Simulation of Scenarios to Meet Dissolved Oxygen Standards in the Chicago Waterway System

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SIMULATION OF SCENARIOS TO MEET DISSOLVED OXYGEN
STANDARDS IN
THE CHICAGO WATERWAY SYSTEM

by

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ABSTRACT

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Although most reaches of the Chicago Waterway System (CWS) meet the General Use Water Quality Standards a high percentage of the time, dissolved oxygen (DO) standards are not met in the CWS during some periods for both the WYs 2001 and 2003 as representative of wet and dry years. Several methods were used to solve this problem; however, they were inadequate for achieving the proposed DO standards. Therefore, a method of integrating the alternative DO remediation methods into one integrated strategy for improving water quality is considered in this study.

The main purpose of this study is the application of the DUFLOW model to improve DO concentrations in the CWS during the WYs 2001 and 2003. Two sets of DO standards needed to be achieved: 90 and 100% compliance with the IEPA's proposed DO standards, and the MWRDGC's proposed DO standards. In order to meet both standards, the following DO remediation methods were considered: 1) flow augmentation practices on the NSC, Bubbly Creek, and the Little Calumet River (north); 2) Side-stream Elevated Pool Aeration (SEPA) stations operational adjustments; and 3) the supplemental aeration stations on the CWS.

The results show that flow augmentation on the NSC and on Bubbly Creek can be combined to achieve 90% compliance with the IEPA's proposed DO standards for both years. However, the combination of flow augmentation, operational changes for the existing SEPA stations, and new aeration stations were required to meet 100% compliance with the IEPA's standards. For WYs 2001 and 2003, additional new aeration stations with the maximum DO loads of 80 or 100 g/s were needed along the CWS. For the MWRDGC's standards, a method of combing a 24 MGD transfer of aerated flow on the NSC with adjustment of the operating hours of the Devon Avenue in-stream aeration station and 2 new aeration stations on the SBCR can be an effective management, whereas only 24 MGD of aerated flow augmentation plus 1 new aeration station on the SBCR can meet the MWRDGC standards for WY 2003. A maximum oxygen load of 80 g/s is applied for three new aeration stations.

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CHAPTER 1: INTRODUCTION

1.1. Introduction

The Chicago Waterway System (CWS) starts from Lake Michigan at the Wilmette Pumping Station on the north and follows a path consisting of the North Shore Channel (NSC), lower portion of the North Branch Chicago River (NBCR), South Branch Chicago River (SBCR), and the Chicago Ship and Sanitary Channel (CSSC). The Chicago River Main Stem flows into the SBCR, and the Calumet-Sag Channel and Little Calumet River flows into the CSSC composing the CWS. Totally, the CWS is a 76.3 mile branching network of navigable waterways controlled by hydraulic structures. It flows through downtown Chicago and it has played a quite important role in the history of Chicago. The Calumet and Chicago River Systems are shown in Figure 1.1.

Originally, the Chicago River flowed into Lake Michigan taking municipal pollution to the lake. However, with the growth of Chicago, the city required removal of municipal sewage and other contamination from Lake Michigan. At the end of the 19th century, in order to clean Lake Michigan, the river's flow direction was reversed away from Lake Michigan, toward the Mississippi River by developing the CSSC. Thus, a 28-mile man-made canal was built to link the SBCR to Lockport and it was completed in 1900. Subsequently, two more man-made canals: North Shore Channel (1910) and Calumet-Sag Channel (1922) were built to complete the CWS. Then, commercial and recreational activities and urban drainage, i.e. discharge of

stormwater runoff, sanitary wastewater, and combined sewer overflows (CSOs) after rainstorms, are the major uses of the CWS.

