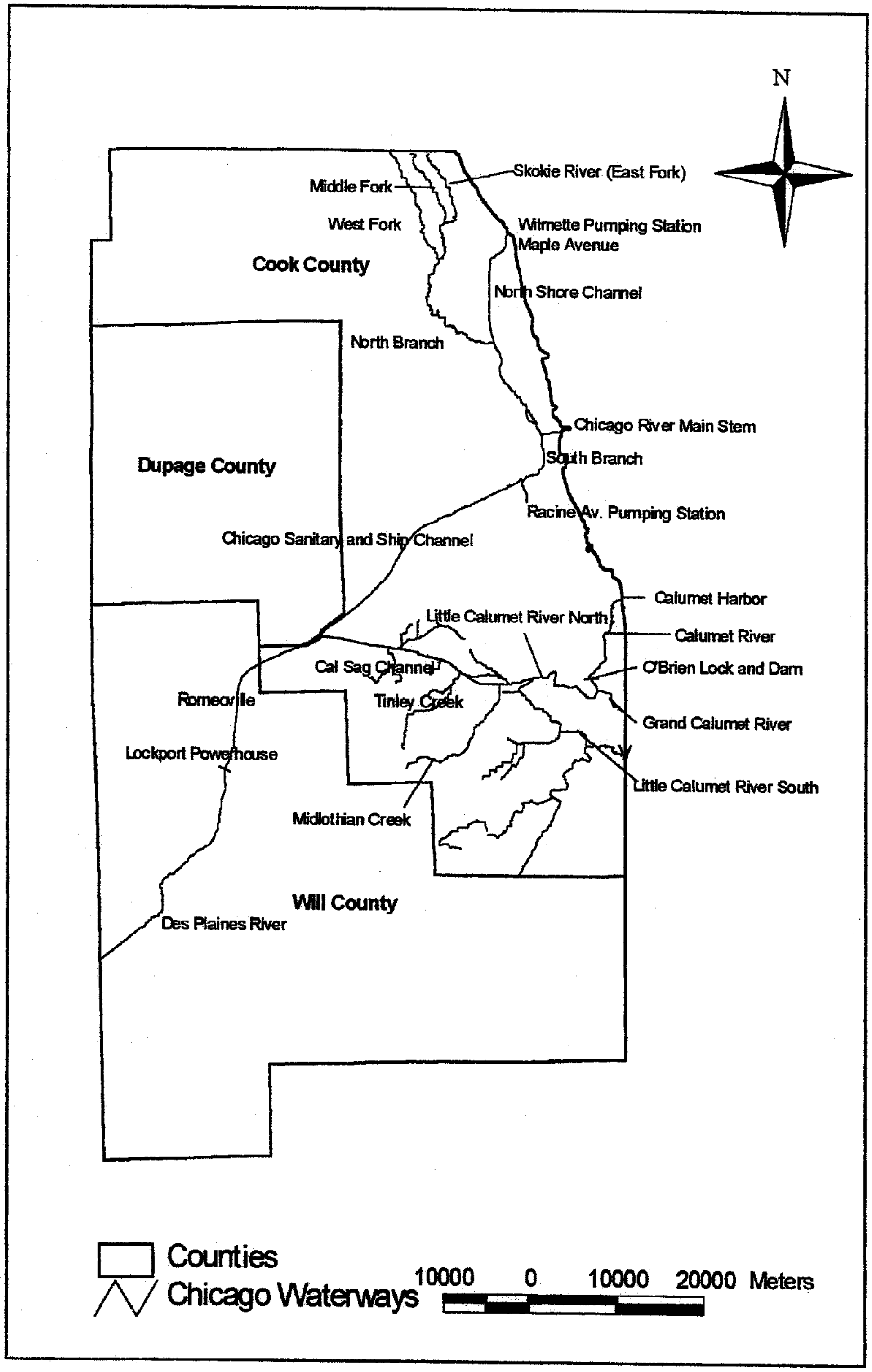


Figure 1.1. The Calumet and the Chicago River System

In 1887, it was decided to attempt a bold engineering feat and reverse the Chicago River. Rudolph Hering, chief engineer of the drainage and water supply commission, proposed to excavate a canal from the southerly tip of the South Branch of the Chicago River and carry the wastes away from the lake and down to the Mississippi River through the Des Plaines and Illinois Rivers.

To reverse the flow of the Chicago River, a 28-mile canal was built from the South Branch of the river through the low summit and down to Lockport. It was completed in 1900. Today the flow in this canal, commonly known as the Chicago Sanitary and Ship Canal (CSSC), is controlled by lock gates and sluice gates at the mouth of the Chicago River and at Lockport. Thus, Chicago had built the first of its own rivers to dispose of effluents.

In 1910, another small artificial river was completed by building a dam, lock, and pumping plant at Wilmette and by digging the 8-mile long North Shore Channel, connecting Lake Michigan with the North Branch Chicago River. The wastes from the north suburban communities of Evanston, Wilmette, Winnetka, and others were diverted away from the lake and drained through the newly created Channel and to and thorough the CSSC.

In 1922, the third of Chicago's artificial rivers was created. This river, the Calumet-Sag (Cal-Sag) Channel, extends 16 miles westward from the Little Calumet River at Blue Island to a junction with the CSSC. Here again, the flow of a natural river was diverted away from Lake Michigan and into the main drainage system flowing to the west. Today the entire CWS consists of 78 miles of canals, channels, and rivers.

1.2 Objectives

The Water-Quality Model QUAL2E (Brown and Barnwell, 1987) was applied in the late 1980's and early 1990's to the Chicago Waterway and Upper Illinois River Systems, (CDM, 1992). The QUAL2E model has been used for water-quality planning and management purposes for the CWS and upper Illinois River. However, QUAL2E has several limitations that decrease its usefulness in simulating water quality in the CWS. The primary limitation is that QUAL2E only is applicable for steady, low flows commonly of interest in the development of traditional waste-load allocations wherein summer low flows commonly result in the critical water-quality conditions.

Steady-flow analysis typically does not consider all the forces acting on the flow and only partially accounts for channel-storage effects. The approximate solutions for piecewise steady-flow analysis are adequate for certain simplified planning or design problems but are inadequate for many others (for example, streams with rapidly rising and falling stage and flat slopes). In unsteady flow, some aspect of the flow (velocity, depth, pressure, or another characteristic) is changing with time. In 1-D flow, longitudinal acceleration is significant, whereas transverse and vertical accelerations are negligible. With the recent increases in the calculation speed and storage capabilities of computers, simulation of unsteady flow in a complex stream system with many hydraulic structures has become practicable.

The flow and water-quality processes in the CWS are very complex and water-quality conditions vary under a wide range of flows. Recent intensive sampling of dissolved oxygen (DO) throughout the CWS done by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) has found that the worst DO conditions result during storms, thus, stressing the need for simulation of unsteady flow conditions. Also, future scenarios of interest to the MWRDGC such as flow reversal on the Chicago River mainstem, reduced discretionary diversions from Lake Michigan, changes in runoff and nonpoint source loads resulting from the Tunnel and Reservoir Plan (TARP), etc. require analysis of unsteady-flow conditions. A water-quality model for unsteady-flow conditions gives the time series of water quality and flow in the system. The flow and water-quality condition for any low and high flow can be predicted.

The objective of this study is to develop, calibrate, and verify a hydraulic model for unsteady-flow conditions. This model of unsteady flow hydraulics will be coupled with a dynamic model of water quality to simulate changes in water quality in the CWS as a result of unsteady flow conditions.

1.3 Scope of this Report

The purpose of this report is to document the implementation, calibration, and verification of an unsteady flow model for the CWS. The ability to reproduce a period of unsteady flow with the calibrated model is demonstrated by comparing the simulation results for eight different periods between August 1, 1998 and July 31, 1999 to measured stage and discharge data for those periods. The model was calibrated using hourly stage data at three gages operated by the MWRDGC along the CSSC and at the downstream boundary at Romeoville operated by the U.S. Geological Survey (USGS), and using daily flow data collected by the USGS near the Chicago River Controlling Works (CRCW) and O'Brien Lock and Dam upstream boundaries.

The ungaged tributaries and watersheds as well as the effect of the combined sewer overflow (CSO) flows on the CWS was studied. Those unmeasured flows were estimated by a suitable mathematical approach. Since this report is only focused on the hydraulic model, it will not study the impact on water quality.

English units are used for the description of the channels. However, simulation output of water level (stage) and flow are in metric units because the selected model DUFLOW works in metric units. One meter is equivalent to 3.28 feet and one cubic meter per second (m^3/s) is equivalent to 35.3 cubic feet per second.

CHAPTER TWO: MODELING CONCEPTS

2.1 Role of Modeling in Water Quality Analyses

Models are used to predict or compare the future performance of a new system, a modified system, or an existing system under new conditions. In the broadest context, a model can be defined as any organized procedure for the analysis of a problem. The U.S. Environmental Protection Agency (EPA) defines models as processes which are "used to increase the level of understanding of (natural or man-made) systems and the way in which they react to varying conditions." Computer models use the computational power of computers to automate tedious and time-consuming manual calculations. Most models also include extensive routines for data management, including input and output procedures, and possibly including graphics and statistical capabilities. Computer models allow some types of simulations to be performed that could rarely be performed otherwise. Although modeling is generally cheaper than data collection, the uncertainties involved, especially in water-quality simulation, mandate the collection of data for model calibration and verification.

The fate of pollutants or chemicals in the environment is determined by the complex interaction of numerous factors including physical, chemical, and/or biological transformations of the pollutant or chemical within the environment; the characteristics of the surface and/or subsurface media through which transport occurs; and climatological and other external environmental conditions. Mathematical models can provide a mechanism to evaluate the effects of these factors. Further, modeling allows such evaluations to be performed in a time and cost effective manner when compared to conducting resource intensive field monitoring studies.

Finally, modern computer-based tools and approaches to environmental evaluations are appropriate to support activities including the following:

- Understanding key "cause and effect" processes within the natural environment,
- Understanding the role of anthropogenic versus autochthonous or background pollutant or chemical inputs
- Development of waste load allocations and total maximum daily load (TMDL) analyses by simulation of the effects of proposed remedial actions,
- Assisting in outfall siting and diffuser design,
- Determine the time to recovery for a water body after the implementation of a contaminant reduction program,
- Assisting in the design and development of field sampling programs and laboratory bench-scale studies.

2.2 Model Selection

As described previously, mathematical water-quality models are key tools that can be used by water-quality managers in developing management plans for a watershed. Due to the increased computational power of modern computers and expansion of mathematical modeling codes, water-quality managers and modeling practitioners often are faced with a wide choice of model and modeling frameworks with which to evaluate environmental problems. It is very important to make an appropriate choice for water-quality modeling tools for their specific problems.

A number of models are available for simulation of water quality under unsteady-flow conditions. Some models have been developed by U.S. government agencies, for example, the

Water-Quality Analysis and Simulation Program Version 5 (WASP5, Ambrose et. al., 1993), developed by the EPA and the Branched Lagrangian Transport Model (BLTM), (Jobson and Schoellhamer, 1987; Jobson, 1997), developed by the USGS. The water-quality capabilities of these models are quite robust. However, the hydrodynamic portions of these models are less efficient. The hydrodynamic model suggested for coupling with WASP5 has a history of not performing well for one-dimensional unsteady flows in river systems. BLTM requires the development of a separate hydrodynamic model for the river system, and the computed stages and velocities must be transformed from the hydrodynamic-model output to the water-quality model input.

The DUFLOW Model (DUFLOW 3.3, 2000) was jointly developed in The Netherlands by the Rijkswaterstaat, International Institute for Hydraulic and Environmental Engineering (IHE) of the Delft University of Technology, STOWA (Dutch acronym for the Foundation for Applied Water Management Research), and the Agricultural University of Wageningen. DUFLOW may be a reasonable alternative to WASP and BLTM. DUFLOW has been applied with great success to several European river systems (e.g., Manache et al., 2000). It allows several options for the simulation of water quality in stream systems. Finally, its compatibility with Geographical Information Systems (GIS) facilitates representation and display of the river system, its compatibility with Microsoft Windows facilities ease of use, and its relatively low license cost ($1,000 for academic and $ 2,000 for nonacademic use) makes it affordable for many applications. Given these capabilities and advantages, DUFLOW was selected for modeling of the CWS.

2.3 Basic Features of the DUFLOW Model

The DUFLOW modeling system provides the water manager with a set of integrated tools, to quickly perform simple analyses. But the system is equally suitable for conducting expensive, integral studies. It enables water managers to calculate unsteady flows in networks of canals, rivers, and channels. It also is useful for simulating the transport of substances in free-surface flow. More complex water-quality processes can be simulated as well.

In 1988, DUFLOW 1.0 was developed by a collaborative effort of the International Institute for Hydraulic and Environmental Engineering (IHE), the Faculty of Civil Engineering at the Delft University of Technology, and the Public Works Department (Rijkswaterstaat), Tidal Waters Division (now RIKZ).

In 1992 version 2.0 was completed by order of the STOWA, the Agricultural University of Wageningen, Department of Nature Conservation extended the program with water- quality modeling, called DUPROL. Since the relation between quality and flow receives special attention nowadays, a program suitable for modeling both aspects makes DUFLOW a useful tool in water-quality management. In the water-quality part of the model users can supply the process descriptions, or select from two predefined sets of water-quality simulation routines (see Section 2.3.2).

Because users often also need the ability to model the precipitation-runoff process, the precipitation-runoff module RAM was developed by Witteveen + Bos and MX Systems, by order of the STOWA in 1998 leading to DUFLOW 3.0. Furthermore, by order of KIWA (a Dutch consulting firm) the program MODUFLOW was developed. MODUFLOW combines the ground- water model MODFLOW (Harbaugh and McDonald, 1996) and DUFLOW. The product as a whole is called the DUFLOW Modeling Studio (DMS).

DUFLOW is jointly owned by the Rijkswaterstaat, IHE, the Delft University of Technology, STOWA, and the Agricultural University of Wageningen.

2.3.1 Types of Users

The DUFLOW product is designed for various categories of users. The model can be used by water authorities, designers, and educational institutions. DUFLOW runs on a personal computer with a graphical user interface. It can, therefore, be operated in almost every scientific or engineering environment.

In water management the model can be used to simulate the behavior of a system due to operational measures such as opening of sluices, switching on pump stations, or reduction of pollutant loads, etc., and, thus, to optimize the day to day management decisions and evaluate management strategies. In a consultancy environment, the model can be used in the design of hydraulic structures, flood prevention, and river training measures.

The major advantage in engineering education is the short learning time, which is due to its program structure and user oriented input and output.

2.3.2 Design Considerations

DUFLOW is designed to cover a large range of applications, such as propagation of tidal waves in estuaries, flood waves in rivers, operation of irrigation and drainage systems, etc. Basically, free flow in open channel systems is simulated, where control structures like weirs, pumps, culverts, and siphons can be included.

In many water management problems, the runoff from catchment areas is important, and thus, a simple precipitation-runoff model is part of the model set-up in the DMS. With the DMS component RAM the precipitation-runoff processes can be described in detail. The results of a RAM calculation can be used as input for a DUFLOW calculation. In this study, neither the simple precipitation-runoff model nor RAM was applied to estimate runoff from ungaged areas or combined sewer overflows. The estimation of ungaged flows in this study is described in Section 3.2.3.

DUFLOW allows for a number of processes affecting water quality to be simulated, such as algal booms, contaminated silts, salt intrusions, etc., to describe the water quality and to be able to model the interactions between these constituents. There are two water quality models included in DUFLOW as EUTROF1 and EUTROF2. EUTROF1 calculates the cycling of nitrogen, phosphorus, and oxygen. The model is particularly suitable to study the short-term behavior of systems. In case the long term functioning of a system is of interest the other eutrophication model EUTROF2 is more appropriate. In EUTROF2, three algal species can be defined, and the model also describes the interaction between the sediment and the overlying water column. In addition to these two water-quality models there is an abundance of formulations proposed in the literature. DUFLOW gives great freedom to the user in formulating the production or destruction of biological or chemical matters because users may write their own water-quality simulation routines and easily incorporate them with DUFLOW.

An important topic in water-quality problems also is the interaction between the bottom layer and the water mass above. DUFLOW distinguishes among transported material that flows with the water, and bottom materials that are not transported materials that flow with the water and bottom materials that are not transported but that can be subject to similar interactions to those for the water column.

DUFLOW is efficient both in terms of computational time and required memory, thus allowing the processing of large models. Computational time is usually in the range of minutes up to one hour. In the case of the CWS with the use of a Pentium III computer with a speed of 1 GHZ, it took less than 3 minutes to simulate flows and stages for a 4 months period at a 15-min. time step. For immediate analysis, the results can be graphically displayed on the screen in time or space. Optionally output is given in the form of tables, while all output could be directed to a (graphical) printer.

2.3.3 Options and Elements

In DUFLOW a model, representing a specific application, can be put together from a range of elements. Types of elements, which are available, are open channel sections (both river and canal sections), and control sections or structures such as weirs, culverts, siphons, and pumps.

Boundary conditions can be specified as:

- Water levels or discharges, either constant or in the form of time series or Fourier series;
- Additional or external flow into the network can be specified as a time dependent discharge or can be computed for a given rainfall, using the simple precipitation runoff relation of DUFLOW or the extended precipitation-runoff module RAM;
- Discharge-level relations (rating curves) in tabular form;

2.3.4 Structure of the DUFLOW Modeling Studio

The Duflow Modeling Studio (DMS) is developed under the Windows 95/98/00/ windows NT operating system. The graphical user interface gives the user the possibility to manipulate and activate the objects of the model directly.

The program consists of a Scenario Manager with which the user can define several different scenarios. The network Editor enables the user to create a network of water courses by dragging and dropping the elements from the Network Palette. The network can be presented on a geographical background.

The DMS consists of the following parts:

1. DUFLOW water quantity

   With this program one can perform unsteady-flow computations in networks of open water courses. The calibration of this part for the CWS is the focus of this report.

2. DUFLOW water quality

   This program is useful in simulating the transport of substances in free surface flow and can simulate more complex water-quality processes.

3. RAM precipitation runoff module

   With RAM one can calculate the supply of rainfall to the surface flow. RAM calculates the losses and delays that occur before the precipitation has reached the surface flow.

4. MODUFLOW

This program simulates an integrated ground water and surface-water problem by combining the ground water model MODFLOW (Harbaugh and McDonald, 1996) and DUFLOW.

2.4 Physical and Mathematical Background of DUFLOW

The basic equations used in the water quantity part of DUFLOW and the numerical procedures used to discretize and solve these equations are described in this section. The numerical method is based on the use of both the mass conservation equation and the equation of motion in the section, and the use of the conservation equation (stating that the sum of the discharges is 0) at the nodes.

2.4.1 Flow

DUFLOW enables the water manager to calculate unsteady flows in networks made up of canals, rivers, and channels. Example applications include:

- Prediction of the behavior of flood waves;
- Judging the effect that infrastructural changes have on the water balance;
- Studying the effects of changes in management on the water level.

2.4.1.1 The Unsteady Flow Equations

Three conservation principles--conservation of water mass, conservation of the mechanical-energy content of the water, and conservation of the momentum content of the water--are available for analysis of 1-D unsteady flow. Conservation of thermal energy is not considered because temperature-change and heat-transfer effects do not affect flow depth and discharge. The first principle selected is the conservation of water mass, which becomes the conservation of water volume if the density is constant. Equations derived from application of the conservation of mass principle are often referred to as "continuity equations."

DUFLOW is based on the one-dimensional partial differential equations that describe non-stationary flow in open channels (Abbott, 1979; Dronkers, 1964). These equations, which are the mathematical translation of the laws of conservation of mass and of momentum read:

$$\frac{\partial B}{\partial t}+\frac{\partial Q}{\partial x}=0 \qquad (1)$$

and

$$\frac{\partial Q}{\partial t}+\frac{\partial(\beta QV)}{\partial x}+gA\frac{\partial H}{\partial x}+\frac{g|Q|Q}{C^2AR}=a\gamma w^2\cos(\Phi-\phi) \qquad (2)$$

While the relation:

$$Q = v\,A \qquad (3)$$

holds and where:

t	time
x	distance as measured along the channel axis [m]
H(x, t)	water level with respect to a reference level [m]
V(x, t)	mean velocity (averaged over the cross-sectional area) [m/s]
Q(x, t)	discharge at location x and at time t [m^3/s]
R(x, H)	hydraulic radius of cross-section [m]
a(x, H)	cross-sectional flow width [m]
A(x, H)	cross-sectional flow area [m^2]

B(x, H)	cross-sectional storage area [m^2]
g	acceleration due to gravity [m/s^2]
C(x, H)	coefficient of De Chezy [$m^{1/2}/s$]
w(t)	wind velocity [m/s]
$\Phi(t)$	wind direction in degrees [degrees]
$\varphi(x)$	direction of channel axis in degrees, measured clockwise from the north [degrees]
$\gamma(x)$	wind conversion coefficient [-]
β	correction factor for non-uniformity of the velocity distribution in the advection term, defined as:

$$\beta = \frac{A}{Q^2}\int v(y,z)^2 dydz$$

where the integral is taken over the cross-section A

The continuity equation (1) states that if the water level changes at some location this will be the net result of inflow minus outflow at this location. The momentum equation (2) expresses that the net change of momentum is the result of exterior forces like friction, wind, and gravity. For the derivation of these equations it has been assumed that the fluid is well-mixed, and, hence, the density may be considered to be constant. The combination of equations 1 and 2 is known as the de Saint Venant or dynamic-wave equations.

The advection term in the momentum equation:

$$\frac{\partial(\beta Q v)}{\partial x} \tag{4a}$$

can be broken into

$$\beta\left(2\frac{Q}{A}\frac{\partial Q}{\partial x} - \frac{Q^2}{A^2}\frac{\partial A}{\partial x}\right) \tag{4b}$$

The first term represents the impact of the change in discharge. The second term, which expresses the effect of change in cross-sectional flow area, is called the Froude term. In case of abrupt changes in cross section this Froude term may lead to computational instabilities.

2.4.1.2 Discretization of the Unsteady Flow Equations

Equations (1) and (2) are discretized in space and time using the four point implicit Preissmann scheme. Defining a section Δx_i from node x_i to x_{i+1} and a time interval Δt from time $t = t^n$ to time $t = t^{n+1}$, the discretization of the water level H can be expressed as:

$$H^n_{i+1/2} = (1-\theta)H^n_i + \theta H^{n+1}_i \tag{5}$$

at node x_i and time $t+\theta\Delta t$

and

$$H^n_{i+1/2} = \frac{H^n_{i+1} + H^n_i}{2} \tag{6}$$

in between nodes x_i and x_{i+1} at time t.

The transformed partial differential equations can be written as a system of algebric equations by replacing the derivatives by finite difference expressions. These expressions approximate the derivatives at the point of reference ($x_{i+1/2}$, $t^{n+\theta}$) as shown in Figure 2.1.

With Initially: $H^0_{i+1/2} = H^n_{i+1/2}$

$B^n_{i+1/2} = B_{i+1/2}(H^n_{i+1/2})$

$b^n_{i+1/2} = b_{i+1/2}(H^n_{i+1/2})$

$B^{n,*}_{i+1/2} = B^n_{i+1/2} - b^n_{i+1/2}H^n_{i+1/2}$

equation (1) is transformed into:

$$\frac{B^{*,n+1}_{i+1/2} + b^{n+1}_{i+1/2}H^{n+1}_{i+1/2} - B^n_{i+1/2}}{\Delta t} + \frac{Q^{n+\theta}_{i+1} - Q^{n+\theta}_i}{\Delta x_i} = 0 \tag{7}$$

and equation (2) into:

$$\frac{Q_{i+1/2}^{n+1} - Q_{i+1/2}^{n}}{\Delta t} + \frac{gA_{i+1/2}^{*}(H_{i+1}^{n+\theta} - H_{i}^{n+\theta})}{\Delta x_i} + \frac{\beta(\frac{Q_{i+1}^{n}}{A_{1+1}^{*}} Q_{i+1}^{n+1} - \frac{Q_{i}^{n}}{A_{i}^{*}} Q_{i}^{n+1})}{\Delta x_i} \tag{8}$$
$$+ \frac{g\left(Q_{1+1/2}^{n+1} / Q_{i+1/2}^{n}\right)}{(C^2 AR)_{i+1/2}^{*}} = a^n \gamma(w_{i+1/2}^{n+1}) \cos(\Phi^{n+1} - \phi)$$

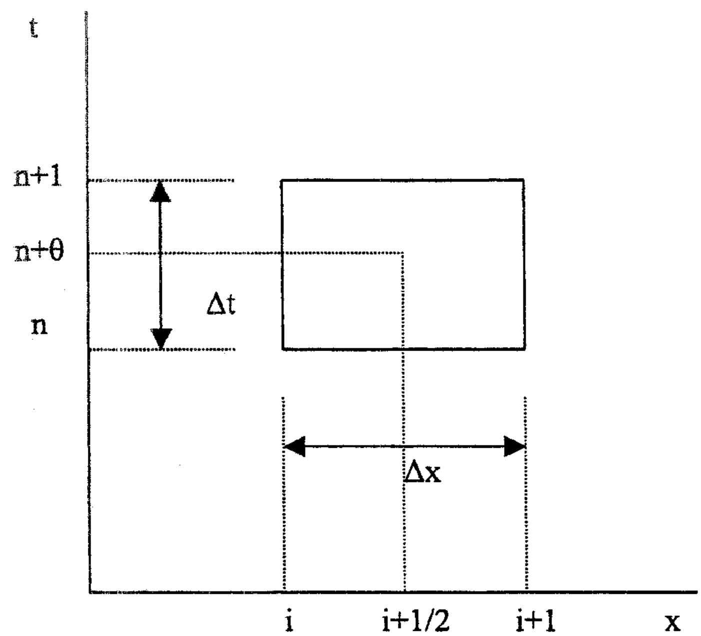


Figure 2.1. The Four-Point Preisman scheme

A mass conservation scheme for water movement is essential for proper water-quality simulation. If the continuity equation is not properly taken into account, the calculated concentrations will not match the actual concentration. The mass conservation scheme is based on the fact that the error made in the continuity equation will be corrected in the next time step.

The * (like in $A^{*}_{i+1/2}$) expresses that these values are approximately at time $t^{n+\theta}$. This descretization is of second order in time and space if the value $\theta = 0.5$ and it can be shown that in this case the descretized system is mass conservative. In most applications, a somewhat larger θvalue, such as 0.55 is used in order to obtain better stability (Franz and Melching, 1997, p. 63-64).

The values indicated with (*) are computed using an iterative process. For example, a first approximation of A is

$A^* = A^n$

Which is adjusted in subsequent iteration steps:

$$A^* = \frac{\left(A^n + A^{n+1,*}\right)}{2}$$

where $A^{n+1,*}$ is the new computed value of A^{n+1}.

So finally, for all channel sections in the network two equations are formed which have Q and H as unknowns on the new time level t^{n+1}:

$$Q_i^{n+1} = N_{11}H_i^{n+1} + N_{12}H_{i+1}^{n+1} + N_{13} \tag{9a}$$

$$Q_{i+1}^{n+1} = N_{21}H_i^{n+1} + N_{22}H_{i+1}^{n+1} + N_{23} \tag{9b}$$

2.4.2 Boundary and Initial Conditions

For a unique solution of the set of equations additional conditions have to be specified at the physical boundaries of the network and at the sections defined as hydraulic structures. The user-defined conditions at the physical boundaries may be specified as levels, discharge, or a relation between both.

At internal junctions the (implicit) condition states that the water level is continuous over such a junction node, and that the flows towards the junction are in balance since continuity requires:

$$\sum_{J=1}^{JJ} Q_{j,i} + q_i = 0 \tag{10}$$

where:

i indication for the junction node,

$Q_{j,i}$ discharge from node j to node i (note: this value is negative for flows moving downstream from node i),

q_i tributary or lateral flow entering the stream network at node i (e.g., tributary inflow from a water reclamation plant or tributary stream).

2.5 Assumptions in Unsteady Flow Analysis

Analysis of 1-D unsteady flow in open channels requires many assumptions. The major assumptions are the following (Franz and Melching, 1997, p. 4):

1. The wavelength of the disturbance of the flow is very long relative to the depth of the flow. This "shallow water wave assumption" implies that the flow is principally 1-D and basically parallel to the walls and bottom forming the channel. Thus, streamline curvature is small; lateral and vertical accelerations are negligible relative to the longitudinal accelerations; and, therefore, the pressure distribution is hydrostatic.

2. The channel geometry is fixed so that the effect of deposition or scour of sediment is small.

3. The effect of boundary friction force can be estimated with a relation derived from steady uniform flow. Nonuniformity and unsteadiness are assumed to have only a small effect on the frictional losses.

4. Channel alignment with respect to the effect of directional changes on the conservation of momentum principle may be treated as if it were rectilinear even though the channel is

curvilinear. Thus, the water surface in any cross section of the stream is assumed to be horizontal. Super-elevation effects on the water surface in channel bends are not considered in the analysis and are assumed to have a small effect on the results.

5. The fluxes of momentum and energy along the cross section resulting from nonuniform velocity distribution may be estimated by means of average velocities and flux-correction coefficients that are functions of location along the stream and water-surface elevation.

6. The flowing fluid is homogeneous (constant density).

2.6 Limitations of the Model

The equations are for one-dimensional flow. Therefore, DUFLOW is not suitable for performing calculations of flows in which an extra spatial dimension is of interest. Water bodies with significantly different velocities in the vertical can, therefore, not be modeled. Vertical density differences also are not taken into account; also horizontal density differences are not modeled because the density is assumed to be constant throughout.

Although the equations underlying the model are valid in case of supercritical flow, the numerical solution method does not support supercritical flow. Because subcritical flow is assumed there must be one boundary condition at each of the boundaries of the network. Supercritical flow is extremely unlikely in the CWS.

As discussed earlier, the dynamic wave equations are a combination of conservation of mass and momentum equations. The accuracy of the momentum equation is based on how well the various flow parameters and variables can be approximated and how well each particular principle works when only approximations to physical reality are possible. The precise knowledge of these is never possible.

CHAPTER THREE: DESCRIPTION OF STUDY AREA

3.1 Description of Chicago Waterway System

The CWS is one of the major water transportation systems in the world. The Chicago River's northernmost headwaters are in Lake County near Park City, Illinois. There are three northern tributaries named the West Fork, the Middle Fork, and the Skokie River (East Fork). These three flow south, basically parallel to each other, and meet to become the North Branch Chicago River. The North Branch continues to flow south and east through the northwest side of the City of Chicago where it joins the completely manmade North Shore Channel, which diverts water from Lake Michigan at Wilmette Harbor and conveys other point and nonpoint discharges as shown in Figure 1.1.