The Illinois Environmental Protection Agency (IEPA) launched several studies on the water quality in the CWS in the past and the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) has been responsible for protecting water quality along the CWS. In 1992, Camp, Dresser & McKee (CDM, 1992) used the QUAL2EU model (Brown and Barnwell, 1987) to simulate dissolved oxygen (DO) on the CWS and Upper Illinois River for the MWRDGC, because of the model's ability to accurately simulate the complex waterway and because the model is widely accepted(CDM, 1992 p. 2-2). Based on the long-term vision and development shared by many of the stakeholder agencies, CDM (2007) completed an evaluation of water quality problems and potential use designations as part of a Use Attainability Analysis (UAA) program for the IEPA in order to achieve the highest attainable uses consistent with Clean Water Act goals.
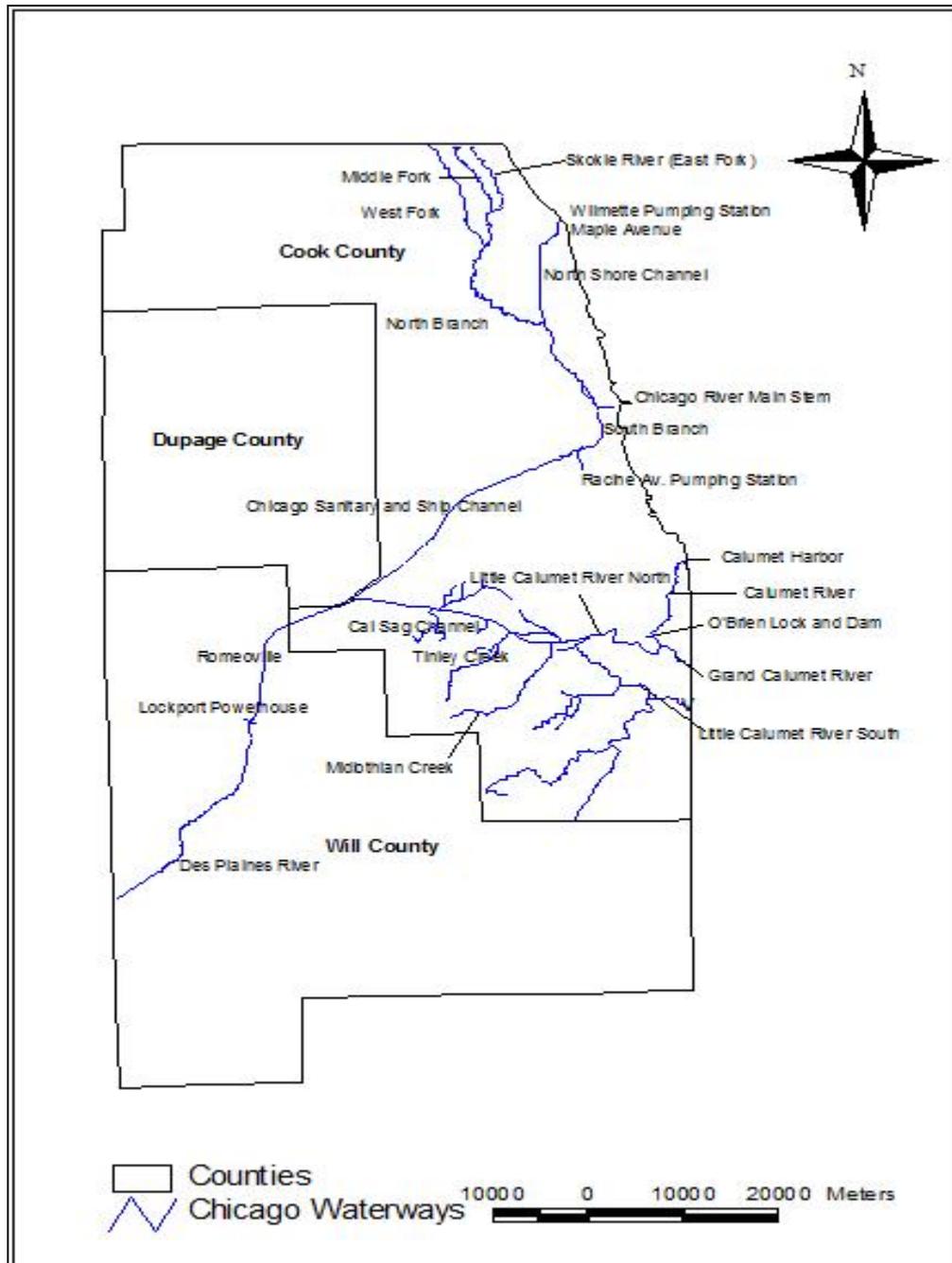
Figure 1. 1 The Calumet and the Chicago River Systems

In this thesis, because the flow and water-quality processes in the CWS are very complex and water-quality conditions vary under a wide range of flows, the DUFLOW water quality model developed in the Netherlands (DUFLOW, 2000) was applied to hydraulic and water quality simulation of the CWS for several reasons as

follows: 1) The QUAL2E model has several limitations that make it inadequate to simulate water quality in the CWS. The primary limitation is that QUAL2E is only applicable for steady, low flows, which is commonly of interest in the development of traditional waste-load allocations wherein summer low flows commonly result in the critical water-quality conditions (Shrestha, 2003). However, the previous research done by the MWRDGC have shown that the worst DO conditions result during storms, thus, simulation of unsteady flow conditions was needed. 2) The DUFLOW model has been applied to several projects on the CWS: i) Alp and Melching (2004) used the DUFLOW model to investigate the possible effects of a change in navigational water level requirements and the navigation make-up diversion of water from Lake Michigan during storm events; ii) Neugebauer and Melching (2005) developed a method to verify the calibrated DUFLOW model under uncertain storm loads; iii) Manache and Melching (2005) applied the DUFLOW model to simulate fecal coliform concentrations in the CWS under unsteady flow conditions; and iv) Alp and Melching (2006) evaluated the effectiveness of flow augmentation, supplemental aeration, and CSO treatment acting individually to improve DO conditions in the CWS. This thesis extends the work of Alp and Melching (2006) applying the DUFLOW model to simulate scenarios combining flow augmentation and supplemental aeration to meet different proposed DO standards for the CWS.

The periods of October 1, 2000 to September 30, 2001 (wet year) and October 1, 2002 to September 30, 2003(dry year) were selected to develop suitable combinations of flow augmentation and supplemental aeration (see section 1.2 for details).

Hydraulic verification and water-quality calibration of the DUFLOW (2000)

model were done by Marquette University (Alp, 2009) under unsteady flow

conditions for the periods of October 1, 2000 to September 30, 2001 and October 1,

2002 to September 30, 2003.

1.2. Selection of Representative Wet and Dry Years

Consideration of "wet" and "dry" weather years is important for the

development of integrated strategies that are sufficient to improve deficient DO

concentrations in the CWS. Normally, representative "wet" and "dry" years should be

decided based on their flows. However, representative flow data for the CSO drainage

areas to the CWS are not available. Thus, precipitation data and CSO pump station

operation data were used to select the representative "wet" and "dry" years (Melching

et al., in preparation). In order to show a long-term perspective, precipitation data

from the National Weather Service for O'Hare Airport and Midway Airport with a

wide range of years were considered (approximately from the 1951 to 2007 Water

Years, Figure 1.2). Meanwhile, to give an area wide perspective the average

measured precipitation data at the 25 precipitation gages spread over the CSO

drainage area in Cook County established by the U.S. Army Corps of Engineers and

operated by Illinois State Water Survey (ISWS) for use in the Lake Michigan

Diversion Accounting (since 1990) also were considered (Figure 1.2). Because at the

start of this project 1) hourly water reclamation plant flow data were available merely

from Water Years 1997 to 2007, and 2) the continuous temperature and DO monitors

along the CWS began collecting data beginning from August 1998, Water Years for possible study range from 1999 to 2007.

The selection of a "wet" year was found to be much more difficult than a "dry" year in the data analysis among the candidate years. The two selection criteria, the annual CSO volume at the pumping stations, the quartile ranking of annual precipitation and were compared and evaluated. Table 1.1 lists the total annual precipitation at O'Hare Airport, Midway Airport, and for the ISWS network average, and the ranking from the highest precipitation over the period of record for each data series for the Water Years between 1997 and 2007. The long term average annual precipitation was 34.57, 35.55, and 35.94 in. at O'Hare Airport, Midway Airport, and for the 25 gage ISWS network, respectively. Five of the eleven years had above average precipitation at O'Hare Airport, three of the eleven years had above average precipitation at Midway Airport, and four of the eleven years had above average precipitation for the 25 gage ISWS network.