Downstream from the junction of the North Branch and North Shore Channel the larger, wider river, still called the North Branch, flows south through the city's north side to the west of Western Avenue. Near Belmont Avenue it continues southeast until it reaches Kinzie Street and there joins the Chicago River Main Stem, which originally flowed into Lake Michigan but now typically flows west through downtown Chicago. Together, these two rivers form the South Branch, which flows south to 18th Street, then southwest to the beginning of the Chicago Sanitary and Ship Canal (CSSC), which begins near South Damen Avenue. The CSSC flows southwest out of Chicago until it meets up with the Des Plaines River near Lockport. The widest point of the North Branch is the North Avenue turning basin, which is about 800 ft across. The deepest point is about 26 ft in the Chicago River Main Stem; this depth is man made and was required for the many ships that once made port in downtown Chicago.

The Calumet River System is another part of the CWS. The prominent branch of this system is the Little Calumet River, which is divided into two segments, North and South. The tributaries of the Little Calumet River South originate in the State of Indiana and Will County, Illinois, and generally flow north. The Little Calumet River South flows westerly to Blue Island, Illinois, where it abruptly turns east before joining the Cal-Sag Channel at Calumet Junction. The Little Calumet River South is more of a natural river channel and is upstream of the deepened and widened channel used for commercial navigation, referred to as the Little Calumet River North. Tributary to the Little Calumet River North is the Grand Calumet River, flowing west from the State of Indiana. Near this confluence is the O'Brien Lock and Dam, another upstream end and control of the CWS. Lake-ward of the O'Brien Lock and Dam is the Calumet River, connecting to Lake Michigan at Calumet Harbor. The Calumet River is not part of the controlled CWS and is not included in the model.

The Little Calumet River North flows west into the Cal-Sag Channel at Calumet Junction, where the flow of both the North and South parts of the Little Calumet River continue in a westerly direction through the Cal-Sag Channel to where it joins and the CSSC at Cal-Sag Junction. Several small streams tributary to the Cal-Sag Channel include Midlothian Creek, Tinley Creek, and Stony Creek.

Upper reaches of the CWS are generally much narrower and shallower than the lower reaches. Upper reaches also are above most point-source discharges, and the watershed is less intensively developed. The remainder of the CWS from the North Shore Channel in Wilmette to Lockport and eastward through the Cal-Sag Channel to the Calumet Harbor has been repeatedly dredged and deepened for commercial navigation. Most of the treated municipal and industrial wastewater from the Metropolitan Chicago area are conveyed by the CWS. The different reaches of the CWS are listed in Table 3.1.

The study area, inflow locations, boundary conditions, and measured stage locations available for CWS are shown in Figure 3.1. A detail drawing consisting of the DO measurement locations, inflow locations, stage gages, water-quality stations, and instream aeration stations is shown in Appendix I. The river channel cross sectional data obtained from the U.S. Army Corps of Engineers, Chicago Districts, (Corps) and used in the UNET model (Barkau, 1991) were used in this study. The longitudinal bottom profiles of the CWS from Wilmette to Romeoville, Little Calumet River North, and Little Calumet River South are shown in Figures 3.2 - 3.4, respectively. Since the bottom elevation of Cal-Sag Channel is nearly constant from the Cal-Sag Junction to the Calumet Junction 16 miles upstream, this profile is not shown. The longitudinal profile shown in Figure 3.3 is from the Calumet Junction to the O'Brien Lock and Dam.

Table 3.1. Description of Reaches in the Chicago Waterway System simulated in this study. (Note: river miles are measured from the Lockport Lock and Dam at Lockport, IL)

Reach	Description	Mileage		Length	Width	Depth
		Start	End	miles	ft	ft
1	North Shore Channel	49.6	42.5	7.1	100	8
2	North Branch Chicago River	42.5	34.6	7.9	150-200	9-21
3	Chicago River	36.1	34.6	1.5	180-400	21
4	South Branch Chicago River	34.6	30.7	3.9	150	17
5	Chicago Sanitary and Ship Channel (CSSC) upstream of Cal-Sag Junction	30.7	12.6	18.1	150-300	17-23
6	CSSC from Cal-Sag Junction to Romeoville	12.6	5.2	7.4	160-200	23
7	Cal-Sag Channel	28.6	12.6	16	300-450	9-27
8	Little Calumet River North	35.5	28.6	6.9	300-450	9-27
9	Little Calumet River South	35.4	28.6	6.8	*	*

*Reaches 1-8 essentially are constructed channels with relatively fixed geometry whereas reach 9 is more of a natural stream, and, thus, the width and depth are highly variable.

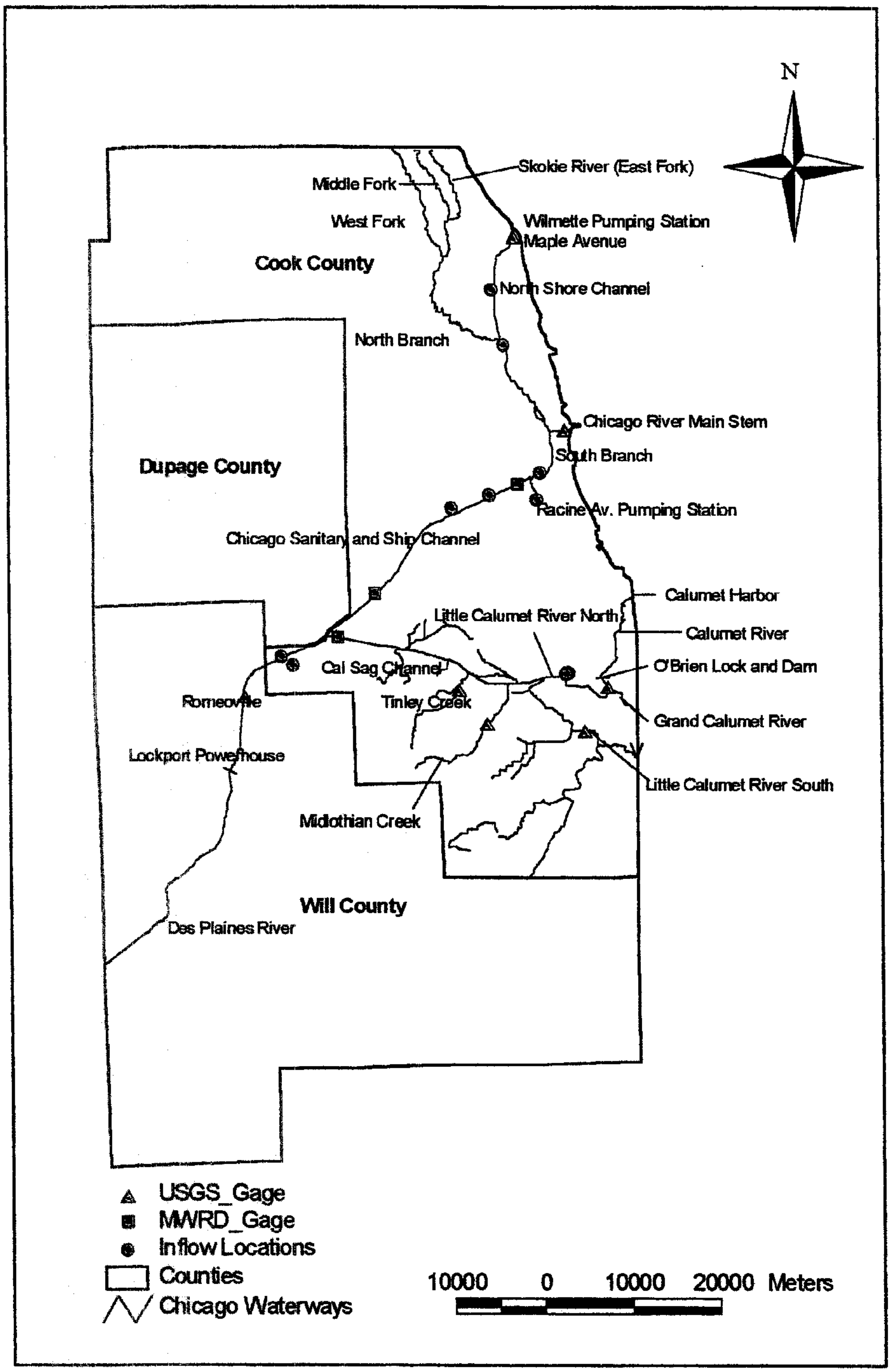


Figure 3.1. The study area, inflow locations, and stages of Chicago Waterway System (See map in Appendix I for detailed description of data locations and note that the U.S. Geological Survey Gage Grand Calumet River at Hohman Avenue at Hammond, Ind., is just cast of the Illinois-Indiana border and just outside the extent of this map)

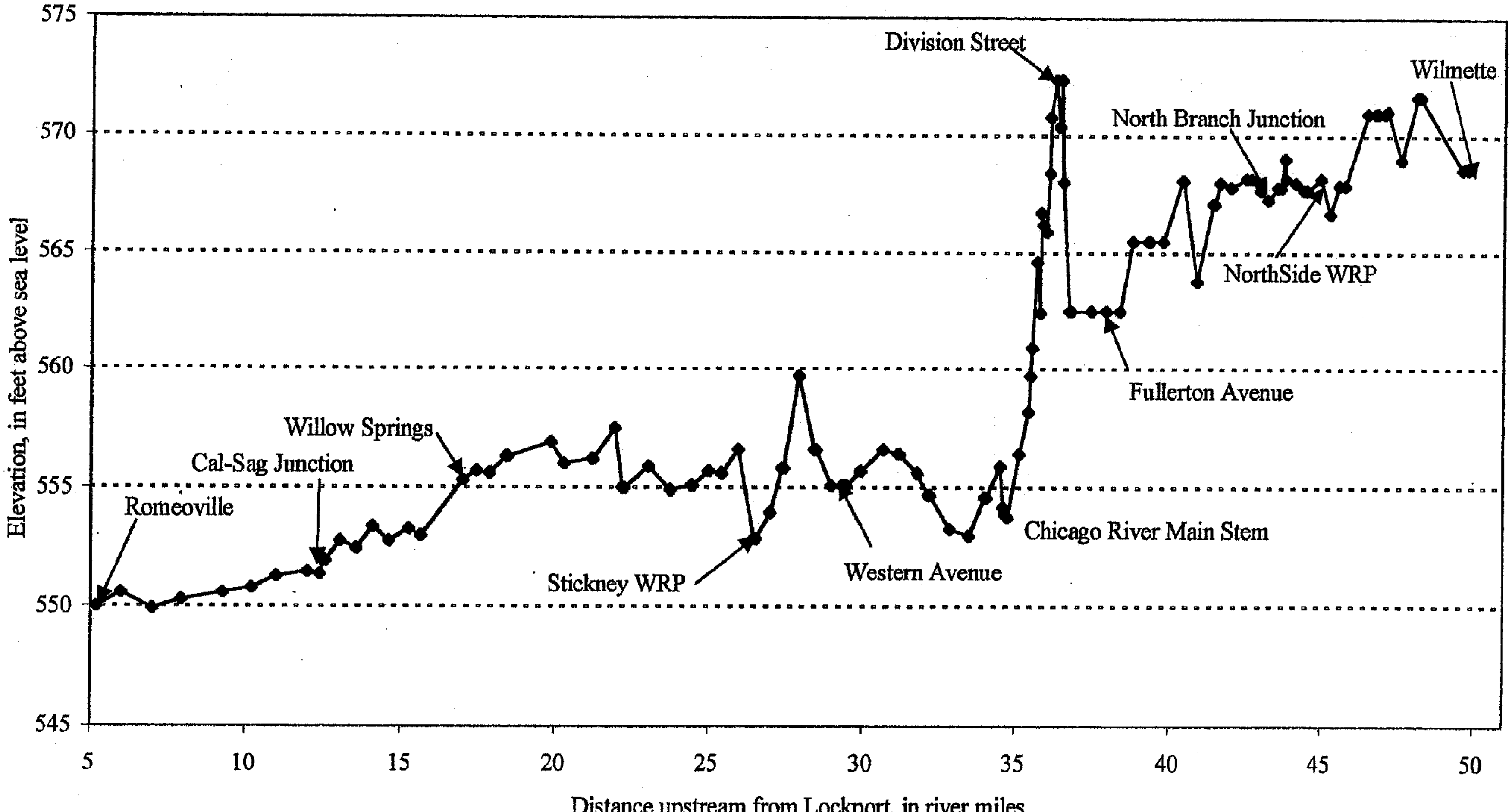


Figure 3.2. The longitudinal bottom profile of the Chicago Waterway System from Wilmette to Romeoville

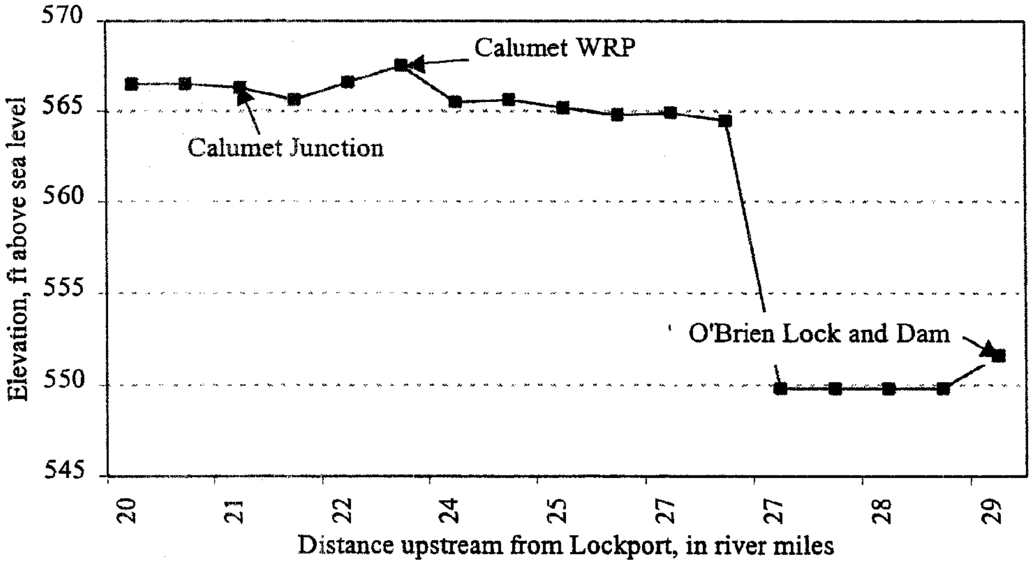


Figure 3.3. Longitudinal bottom profile of the Little Calumet River Nort (Note: the Cal-Sag Channel maintains a nearly constant elevation from the Calumet Junction to the Cal-Sag Junction with the CSSC)

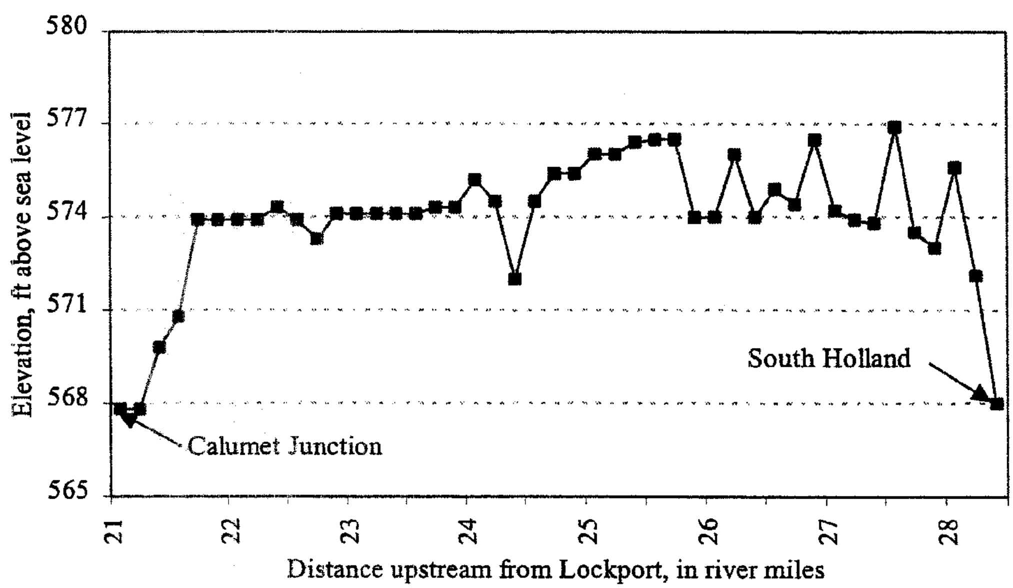


Figure 3.4. Longitudinal bottom profile of the Little Calumet River Sout

3.2 Hydraulic Data Used for the Model Input

All the hydraulic data needed for model input are not measured. Some information is missing, such as discontinuous data, ungaged tributaries, and ungaged watersheds. It is difficult to estimate such data. The following subsections describe the available flow and stage (water-surface elevation) data and methods used to estimate some of the missing data/information.

3.2.1 Measured Inflows, Outflows, and Water Surface Elevations

The hydraulic data available for the CWS have been compiled from different agencies and used in this study. The USGS has established discharge and stage gages at the three primary points where water is diverted from Lake Michigan into the CWS. These locations are:

1) the Chicago River Main Stem at Columbus Drive, USGS gage # 05536123
2) the Calumet River at the O'Brien Lock and Dam, USGS, gage # 05536357
3) the North Shore Channel at Maple Avenue, USGS gage # 05536101

The data from these gages are used as the primary upstream elevation versus time boundary conditions for the unsteady-flow model, and their time step is listed in Table 3.2. Further, flow versus time data (on a 15-minute basis) from the USGS gage on the CSSC at Romeoville (USGS gage # 05536995) are used as the downstream boundary condition for the model. The data from the USGS gage on the Little Calumet River South at South Holland (USGS gage # 05536290) provide a flow versus time upstream boundary condition for the model. Two tributaries to the Cal-Sag Channel are gaged by the USGS, Tinley Creek near Palos Park (USGS gage # 05536500) and Midlothian Creek at Oak Forest (USGS gage # 05536340), and are considered as tributary flows in the modeling of the CWS. The USGS gage on the Grand

Calumet River at Hohman Avenue at Hammond, Ind. (USGS gage # 05536357) is considered as the tributary flow for the Little Calumet River North.

Table 3.2. Water-Surface Elevation boundary conditions used in the model

S. No.	Locations	Series
1.	Wilmette Pumping Station*	Hourly
2.	Chicago River Controlling Works	Hourly**
3.	O'Brien Lock and Dam	Hourly**

* The USGS elevation gage at this location was not established until September 1999, which is outside the simulation period for this study. Thus, hourly water-surface elevation data from the MWRDGC were used at this site.

** 5-minute data are available from the USGS but sample computations found that the results changed little when 5-minute values were used. Thus, hourly values were used to facilitate filling in missing records with hourly data available from the MWRDGC.

Inflows to the CWS also come from the facilities of the MWRDGC. Flow data are available from the MWRDGC for the treated effluent discharged to the CWS by each of four Water Reclamation Plants (WRPs). In addition, flows discharged to the CWS at two Combined Sewer Overflow (CSO) pumping stations were estimated from the operating logs of these stations. The points where measured inflow to the CWS is available and the time step at which these data are input to the model are listed in Table 3.3.

There are numerous points at which CSOs occur and numerous ungaged tributary streams, but no flow data are available. The methods used to estimate flows for the ungaged tributaries and CSOs are described in section 3.2.3

Table 3.3. Major input flow locations to Chicago Waterway System

S.No.	Locations	Series
1.	North Side Water Reclamation Plant	Hourly
2.	North Branch + Pump*	Hourly
3.	Racine Pump Station	Hourly
4.	Stickney Water Reclamation Plant	Hourly
5.	Lemont Water Reclamation Plant	Daily
6.	Citgo Petroleum	Daily
7.	Tinley Creek	15 minutes
8.	Midlothian Creek	15 minutes
9.	Calumet Water Reclamation Plant	Hourly**
10.	Little Calumet River at South Holland	15 minutes
11.	Grand Calumet River	Hourly

* This is a combination of streamflow from the North Branch Chicago River at Albany Avenue at Chicago (USGS gage # 05536105) and flows from the North Branch Pumping Station.

** Hourly flows for the Calumet Water Reclamation Plant were determined from values recorded every 8 hours as described in Section 3.2.2(F).

3.2.2 Data Estimation for Missing Data

As discussed earlier, the modeling required data collection at many locations on a continuous basis at an often short time step. It was found that much data at gaged sites was missing, discontinued, or at an unsuitable time step. The following approaches were used to fill in the missing, incomplete, and discontinued data.

A) Estimation of discontinued data for the North Branch at Albany Avenue gage:

The USGS gage North Branch Chicago River at Albany Avenue at Chicago (USGS gage # 05536105) was discontinued during the following period.

Discontinued period: 1/22/99:300 min – 6/23/00:675 min

The hourly flow of the North Branch Chicago River at Albany Avenue was estimated using the flow recorded at the USGS gage North Branch Chicago River at Niles (USGS gage # 05536000). This gauge is located 8 miles upstream from Albany Avenue and drainage area upstream from this gage is equal to 100 mi^2, whereas the gage located at Albany Avenue has a drainage area of 113 mi^2.

The 15-minute data available from October 1993 to December 1999 for the Albany Avenue and Niles gages was compiled and compared. There also are some missing data at these locations. Only periods with the data available at both locations are considered for comparison to establish a relation between them. During comparison it was found that there is time lag in flows between the two sites during storms. The stage-discharge relation for the Albany Avenue and Niles gages are not the same at all flows. A typical comparison of flow between Albany Avenue and Niles is shown in Figure 3.5. For the estimation of the flow during the discontinued period, the measured data at Albany Avenue and Niles were divided into three different flow regimes and an estimation equation was determined for each regime on the basis of data at each gage for the period October 1993 to December 1999. Flows for the period 1/22/1999 to 7/31/1999 then were estimated using the relations described in the following.

1. Low flow: The low flow is considered as flows less than 100 cfs. In the case of storm flow, the low flow is assumed after the time where decreasing flow showed the normal pattern that resulted before the beginning of storm flow, i.e. just before the start of the hydrograph rising limb leading to a peak flow. For the estimation of discontinued data for this type of flow, all the available data of Albany Avenue and Niles in this category were summed and a ratio was determined. The relation shows that the flow at Albany Avenue is comparatively greater than Niles. This result is obvious because Albany Avenue is downstream of Niles and has a larger drainage area. The relation is as follows:

$$Q_{Albany,t} = 1.1214 * Q_{Niles,t}$$

Note: The flow ratio between these two gages for low flow nearly equals the area ratio (1.13) between these two gages.

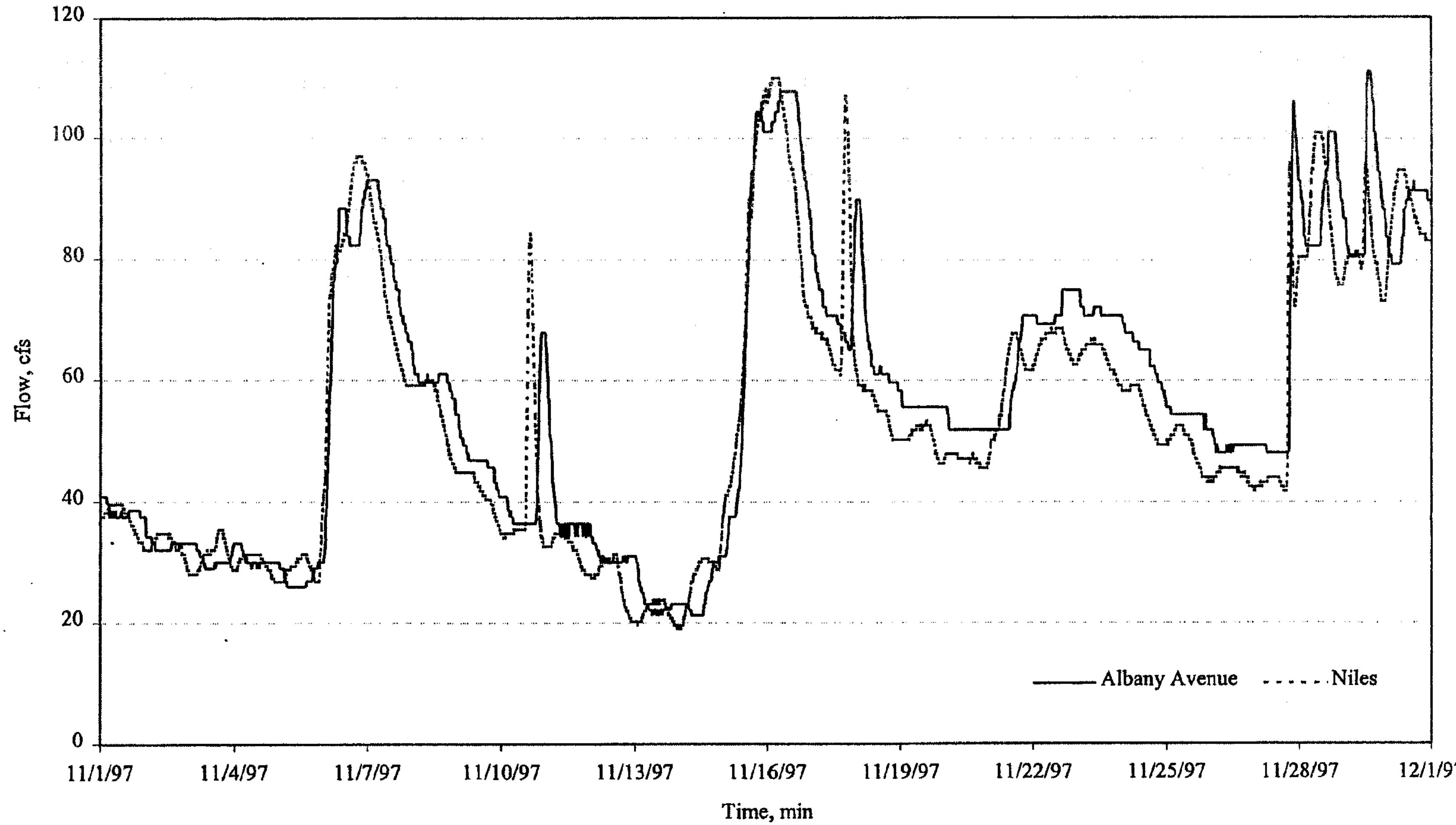


Figure 3.5. Comparison of flow on the North Branch Chicago River at Albany Avenue and Niles for November 1997

2. Minor storm flow: When the pattern of flow shows a storm peak flow in the range of 100 cfs to 200 cfs, it was considered a minor storm flow. The time lag of flow and the regression equation were determined from the measured flow at both gages. The best time lag of flow between the two locations on minor storm flow was determined by regression for different time lags. Statistical measures of fit quality of regression equations for minor storm flow on different time lags are listed in Table 3.4. The best relation between the two gages was found as:

$$Q_{Albany,t} = 0.8507 * Q_{Niles,t-240} + 23.516$$

The t-240 indicates that there is a 4 hour time lag between Niles and Albany Avenue for minor storms. The statistical analysis has been done on intervals of 30 minutes and the results listed in Table 3.4 show the best fit results for a time lag of 4 hours for minor storm flow.

Table 3.4. Statistical comparison for different time lag for minor storm flow

Function	t-180	t-210	t-240	t-270
Multiple R	0.83129	0.83918	0.84584	0.83873
R Square	0.69104	0.70421	0.71544	0.70347
Adjusted R Square	0.69102	0.70419	0.71542	0.70345
Standard Error	21.92078	21.43624	21.01169	21.50535
Observations	14248	14144	14040	13936
Intercept	25.047	24.232	23.516	24.262

3. Major storm flow: When the storm peak flow is greater than 200 cfs, the flow is considered a major storm flow. The same procedure was applied for the major storm flow to determine the time lag and relation to determine discontinued data at Albany Avenue in relation to Niles. Statistical measures of fit quality of regression equations for major storm flow on different time lags are listed in Table 3.5. The relation between the two gages for major storms was found as:

$$Q_{Albany,t} = 0.9987 * Q_{Niles,t-60} + 12.178$$

The t-60 indicates that there is a 1-hour time lag between Niles and Albany Avenue for major storms.

The comparison of estimated and observed flows for April 1998 is shown in Figure 3.6. The estimated flow is sometimes lower or higher than the observed flow.