On the basis of precipitation, Water Year 2007 would appear to be an excellent representative "wet" year as it ranks in the top 15% at O'Hare Airport (over 45 years) and the second among 18 years for the ISWS Network, but only in the top 40% at Midway Airport (over 57 years). The goal of representative is to be in the top (or bottom) quartile of years, but not being the wettest or driest year. However, if the volume of CSO flow at the pumping stations is considered, Water Year 2007 ranks only 9th among the 16 years beginning in Water Year 1992 (Figure 1.3) spread over 35 pumping incidents (where an incident is defined as a pumping station operating on individual or consecutive days, if there is more than one day between pump

operations a new incident is recorded). Because the "wet" year should be defined on

the basis of high flows having a substantial impact on the water quality in the CWS,

Water Year 2007 would not be a representative "wet" year.
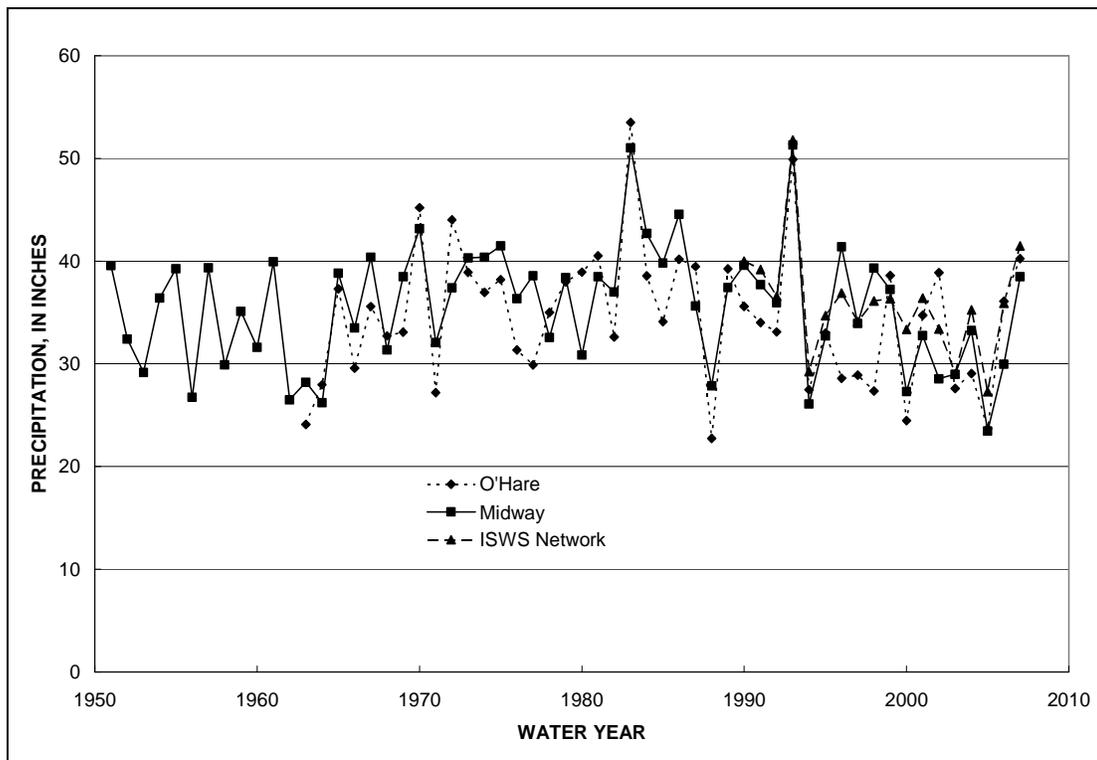
Figure 1. 2 Annual Precipitation by Water Year at O'Hare Airport, Midway Airport, and for the Illinois State Water Survey (ISWS) 25 gage network in Cook County, IL.

On the basis of pump station CSO flow volume, Water Year 1999 has the

largest volume, spread over 33 incidents, among the candidate years for this study

ranking 4th among the 16 years beginning in 1992. In terms of rainfall, Water Year

1999 was 4.03, 1.68, and 0.39 in. higher than average at O'Hare Airport, Midway

Airport, and for the ISWS network. In terms of percentile rankings, Water Year 1999

was in the upper 30%, 50%, and 45% at O'Hare Airport, Midway Airport, and for the

ISWS network. Thus, the goal to be in the upper quartile in terms of precipitation

would not be achieved if Water Year 1999 were selected. Water Year 1999 would also pose a substantial practical problem for the water-quality modeling because during that year no dissolved oxygen and temperature monitors were in the Little Calumet River (north) – Calumet-Sag Channel (Calumet system) reaches of the CWS. Thus, it would be difficult to have accurate temperature values for use in these reaches.

On the basis volume of pump station CSO flow volume, Water Year 2001 had the second largest volume (only 3% less than Water Year 1999 and 40% higher than Water Year 2007), spread over 32 incidents. Water Year 2001 ranked 5th among the 16 years beginning in 1992. In terms of rainfall, Water Year 2001 was 0.14 and 0.45 in. higher than average at O'Hare Airport and for the ISWS network, but was 2.81 in. below average at Midway Airport. In terms of percentile rankings, Water Year 2001 was the median at O'Hare Airport, in the lower 35% at Midway Airport, and the upper 40% for the ISWS network. Thus, the goal to be in the upper quartile in terms of precipitation would not be achieved if Water Year 2001 were selected. However, given the higher CSO volume at the pumping stations in Water Year 2001, the lack of high precipitation in the other candidate years, and the lack of temperature data for the Calumet system for Water Year 1999, Water Year 2001 was selected as the representative "wet" year for the development of an integrated strategy for dissolved oxygen improvement in the CWS.

Table 1. 1 Annual precipitation depth and rank from the highest among the recorded years for O'Hare Airport, Midway Airport, and the Illinois State Water Survey (ISWS) 25 gage network in Cook County, IL.

Water Year	O'Hare Airport		Midway Airport		ISWS Network	
	Depth	Rank among 45	Depth	Rank among 57	Depth	Rank among 18
2007	40.23	6	38.47	22	41.47	2
2001	34.71	23	32.74	37	36.39	7
1999	38.60	13	37.23	27	36.33	8
1998	27.35	40	39.30	16	36.12	9
2006	36.07	19	29.96	45	35.89	10
2004	29.05	34	33.23	36	35.24	11
1997	28.89	35	33.90	34	34.09	13
2002	38.86	12	28.53	49	33.37	14
2000	24.47	42	27.28	52	33.33	15
2003	27.58	38	28.97	48	29.03	17
2005	23.68	44	23.45	57	27.29	18
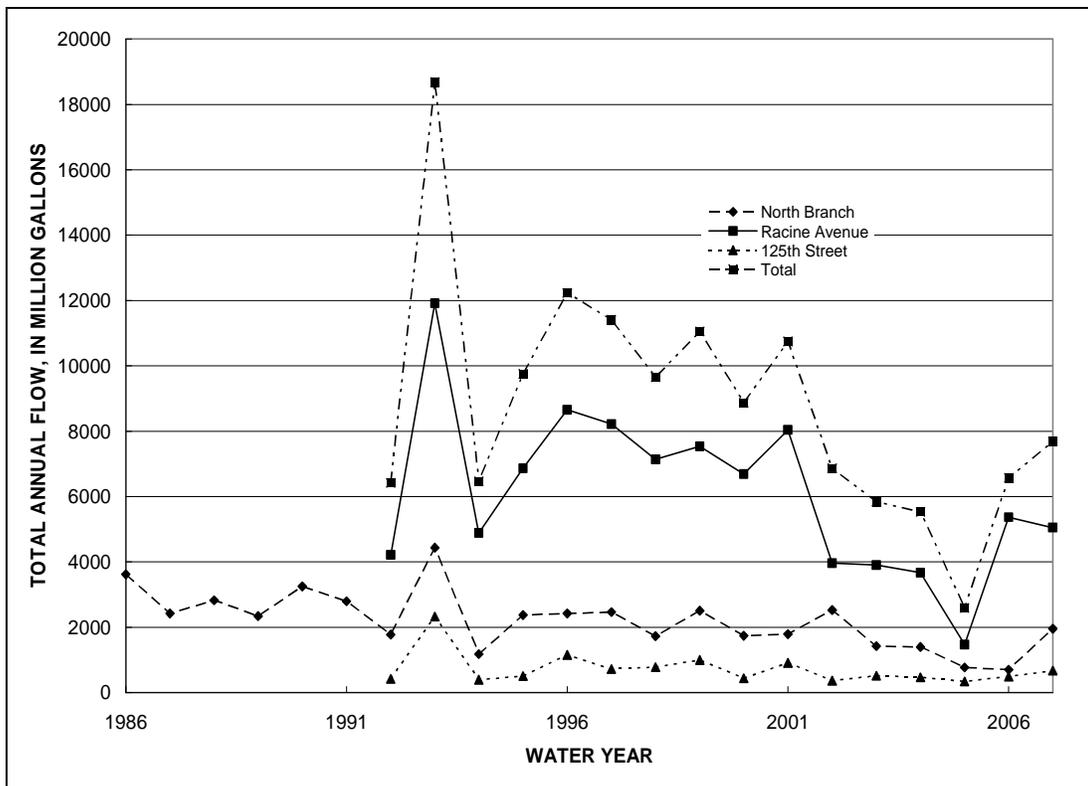
Figure 1. 3 Volume of annual combined sewer overflow at the North Branch, Racine Avenue, and 125th Street Pumping Stations

The selection of the representative "dry" year was much easier. Water Year 2005 probably is the driest year in the last 50 years as it ranks last in annual rainfall at Midway (over 57 years), second to last at O'Hare Airport over 45 years, and last for the ISWS network over 18 years. Further, it yielded the smallest volume, over 16 incidents, of CSO flow at the pumping station among the 16 years beginning from Water Year 1992. However, the representative "dry" year should not be the driest year. Water Year 2004 has the second smallest CSO volume at the pumping stations, but its rainfall is around the 40th percentile from the bottom at Midway Airport and for the ISWS network.

Water Year 2003 has a 6 % larger CSO volume at the pumping stations than Water Year 2004. Water Year 2003 ranks third smallest in CSO volume at the

pumping stations among 16 years (lower 20%) and it ranks in the lower 16% of years

in terms of precipitation at O'Hare Airport and Midway Airport, and the lower 6% for

the ISWS network (i.e. second smallest). Water Year 2003 only had 23 CSO

pumping station incidents whereas Water Year 2004 had 27 CSO pumping station

incidents. Finally, during Water Year 2004 (March 2004) data collection was

discontinued by the Metropolitan Water Reclamation District of Greater Chicago at

14 dissolved oxygen and temperature monitoring stations. Thus, use of Water Year

2003 allows a more complete verification of the water-quality model before it is

applied to evaluating the integrated strategy.

Therefore, based on these facts, Water Years 2001 and 2003 were selected as

the representative "wet" and "dry" years, respectively, for the development of an

integrated strategy for DO improvement in the CWS.

1.3. Objectives of Thesis

The IEPA proposed DO concentration standards for Chicago Area Waterways

aquatic life use designations, which are part of the IEPA's proposal to the Illinois

Pollution Control Board (IPCB) for rulemaking (IEPA, 2007). The MWRDGC has

proposed alternate DO standards for the CWS. Based on the different proposed DO

standards along the CWS, this thesis describes the development, evaluation and

simulation of effective integrated management plans of flow augmentation and

addition of supplemental aeration stations to meet the various DO standards. The

results of this study will be used by AECOM-CTE to develop cost estimates as part of

the IPCB rulemaking.