Table 3.5. Statistical comparison for different time lag for major storm flow

Function	t-45	t-60	t-75	t-90
Multiple R	0.96744	0.96825	0.97746	0.98882
R Square	0.93594	0.93750	0.93692	0.93562
Adjusted R Square	0.93594	0.93750	0.93692	0.93562
Standard Error	62.52854	61.76930	61.87302	61.98982
Observations	52945	52873	52809	50367
Intercept	12.345	12.178	12.352	12.686

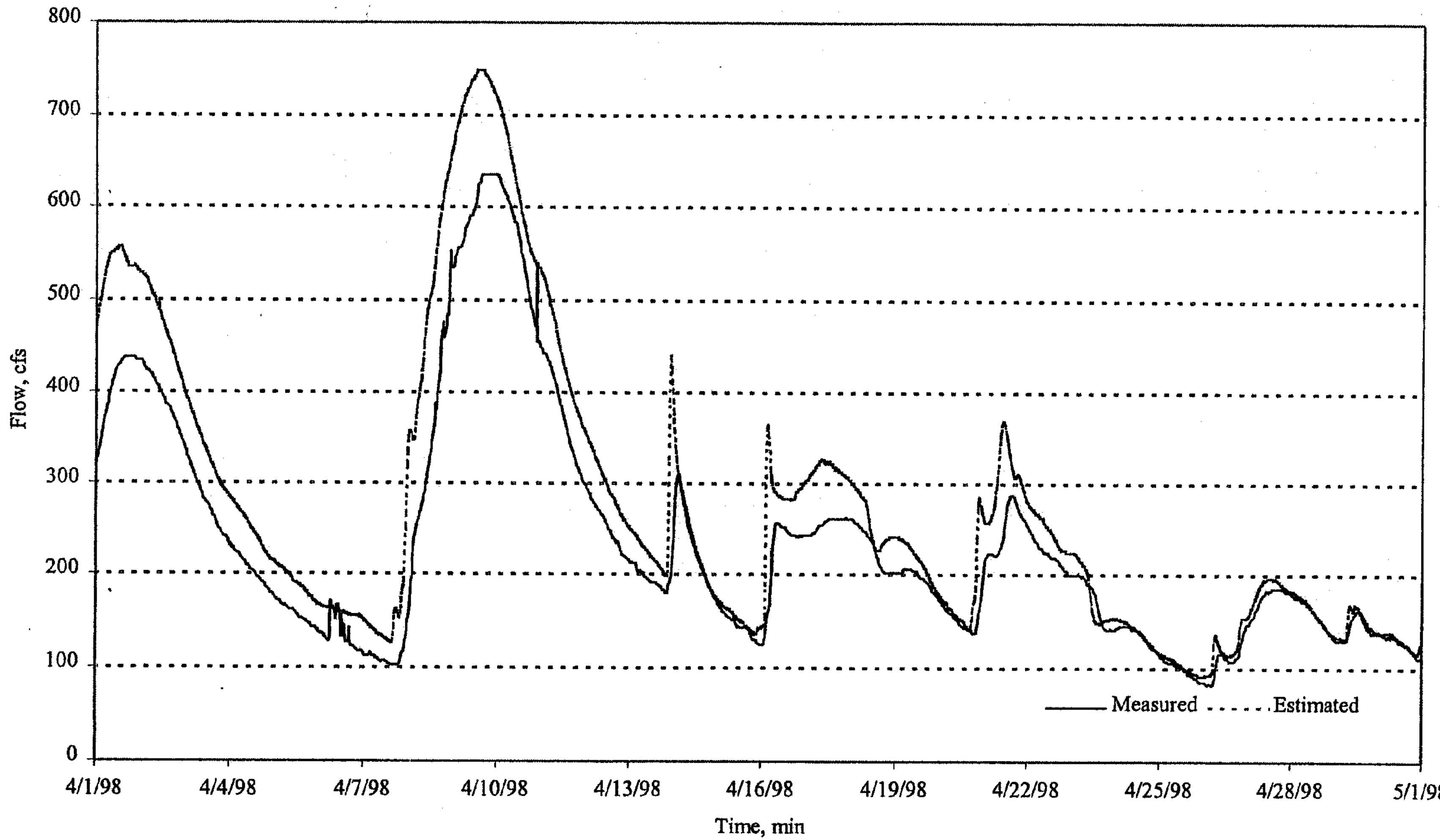


Figure 3.6. Comparison of measured and estimated flows for the North Branch Chicago River at Albany Avenue for April 1998

B) Columbus Drive – Chicago River Controlling Works

Flow and water-surface elevation data are available from the USGS at a 5-minute interval. For modeling purposes a 1-hour interval was used to facilitate use of water-surface elevation data from the MWRDGC to fill in missing data. Missing data were estimated from hourly Chicago River water-surface elevation data collected by the MWRDGC applying a correction of +0.033 ft. The period January – February 1999 was simulated using the 5-minute stage data at this location and it was found that using hourly values of water-surface elevation (which are linearly interpolated to a 15-minute time step during computation) did not adversely affect simulation results.

C) Little Calumet River

Flow data are available from the USGS at a 15-minute interval. Most of the missing data were estimated from Midlothian Creek flow data using a ratio in terms of the watershed drainage area. However, for the period 1/1/99 – 1/16/99, the flow data also are missing for Midlothian Creek and Tinley Creek because of ice conditions on the south side of Chicago. Only the USGS gage Thorn Creek at Glenwood (USGS gage # 05536215) was operational during this period. All flows during this period were estimated for the Little Calumet River, Midlothian, and Tinley Creek from data at Thorn Creek at Glenwood. Because this was a low flow period the USGS estimated daily mean discharge was applied to each 15-minute value at each location.

D) O'Brien Lock and Dam

Flow and water-surface elevation data are available from the USGS at a 5-minute interval. However, for modeling purposes it was changed into a 1-hour interval for the same reason as for the CRCW. Missing data were estimated from hourly water-surface elevation data at this location collected by the MWRDGC applying a correction of +0.118 ft.

Note: The water-surface elevation value at O'Brien Lock and Dam obtained from the MWRDGC is at times confusing. For most of the time the difference between the two measurements ranges between 0.1 and 0.15 ft, but then on occasion the MWRDGC value will become positive for a few hours while the USGS value remains around –1.8 ft City of Chicago Datum (CCD) (579.48 ft National Geodetic Vertical Datum of 1929).

E) Wilmette Pump Station

Flow and water-surface elevation data are available from the USGS at a 5-minute interval. However, a 1-hour interval was used for modeling purposes. The USGS gage was established in September 1999 and the comparison between MWRDGC and USGS data is good (MWRDGC values on average 0.035 ft lower than USGS values). The MWRDGC hourly elevation data are used throughout the study reported here.

F) Calumet Water Reclamation Plant (CWRP)

The discharge from the CWRP is measured daily at three different times: 6:30 a.m., 2:30 p.m., and 10:30 p.m. (essentially times of a shift change in operating personnel). Some hourly data were available for December 2000 and January 2001. With the available hourly data, an average weighting of hourly flow in each day was established. Most of the time this weighting was applied to calculate the hourly flow for the required time period but in some cases linear interpolation between the measured values was done for times when weighting does not look appropriate (i.e. large differences between measured values).

G) Others

For other missing periods for Midlothian Creek, Tinley Creek, and the North Side Water Reclamation Plant, linear interpolation was done for short periods to complete the data. For short periods of missing data at Romeoville, linear interpolation also was applied. However, for longer periods of missing record at Romeoville the missing values could not be reliably estimated, and since the downstream boundary flow is the primary driving force for simulated flow conditions, the simulations were limited to those days where the Romeoville gage was operated for the entire day. Thus, the period of August 1, 1998 to July 31, 1999 was divided into 8 simulation periods as listed in Table 3.6.

Table 3.6. Simulation periods for the August 1, 1998 to July 31, 1999 study period.

Periods	Time
1	08/01/1998 – 08/14/1998
2	08/18/1998 – 09/05/1998
3	09/11/1998 – 12/30/1998
4	01/07/1999 – 02/03/1999
5	02/05/1999 – 05/24/1999
6	05/27/1999 – 06/12/1999
7	06/15/1999 – 07/18/1999
8	07/22/1999 – 07/28/1999

3.2.3 Estimation of Flow for Ungaged Tributaries and Combined Sewer Overflows

It is necessary to estimate the inflows from ungaged-tributary watersheds. The drainage areas of ungaged tributary watershed have been overlaid with CSO drainage areas in a GIS system developed for this study. This overlaying of drainage areas allowed ungaged, separately sewered areas to be identified (referred to as ungaged tributaries). The continuous time series of flows for these ungaged tributaries has been estimated by considering Tinley Creek and Midlothian Creek as possibly hydrologically similar to each ungaged tributary, i.e. assuming that the topography of the drainage area, rainfall and runoff patterns, and other physical and hydrologic characteristic of the ungaged watershed area are same as that for the gaged watershed. The flow calculated for each ungaged tributary using an area ratio with Midlothian Creek and Tinley Creek was summed with all other inflows and compared with the total outflows at Romeoville. During this flow balance, the flow calculated using Midlothian Creek as representative of ungaged tributaries was found to be more reliable. The calculation of drainage area ratios compared to the Midlothian Creek drainage area is shown in Table 3.7.

During storm periods, the Racine Avenue and North Branch Pumping Stations may be operating. Flow from these pumping stations can be estimated from pump operation records and also is taken in consideration for flow balance calculation (Section 3.6). The flow from other CSO drainage areas during storms has a substantial effect on the CWS. It was realized during the initial calibration of the model that when these flows were not considered, simulated stages at Romeoville would be far less than observed stages as DUFLOW artificially lowered the slope in the CSSC to increase the flow to match observed values. The CSO volume then was approximated as the amount of water needed to get simulated and observed stage to agree during storms. The results of this CSO estimation are discussed in Chapter 4. In order to properly distribute the CSO volume, the CSO drainage area was calculated approximately for each reach and the ratio of the CSO area of each reach to the total CSO area (less the pumping stations drainage area) listed in Table 3.8 was used to distribute the CSO volume during the operation periods of the North Branch Pumping Station.

Table 3.7. Calculation of ungaged tributaries and watersheds

S. No.	Stream Ungaged	Area Sq. Miles	Area considered in CSO	Ratio with Midlothian*
1	Mill Creek	11.10		0.555
2	Stony Creek West	24.90	3.18	1.086
3	Cal-Sag Watershed East	4.93		0.246
4	Navajo Creek	2.74		0.137
5	Stony Creek East	17.72	8.00	0.486
6	Des Plaines Watershed	14.05		0.703
7	Calumet Union Ditch	23.37		1.168
8	Cal-Sag Watershed West	19.82		0.991

*The gaged Midlothian Creek drainage area is 12.6 mi^2, but these ratios are computed relative to the total Midlothian Creek drainage area of 20 mi^2. The total flow for both Midlothian and Tinley Creeks was determined by area ratio of the total drainage area to the gaged drainage area, 12.6 mi^2 and 11.2 mi^2 for Midlothian and Tinley Creeks, respectively.

Table 3.8. Combined Sewer Overflow (CSO) drainage area for different reaches in the DUFLOW model

S. No.	Name of Reaches	CSO Area (mi^2)	Area considered in CSO (mi^2)	% of CSO Area
1	North Shore Channel	44.57	44.57	0.197
2	North Branch	37.8	21.98*	0.097
3	Chicago R+South Branch	61.631	29.24*	0.129
4	CSSC	66.56	66.56	0.294
5	Cal-Sag Channel	55.4	55.4	0.245
6	Little Calumet River North	8.73	8.73	0.039
	Total	274.691	226.48	1.000

*Areas for the Racine Avenue and North Branch Pumping Stations areas are excluded.

3.2.4 Summary of Boundary Conditions and Tributary Inflows

Boundary and initial conditions for the calibration periods were set by the data collected by the USGS and the MWRDGC at the three lake front control structures and USGS data at Romeoville and for the tributary flows. The data collected by the MWRDGC for the discharges from different water reclamation plants also were used. The major flows into the CWS have been identified as follows:

a. North Side Water Reclamation Plant

b. Stickney Water Reclamation Plant

c. Calumet Water Reclamation plant

And the minor flows in the CWS are from:

a. North Branch Chicago River + North Branch Pumping Station

b. Racine Avenue Pumping Station

c. Lemont Water Reclamation Plant

d. Citgo Petroleum

e. Tinley Creek + Navajo Creek (i.e. Navajo Creek estimated based on area ratio with Midlothian Creek and added with nearby Tinley Creek)

f. Midlothian Creek

g. Little Calumet River South

h. Grand Calumet River

i. Mill + Stony Creek (West)*

j. Stony Creek (East)*

k. Des Plaines Watershed*

l. Calumet Union Ditch*

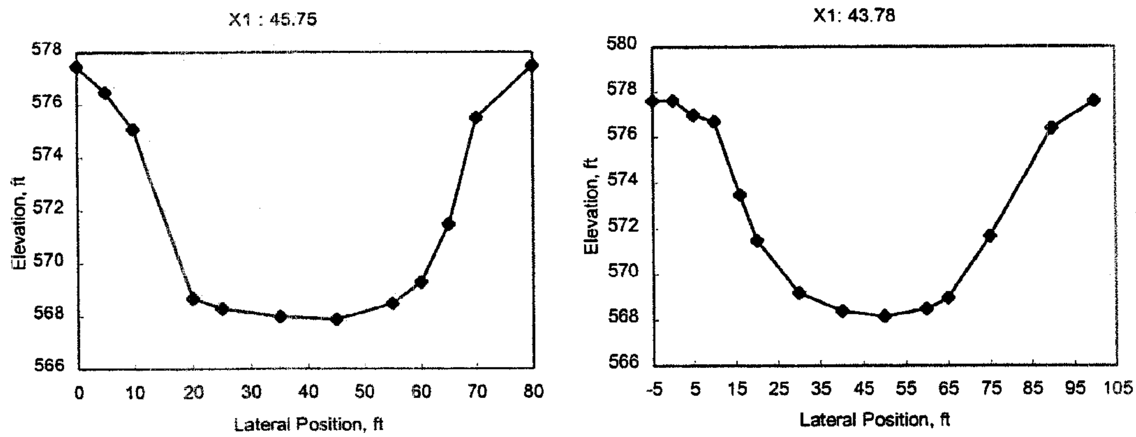
m. Cal-Sag Watershed West*

The * indicates these flows were estimated on the basis of an area ratio with Midlothian Creek. Measured inflows at the three-lakefront control structures have also been considered in the water balances, but flows are computed at these stage boundaries during simulation. Most of the time flows from the previously listed sources of minor flow are significantly low. But in case of storms, the Little Calumet River South and Racine Avenue Pumping Station have a significant effect on the system.

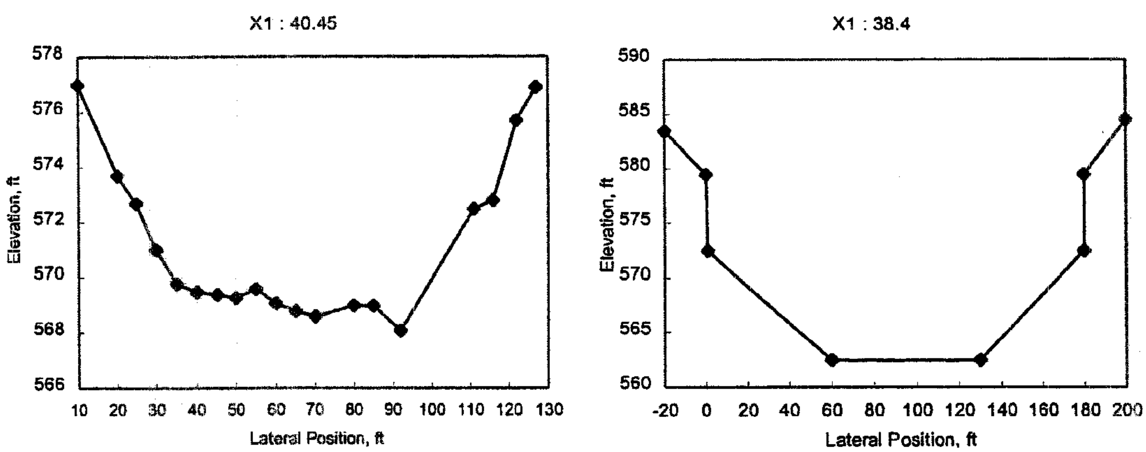
In this model, the upstream boundary conditions are water-surface elevation at three locations and flow at a fourth location and the downstream boundary condition is discharge. Using these types of boundary conditions the discharge computed at the upstream boundaries can be used for comparison with the measured flows, and the stage computed at the downstream boundary can be compared with the measured stage.

3.3 Channel Geometry

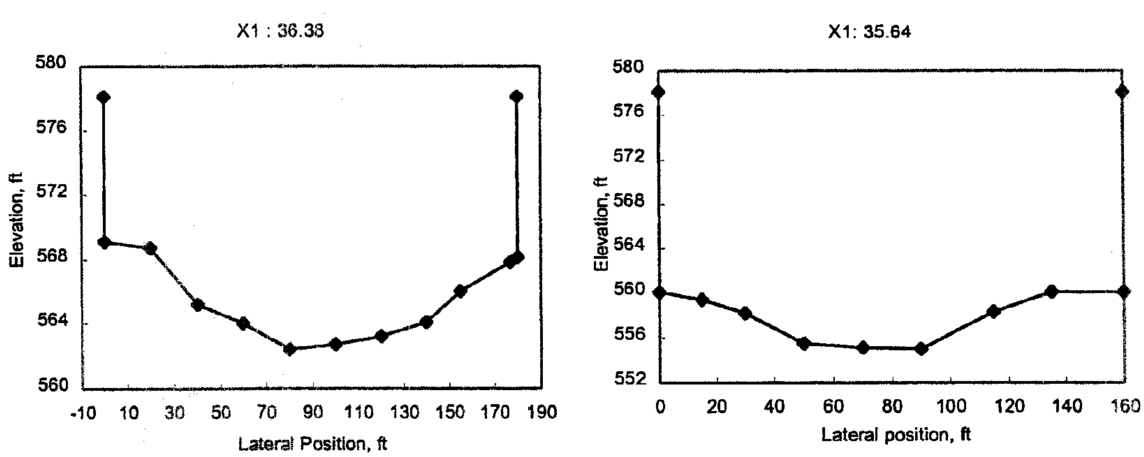
The description of the size and shape of the channels in which water flows often is given cursory treatment in modeling documentation although it forms the foundation of any analysis of open-channel hydraulics. The channel geometry is represented as a series of 193 measured cross sections. For the input of cross sectional data, each measured datum was plotted and width and height of the channel at each 2 ft increment of height were interpolated and the corresponding values were input in the model. The cross sectional data were obtained from surveys carried out by the Corps in the late 1980s and early 1990s. The water surface elevation is not shown because it varied during the study. Since the channel is constructed and used for navigational purposes, the majority of the CWS did not flow over bank during the study period. Little Calumet River South reach is more of a natural river and involved floodplain flows as well as main channel flows. In DUFLOW the cross section of the Little Calumet River South was subdivided into the flow width and maximum channel width. Some typical measured channel cross sections for the CWS are shown in Figure 3.7.



North Shore Channel

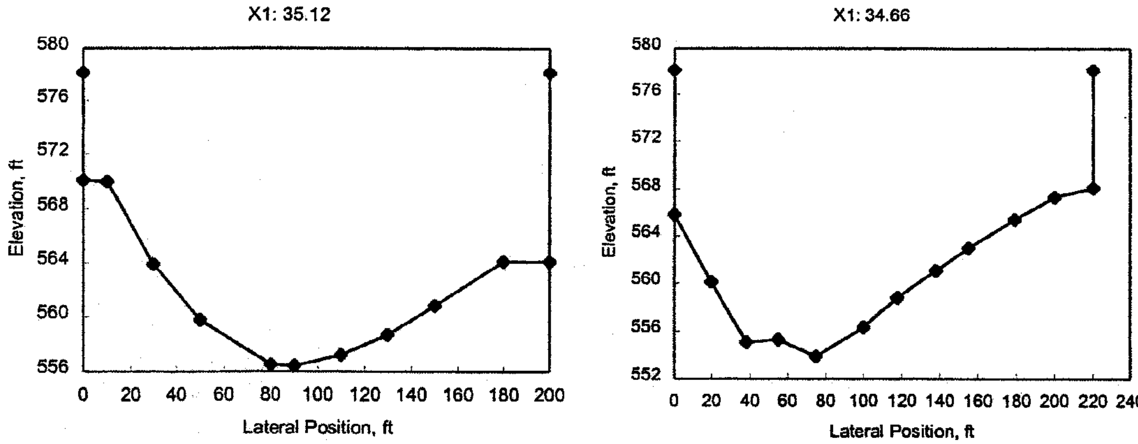


North Branch

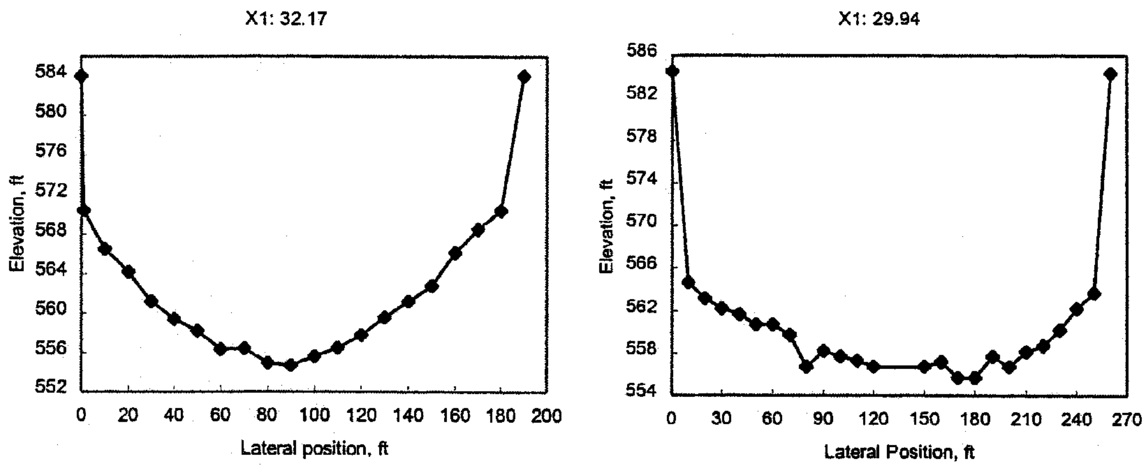


North Branch

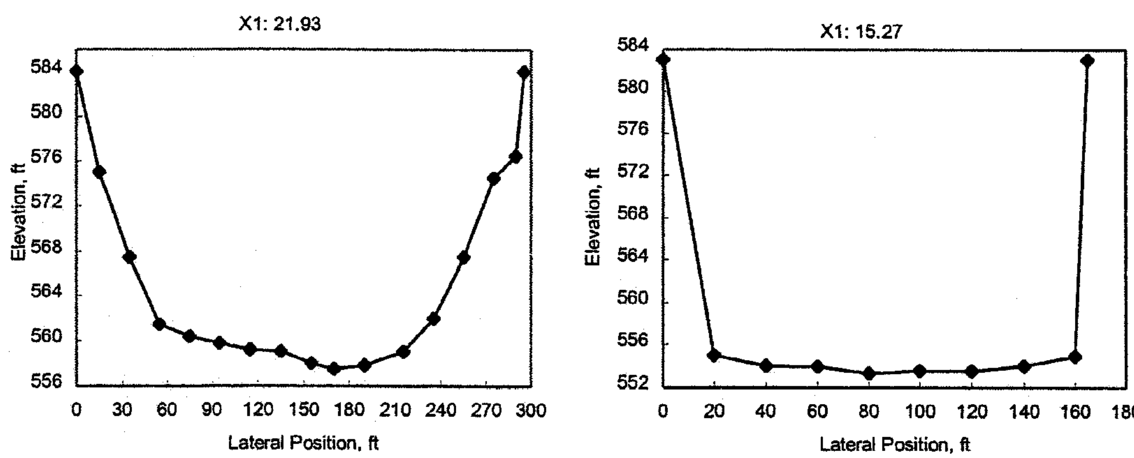
Figure 3.7. Typical cross sections in various reaches of the Chicago Waterway System (Note: X1 is the position in river miles from the Lockport Lock and Dam)



South Branch

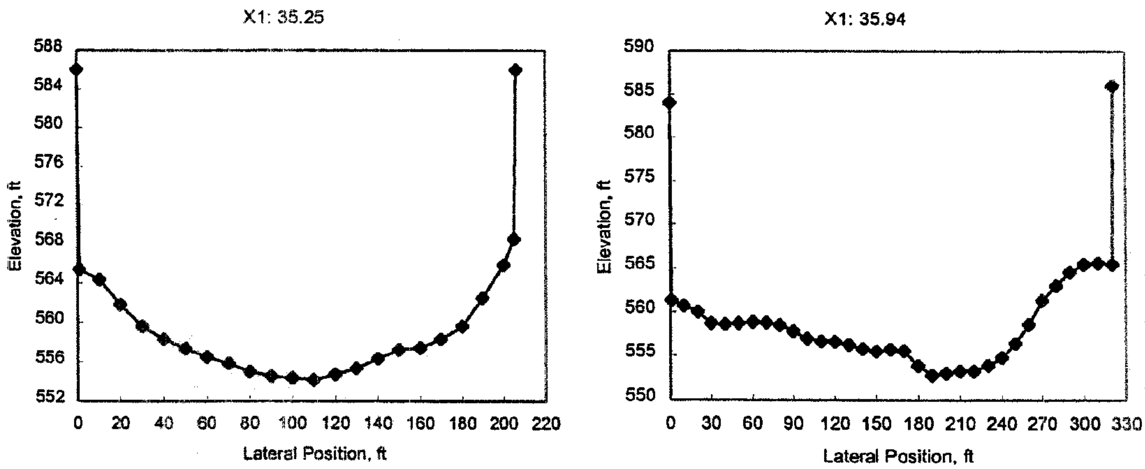


Chicago Sanitary and Ship Channel

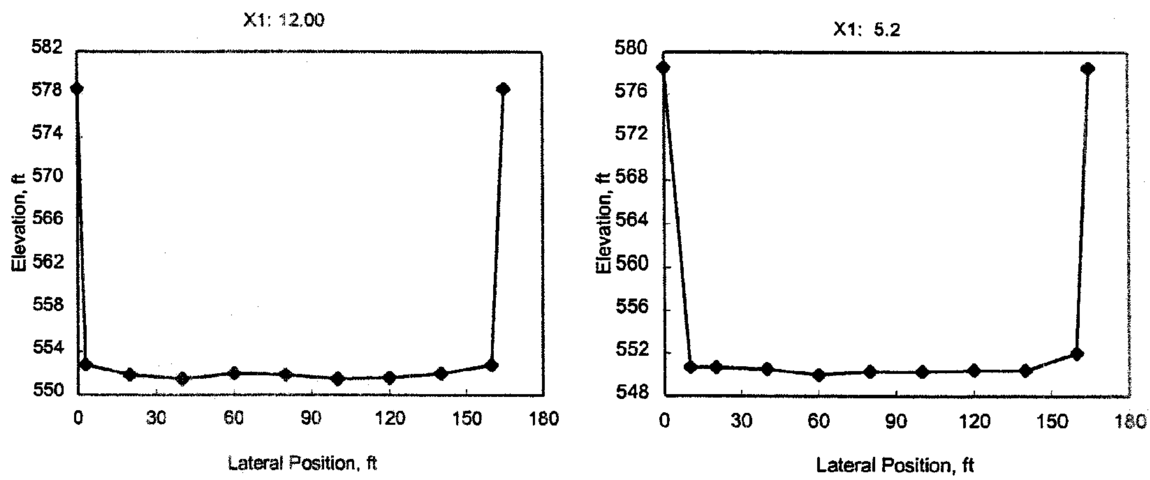


Chicago Sanitary and Ship Channel

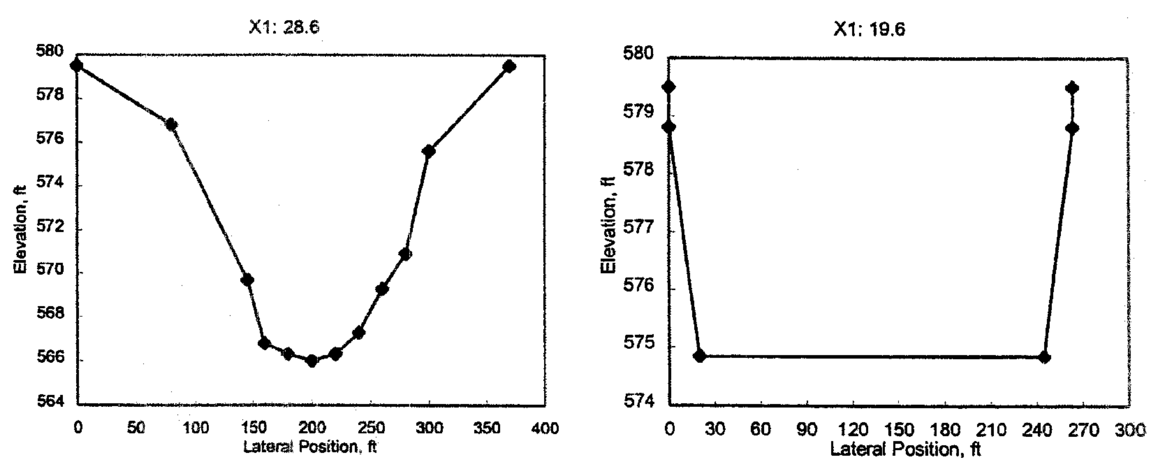
Figure 3.7. Typical cross sections in various reaches of the Chicago Waterway System (Note: X1 is the position in river miles from Lockport Lock and Dam) – cont



Chicago River (Main Stem)

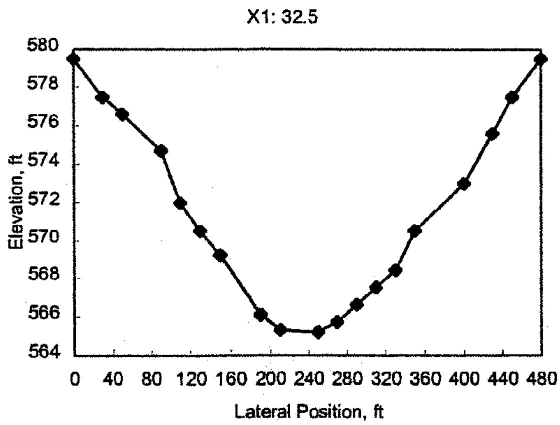


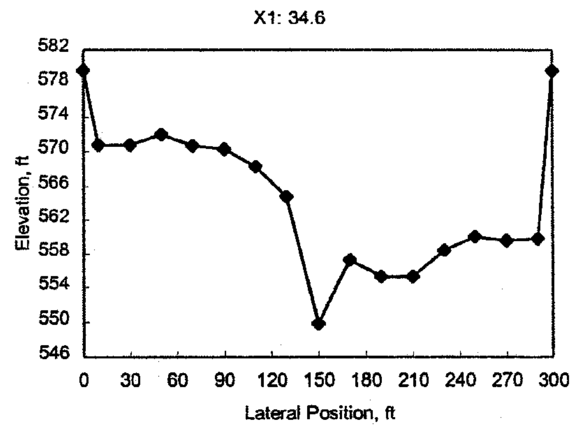
CSSC from Cal-Sag Junction to Romeoville



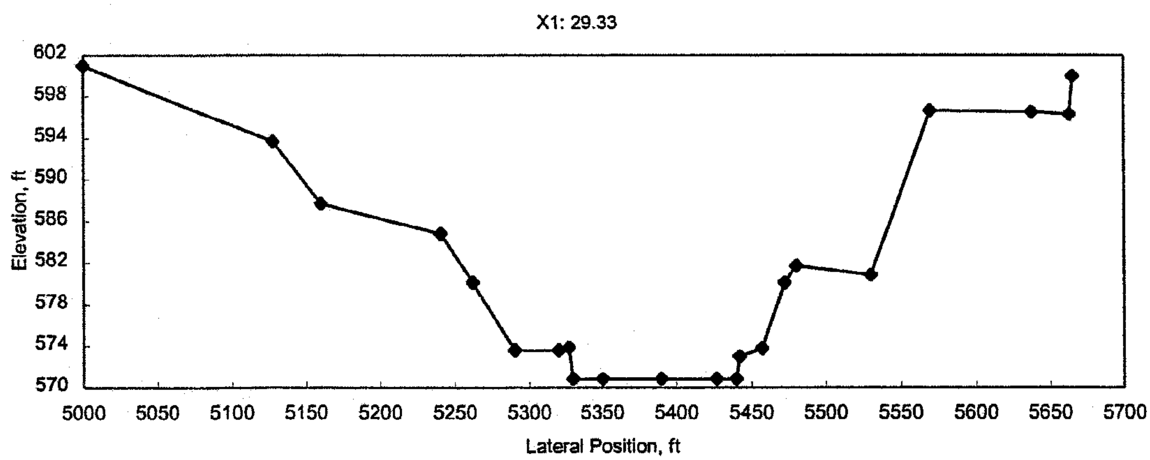
Cal-Sag Channel

Figure 3.7. Typical cross sections in various reaches of the Chicago Waterway System (Note: X1 is the position in river miles from Lockport Lock and Dam) – cont

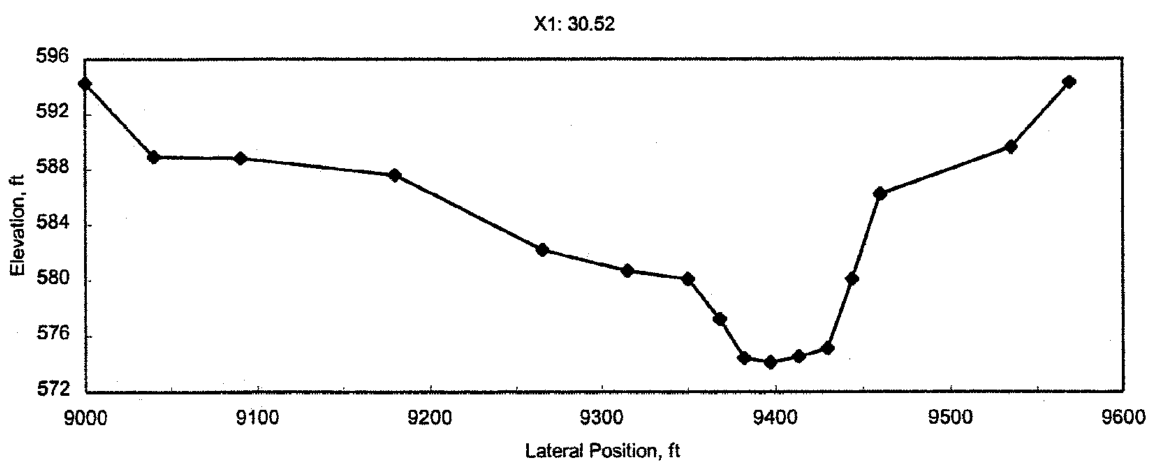




Little Calumet River North



Little Calumet River South



Little Calumet River South

Figure 3.7. Typical cross sections in various reaches of the Chicago Waterway System (Note: X1 is the position in river miles from Lockport Lock and Dam) – cont

3.4 Hydraulic Data Used for the Model Calibration and Verification

Although flows in the various branches of the CWS are not measured, water-surface elevation recorded at different locations was used for the calibration and verification of the model. The water-surface elevation recorded at Western Avenue (river mile (RM) 29.5), Willow Springs (RM 16.8), Cal-Sag Junction (RM 12.6), and Ashland Avenue on the Little Calumet River South (RM 29.3) by the MWRDGC and at Romeoville (RM 5.2) by the USGS will be used for model calibration and verification. In addition, daily flow data at the Chicago River Controlling Works and O'Brien Lock and Dam measured by the USGS will be used for calibration and verification of the model.