Although DO concentrations of most reaches in the CWS meet General Use Water Quality standards proposed by the IEPA a high percentage of time, DO problems exist in some waterway reaches during some periods for both of the selected water years (WYs 2001 and 2003). In order to attain more effective DO improvements at lower cost, a method of integrating the alternative methods into one integrated strategy for improving water quality is considered in this study with goals of 90% and 100% compliance with the IEPA proposed standards and 100% compliance with MWRDGC standards for the selected water years.

CHAPTER 2: THE MODELING CONCEPTS

2.1. Model Selection

From the early years of the Twentieth Century, water quality modeling has evolved appreciably since its beginnings (Chapra, 1997, p. 14). With the development of computers, a variety of mathematical simulation models have been applied to solve comprehensive water quality management problems. The first important step for having accurate simulation results is to choose an appropriate water quality management model for the specific water quality problems of interest. It is unwise to choose a model without elaborative thinking and analysis due to the fact that too simple or too complicated models may cause unreliable evaluation of water quality. Therefore, the selection of a water quality model should be based on a good balance among elements: model complexity, uncertainty, and the available amount of data (Manache, 2001).

In this thesis, the DUFLOW water quality model (DUFLOW, 2000) was selected as a tool to achieve water quality objectives in the CWS. It is considered a useful software product for river water quality modeling under unsteady-flow conditions (Manache and Melching, 2004). It was developed collaboratively by the International Institute for Hydraulic and Environmental Engineering (IHE), the Faculty of Civil Engineering at Delft University, the Dutch Public Works Department (Rijkswaterstaat), Tidal Waters Division (now RIKZ), STOWA (Dutch acronym for the Foundation for Applied Water Management Research) and the Agricultural University of Wageningen (DUFLOW, 2000) in the Netherlands. In addition,

DUFLOW software can be run under popular operation systems (e.g., Windows XP) on personal or micro computers with relatively low cost so that it is convenient for anyone who wants to do simulation work for water management and hydraulic engineering. Meanwhile, it is compatible with Geographical Information Systems (GIS) products, like ArcGIS produced by Environmental System Research Institute (ESRI), which can show detailed geographical information of objects of study. Several successful projects have applied the DUFLOW model in simulation to solve water quality problems in European rivers (e.g., Manache and Melching, 2004). According to these advantages, applying the DUFLOW model to the CWS is reasonable and sufficient.

2.2. The DUFLOW Model Concepts

The DUFLOW model is a software package for simulating one-dimensional unsteady flow and water quality in open-channel systems, designed for simple networks of channels with simple structures (DUFLOW, 2000). The model can be operated by different users and has a large range of applications. It provides a powerful tool to make day-to-day management decisions and to evaluate management since it can simulate the behavior of a system by operational measures, such as opening or closing of sluices, switching on pumping stations, reduction of pollutant loads, etc. In addition, it can be used for the design of hydraulic structures, flood prevention, operation of irrigation and drainage system, and other water-management-based objectives (DUFLOW, 2000).

The DUFLOW model allows for a rather large time step in the computation and for choosing different lengths of the elementary sections. To simulate factors (e.g., algal blooms, contaminated silt, and salt intrusion) in DUFLOW, two predefined eutrophication models are included in DUFLOW: EUTROF1 and EUTROF2. EUTROF 1 is a relatively simple model compared to EUTROF2. It simulates the cycling of nitrogen, phosphorus, and oxygen. It also simulates the growth of one phytoplankton species. However, the interaction between the sediment and the overlying water column is not included in a dynamic way. Thus, EUTROF 1 typically is specified for the study of short-term behavior of systems. EUTROF 2 is more suitable for studying long-term functioning of systems because EUTROF 2 defines three algal species and includes the interaction between the sediment and the overlying water column. Moreover, DUFLOW allows users to describe water quality processes by themselves according to their needs, so that users can create their own water quality models. In the following sections, basic equations used in DUFLOW are given.

2.2.1. The Unsteady-Flow Equations

The mass conservation equation and the momentum equation are used in the mathematical method in DUFLOW. In the hydromechanic part, DUFLOW is based on one-dimensional partial differential equations that describe unsteady flow in open channels (Abbott, 1979; Dronkers, 1964), such as the de Saint-Venant equations. These equations, which are the mathematical translation of the laws of conservation of mass and of momentum conservation read as follows:

$$\frac{\partial B}{\partial t} + \frac{\partial Q}{\partial x} = 0$$

(2.1)

and

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

(2.2)

where:

t = time [s]

x = distance as measured along the channel axis [m]

$H(x, t)$ = water level with respect to a reference level at location x and at time t [m]

$v(x, t)$ = mean velocity (averaged over the cross-sectional area) at location x and at time t [m/s]

$Q(x, t)$ = discharge at location x and at time t [m^3/s]

$R(x, H)$ = hydraulic radius of the cross section at location x for water level H [m]

$a(x, H)$ = cross-sectional flow width at location x for water level H [m]

$A(x, H)$ = cross-sectional flow area at location x for water level H [m^2]

$B(x, H)$ = cross-sectional storage area at location x for water level H [m^2]

g = acceleration due to gravity [m/s^2]

$C(x, H)$ = coefficient of De Chezy at location x for water level H [m$^{1/2}$/s]

There are two methods which can calculate the coefficient of De Chezy:

$$C = \frac{v}{\sqrt{RS}}$$

(a)

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

(b)

where:

S = the slope of the water surface

n $\quad=\quad$ the Manning's n

k $\quad=\quad$ a constant equal to 1.486 for U.S. customary units and 1.0 for S.I. units

w (t) $\quad=\quad$ wind velocity at time t [m/s]

Φ (t) $\quad=\quad$ wind direction in degrees at time t, measured clockwise from the north

[degrees]

φ (x) $\quad=\quad$ direction of channel axis in degrees at location x, measured clockwise

from the north [degrees]

γ (x) $\quad=\quad$ wind conversion coefficient at location x [-]

β $\quad=\quad$ correction factor for non-uniformity of the velocity distribution in the

advection term, defined as:

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

where the integral is taken over the cross section A[m^2]

The continuity eq. 2.1 states that if the water level changes at some locations, then eq. 2.1 will be the net result of inflow minus outflow at this location. The momentum equation (eq. 2.2) expresses that the net change of momentum is the result of exterior and interior forces. Assumptions for application of these equations include: the fluid is mixed well, and hence, the density may be considered to be constant.

2.2.2. The Mass Transport Equation

The quality part of the DUFLOW package depends on the one-dimensional (1-D) transport equation. This partial differential equation describes the concentration of a constituent in a 1-D system as function of time and space.

$$\frac{\partial(\mathrm{B}c)}{\partial t} = -\frac{\partial(Qc)}{\partial x} + \frac{\partial}{\partial x}\left(AD\frac{\partial c}{\partial x}\right) + P \tag{2.3}$$

where:

c =   constituent concentration [g/m^3]

D   =   Dispersion coefficient [m^2/s]

P =   production of the constituent per unit length of the section [g/s]

         The production term of the equation includes all physical, chemical and biological processes which a specific contaminant is subject to. In order to solve eq. 2.3, a numerical method is applied in the following form:

$$\frac{\partial S}{\partial x} + \frac{\partial(Bc)}{\partial t} - P = 0 \tag{2.4}$$

where S is the transport (quantity of the contaminant passing a cross section per unit of time):

$$S = Qc - AD\frac{\partial c}{\partial x} \tag{2.5}$$

         Equation 2.5 describes the transport by advection and dispersion. Equation 2.4 is the mathematical formulation of the mass conservation law, which states that the accumulation at a certain location, x, is equal to the net production rate minus the transport gradient.

2.2.3. Water-quality Processes

         In this thesis, EUTROF 2 was selected for CWS water quality model because: (1) this study needs to evaluate long-term behavior of the CWS; and (2) the sediment top layer is used in this model to describe the flux dynamics across the sediment-water interface which is considered to be important in the CWS. Many conventional

state variables for both the water column and sediment pore water are included in the

model. The conventional state variables are listed in Table 2.1.

Table 2. 1 List of state variables included in EUTROF 2

State Variables	Definition	Unit
A1, A2, A3	Algal biomass species 1, 2, and 3	mg C/l
Ab	Total algal biomass in the sediment	mg C/l
SS_w	Suspended solids concentration	mg/l
SS_B	Solid concentration in the sediment	mg/l
TIP_w	Total inorganic phosphorus in the water column	mg P/l
TIP_B	Total inorganic phosphorus in the sediment	mg P/l
TOP_w	Total organic phosphorus in the water column	mg P/l
TOP_B	Total organic phosphorus in the sediment	mg P/l
TON_w	Total organic nitrogen in the water column	mg N/l
TON_B	Total organic nitrogen in the sediment	mg N/l
$NH4_w$	Ammonia nitrogen in the water column	mg N/l
$NH4_B$	Ammonia nitrogen in the sediment	mg N/l
$NO3_w$	Nitrate nitrogen in the water column	mg N/l
$NO3_B$	Nitrate nitrogen in the sediment	mg N/l
$O2_w$	Oxygen in the water column	mg/l
$O2_B$	Oxygen in the sediment	mg/l
BOD_w	Biochemical oxygen demand in the water column	mg/l
BOD_B	Biochemical oxygen demand in the sediment	mg/l

2.2.4. Algae

Algae are eukaryotic organisms in Protista ranging from unicellular to

multicellular forms, including simple aquatic plants and bacteria. They live mainly in

the aquatic environment. In a water body, the growth of algae is determined by water

temperature, nutrients, and solar radiation. In EUTROF 2, three algae species are modeled and algal growth, respiration, and settling cycle processes are included in the model. However, only one algal species is considered in the DUFLOW model of the CWS. So the succession and dynamics of the composition of the algae population can be simulated to certain extent. The overall growth equation for each species is given by:

$$\frac{dA_{w,i}}{dt} = \left[\mu_{\max,i} F_{T,i} F_{N,i} F_{L,i} \right] A_{w,i} - \left[k_{die,i} + k_{res,i} \theta_{ra,i}^{(T-20)} - \frac{v_{sa,i}}{Z} \right] A_{w,i} \tag{2.6}$$

where:

$A_{w,i}$ =   Algal biomass in the water column for algal species i [mg C/l]

$\mu_{max,i}$ =   Maximum specific growth rate of algae for algal species i [l/d]

$k_{res,i}$ =   Algal respiration rate constant for algal species i [1/d]

$\theta_{ra,i}$ =   Temperature coefficient for respiration for algal species i

T =   Water temperature [^{0}C]

$k_{die,i}$ =   Algal die-off rate constant for algal species i [1/d]

$v_{sa,i}$ =   Settling velocity for algal species i [m/d]

Z =   Water depth [m]

The growth is considered to be limited by nutrients, light, and temperature. The main nutrients needed for algae growth include nitrogen and phosphorus. Therefore, nutrient limitation is described as:

$$F_{N,i} = \min \left[\frac{DIP_w}{DIP_w + k_{p,i}}, \frac{DIN_w}{DIN_w + k_{n,i}} \right] \tag{2.7}$$

where:

DIP_w = Total dissolved inorganic phosphorus concentration in the water

column [mg P/L]

DIN_w = Total inorganic nitrogen (sum of nitrate and ammonia) concentration

in the water column [mg N/L]

$k_{p,i}$ = Monod constant for phosphorus for algal species i [mg P/L]

$k_{n,i}$ = Monod constant for nitrogen for algal species i [mg N/L]

In eq. 2.7, the reduction of the maximum growth rate is controlled by the most

limiting factor. It is assumed that algae can use inorganic phosphorus determined by

the phosphorus cycle subroutines and ammonia and nitrate concentrations determined

by the nitrogen cycle subroutines for their growth.