Flow data are available from the USGS at 5-minute intervals at the CRCW and the O'Brien Lock and Dam. Therefore, it is possible to compare results from DUFLOW and measured values at these locations at the 15-minute computational time step. This was not done for two reasons. First, DUFLOW calculates instantaneous flow values, whereas the acoustical velocity meters used by the USGS report the average flow rate over a 5-minute period based on the average velocity over this period. Thus, the USGS includes some smoothing of the highly variable data at these boundaries. Second, the flow velocities measured at both locations are very small even for higher flows. Under this flow condition the USGS discharge measurements are unbiased, but highly uncertain (Duncker, Gonzalez, and Over, 2003, Computation of Discharge and Error Analysis for the Lake Michigan Diversion Project—Lakefront Accounting Streamflow-Gaging Stations, U.S. Geological Survey Water-Resources Investigations Report, in preparation) making comparison of the daily values more reliable. Thirdly, because of the complex hydraulics of the CWS the flows at these boundaries are highly variable as shown in Figure 3.8, which shows a comparison of measured and simulated flows at a 15-min. time step over an example 2-day period. Considering the comparison in Figure 3.8, it is clear that a daily comparison of simulated and observed flows at the boundaries probably is most useful.

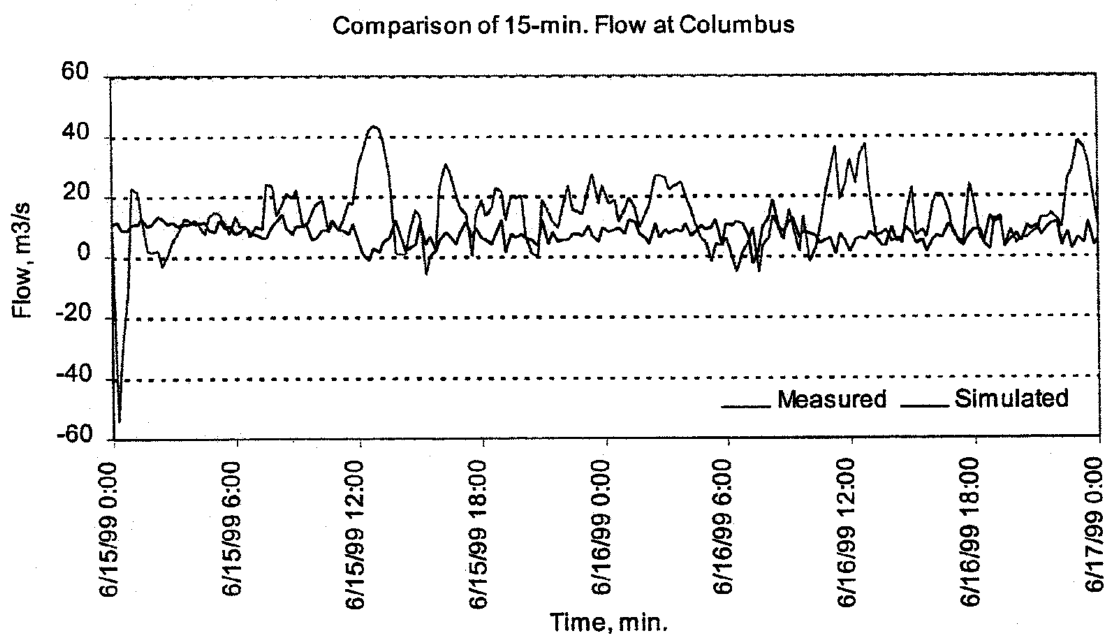


Figure 3.8. Comparison of measured and simulated (at a 5-minute time step) flows at a 15-minute time step at Columbus Drive.

3.5 Water-Quality Data Available for Model Calibration and Verification

In the case of water quality, data for calibration of the water-quality part of the model will be provided by the MWRDGC, and to a lesser extent by the Illinois Environmental Protection Agency (IEPA) and the Midwest Generation Power Company. The MWRDGC regularly samples constituent concentrations discharged from its WRPs discharging to the CWS: the North Side, Stickney, Lemont, and Calumet WRPs. The MWRDGC also takes approximately

12 grab samples of water quality per year at 28 locations in the CWS. The names and positions of the locations are given in Appendix II.

In 1998, the district installed a network of 19 continuous temperature and dissolved oxygen (DO) monitors along the North Shore Channel, North and South Branches of the Chicago River, Chicago River Main Stem, and the CSSC. Another 15 continuous temperature and DO monitoring stations were established in 2000 on the Chicago River Main Stem and on the Little Calumet River and Cal-Sag Channel. The details of the locations of all continuous monitors are given in Appendix III. These data will allow rigorous calibration of the water-quality model. The additional new data collection sites are listed in Appendix IV. Although these water-quality data are not used in this report, this information is included here to provide a more complete overview of the modeling project.

3.6 Flow Balance

The inflow to the CWS is comprised of flows from tributaries, water reclamation plants, pumping stations, CSOs, and from Lake Michigan at the controlling structures. All the inflow to the system are measured as outflow at Romeoville. Missing data from gaged sites, ungaged tributaries, and CSO flows have been estimated by various mathematical and statistical approaches. In order to check the reasonableness of these estimates and the potential accuracy of the modeling, the measured and estimated inflows were summed and compared to the outflow at Romeoville for each of the study periods. During the calculation of this flow balance, it is assumed that the difference in water balance due to the travel time and change in storage are negligible. The difference between all inflows to the system and flows at Romeoville for each of the periods studied is listed in the Table 3.9.

During the normal flow in the CWS, more than 60 % of flow is due to the water reclamation plants. The minimum average daily flow varies from 70 m^3/s in September – December to maximum of 136 m^3/s for the early August period. The maximum water drawn from the three intake controlling works is in summer and the minimum is in winter. The flow in the CWS varies widely from the average daily minimum less than 40 m^3/s to maximum daily flow of more than 300 m^3/s.

The flow balance of the CWS for the water year 1999 has been divided into eight different periods due to the discontinuous data at Romeoville. The total inflows are always higher than the outflows. The flow ranging from 1.24% higher for the May – June period to 7.66% higher for the August – September period. At first it might appear that this constant overestimation indicates a problem with the estimation of ungaged flows. However, for 5 of the 8 periods the **measured inflow** exceeded the measured outflow at Romeoville. Given the level of the quality assurance and quality control applied at USGS gages, particularly the Romeoville gage, it is suspected that the flows from the WRPs might be overestimated. Another possibility for the flow imbalance could be consumptive use at the Midwest Generation Fisk and Crawford Power Plants where water is withdrawn from the CWS for once through/run-of-the-river cooling water at the plants and immediately returned to the CWS. However, it was decided in discussions with the MWRDGC to not adjust the inflows from tributaries and WRPs or to try to account for consumptive use at the power plants and to let the model reduce the flows at the boundaries, as necessary. The method of flow adjustment would have little effect on the performance of the hydraulic model of the CWS.

Table 3.9. Balance of average daily flows in m^3/s for each indicated period for the Chicago Waterway System

Inflows	08/01-08/14	08/18-09/05	09/11-12/30	01/07-02/03	02/05-05/24	05/27-06/12	06/15-07/18	07/22-07/28
Wilmette	2.82	3.547	0.88	0.016749	0.25812	0.741454	2.333	2.476
North Side WRP	13.23	12.03	10.72	13.95	13.33	12.37	11.36	12.19
North Branch Pump St.*	4.66	2.07	2.99	10.13	7.75	4.51	2.92	2.56
Racine Pump St.	5.45	0.00	0.65	3.21	0.87	3.39	0.00	0.75
Stickney WRP	40.40	35.36	29.70	40.20	37.87	39.55	35.84	42.72
Lemont WRP	0.09	0.07	0.08	0.11	0.11	0.10	0.08	0.10
Citgo	0.10	0.10	0.10	0.06	0.13	0.13	0.11	0.10
CRCW	21.61	21.61	5.23	0.70	0.95	6.22	10.19	15.13
O'Brien L&D	14.28	16.68	4.80	0.97	1.96	6.04	9.18	13.86
Calumet WRP	12.03	10.27	10.48	13.73	13.16	12.82	11.77	12.02
Grand Calumet	1.70	1.55	0.63	0.50	0.71	0.61	0.45	0.60
Little Calumet	11.65	1.94	2.03	13.01	7.39	8.24	2.60	2.04
Midlothian	1.04	0.18	0.31	1.48	0.79	1.16	0.63	0.61
Stoney Creek E.	0.50	0.09	0.15	0.72	0.38	0.56	0.31	0.29
Tinley+Navajo	1.34	0.16	0.32	2.31	1.03	1.45	0.78	0.41
Mill+ Stoney W.	1.70	0.30	0.50	2.43	1.29	1.90	1.03	1.00
CalSag End Wshed	1.03	0.18	0.30	1.47	0.78	1.15	0.62	0.60
Calumet Union Ditch	1.21	0.21	0.36	1.73	0.92	1.35	0.73	0.71
DesPlaines Wshed	0.73	0.13	0.22	1.04	0.55	0.81	0.44	0.43
Romeoville	-128.97	-98.90	-66.43	-104.03	-88.06	-101.83	-84.92	-101.39
Total from Inflow	135.58	106.47	70.44	107.77	90.24	103.09	91.36	108.59
Flow Diff	6.60	7.58	4.01	3.74	2.18	1.27	6.44	7.20
%	-5.12	-7.66	-6.03	-3.60	-2.47	-1.24	-7.58	-7.10

Note: The bold letters indicated the measured flow locations.

* Sum of North Branch Chiacgo River and North Branch Pump Station Flows.

CHAPTER FOUR

CALIBRATION AND VERIFICATION OF THE HYDRAULIC MODEL

4.1 Model Formulation

The network editor needs to be prepared to run the model. The network editor is a graphical editor that enables the network schematization to be interactively drawn. The map layers for different features have been imported from ArcView GIS. The schematization is built up of nodes and sections. The schematization diagram is set up by defining the structures, discharge points, cross sectional data, and schematization points. The schematization of Chicago Waterway System (CWS) is shown in Figure 4.1.

The selection of the nodes is based on the data available for comparison with simulation results, upstream and downstream boundary locations, and the locations where there is interest in the simulation output result. The calculation nodes and the sections of the Chicago Waterway System are shown in Figure 4.2.

To schematize a river for modeling, it is necessary to split the river conceptually into reaches of gradually varying flow where head loss per length is relatively constant (for example, losses due to channel friction). The reaches and their number and order applied in DUFLOW are the same as those for the UNET model (Barkau, 1991, 1992) applied by the Corps to the CWS. The reaches and the inflow locations are shown in Figure 4.3. The reaches indicated in Figure 4.3 are different from those listed in Table 3.1 because Table 3.1 lists physically similar reaches of the CWS, whereas Figure 4.3 includes computational details of the CWS, such as the path around Goose Island.

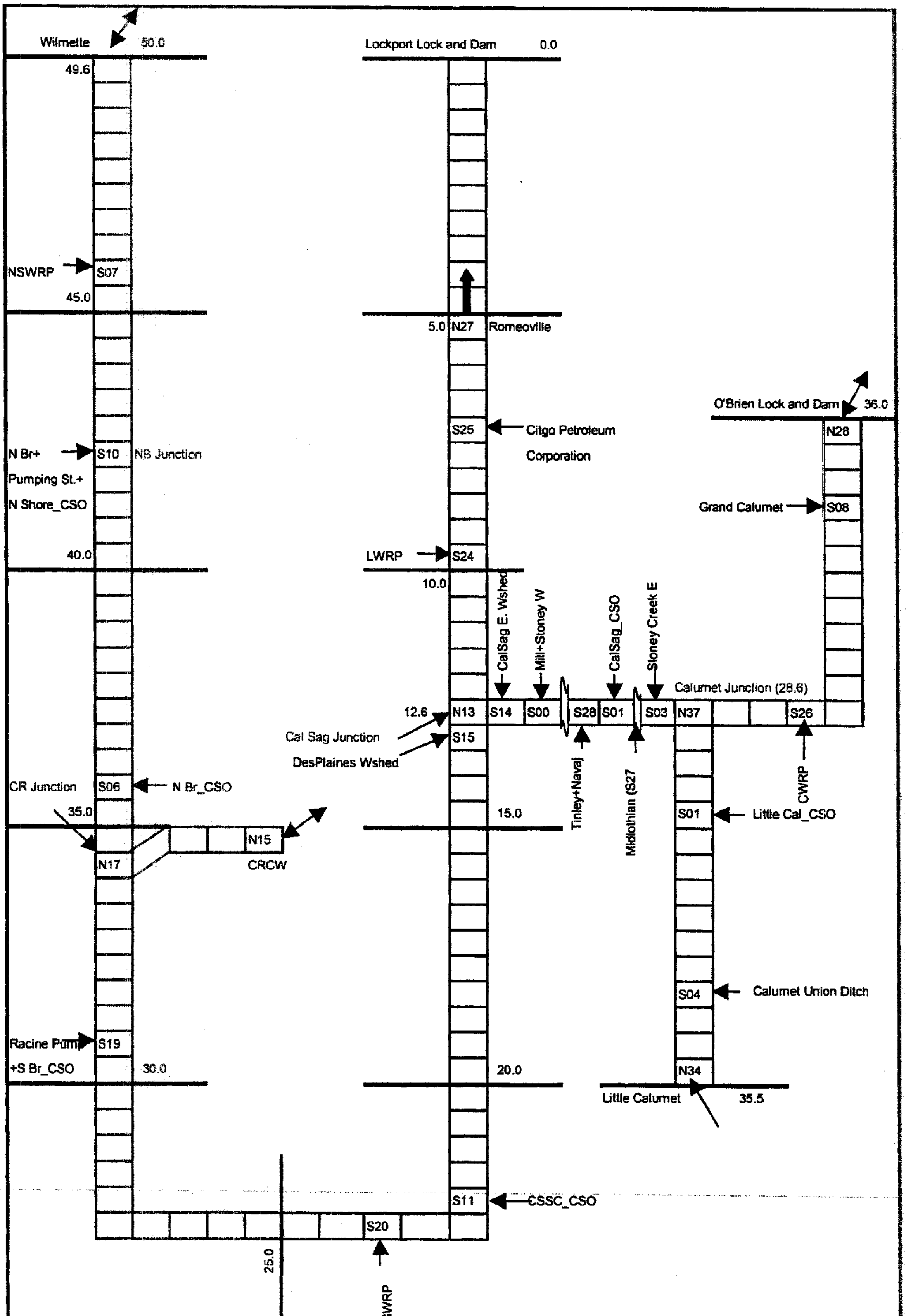


Figure 4.1. Model Schematic of the Chicago Waterway System

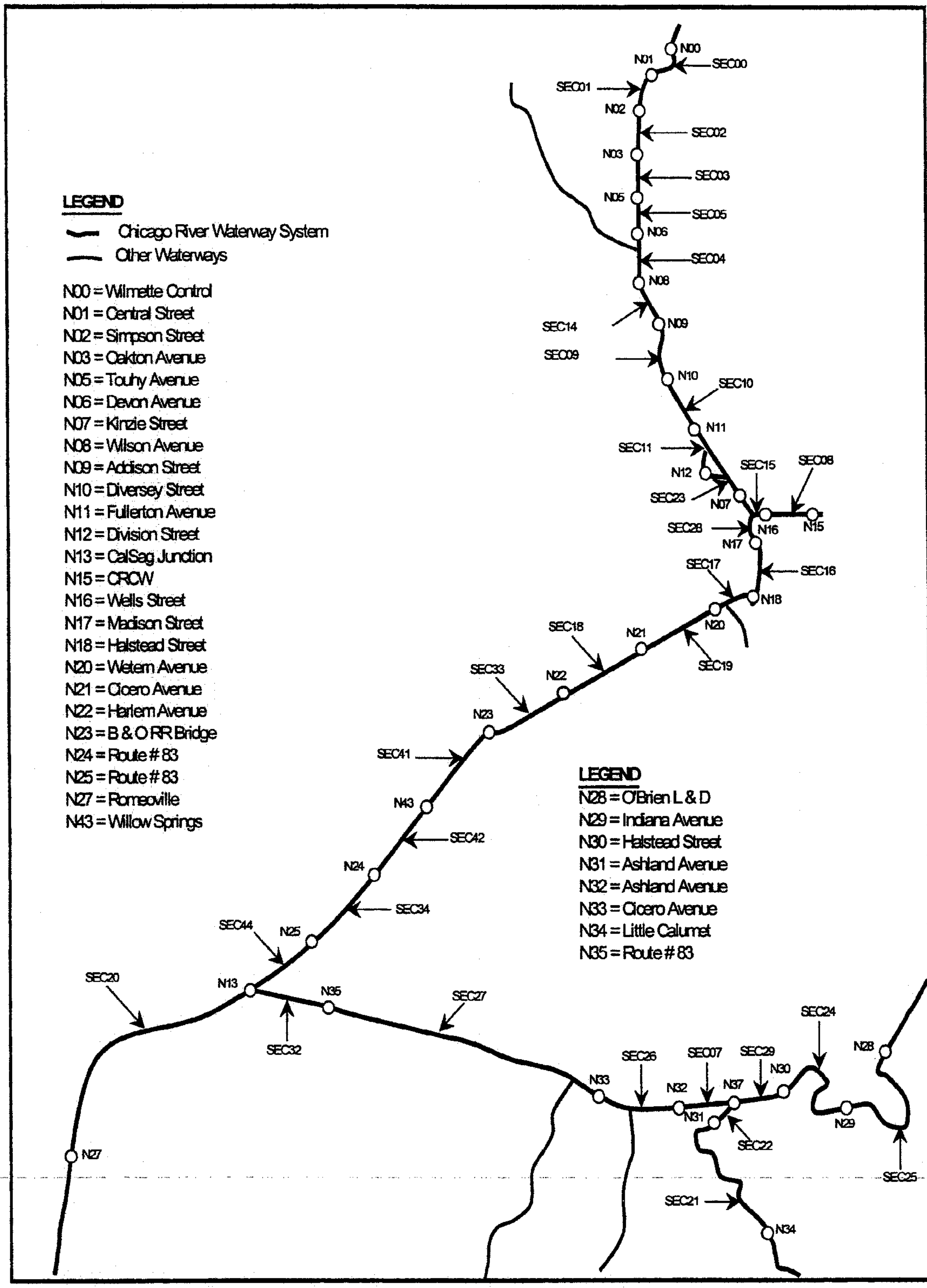


Figure 4.2. Calculation nodes and sections for the Chicago Waterway System

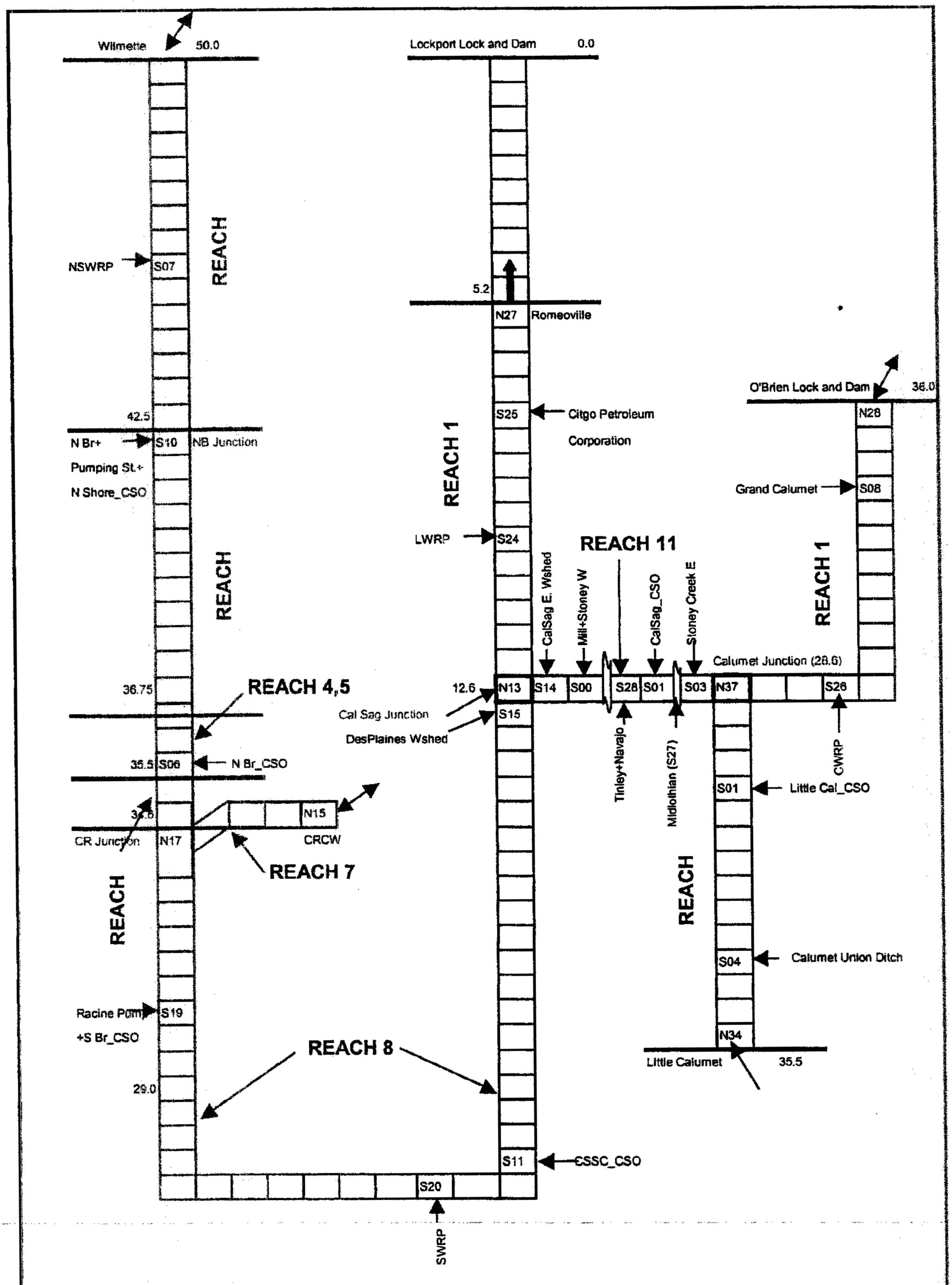


Figure 4.3. Reaches and Inflow Locations in the Chicago Waterway System

Four different locations are considered as the upstream boundary conditions of the model. They are:

1) the Chicago River Main Stem at Columbus Drive
2) the Calumet River at the O'Brien Lock and Dam
3) the North Shore Channel at Maple Avenue near Wilmette Pumping Station
4) the Little Calumet River at South Holland

Romeoville is considered as the downstream boundary condition. There are several input locations for measured flows to the CWS and for unmeasured drainage watersheds and tributaries. These unmeasured drainage and tributaries are represented by suitable methods as previously discussed and their entry locations to the CWS are shown in Figure 4.3. For the CWS, the CSO flow during heavy rainfall has a significant effect on the water-surface elevation as well as on the discharge of the system. CSO volumes were distributed to each reach by drainage area and distributed in time as per the operation of North Branch Pumping Station. The ratio of CSO area in each reach to the total CSO area as listed in Table 3.8. The calibrated CSO volumes and time periods are listed in Table 4.1. The locations of the 6 representative CSO inflow points also are shown in Figure 4.3.

Because of the discontinuity of the measured data for different periods in the downstream boundary condition at Romeoville, the study period is divided into 8 different periods between August 1, 1998 and July 31, 1999. These periods are defined in Table 3.6. The period of modeling varies from one week to four months.

Table 4.1. Calibrated CSO volumes for the entire Chicago Waterway System.

Date		CSO_Volume	
Start time	End time	Gallons	cu.m.
08/04/1998 14:22	08/04/1998 19:44	72900000	275956.47
08/07/1998 17:08	08/08/1998 23:56	131400000	497403.02
10/17/1998 17:02	10/18/1998 9:11	291600000	1103825.88
01/23/1999 4:05	01/24/1999 4:11	313200000	1185590.76
04/09/1999 0:20	04/09/1999 7:15	153900000	582574.77
04/22/1999 13:45	04/23/1999 18:02	469800000	1778386.14
04/27/1999 17:55	04/28/1999 5:13	129600000	490589.28
06/02/1999 21:56	06/03/1999 3:31	133500000	505352.38
06/12/1999 18:32	06/12/1999 22:00	44400000	168072.25
07/24/1999 0:33	07/24/1999 2:26	15250000	57727.52

4.2 Model Calibration

Model calibration is the testing or tuning of a model to a set of field data. Model calibration refers to the process of adjusting model parameters so predictions acceptably match field data. Model calibration consists of changing values of model input parameters in an attempt to match field conditions within some acceptable criteria. In the case of hydraulic modeling for the CWS, the model input parameter that was adjusted is Chezy's roughness coefficient (C), which is similar to Manning's roughness coefficient (as described in Section 4.3). The CSO volumes also were calibrated such that water-surface elevations throughout the CWS were matched during storm events. The primary fit criterion was to get the simulated water-surface elevations to within 1 percent of the measured water-surface elevations relative to the depth of the channel at the calibration point. It also was hoped to get the simulated daily flows at the upstream boundaries to within 10 percent of the measured flows at these points. However, because of the flow inflow-outflow imbalance discussed in Section 3.6 this could not be achieved.

The period from 01/07/1999 to 02/03/1999 was selected for calibration purposes. The selection of this period mainly results because the storm flow and low flow during this period made it a relatively average period representative of the other study periods. CSO overflow also occurred in this period. Once the model is calibrated, the model was verified by application to the other periods of measured data.

In this study, there are all together 11 reaches. Among them reach number 8, Chicago Sanitary and Ship Channel (CSSC) and reach number 11, Cal Sag channel are the longest. The reach numbers 8 and 11 are 22 and 16 miles long, respectively. Due to the higher discharge in these reaches, a change in Chezy's coefficient during the calibration in these reaches effects to the whole CWS. The model output is more sensitive to conditions in these, so the main focus was given to these reaches during calibration. The calculation distance between two nodes is taken 1000 meters per pixel.

4.3 Roughness Coefficient Selection

Manning's roughness coefficient, n, commonly is used to represent flow resistance for hydraulic computations of flow in open channels. The procedure for selecting *n* values is subjective and requires judgment and skill that is developed primarily through experience. In this modeling the initial estimates of Manning's n were derived from the UNET model (Barkau, 1991) of the CWS developed by the Corps. The value of Manning's coefficient, n, is related to the channel-boundary friction. The DUFLOW model uses Chezy's roughness coefficient, C, to calculate the hydraulic resistance, whereas UNET uses Manning's roughness coefficient, n. Thus, the following relation was used to convert the Manning's, n, to Chezy's, C.

$$C = \frac{R^{1/6}}{n}$$

where:

C = Chezy's Coefficient

R = Hydraulic Radius (taken as equal to a representative flow depth in each reach)

The conversation of Manning's n to the Chezy's C and the modified C for the calibrations of the model are listed in Table 4.2. The initial value of C obtained by conversion of Manning's n used in UNET was applied to the initial model. Initially simulated stages and discharges were very much different compared to the measured data. So, adjustments for the C value were made for calibration purposes. The change of the C value in reaches 2, 8, and 11 substantially affected the discharges and stages. The change of C value in the Chicago River main stem to North Branch does not have a significant effect on the results. The Manning's n value from the UNET model is higher in all reaches compared to DUFLOW calibrated value.

At the Romeoville site Gonzalez et al. (1996) estimated the at-a-point-Manning's n (Manning's n is normally defined as a roughness coefficient for an entire reach rather than a single point) to be 0.03 (Chezy C = 45) on the basis of velocity distributions measured with an acoustic Doppler current profiler. Further, the Manning's n values found in DUFLOW calibration agree reasonably well with the pictorial representation of Manning's n given by Chow (1959, p. 117 – 120). The high Manning's n value for the Little Calumet River South reflects the composite roughness of the channel and the floodplain.

Table 4.2. Comparison of Manning's and Chezy's coefficients, n and C, respectively, for the U.S. Army Corps of Engineers UNET model (Barkau, 1991) and the DUFLOW calibrated values.

Reach No.	Reach Name	Hyd. Radius (meters)	Army Corps		DUFLOW	
			n	C	C	n
2	North Shore Channel	2.37	0.05	23	38	0.030
3	North Branch	3.08	0.05	24	38	0.032
4	Goose Island West	4.86	0.05	26	38	0.034
5	Goose Island East	4.86	0.05	26	38	0.034
6	South Branch	4.86	0.05	26	38	0.034
7	Chicago River Main Stem	5.59	0.03	44	44	0.030
8	Chicago Sanitary and Ship Canal	4.61	0.05	26	60	0.022
9	Little Calumet River South	0.93	0.20	5	6	0.165
10	Little Calumet River North	2.16	0.03	38	50	0.023
11	Cal Sag Canal	2.93	0.03	40	47	0.025
12	Romeoville	6.26	0.04	34	41	0.033

4.4 Verification of the Model

The verification of a mathematical model is essentially a checking process whereby the predicted output from the model is checked against known data in the form of a structured process of comparative analysis. Errors in those elements of data, which are selected or input by the modeler are identified and corrected until the simulated and measured performances agree within acceptable limits. Specifically, verification is the process to find and fix the modeling errors and assure the modeler in his assumptions. The verification often detects *bugs* that require further debugging, or incorrect assumptions that require significant modifications in the model.

For the purpose of verification, the other seven periods were considered. The measured data at different locations were compared with simulated data for these periods.

4.5 Flow Model Results

The evolution of the simulated amount of water flow and the water-surface elevation at different locations on the CWS in a 44.8 mi reach during the period August 1, 1998 to July 31, 1999 and their comparison with measured flow and water-surface elevation is discussed in the following paragraphs. The 44.8 mi reach of the CWS starts from the Wilmette Pumping Station and extends to Romeoville. Another 22.4 mi reach of the CWS also was studied. It starts at the O'Brien Lock and Dam and connects with the Chicago Sanitary and Ship Channel at the Cal-Sag Junction. The CWS is a very complex river system. During the summer, water is withdrawn from Lake Michigan, but during CSO flows water may flow towards Lake Michigan. The water flow in the CWS also is affected by the gate operation at Lockport, 5.2 mi downstream from Romeoville. During normal flow, the simulated flows more or less follow the water-surface elevation pattern of observed flow. But when CSOs occur or significant storm flows are anticipated, gates are operated at Lockport to draw down the CWS increasing flows from the CWS and providing storage for floodwaters in the CWS. During storms higher flow from the tributaries results, and those flows, especially from the Little Calumet River South, substantially affect flow patterns in the CWS. The simulated flow did not agree well with the observed flows at the upstream boundaries even though the model yielded a good fit with observed water-surface elevation data. The results for the different periods are discussed in detail in the following subsections.

To calibrate the model, stage data collected at five different locations of the CWS were compared with the simulation results. These locations are:

a) Western Avenue at river mile 29.5

b) Willow Springs at river mile 16.8

c) Cal-Sag Junction at river mile 12.6

d) Romeoville at river mile 5.2

e) Ashland Avenue at river mile 29.3 on the Little Calumet River South

The last location has a lot of missing data during the study periods; so statistical analysis has not been done to verify the model at this location. However a graphical comparison has been done at this location especially for storm periods when data are available. In addition to the stage measurement locations listed previously, flow measurement at two upstream boundary locations - Chicago River at Columbus Drive and O'Brien Lock and Dam - have been considered in model calibration. The main purpose of comparison of stage and flow measurements at different locations with model simulation results is to get a competent and reliable model.

4.5.1 January – February Period (01/07/99 – 02/03/99)

This period is considered as the calibration period for the model. The reasons for considering this as the calibration period are mainly that it represents an average period, with CSO flow and the higher tributary inflows. Once the considered period is well calibrated, it is assumed that the considered Chezy coefficients will be applicable to all other periods. First, the model was calibrated without considering the CSO flows and simulated values were compared with observed data. The model did not represent a good fit for the storm period. The simulated stage was substantially lower than the measured stage indicating that the model was artificially dewatering the canal to generate outflows greater than inflows so that the observed outflows could be matched.

After considering CSO flows, and appropriate calibration of Chezy's coefficient, the model shows good agreement between the measured and simulated stages along the CWS. The stage comparison at five different locations shows very good agreement between the measured data and simulation results. The comparison of measured and simulated stages is shown in Figure 4.4. Although most of the time the simulated stage is higher than the measured stage, the difference between the measured and simulated stages are nearly all below 2% of the depth of water. More than 87% of the stage differences are below 1% of the depth except at Cal-Sag Junction. The percentage error of stages according to the depth of water flow is shown in Table 4.3.

Table 4.3. Percentage of the hourly stages for which the error in simulated versus measured stages relative to the depth of water flow is less than the specified percentage

Periods	Total no.	Western Av.		Willow		Cal Sag Junction		Romeoville	
	of Stages	<±1%of D	<±2%of D	<±1%of D	<±2%of D	<±1%of D	<±2%of D	<±1%of D	<±2%of D
August	337	96.44	100	86.35	99.70	90.80	97.92	96.44	98.52
Aug_Sep	457	98.25	100	75.27	99.12	92.56	99.56	95.62	100
Sep_Dec	2665	96.19	100	83.90	99.06	84.09	99.17	91.18	99.25
Jan_Feb	673	94.95	100	88.86	100	78.75	100	87.37	99.85
Feb_May	2617	93.74	99.79	85.39	98.94	80.75	99.05	93.43	99.35
May_June	409	94.87	100	81.42	99.51	87.78	97.56	92.42	96.82
June_July	817	97.80	100	85.19	99.88	93.88	99.88	96.70	99.88
July	169	98.22	100	79.88	99.41	87.57	100	92.31	99.41

The average absolute stage difference and depth error are less than 6% and 1%, respectively. The correlation coefficient between measured and simulated hourly stages for this period at different locations varies from 90% to 99%. The statistical analysis for this period is listed in Table 4.4. The January – February period experienced heavy rainfall on January 22 and January 24. There were CSO flows during this period, so a sudden change in the stages resulted at all locations during this period. At upstream locations (Western Avenue and Willow Springs) there is a sudden increase in the stage due to the higher flows on January 22 and there is another increase in stage due to the rainfall event on January 24. With respect to stage, the model is calibrated very well. The comparison of measured and simulated stage at the Ashland Avenue also shows a good fit. During the storm period the simulated value is about 0.25 m higher with about a one-day time lag. During the normal flow, the simulated stage always is lower than the measured stage.

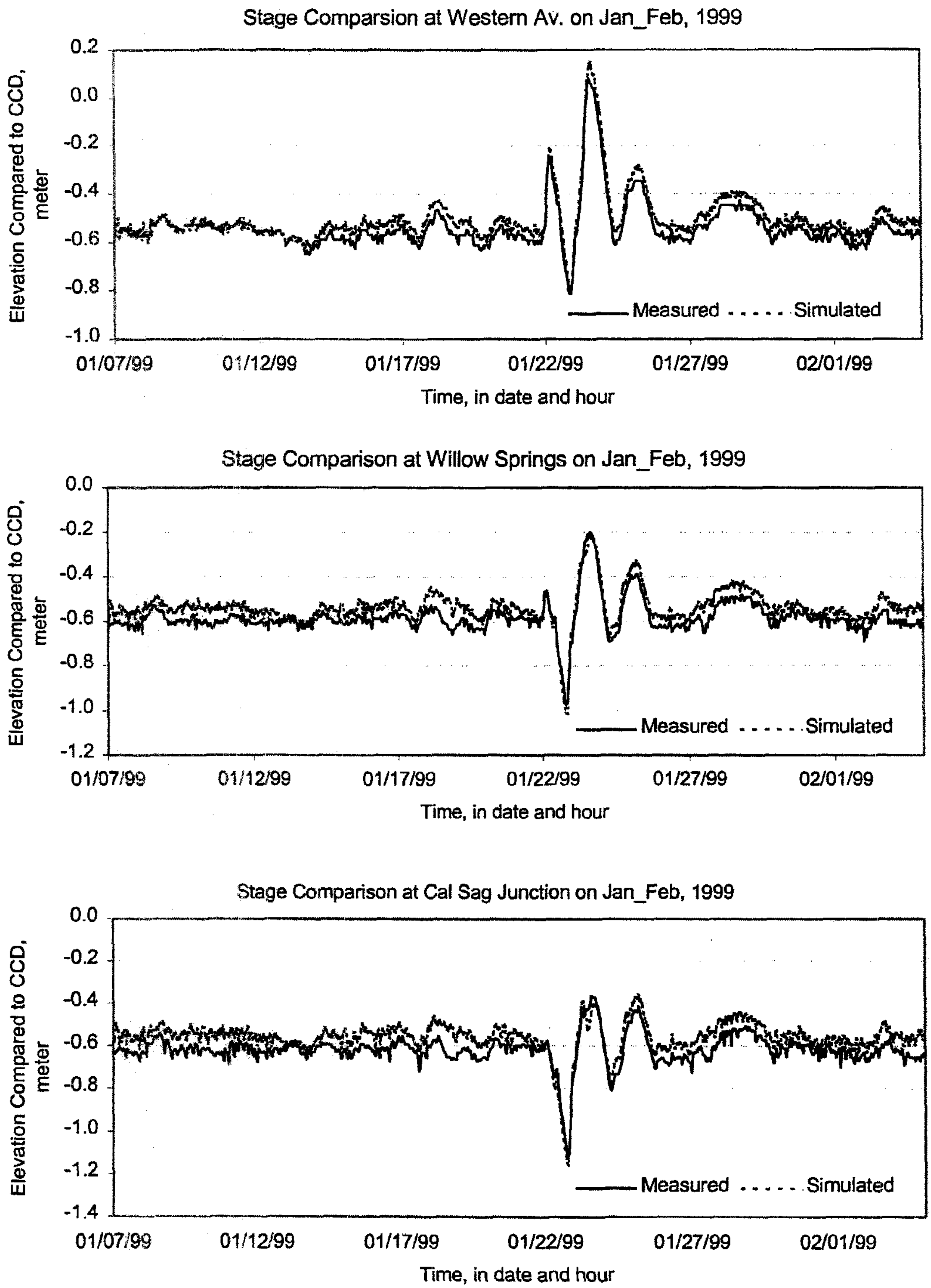
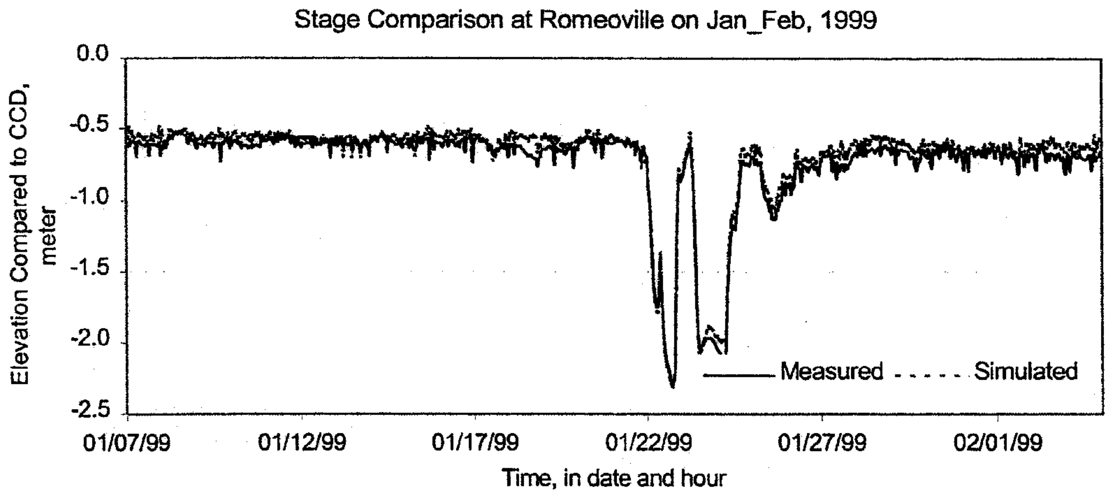


Figure 4.4. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the January – February, 1999 period



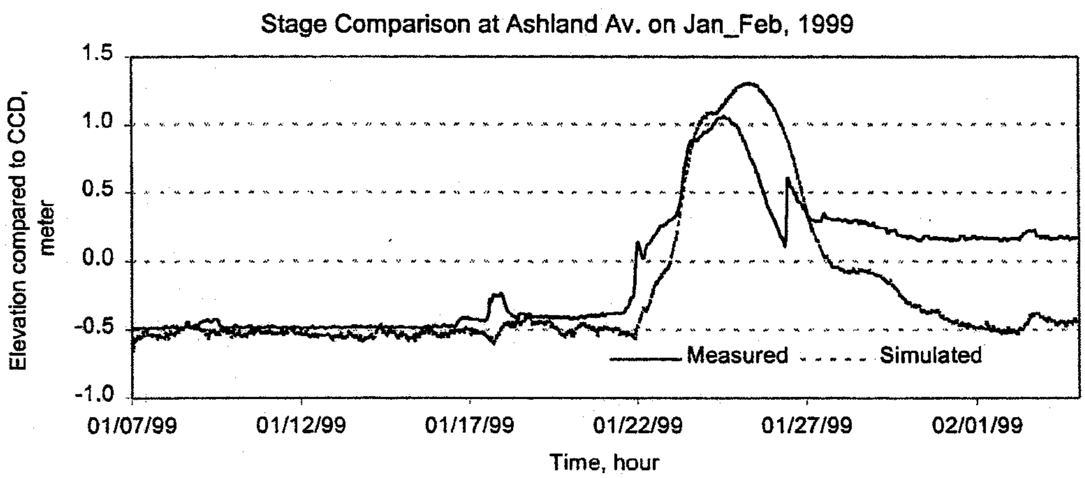


Figure 4.4. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the January – February, 1999 period - cont

Figure 4.5 shows the measured and simulated flows for the January – February, 1999 period. The measured flow at both Columbus Drive and O'Brien Lock and Dam during this period is relatively small throughout the period. The simulated flows for this period do not have a good fit. In the beginning of the period, the Columbus Drive had negative simulated flow (i.e. water flows towards Lake Michigan) while there is more water withdrawn from Lake Michigan at O'Brien Lock and Dam in the simulation than was measured. This phenomenon could result because of different reasons described in the following paragraphs.

During this time most of the water flow (about 65%) in the system is from the water reclamation plants. The discharge measurement from the Calumet Water Reclamation Plant is available three times a day. So, for the model calibration, hourly estimation of discharge is done by linear interpolation or an hourly discharge ratio. The hourly discharge ratio was established by the partial series of hourly discharge measurements taken in December 2000 and January 2001.

There are about 132 mi^2 of ungaged tributaries and watersheds and about 226 mi^2 of ungaged CSO drainage area. The flow from ungaged tributaries and watersheds to the system is estimated from the Midlothian Creek watershed assuming that the topography, rainfall, and runoff pattern is same throughout the area. The CSO volume is determined by matching simulated and measured stages at Romeoville during CSO periods and is uniformly distributed in time and space during the period when the North Branch Pumping Station is in operation.

Table 4.4. Statistical analysis of simulated versus measured hourly stages for the different periods at different locations on the Chicago Waterway System

Periods	Difference (Sim - Obs)					Depth Error					Corr.	Bias	
	Min	Max	Average	Median	Ave. ABS	Min	Max	Mean	Median	Ave. ABS	Coeff.	-ve	+ve
August													
Western	-0.0768	0.1193	0.0046	0.0025	0.0225	-0.0110	0.0174	0.0007	0.0004	0.0033	0.92	160	177
Willow	-0.1163	0.2026	0.0351	0.0377	0.0451	-0.0157	0.0278	0.0047	0.0050	0.0061	0.87	52	285
Cal Sag	-0.2469	0.1579	-0.0074	-0.0005	0.0374	-0.0311	0.0204	-0.0009	-0.0001	0.0047	0.89	172	165
Romeoville	-0.4316	0.0905	0.0008	0.0093	0.0310	-0.0639	0.0112	-0.0001	0.0011	0.0040	0.99	128	209
Aug_Sep													
Western	-0.0682	0.1198	0.0140	0.0134	0.0225	-0.0098	0.0176	0.0020	0.0019	0.0033	0.80	128	329
Willow	-0.0604	0.3000	0.0557	0.0556	0.0576	-0.0082	0.0415	0.0075	0.0074	0.0077	0.79	20	437
Cal Sag	-0.1141	0.1911	0.0173	0.0160	0.0351	-0.0149	0.0246	0.0022	0.0020	0.0044	0.80	155	302
Romeoville	-0.0598	0.1266	0.0338	0.0340	0.0385	-0.0085	0.0156	0.0041	0.0042	0.0047	0.98	57	400
Sep_Dec													
Western	-0.0409	0.1335	0.0301	0.0296	0.0315	-0.0059	0.0197	0.0044	0.0043	0.0046	0.88	174	2212
Willow	-0.0551	0.3605	0.0495	0.0483	0.0505	-0.0075	0.0504	0.0066	0.0065	0.0068	0.72	87	2578
Cal Sag	-0.1138	0.2488	0.0475	0.0489	0.0515	-0.0147	0.0315	0.0060	0.0061	0.0065	0.69	248	2417
Romeoville	-0.1788	0.3043	0.0414	0.0405	0.0453	-0.0248	0.0375	0.0050	0.0049	0.0055	0.95	245	2420
Jan_Feb													
Western	-0.0262	0.0956	0.0435	0.0431	0.0437	-0.0038	0.0140	0.0063	0.0063	0.0063	0.9903	2	513
Willow	-0.0620	0.1313	0.0452	0.0462	0.0477	-0.0085	0.0177	0.0060	0.0062	0.0064	0.9374	32	636
Cal Sag	-0.0942	0.1548	0.0573	0.0606	0.0614	-0.0116	0.0196	0.0072	0.0076	0.0077	0.9069	38	630
Romeoville	-0.0939	0.1762	0.0407	0.0428	0.0477	-0.0137	0.0217	0.0050	0.0053	0.0059	0.9913	93	575

Table 4.4. Statistical analysis of simulated versus measured hourly stages for the different periods at different locations on the Chicago Waterway System - continued

Periods	Difference					Depth Error					Corr.	Bias	
	Min	Max	Average	Median	Ave. ABS	Min	Max	Mean	Median	Ave. ABS	Coeff.	-ve	+ve
Feb_May													
Western	-0.1625	0.1503	0.0300	0.0308	0.0344	-0.0231	0.0222	0.0044	0.0045	0.0050	0.90	266	2065
Willow	-0.1101	0.2249	0.0444	0.0439	0.0997	-0.0167	0.0346	0.0068	0.0067	0.0154	0.75	148	1645
Cal Sag	-0.2268	0.2385	0.0429	0.0454	0.0528	-0.0287	0.0301	0.0054	0.0057	0.0067	0.77	355	1962
Romeoville	-0.3919	0.2768	0.0235	0.0320	0.0370	-0.0576	0.0347	0.0028	0.0028	0.0044	0.98	612	2005
May_June													
Western	-0.0971	0.1021	0.0173	0.0199	0.0289	-0.0131	0.0149	0.0025	0.0029	0.0042	0.91	106	303
Willow	-0.1588	0.1481	0.0362	0.0404	0.0486	-0.0207	0.0201	0.0049	0.0054	0.0065	0.72	62	347
Cal Sag	-0.2260	0.2116	0.0185	0.0238	0.0445	-0.0284	0.0273	0.0023	0.0030	0.0056	0.78	112	297
Romeoville	-0.5948	0.1123	0.0077	0.0209	0.0432	-0.0863	0.0137	0.0007	0.0025	0.0056	0.99	108	301
June July													
Western	-0.1018	0.1163	0.0178	0.0184	0.0238	-0.0146	0.0170	0.0026	0.0027	0.0035	0.80	169	648
Willow	-0.1093	0.1841	0.0456	0.0472	0.0476	-0.0146	0.0251	0.0061	0.0063	0.0064	0.63	50	767
Cal Sag	-0.0973	0.1943	0.0238	0.0248	0.0355	-0.0122	0.0247	0.0030	0.0031	0.0045	0.51	205	612
Romeoville	-0.1108	0.2075	0.0266	0.0282	0.0338	-0.0137	0.0254	0.0032	0.0034	0.0041	0.89	149	668
July													
Western	-0.0557	0.0803	0.0184	0.0195	0.0255	-0.0081	0.0117	0.0027	0.0028	0.0037	0.87	32	137
Willow	-0.0655	0.1651	0.0455	0.0446	0.0504	-0.0087	0.0223	0.0061	0.0060	0.0067	0.76	19	150
Cal Sag	-0.1088	0.1567	0.0224	0.0239	0.0418	-0.0135	0.0198	0.0028	0.0030	0.0052	0.74	47	122
Romeoville	-0.0605	0.1670	0.0289	0.0264	0.0385	-0.0074	0.0204	0.0035	0.0032	0.0047	0.94	30	139

The simulation tendency of getting higher flows toward the Chicago River Controlling Works (CRCW) (i.e. negative flows at Columbus Drive) and flows into the CWS at O'Brien Lock and Dam is affected by the flows at Romeoville and on the Little Calumet River South, respectively. The difference between the USGS measured and simulated average flow for each period is listed in Table 4.5. The average over inflow (i.e. amount by which the measured and estimated inflows not including CSOs to the CWS exceeds the measured outflow at Romeoville) in each period is about the same as the under simulation of flow at the upstream boundaries.

Table 4.5. Summary of observed and simulated average flow.

Periods	Wilmette Avg		Columbus		O'Brien L&D		Flow	
	Obs	Sim	Obs	Sim	Obs	Sim	Under Sim	Over inflow
August	2.82	6.06	21.50	18.13	13.90	7.06	6.97	6.60
Aug_Sep	3.54	5.68	21.84	18.85	16.83	9.82	7.86	7.58
Sep_Dec	0.88	0.36	5.27	3.47	4.83	2.72	4.44	4.01
Jan_Feb	0.02	-0.06	0.70	-2.27	0.98	-1.73	5.75	3.74
Feb_May	0.25	-0.46	0.94	-0.55	1.97	-0.39	4.56	2.18
May_June	0.74	1.72	6.15	7.15	5.85	1.70	2.17	1.27
June_July	2.33	3.48	9.90	7.59	9.04	4.16	6.04	6.44
July	2.48	3.78	14.59	10.42	13.58	10.33	6.12	7.20

(Note: the under simulation includes the effects of CSOs whereas the over inflow does not)

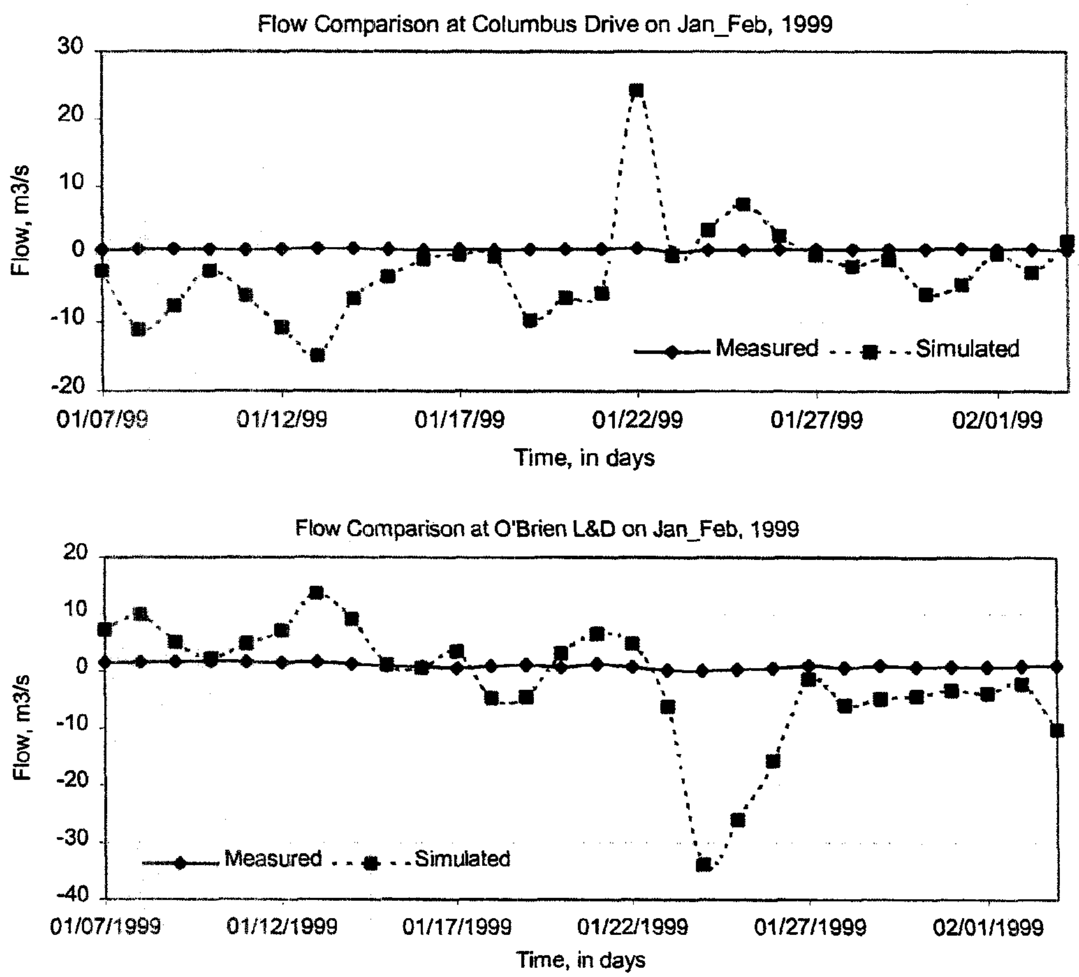


Figure 4.5. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the January – February, 1999 period

In early January the under simulation of inflow at Columbus Drive seems to be directly counteracted by the over simulation of inflow at O'Brien Lock and Dam. Efforts to try and balance these were not successful. In part, this resulted because there was, on average, 3.74 m^3/s more inflow than outflow (not including CSOs). Thus, when the actual leakage only flows at the boundaries are near zero, negative flows will be simulated at the boundaries. Further, the typical flow cross sectional area is 4,500 ft^2 = 418 m^2 at Columbus Drive and 5,100 ft^2 = 474 m^2 at O'Brien Lock and Dam. Thus, a discharge error of 15 m^3/s results in velocity errors of 0.036 and

0.032 m/s at Columbus Drive and O'Brien Lock and Dam, respectively. Given the complexity of the CWS and the relatively small stage differences that drive flows, this velocity accuracy is about as good as can be expected and is similar to the accuracy of the Acoustic Velocity Meters (AVMs) at these locations (Duncker, Gonzalez, and Over, 2003, Computation of Discharge and Error Analysis for the Lake Michigan Diversion Project—Lakefront Accounting Streamflow-Gaging Stations, U.S. Geological Survey Water-Resources Investigations Report, in preparation).

The high negative flows on January 24 – 26 at O'Brien Lock and Dam correspond to a period of high flows on the Little Calumet River South. Figure 4.6 shows the measured water-surface elevation at O'Brien Lock and Dam, Cal-Sag Junction, and Ashland Avenue on the Little Calumet River South. After January 22, there is a 0.5 m or more gradient between Ashland Avenue on the Little Calumet River South and O'Brien Lock and Dam. Thus, it seems reasonable to assume that under these conditions the Little Calumet River North seeks to follow its natural course toward Lake Michigan. Of course the lock and dam prevents the water from going to the lake.

It is suspected that the high negative flows simulated at O'Brien Lock and Dam reflect actual high negative flows up the unmodeled lower Grand Calumet River. The Hohman Avenue gaging station on the Grand Calumet River is approximately 3 miles upstream from the O'Brien Lock and Dam, and the negative flow is thought to fill channel storage downstream from the Hohmann Avenue gage.