At the same time, the limiting factor for light should be considered. In

EUTROF 2, a daylight average light limitation function is used since EUTROF 2 is

for simulation of long time periods. The depth integrated Steele equation is integrated

over the daylight periods. This indicates that EUTROF 2 is not able to describe

diurnal variations in algal growth. The light limitation factor is written as:

$$F_{L,i} = \frac{ef}{\varepsilon_{tot}Z}\left[\exp\left(-\alpha_{1,i}\right) - \exp\left(-\alpha_{0,i}\right)\right]$$

(2.8)

in which:

$$\alpha_{1,i} = \alpha_{0,i}e^{(-\varepsilon_{tot}Z)}$$

(2.8.1)

and

$$\alpha_{0,i} = \frac{I_a}{I_{s,i}}$$

(2.8.2)

where:

e = Neperian number

f =   Fraction of daylight during the day

I_a =   Average light intensity during the daylight period [W/m^2]

$I_{s,i}$ =   Optimal light intensity for algal species i [W/m^2]

ε_{tot} =   Total light extinction coefficient

The total extinction coefficient (ε_{tot}) is determined by the background extinction of the water and the contributions of chlorophyll and suspended solids to the vertical light attenuation as computed below.

$$\varepsilon_{tot} = \varepsilon_0 + \varepsilon_{a\,lg}Chl-a + \varepsilon_{ss}SS \tag{2.9}$$

where:

ε_0 =   Background light extinction coefficient [1/m]

$\varepsilon_{a\,lg}$ =   Specific light extinction coefficient for chlorophyll [L/(µg Chl-a m)]

ε_{ss} =   Specific light extinction coefficient for suspended solids [L/(mg SS m)]

Chl-a =   Algae concentration [µg Chl-a/L]

SS =   Suspended solids concentration [mg/L]

For internal computational purposes algal carbon is used as a measure for the biomass. The algal carbon concentration is converted to chlorophyll-a using a fixed chlorophyll to carbon ratio for each species. The total chlorophyll concentration can be described as:

$$Chl-a = \sum_{i=1}^{3} a_{ChlaC,i}A_{W,i} \tag{2.10}$$

where:

$a_{ChlaC,i}$ =   Ratio of chlorophyll to carbon for algal species i

The temperature limitation is included in EUTROF 2. For the individual species an optimum curve is used to simulated temperature dependent growth. The temperature limitation factor is expressed as:

$$F_{T,i} = \frac{T_{CS,i} - T}{T_{CS,i} - T_{OS,i}} \exp\left(1 - \frac{T_{CS,i} - T}{T_{CS,i} - T_{OS,i}}\right) \tag{2.11}$$

where:

$T_{cs,i}$ =   Critical temperature for algal species i [$^{\circ}$C]

$T_{os,i}$ =   Optimal temperature for algal species i [$^{\circ}$C]

If the water temperature exceeds the critical temperature for growth, $F_{T,i} = 0$.

Three loss processes are included in the algal balance eq. 2.7. The endogenous respiration is considered to be temperature dependent. The second loss term represents the die-off and the effects of grazing and is regarded to be constant. Finally, the sedimentation of algae is included. Although the sedimentation velocity of algae is low, the total load settling to the sediment can be substantial. Together with the sedimentation of dead organic matter (detritus and from man-made sources) it determines the organic and nutrient load of the sediment and controls the resulting interaction between the sediment and the overlying water column. Once settled into the sediment the algae are converted to benthic organic carbon and subject to anaerobic decomposition. There is no transport of living algae from the sediment to the water column. As the stoichiometric ratio for all algae species are considered to be the same for the benthic algal carbon concentration only one state variable has to be defined. The following equation is used to express the algae concentration in the

sediment:

$$\frac{dA_B}{dt} = -K_{daB}\theta_{daB}^{(T-20)}A_B \tag{2.12}$$

where:

A_B =   Algal biomass in the sediment [mg C/L]

K_{daB} =   Anaerobic decay rate constant for algal sediment [1/d]

θ_{daB} =   Temperature coefficient for anaerobic decomposition of algal sediment

2.2.5. Organic Phosphorus

Phosphorus, as a kind of nutrient, plays a significant role in all life. In a water body, phosphorus present as soluble and/or particulate forms. During respiration and die-off of algae, part of the associated phosphorus is released as organic phosphorus. The remaining phosphorus is distributed to the inorganic phosphorus pool. The phosphorus to carbon ratio is assumed to be constant and it is the same for all three algae species. Due to aerobic mineralization in the water column organic phosphorus is converted to the inorganic form. The following equation is used to describe the total organic phosphorus concentration in the water column:

$$\frac{dTOP_w}{dt} = -K_{min}\theta_{min}^{(T-20)}TOP_w + f_{porg}a_{pc}\sum_{i=1}^{3}\left[\left(k_{die,i} + k_{res,i}\theta_{ra,i}\right)A_{w,i}\right] \tag{2.13}$$

where:

TOP_w =   Total organic phosphorus concentration in the water column [mg P/L]

K_{min} =   Mineralization rate constant for organic matter in the water column

            [1/d]

θ_{min} =   Temperature coefficient for mineralization

f_{porg} =   Fraction algal phosphorus released as organic phosphorus

a_{pc} =   Algal phosphorus to carbon ratio [mg P/mg C]

       In the sediment organic phosphorus is only subject to anaerobic

decomposition. The total organic phosphorus in the sediment top layer is given by:

$$\frac{dTOP_B}{dt} = -K_{min\,B}\theta_{min\,B}^{(T-20)}TOP_B + a_{pc}K_{daB}\theta_{daB}^{(T-20)}A_B \tag{2.14}$$

where:

TOP_B =   Total organic phosphorus concentration in the sediment [mg P/L]

$K_{min\,B}$ =   Mineralization rate constant for organic matter in the sediment [1/d]

$\theta_{min\,B}$ =   Temperature coefficient for mineralization in the sediment

$K_{da\,B}$ =   Anaerobic decay rate constant for algal sediment [1/d]