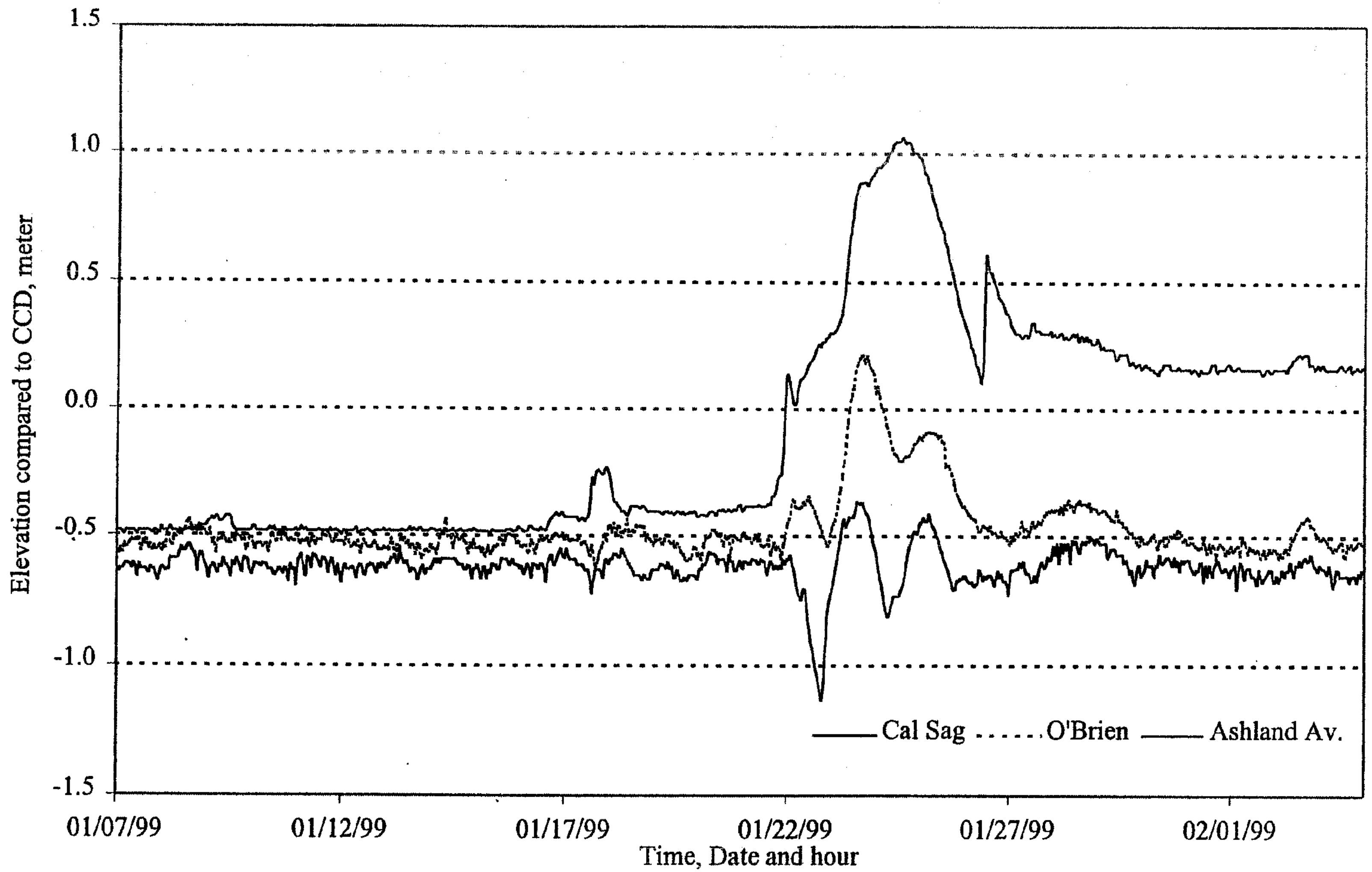


Figure 4.6. Measured stages at Cal-Sag Junction, O'Brien Lock and Dam and the Little Claumet River at Ashland Avenue for January 7 - February 3, 1999

4.5.2 August Period (08/01/98 – 08/14/98)

Figure 4.7 shows the results for the start of the modeling period. The stage at Romeoville late on August 4, 1998 decreases to –2 m because of the opening of gates at Lockport to increase slope in the system and release the higher flows in the system. The comparison of measured and simulated stages at Western Avenue and Romeoville is very good (correlation coefficient > 0.9), compared to the fit at Willow Springs and Cal-Sag Junction (correlation coefficient between 0.87 and 0.89) shown in Table 4.4. The simulated stage is comparatively lower than measured at the Little Calumet River South at Ashland Avenue during low flows (Figure 4.7). During the storm flow on August 4 –8, the simulated stage has the appropriate magnitude but is delayed a few hours. This delay may result from the fact that CSOs are handled on an area-wide basis rather than locally. For the CSSC and South Branch Chicago River locations, average absolute difference ranges from 2% to 4.5%. The average absolute difference with respect to depth is less than 1% as shown in Table 4.3. Ninety-eight percent of the errors in stage relative to the depth of water are less than 2% at all locations on the CSSC and South Branch Chicago River.

The flow comparison between measured and simulated flows at Columbus Drive and O'Brien Lock and Dam is shown in Figure 4.8. In the case of the flow comparison at Columbus Drive, the pattern of simulated and measured flow is identical most of the time. The simulated flow is lower than measured except in the case of the peak flow, reflecting the 6.6 m^3/s over inflow to the CWS during this period. In the case of O'Brien Lock and Dam, the flow pattern of simulated flow follows the measured flows except for a few times. There is high negative flow on 08/05/1998. During this time high flow was measured on the Little Calumet River South. As discussed previously, whenever there is high flow on the Little Calumet River South, the model gives negative flow at O'Brien Lock and Dam that most likely goes into channel storage on the

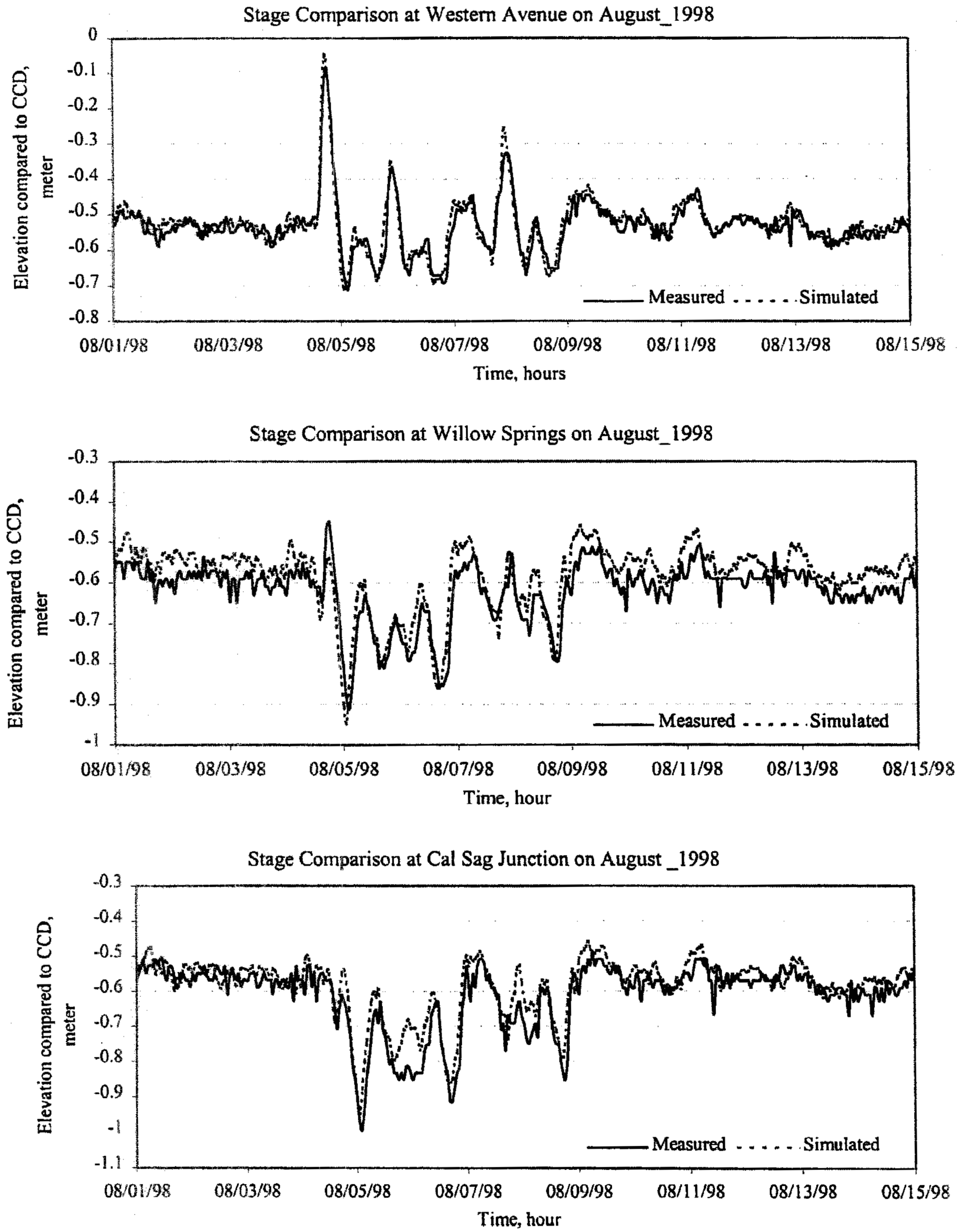


Figure 4.7. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the August 1998 period

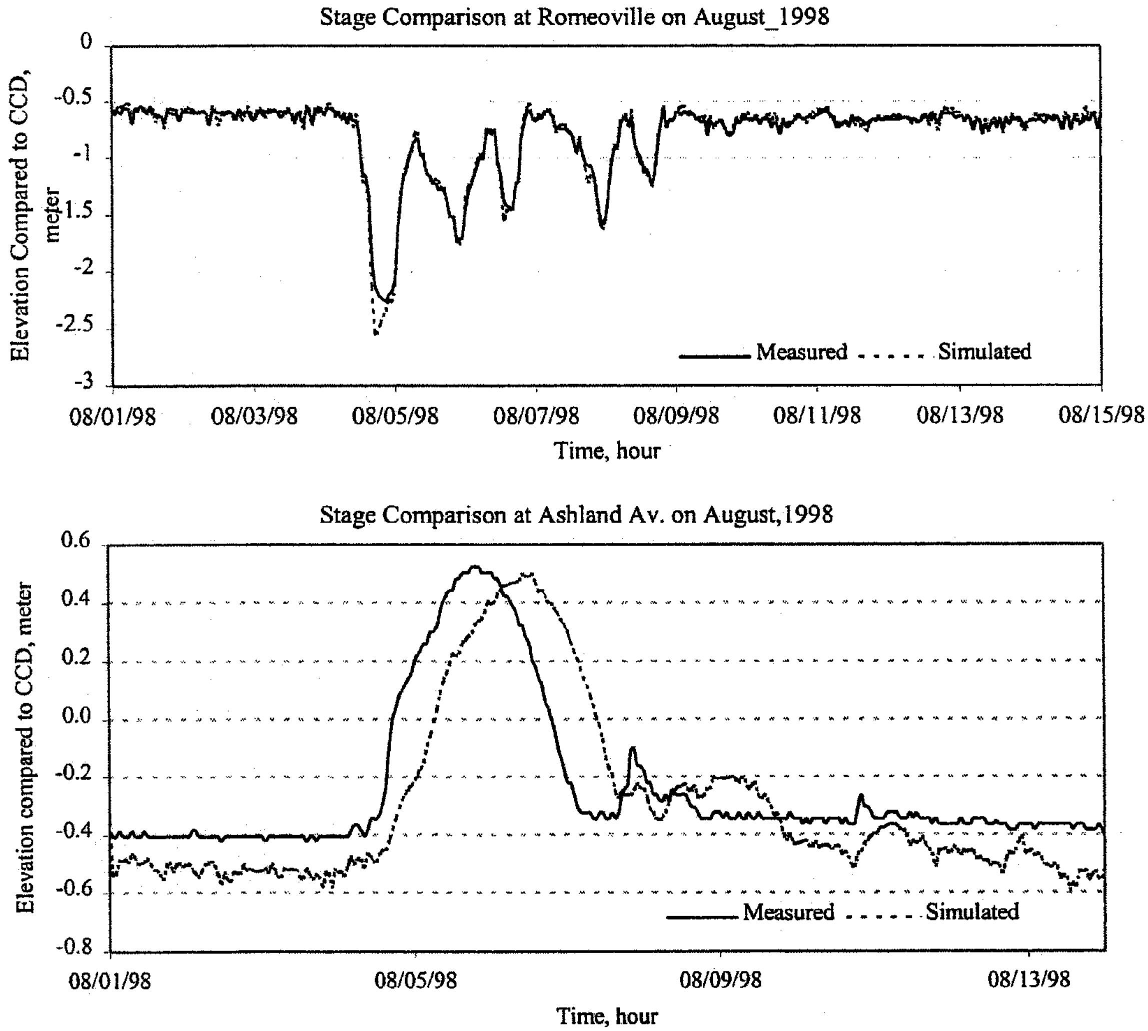
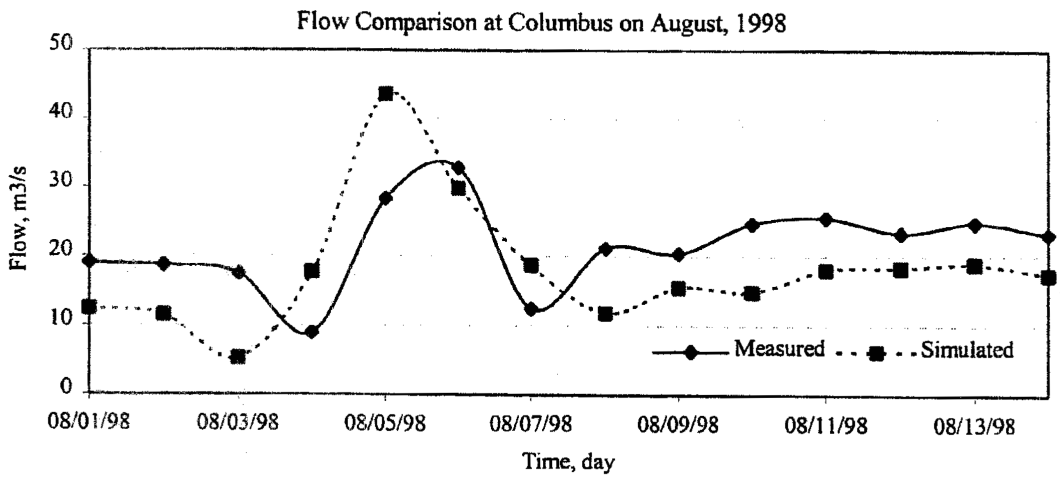


Figure 4.7. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the August 1998 period – cont

lower Grand Calumet River.



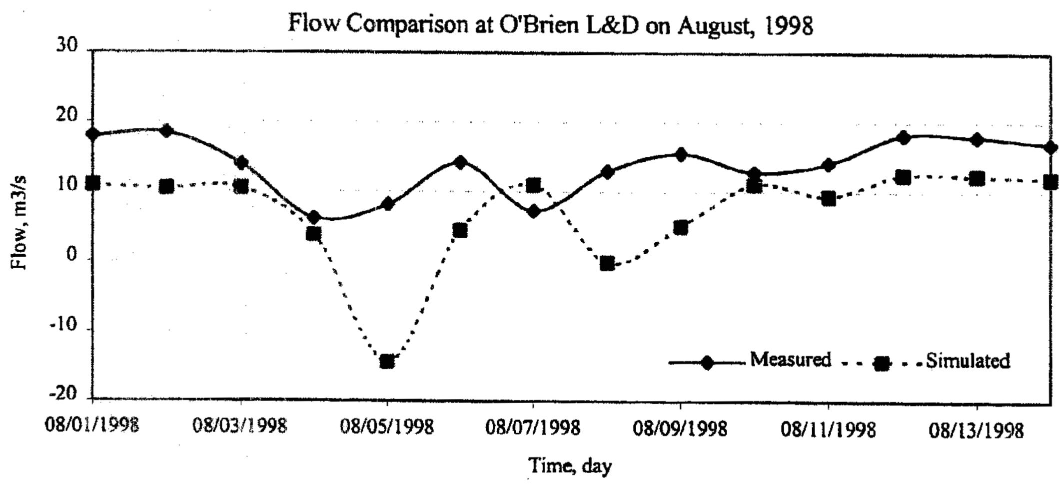


Figure 4.8. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the August 1998 Period

4.5.3 August – September Period (08/18/98 – 09/05/98)

The comparison of measured and simulated stages at different locations is shown in Figure 4.9. During this period more than 50% of the discharge in the CWS is from the water reclamation plants. For the purpose of maintaining the better water quality in the CWS, water also is drawn from Lake Michigan. The daily average water drawn during this period at the Chicago River Controlling Works and O'Brien Lock and Dam is about 22 m^3/s and 17 m^3/s, respectively. The water quantity drawn from these two locations is significant, and helps to maintain the water level in the CWS. There also is a relatively small amount of water drawn (about 3.5 m^3/s) from the Wilmette Pumping Station. In comparison to all other study periods, the maximum quantity of water is withdrawn from Lake Michigan in this period.

The comparisons of measured and simulated stages have a very good fit. The simulated stage has an equally good fit for high and low stages. The simulated stages at Willow Springs have a tendency to be higher than the measured stages most of the time, but for other locations a relatively good fit was obtained. For Willow Springs, the percentage error of stages relative to the depth of water flow are less than 1% for 75% of stages while for other locations this percentage is more than 90%. More than 99% of stages are within 2% of the depth as shown in Table 4.3. The average absolute differences between measured and simulated stages range from 2% to 6%.

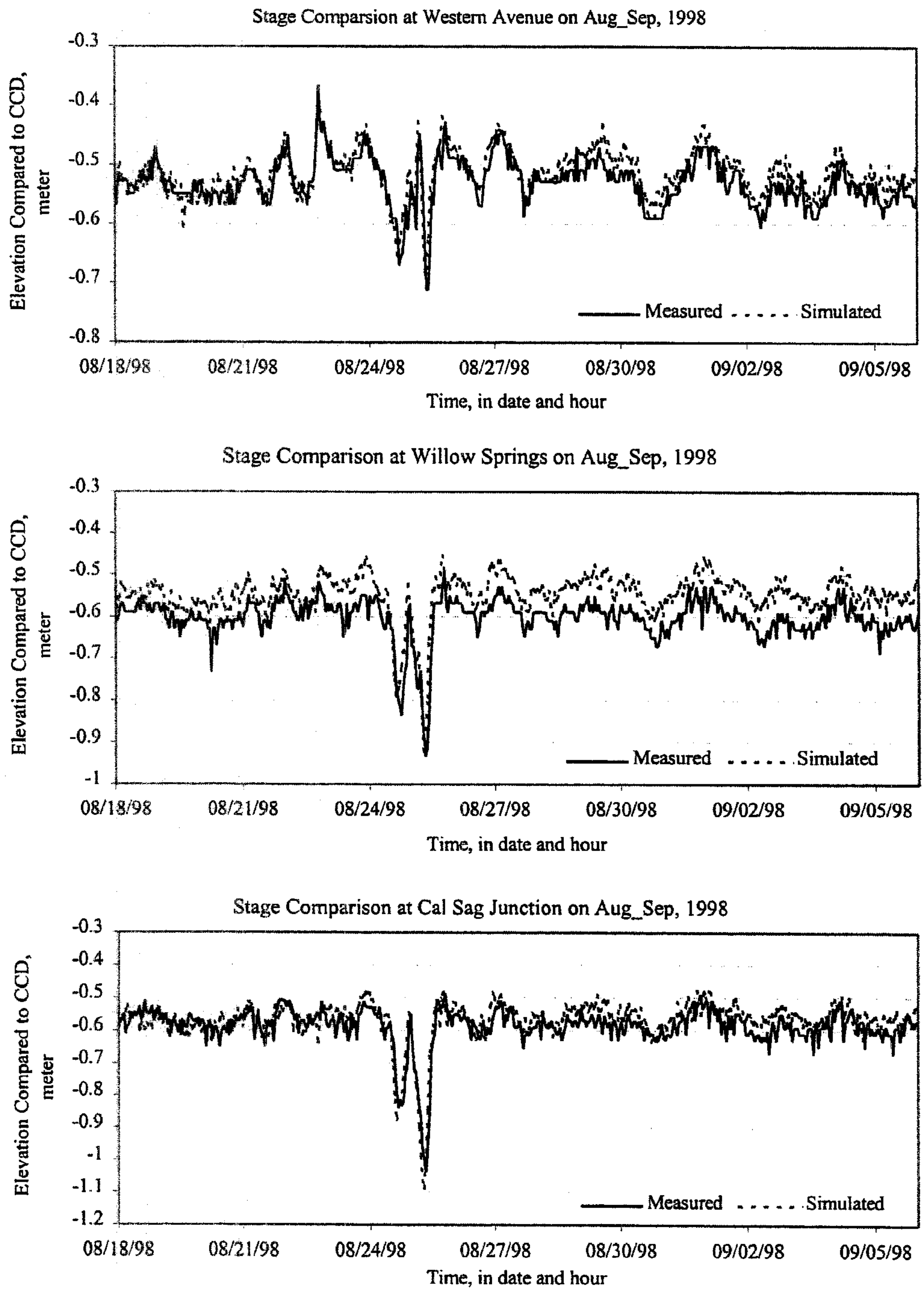


Figure 4.9. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the August – September 1998 period

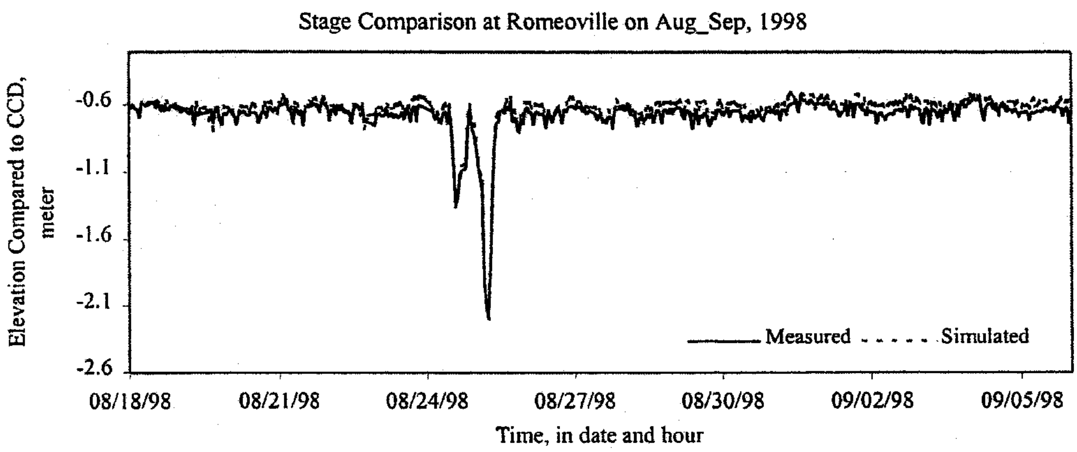


Figure 4.9. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the August – September 1998 period

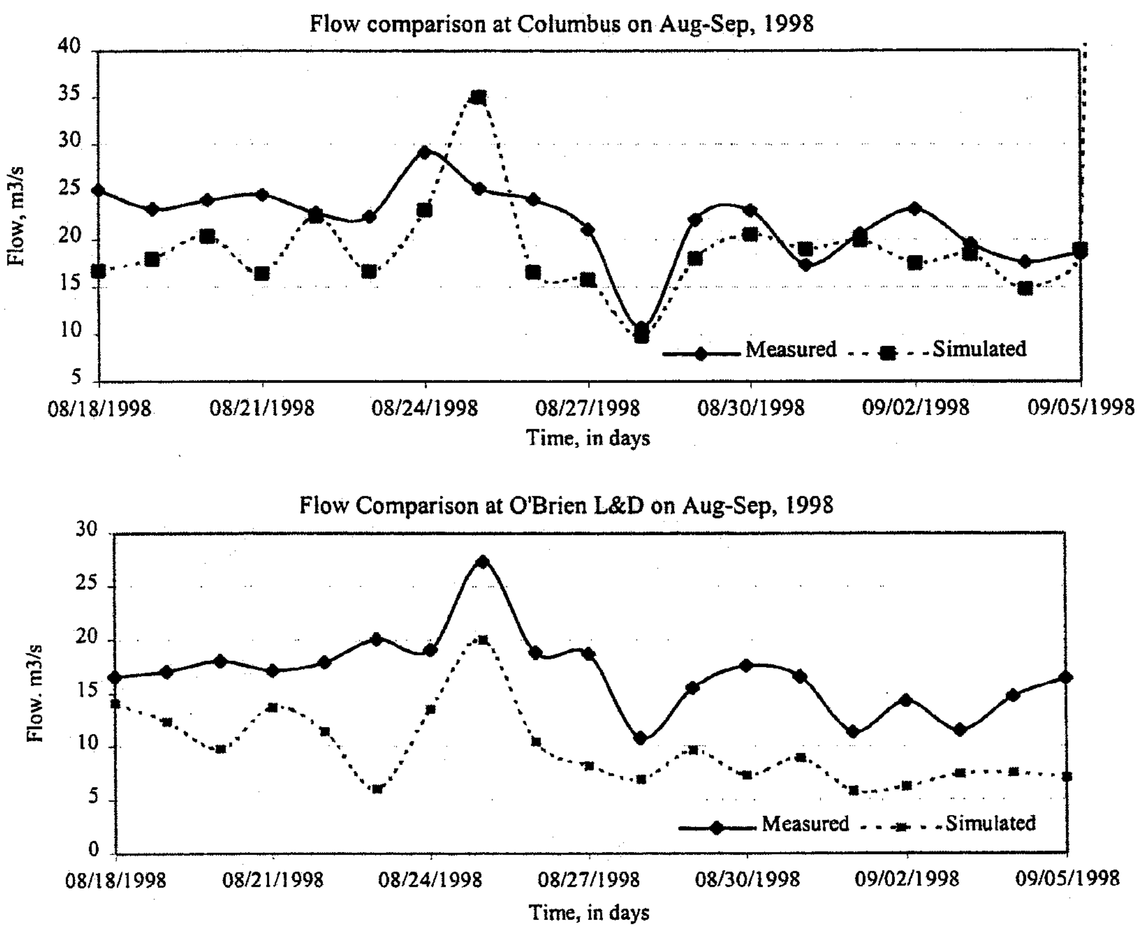


Figure 4.10. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the for August - September 1998 period – cont

The flow comparison at Columbus Drive and O'Brien Lock and Dam is shown in Figure 4.10. According to the flow balance, the total inflow is always higher than the outflows at Romeoville (7.58 m^3/s on average). This results in simulated flows that are always lower than the measured flows at the boundaries except one day at Columbus Drive.

4.5.4 September – December Period (09/11/98 – 12/30/98)

This is one of the longest periods with continuous data considered for the verification of the model. Figure 4.11 shows the comparison of measured and simulated stage at different locations and Figure 4.12 shows the comparison between the measured and simulated flow at Columbus Drive and O'Brien Lock and Dam. The average daily inflow during this period is lowest among all the periods considered in this study. The flows from the Lake Michigan intake controlling works are comparatively very low. During the early part of the period, the flow at the upstream boundaries has a significant effect on the stage of CWS, until the end of October. After then, there is less than 1 m^3/s discharge from each controlling works to the system. About 75% of the flow in the system comes from the water reclamation plants. There is no significant flow from the tributaries except during the storm times.

The comparison between the measured and simulated stages shows a good fit. The correlation coefficient ranges from 69% to 95%. The lowest correlation coefficient is at Cal Sag Junction (0.69) and highest at Romeoville (0.95). The average absolute error for the difference between measured and simulated stages is less than 5% as shown Table 4.4.

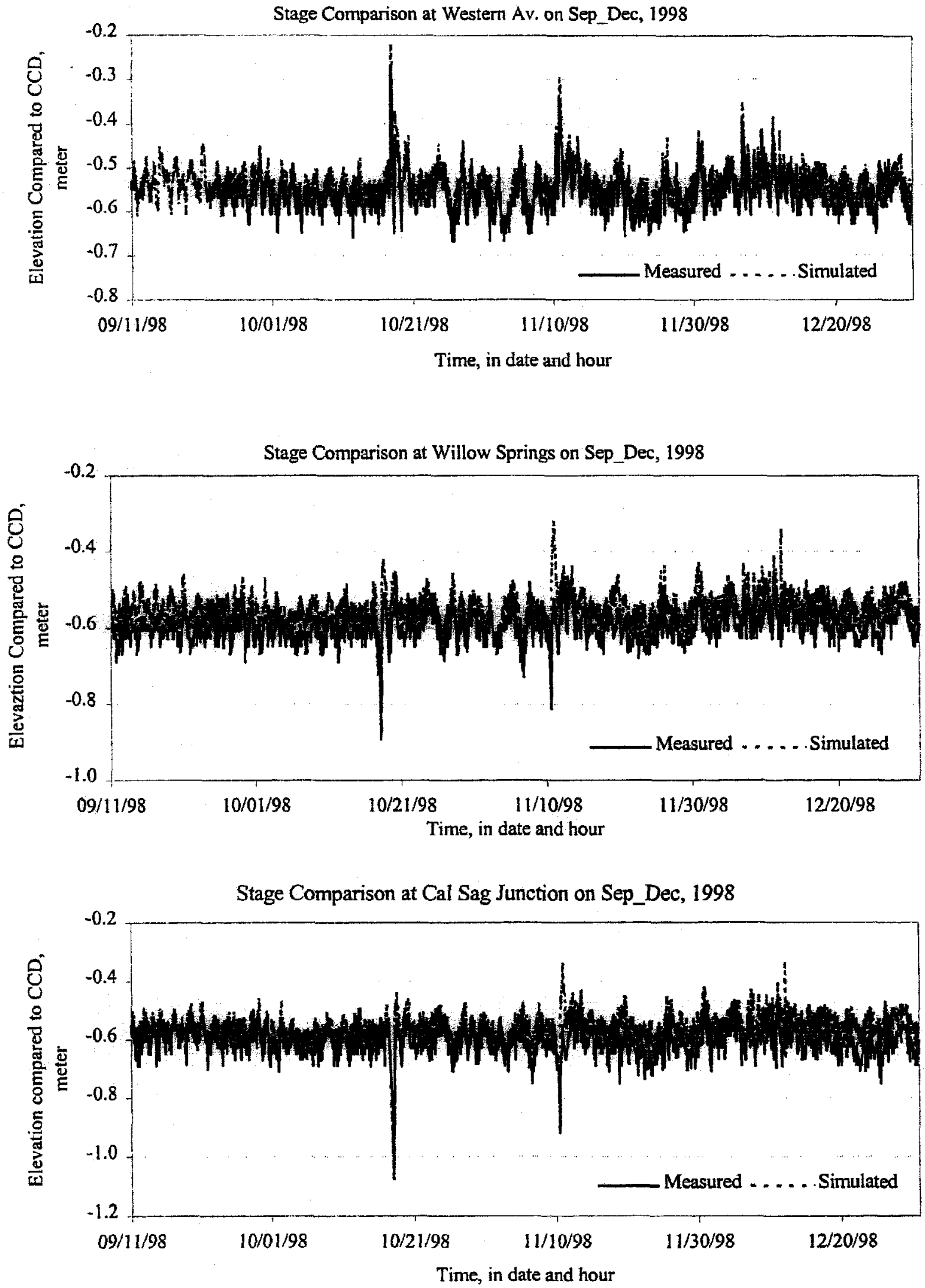


Figure 4.11. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the September - December 1998 period

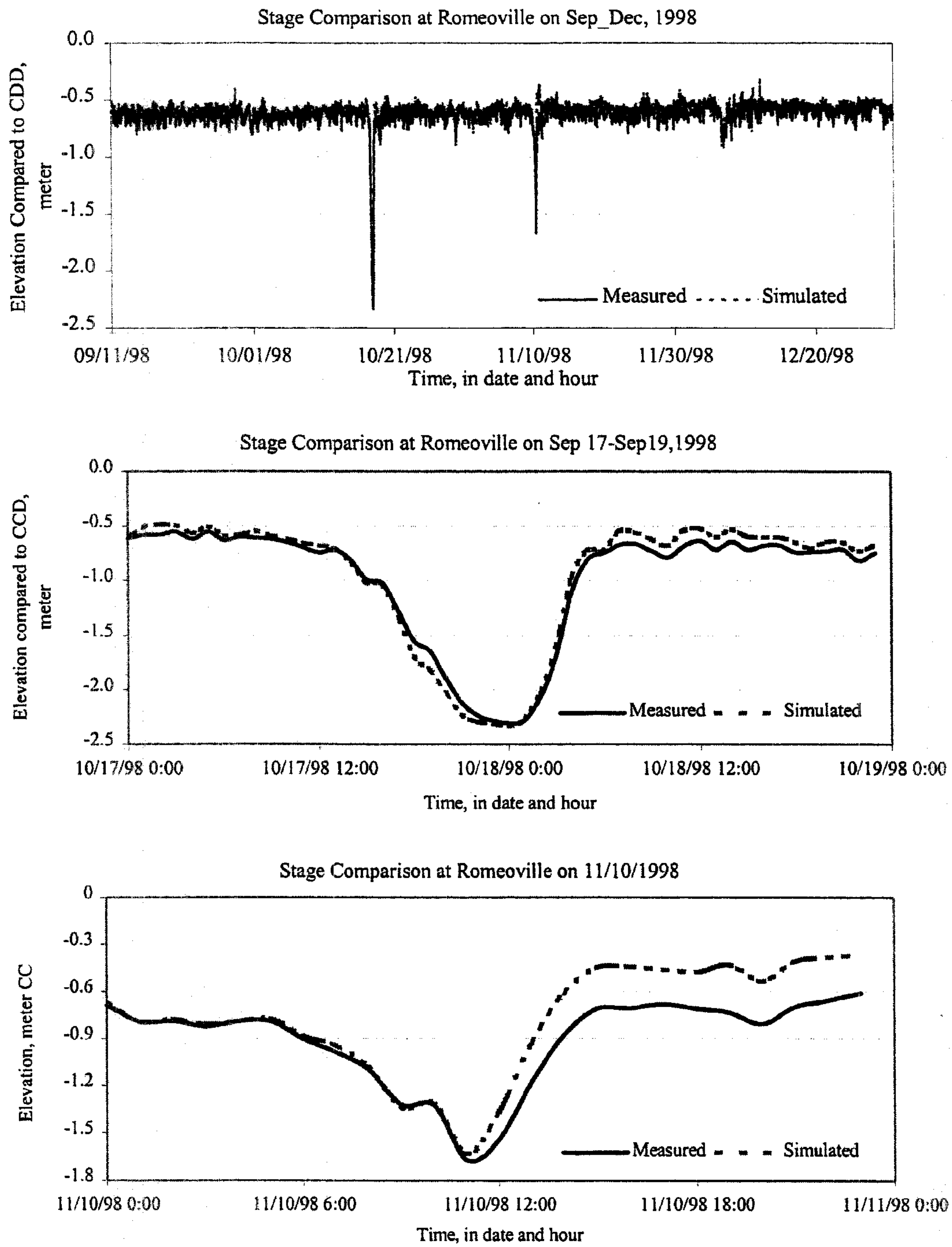
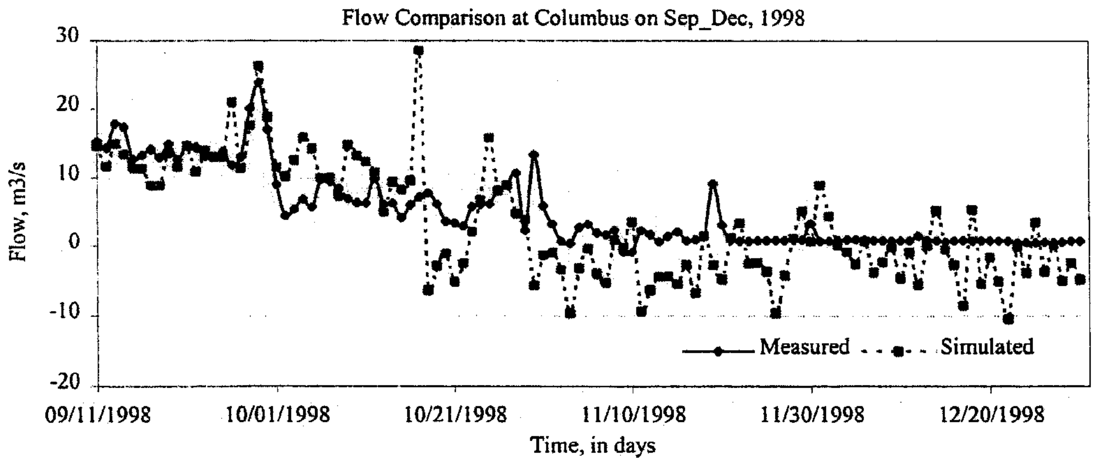


Figure 4.11. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the September - December 1998 period – cont



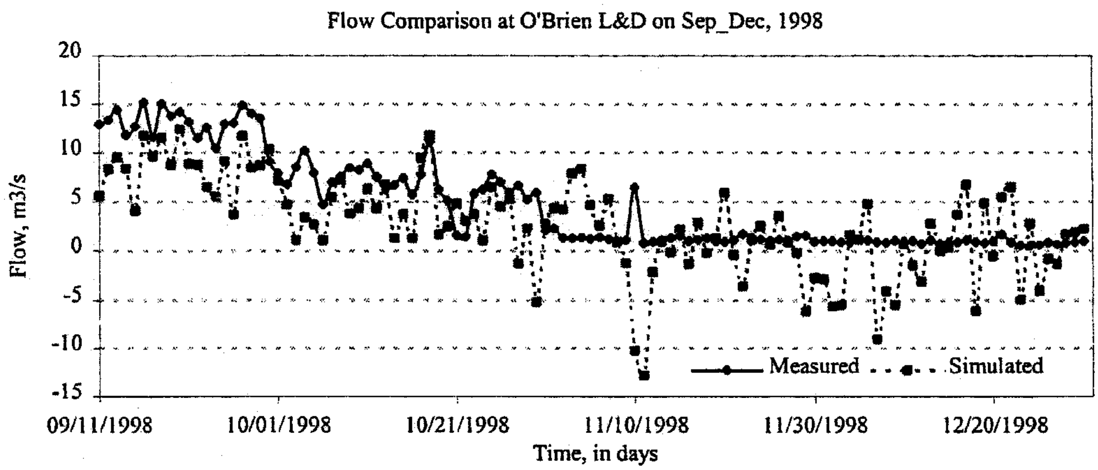


Figure 4.12. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the September - December 1998 period

The comparison of measured and simulated stages during higher flows for shorter time periods at Romeoville also has been shown in Figure 4.12.

As in other periods, the comparison between measured and simulated flows at Columbus Drive and O'Brien Lock and Dam is satisfactory. There are lower simulated flows in most of the times except for a few periods. On 10/17/1998, there is about 30 m^3/s of flow simulated at Columbus Drive into the system. On that day, the measured and estimated inflows are about 49 m^3/s short in the water balance relative to outflow at Romeoville. As discussed earlier there is a tendency for the model to get more water from the CRCW (Columbus Drive) to balance water in the system. This phenomenon could be due to the time lag of flow and inappropriate representation of ungaged tributaries and watersheds and the simplified CSO flow distribution. At present the CSO volume is distributed uniformly in space. Redistribution of CSO volume as a function of rainfall distribution was considered, but substantial rainfall nonuniformity was not detected for most storms, and, thus, redistribution was not done.

4.5.5 February – May Period (02/05/99 – 05/24/99)

This is another long period of continuous data available for model verification purposes. Substantial periods of missing or incorrectly recorded data occurred in the middle of this period except at Romeoville. So, the statistical analysis has been done excluding such data. Figure 4.13 shows the comparison between the measured and simulated stages. The last two figures show the stage comparison during higher flows (lower stages) for short periods. All the stage comparisons show the good agreement at each location between the measured and simulated stages. The average absolute error for the difference between the measured and simulated stages for Willow Springs is about 10% and the correlation coefficient is 75% (Table 4.4). However, more than 85% of stage differences are within 1% and 99% are within 2% with respect to the depth (Table 4.3). The simulated data are very close to measured stages during the sudden changes in stage during the month of April resulting from storms and follow the same tendency as the measured data. So it could be said that the model is well verified in terms of stage comparisons.

The simulated flow at Columbus Drive is relatively better in comparison to measured flow that at O'Brien Lock and Dam (Figure 4.14). The measured flow at Columbus Drive is about 1 m^3/s throughout the study period. During higher measured flows, the model also gave the same tendency but relatively higher flows. The simulated flows at O'Brien Lock and Dam have a tendency to be negative most of the time during the higher measured flows on the Little Calumet River South for reasons previously discussed. Though the average measured flow from O'Brien Lock and Dam during study period is about 2 m^3/s, the model average flow is about –0.4 m^3/s (reflecting the inflow – outflow imbalance of 2.18 m^3/s, Table 4.5).

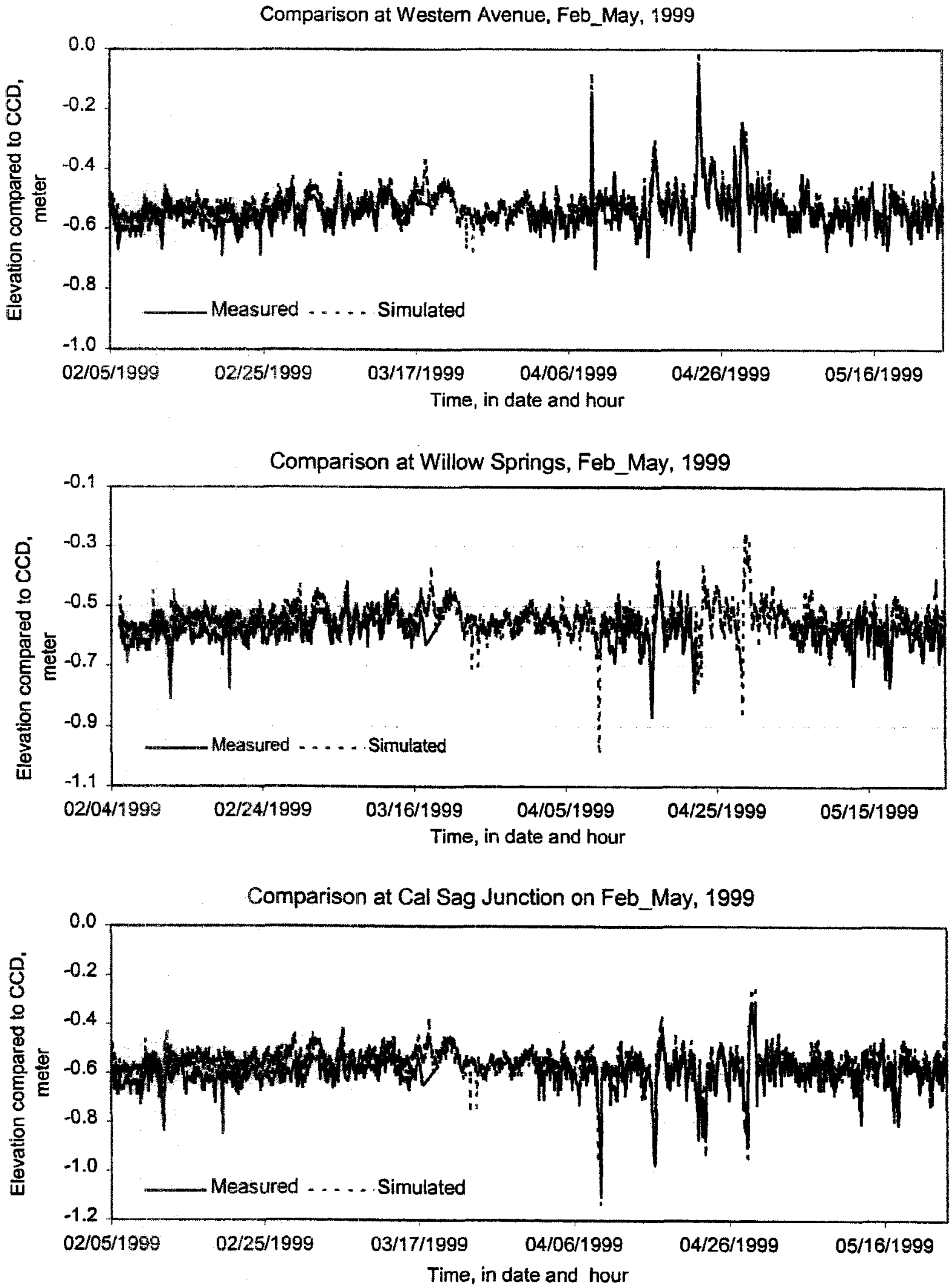
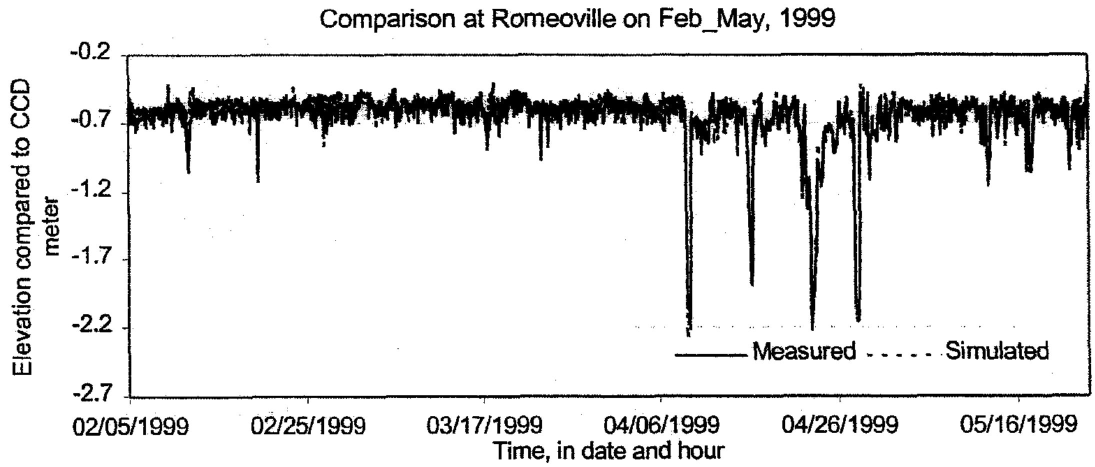
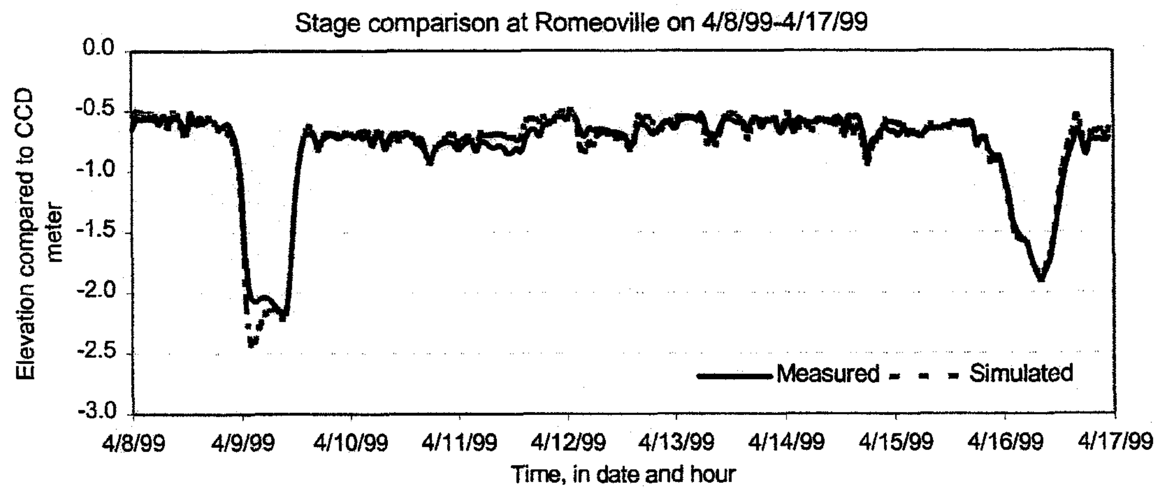


Figure 4.13. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the February – May 1999 period





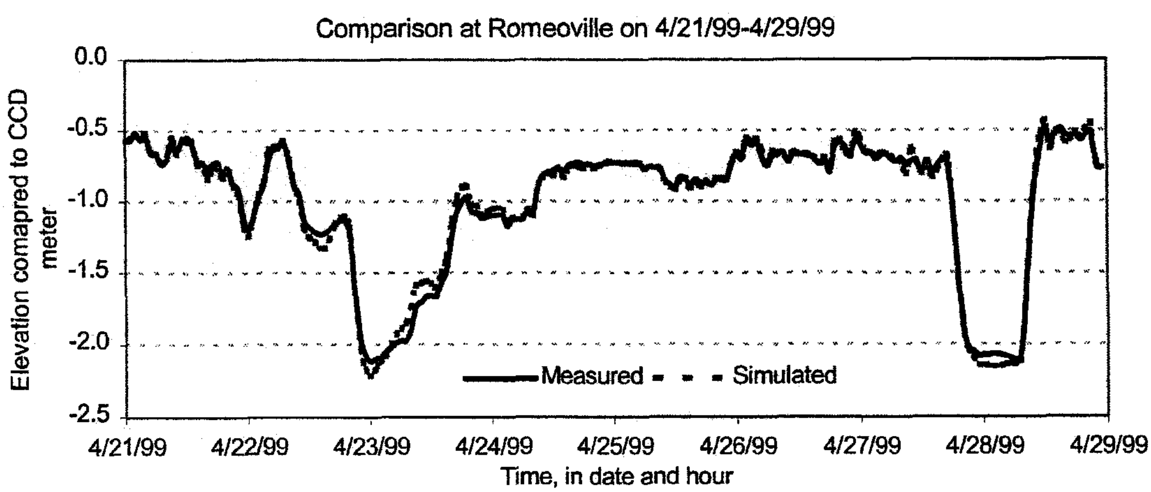


Figure 4.13. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the February - May 1998 period – cont

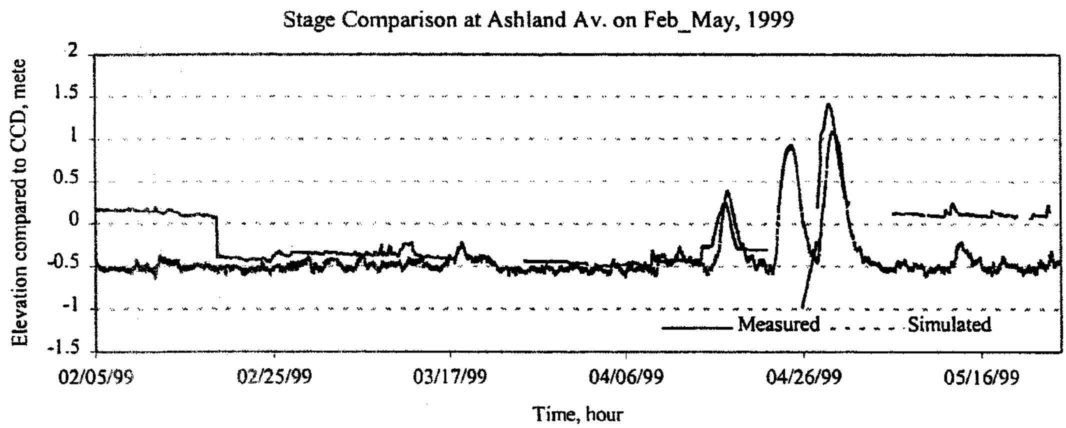
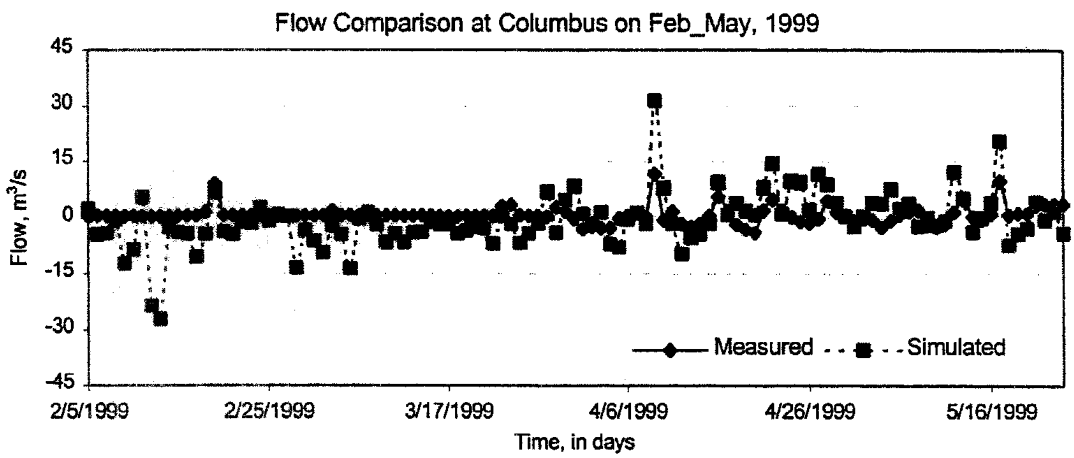


Figure 4.13. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the February - May 1998 period – cont



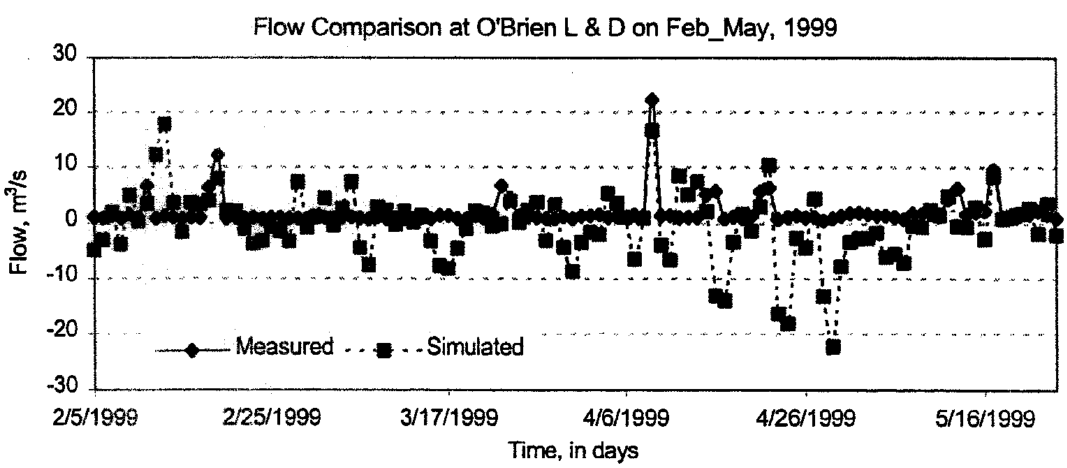


Figure 4.14. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the for February - May 1999 period

4.5.6 May – June Period (05/27/99 – 06/12/99)

Figure 4.15 shows the comparison between the measured and simulated stages at different locations on the CWS. The statistical analysis shows a correlation coefficient of 72% and 78% for Willow Springs and Cal Sag Junction, respectively, and 91% and 99% for Western Avenue and Romeoville, respectively (Table 4.4). Though most of the time, the simulated stages are relatively higher than the measured stages, the simulated stages follow the same pattern as the measured stages during the whole period. Comparatively, Willow Springs shows the higher simulated stages all the time. The average difference between the measured and simulated stages at Willow Springs is about 3.5 cm compared to less than 2.0 cm for other locations. The depth of the water on this location is about 7.5 m. So compared to the total depth of water, the difference in measured and simulated stage is very low. The average absolute depth error is less than 1%.

The flow comparison for this period at Columbus Drive and O'Brien Lock and Dam has a good fit except during the storm of 06/1–3 (Figure 4.16). For this period the inflow-outflow imbalance is only 1.27 m^3/s. The good agreement between simulated and observed daily flows at the boundaries during this period confirms that the inflow-outflow imbalance is the primary cause of poor agreement between measured and simulated flows at the boundaries. The flow at Romeoville is higher before the storm according to the flow balance and the inflows are not equal to the outflow so the model shows water withdrawn from the Chicago River Main Stem at Columbus Drive. The negative flow in simulation at Columbus Drive and O'Brien Lock and Dam could be due to the higher flow from discharge at the Racine Avenue Pumping Station and inflow from the Little Calumet River South, respectively.

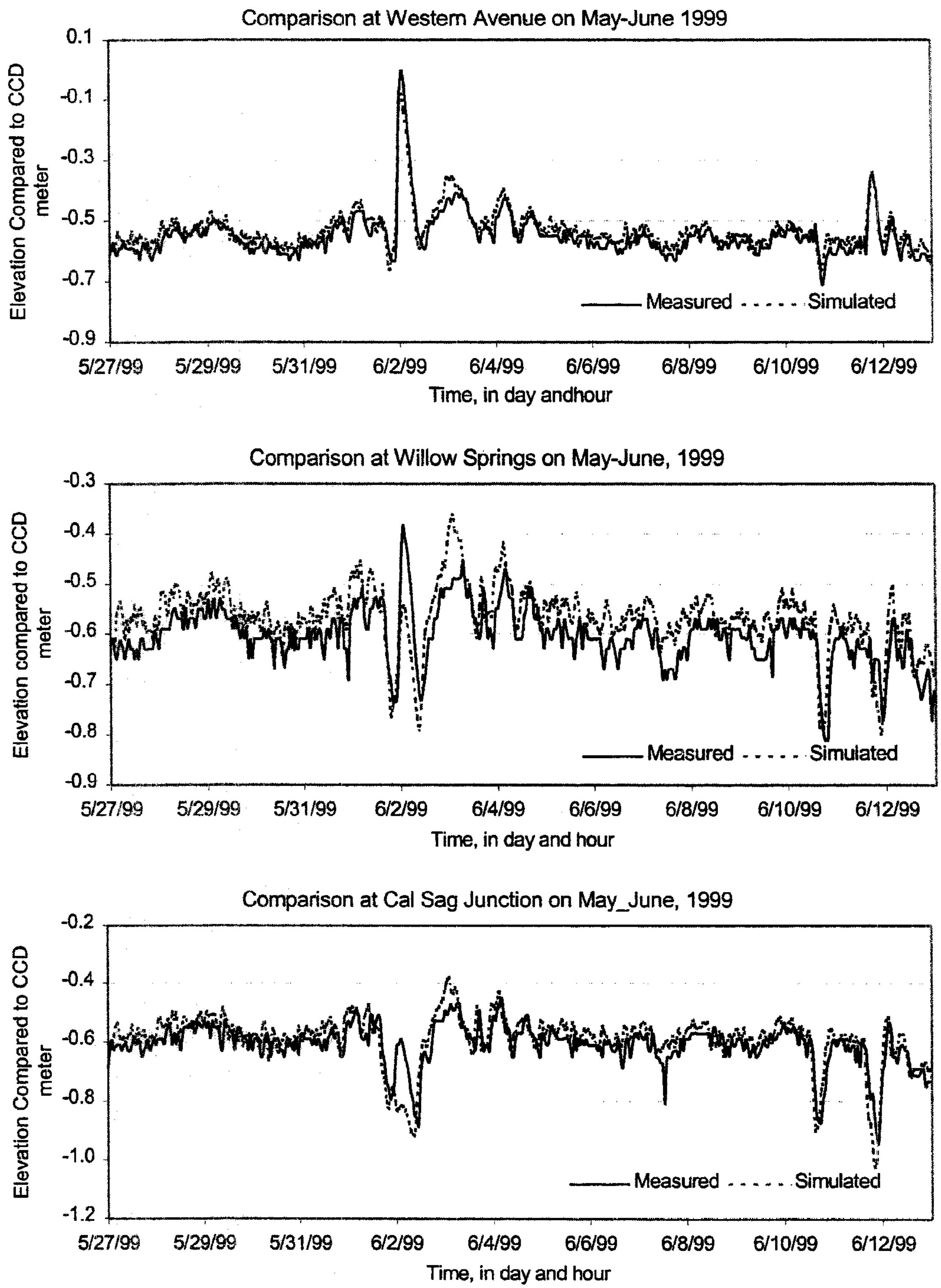


Figure 4.15. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the May - June 1999 period

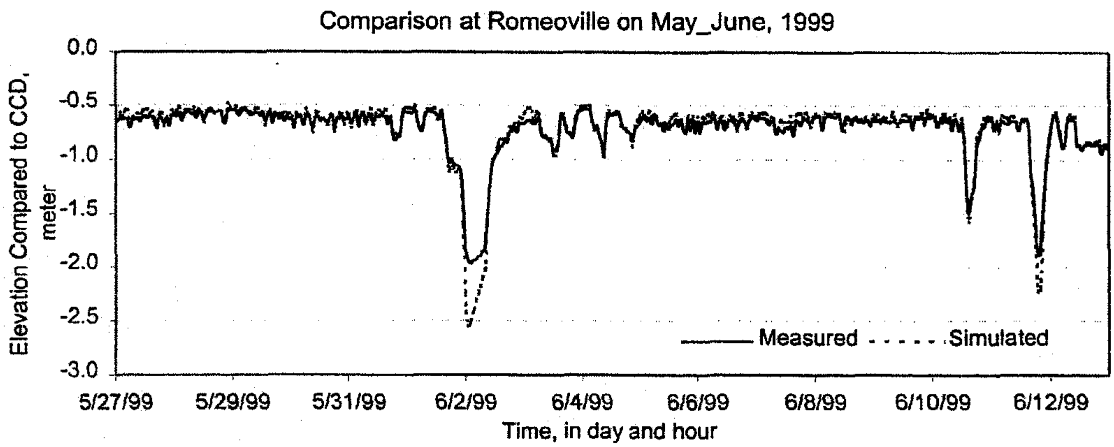
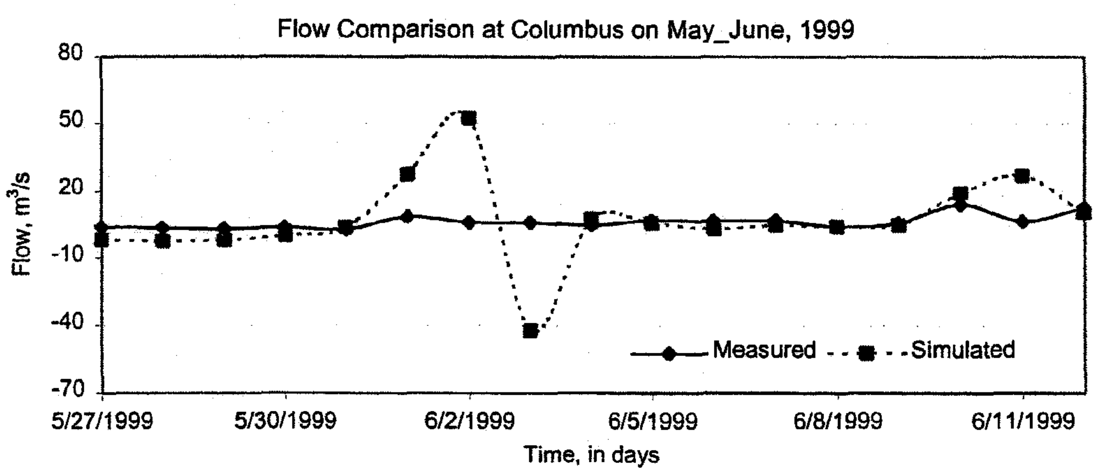


Figure 4.15. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the May - June 1999 period – cont



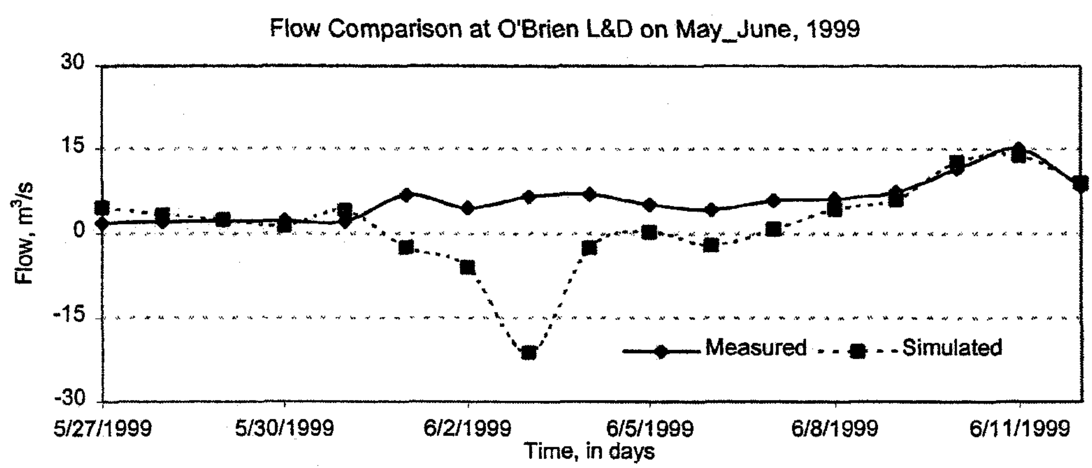


Figure 4.16. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the for May - June 1999 period

4.5.7 June – July Period (06/15/99 – 07/18/99)

As discussed earlier for different periods, this period also has higher simulated stages compared to measured stages most of the time. The comparison is shown in Figure 4.17. The average absolute difference between measured and simulated stages is lowest at Western Avenue (about 2%) and highest at Willow Springs (about 5%). The correlation coefficient at Cal Sag Junction is lowest (0.51) and highest at Romeoville (0.89). Considering the percentage error of stages with respect to the depth of water flow, more than 85% of the stages are in error less than 1% and almost all stages are within 2% (Table 4.3).

The average simulated flow at the upstream boundaries is less than average measured flow in this period (Figure 4.18). In the case of O'Brien Lock and Dam, the simulated flow is always less than the measured flow whereas at Columbus Drive a few simulated flows are higher than the measured flows. When comparing to other simulated periods, there is a tendency for higher flow at Columbus Drive and negative flow at O'Brien Lock and Dam for the same period or vice versa. However, as previously discussed, these boundary flow fluctuations represent very small velocity errors. In this period, there also is a negative flow at both locations during 06/23 to 06/25. It was found that during this period, the measured flow in Midlothian Creek is relatively high. It seems that there was excessive rainfall on the Midlothian Creek watershed during this period

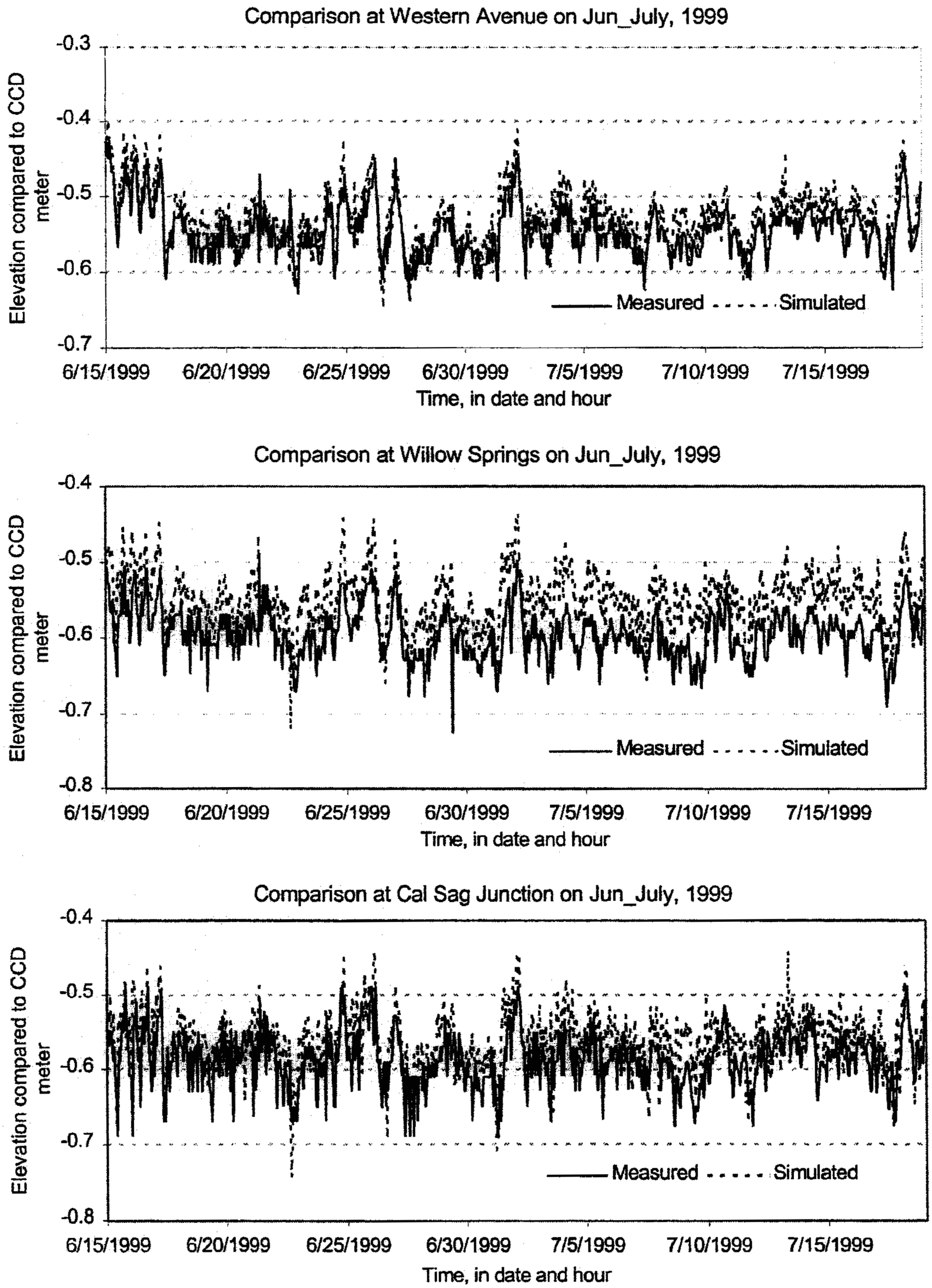


Figure 4.17. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the June - July 1999 period

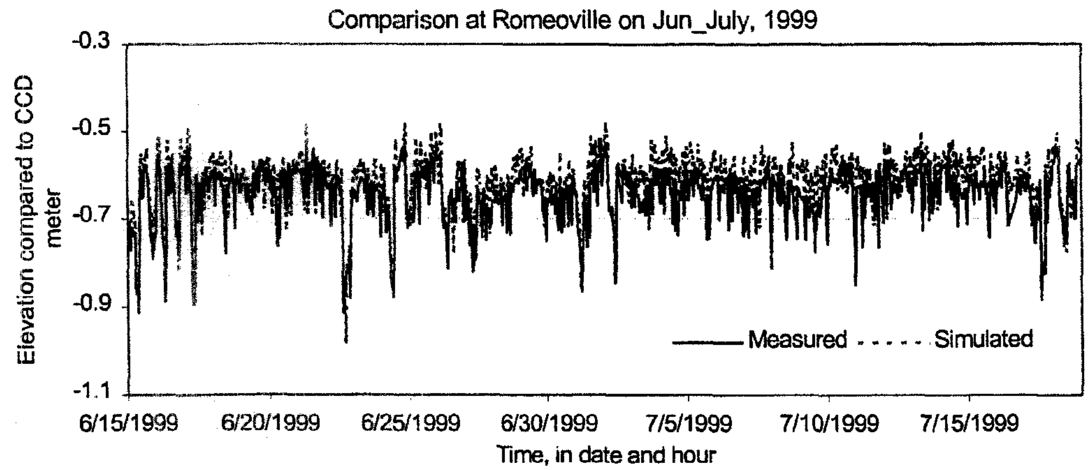
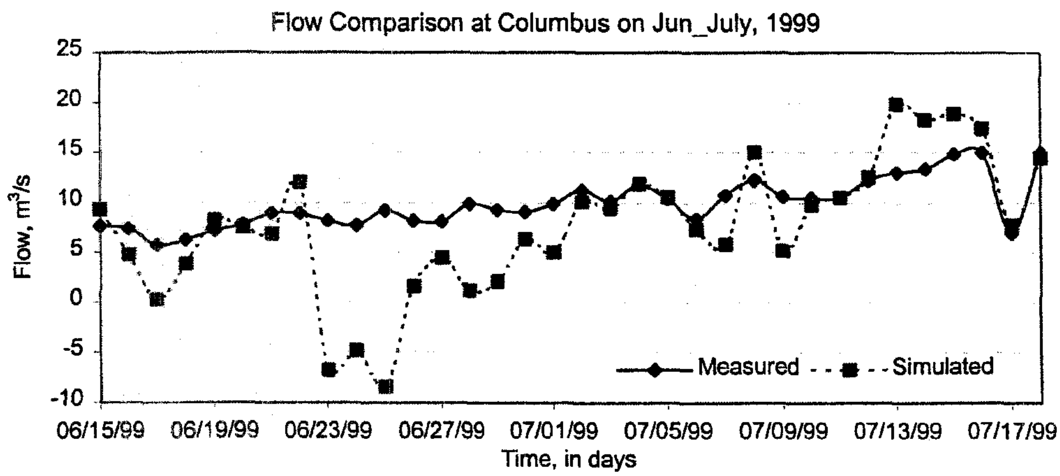


Figure 4.17. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the June - July 1999 period - cont



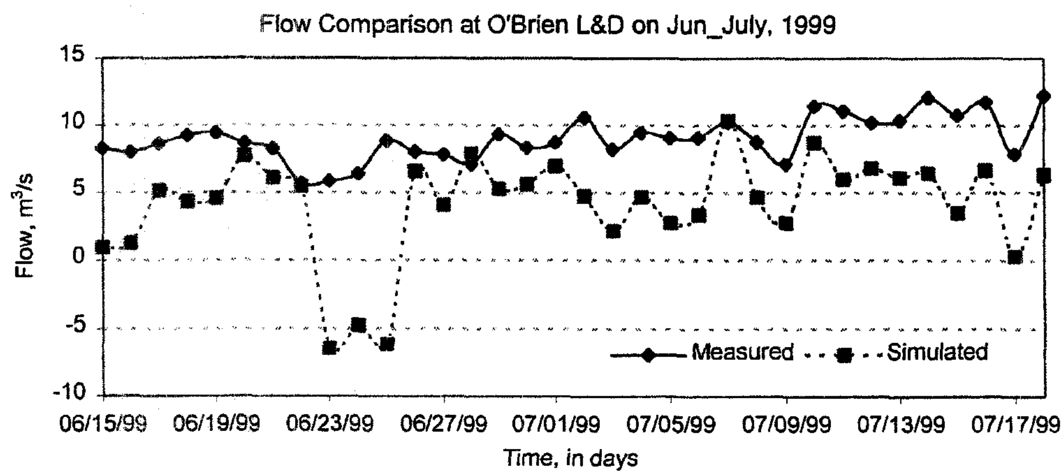


Figure 4.18. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the for June - July 1999 period

relative to the watersheds of the ungaged tributaries draining to the CWS. Since runoff for all of the ungaged tributaries and watersheds are estimated according to the watershed area ratio corresponding to Midlothian Creek, it is believed that there is an overestimate of the runoff during this period, which can be seen in the daily water balance. The total inflows from different sources are higher than outflows during this time.

4.5.8 July Period (07/22/99 – 07/28/99)

Figure 4.19 shows the good fit between the measured and simulated stages. The simulated stages are very close to the measured stages with some time lag on a few occasions. The average absolute error between the measured and simulated stages ranges from 2% to 5% (Table 4.4). During a storm the depth of water changes about 0.5 m. The simulated flows have very good fit during low and high flow in the system.

The simulated flow at both locations is always lower than the measured (Figure 4.20). The reason for this is the higher inflows in the system. That is, the total inflow is about 7% (7.2 m^3/s) higher than the measured outflow (Table 4.5).

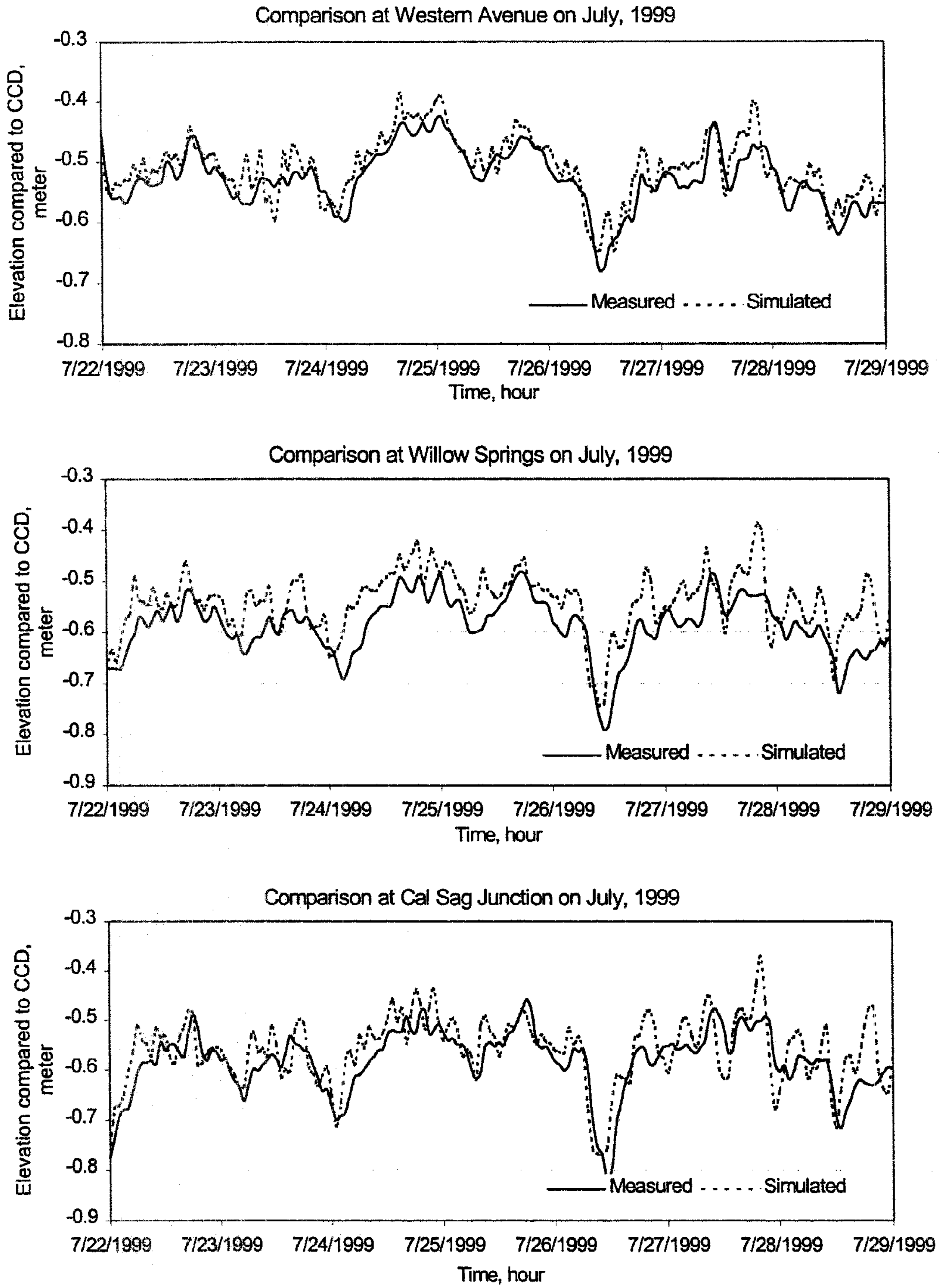


Figure 4.19. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the July 1999 period

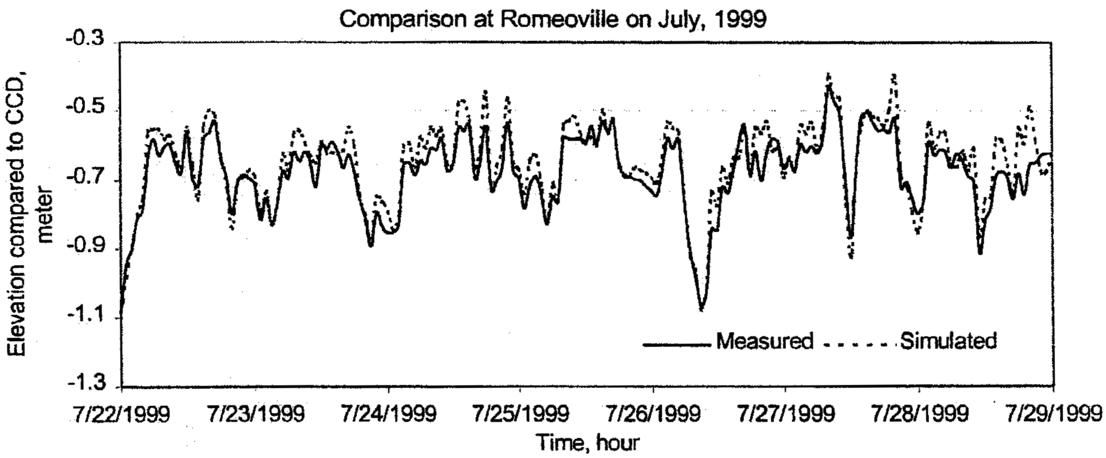


Figure 4.19. Comparison of measured and simulated stage at different locations of the Chicago Waterway System for the July 1999 period – cont

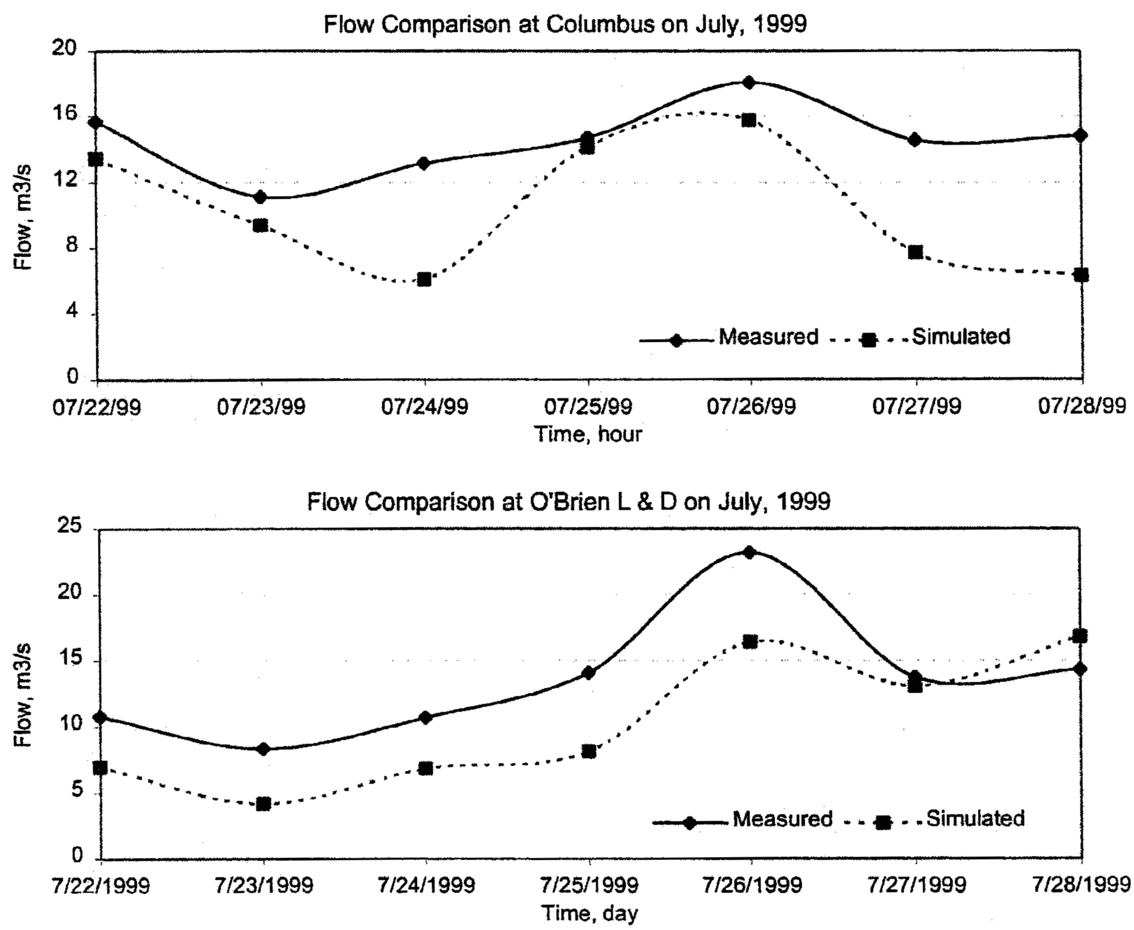


Figure 4.20. Comparison of measured and simulated flow at the upstream boundaries of the Chicago Waterway System for the July 1999 period

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

An unsteady water flow model for the Chicago Waterway System (CWS) has been calibrated to assist water-quality management and planning decision making. An extensive set of flow, stage, and hydraulic geometry data have been used for calibration and verification of an unsteady-flow model for the CWS. The CWS primarily is a constructed system and the primary inflows from the water reclamation plants and other major tributary flows have been well documented. Measured hydraulic cross sections for the waterway also are available, which is helpful for better understanding of the system. Ungaged tributaries, watersheds, and nonpoint sources (Combined Sewer Overflows) to complete the water balance for the CWS have been estimated. These ungaged data have a significant effect on the model calibration.

The model was calibrated using hourly stage data at Western Avenue, Willow Springs, Cal-Sag Junction, and at the downstream boundary at Romeoville, and using daily flow data collected near the CRCW (at Columbus Drive) and O'Brien Lock and Dam upstream boundaries. The model also was calibrated with the stage at Ashland Avenue on the Little Calumet River South. The model was run at a 15-minute time step for several long periods of complete data during the period August 1, 1998 to July 31, 1999. The stage simulation agreed with the measured data nearly always within one percent relative to the depth at each location.

The simulated daily flows do not agree as well with the measured data as do the stages. Most of the time, the simulated flow is less than the measured flow. The reason for this could be overestimates for the ungaged tributaries and watershed flows. However, for five of the eight studied periods the measured inflow exceeded the measured outflow at Romeoville. Thus, there is a general bias for inflows to exceed outflows and this is compensated for at the stage boundaries in the simulation. For the one period when inflow and outflow were nearly equal (May 27 – June 12, 1999) the simulated and measured boundary flows were in good agreement.

When the average measured outflow at Romeoville is higher compared to total inflow according to the flow balance, the flow from the Columbus Drive is always higher. The cause of this should be studied. Higher flow during some times could be due to the time lag of flow. The tendencies of negative flow at O'Brien Lock and Dam during the higher flow on the Little Calumet River South need to be investigated. During higher flow on the Little Calumet River South it is hypothesized that flow backs up into channel storage in the lower Grand Calumet River downstream from the Hohman Avenue gage. The water from the Little Calumet River South flows both east and west because of the higher elevation at Ashland Avenue compared to the Little Calumet River North and Cal-Sag Channel. Since the study did not focus on the stages in the Grand Calumet River and storage in the wetlands, a confirmation of the above hypothesis in needed.

The flow from the ungaged tributaries and watersheds in this study was estimated relative to the drainage area compared to Midlothian Creek flow and drainage area. The result of higher simulated stages at different locations and the higher average inflows could in part be due to the inappropriate representation of these ungaged areas. It would be expensive to establish flows for all such small tributary drainage areas. If the hydraulic results obtained here are not sufficiently accurate, development of an appropriate model for comparing with other similar measured watersheds considering drainage area, rainfall, runoff, and topographic characteristics to represent such ungaged tributaries and watersheds may improve results. For example, inputting results from the corps HSPF model of the watersheds draining to the CWS to DUFLOW may be a useful future activity.

The study to date has focused on the hydraulic unsteady-state model, but the model also is capable of simulating water-quality changes. Although the hydraulic model only gives the flow pattern in the CWS, accurate and reliable simulation of flow in the CWS is necessary to have accurate and reliable simulation of water-quality changes. The effect on stages at different locations during operation of pumping stations and higher runoff during the rainfall now is well understood, which is helpful for future water-quality management understanding and decision making.

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Quality, Ghent, Belgium, September 27-29, 2000, R. Hoeben, Y. Van Herpe, and F.P. De Troch, eds., Laboratory of Hydrology and Water Management, Ghent University

Appendix 1: Location of MWRD Grab WQ Stations

S. No.	Description	River Mile	Latitude	Longitude
1	North Branch Chicago River @ Dempster St.	53	42.0402	87.7865
2	North Branch Chicago River @ Wilson Ave.	41.6	41.9647	87.6953
3	North Branch Chicago River @ Grand Ave.	35	41.8913	87.6407
4	North Branch Chicago River @ Diversey Ave.	39.1	41.9320	87.6823
5	North Branch Chicago River @ Albany Ave.	42.6	41.9746	87.7063
6	North Shore Channel @ Central Ave.	49.4	42.0638	87.6868
7	North Shore Channel @ Oakton Ave.	46.1	42.0115	87.7100
8	North Shore Channel @ Touhy Ave.	45.2	42.0262	87.7097
9	North Shore Channel @ Devon Ave.	44.2	41.9970	87.7102
10	South Branch Chicago River @ Madison St.	34.3	41.8819	87.6356
11	South Branch Chicago River @ Halsted St.	31.8	41.8493	87.6468
12	South Branch Chicago River, South Fork @ Archer Ave.	30.9	41.8389	87.6642
13	CSSC @ Harlem Ave.	22.9	41.8012	87.7847
14	CSSC@ Route # 83	13.1	41.7022	87.9393
15	CSSC @ Stephen St.	9.4	41.6792	88.0114
16	CSSC @ Cicero Ave.	26.2	41.8195	87.7436
17	CSSC@ Lockport Powerhouse Forebay	0	41.5964	87.0686
18	CSSC @ Western Ave.	29.6	41.8380	87.6849
19	Cal Sag Channel @ Route # 83	12.9	41.6968	87.9413
20	Cal Sag Channel @ Ashland Ave.	28	41.6552	87.6607
21	Cal Sag Channel @ Cicero Ave.	24	41.6552	87.7386
22	Little Calumet River (South) @ Wentworth Ave.	41.1	41.5855	87.5299
23	Little Calumet River (South) @ Indiana Ave.	31.4	41.6523	87.5972
24	Little Calumet River (South) @ Ashland Ave.	29.3	41.6517	87.6606
25	Little Calumet River (North) @ Halsted St.	29.1	41.6573	87.6413
26	Little Calumet River (North) @ 130th St.	36	41.6592	87.5725
27	Chicago River @ Outer Drive	35.9	41.8884	87.6144
28	Chicago River Main Stem @ Wells St.	34.8	41.8877	87.6341

Appendix 2: Location of Continuous MWRD_DO Stations

S.No.	Description	River mile	Latitude	Longitude
1	North Branch Chicago River @ Lawrence Ave.	42.1	41.9683	87.7003
2	North Branch Chicago River @ Addison St.	40.4	41.9465	87.6953
3	North Branch Chicago River @ Fullerton Ave.	38.5	41.9253	87.6742
4	North Branch Chicago River @ Division St.	36.3	41.9035	87.6572
5	North Branch Chicago River @ Kinzie St.	34.8	41.8907	87.6388
6	North Shore Channel @ Linden St.	49.8	42.0732	87.6857
7	North Shore Channel @ Simpson St.	48.5	42.0558	87.7067
8	North Shore Channel @ Main St.	46.5	42.0335	87.7095
9	North Shore Channel @ Devon Ave.	44.2	41.997	87.7102
10	South Branch Chicago River @ Jackson Blvd.	33.9	41.8781	87.6378
11	South Branch Chicago River @ Loomis St.	30.8	41.8458	87.6611
12	CSSC @ Cicero Ave.	26.2	41.8195	87.7436
13	CSSC @ B&O R.R. Bridge	21.3	41.7832	87.8257
14	CSSC @ Route #83	13	41.707	87.9292
15	CSSC @ Lockport Powerhouse	0	41.5713	88.0785
16	Cal Sag Channel @ Route #83	12.9	41.6968	87.9413
17	Chicago River Main Stem @ Chicago River Lock	36.1	41.8929	87.6124
18	Chicago River Main Stem @ Michigan Ave.	35.4	41.8889	87.6244
19	Chicago River Main Stem @ Clark St.	34.9	41.8874	87.6316

Appendix 3: Location of New MWRD DO Stations

S.No.	Description	River Mile	Latitude	Longitude
1	Little Calumet River (North) @ Halsted St.	29.1	41.6572	87.6408
2	Little Calumet River (North) @ C&WI Indiana Harbor Belt RR	31.7	41.6504	87.6116
3	Little Calumet River (North) @ Conrail RR	34.3	41.6391	87.5659
4	Little Calumet River (South) @ Ashland Ave.	29.3	41.6518	87.6604
5	Grand Calumet River @ Torrance Ave.	34.9	41.6442	87.5590
6	Calumet River @ 130th Street	36	41.6603	87.5699
7	Cal Sag Channel @ Southwest Highway	19.7	41.6802	87.8107
8	Cal Sag Channel between Harlem & Ridgeland	20.7	41.6771	87.7922
9	Cal Sag Channel @ Cicero Ave.	24	41.6558	87.7386
10	Cal Sag Channel @ Kedzie Ave.	26.1	41.6520	87.6987
11	Cal Sag Channel @ Division Street	27.6	41.6527	87.6708
12	CSSC @ Romeoville Road	5.2	41.6406	88.0606