$\theta_{da\,B}$ =   Temperature coefficient for anaerobic decomposition of algal sediment

A_B =   Algal biomass in the sediment [mg C/L]

2.2.6. Inorganic Phosphorus

       In the water column and sediment, inorganic phosphorus is formed during

aerobic and anaerobic mineralization, respectively. It is also released during the algal

respiration and die-off. The equations describing the inorganic phosphorus

concentration in the water column and the sediment top layer are as follows:

$$\frac{dTIP_w}{dt} = K_{min}\theta_{min}^{(T-20)}TOP_w + a_{pc}\sum_{i=1}^{3}\left[\mu_{max,i}F_{T,i}F_{N,i}F_{L,i}A_{w,i}\right] +$$

$$(1-f_{porg})a_{pc}\sum_{i=1}^{3}\left[\left(k_{die,i} + k_{res,i}\theta_{ra,i}^{(T-20)}\right)A_{w,i}\right] \tag{2.15}$$

where:

TIP_w =   Total inorganic phosphorus concentration in the water column [mg P/L]

and:

$$\frac{dTIP_B}{dt} K_{\min B} \theta_{\min B}^{(T-20)} TOP_B \tag{2.16}$$

where:

TIP_B = Total inorganic phosphorus concentration in the sediment [mg P/L]

The dissolved fraction of inorganic phosphorus in the water column (with subscript W) and in the bottom sediment (with subscript B) is calculated by:

$$f_{dpW} = \frac{1}{1 + K_{pipW} SS_W} \tag{2.17}$$

$$f_{dpB} = \frac{1}{1 + K_{pipB} SS_B} \tag{2.18}$$

where:

K_{pip} = Partition constant for phosphorus [1/mg SS]

f_{dp} = Fraction dissolved organic phosphorus

SS = Suspended solids concentration [mg/L]

Equations 2.18 and 2.19 indicate that it is assumed that the equilibrium is reached instantaneously.

2.2.7. Organic Nitrogen

Nitrogen, as a type of nutrient, is also important in the nitrogen cycle process in natural waters. However, this nutrient can cause water-quality problems directly or indirectly, such as in the nitrification/denitrification process, eutrophication, nitrate pollution, and ammonia toxicity.

The behavior of organic nitrogen is similar to that of organic phosphorus. In the water column, release during algal loss processes and anaerobic mineralization

takes place. In the sediment, the anaerobic mineralization of settled algae and organic nitrogen are the controlling processes. The total organic nitrogen concentration in the water column and sediment top layer are given by:

$$\frac{dTON_W}{dt} = -K_{\min}\theta_{\min}^{(T-20)}TON_W + f_{norg}a_{nc}\sum_{i=1}^{3}\left[\left(k_{die,i} + k_{res,i}\theta_{ra,i}\right)A_{W,i}\right] \tag{2.19}$$

$$\frac{dTON_B}{dt} = -K_{\min B}\theta_{\min B}^{(T-20)}TON_B + a_{nc}K_{daB}\theta_{daB}^{(T-20)}A_B \tag{2.20}$$

where:

TON = Total organic nitrogen [mg N/L]

f_{norg} = Fraction of algal nitrogen released as organic nitrogen

a_{nc} = Nitrogen to carbon ratio [mg N/mg C]

The subscripts W and B again denote the water column and the bottom sediment.

2.2.8. Ammonia Nitrogen

Ammonia is present in two forms in natural waters: ammonium ion (NH_4^+) and ammonia gas (NH_3). During algal respiration and die-off of the algae part of the nitrogen is released as ammonia. The remaining part is added to the pool of organic nitrogen. Both ammonia and nitrate can be used for algal growth. The preference for the nitrogen source used is controlled by the nitrogen preference factor as follows:

$$\begin{aligned} P_{NH4} = NH4_W &\frac{NO_3}{\left(K_{mN} + NH4_W\right)\left(K_{mN} + NO3_w\right)} \\ &+ NH4_W\frac{K_{mN}}{\left(NO3_w + NH4_W\right)\left(K_{mN} + NO3_w\right)} \end{aligned} \tag{2.21}$$

where:

K_{mN} =   The ammonia preference constant [mg N/L]

$NH4$ =   Ammonia nitrogen concentration [mg N/L]

$NO3$ =   Nitrate nitrogen concentration [mg N/L]

The nitrification rate in the water column is controlled by the oxygen concentration, using a Monod type of equation. The equation for ammonia nitrogen in the water column is given by:

$$\frac{dNH4_W}{dt} = -K_{nit}\theta_{nit}^{(T-20)}\frac{O2_w}{(O2_w + K_{no})}NH4_W + K_{\min}\theta_{\min}^{(T-20)}TON_W$$
$$-a_{nc}P_{NH4}\sum_{i=1}^{3}\left[\mu_{\max,i}F_{T,i}F_{N,i}F_{L,i}A_{W,i}\right] + a_{nc}\sum_{i=1}^{3}\left[K_{die,i} + k_{res,i}\theta_{ra,i}^{(T-20)}\right]$$

(2.22)

where:

K_{nit} =   Nitrification rate constant [1/d]

θ_{nit} =   Temperature coefficient for nitrification

K_{no} =   Monod constant for nitrification [mg O_2/L]

Organic nitrogen is hydrolyzed to ammonia by bacterial action within the sediment. Because the decomposition processes in the bottom are assumed to be anaerobic, no nitrification process happens in the bottom sediment. The equation expressing the sediment ammonia concentration is given below:

$$\frac{dNH4_B}{dt} = K_{\min}\theta_{\min}^{(T-20)}TON_B$$

(2.23)

2.2.9. Nitrate Nitrogen

The ultimate result of the nitrification process is nitrate. Depending on the ammonia preference factor nitrate can be used as a nitrogen source for algal growth.

Denitrification, which is also controlled by the oxygen concentration, is included. The nitrate concentration in the water column is described by:

$$\frac{dNO3_W}{dt} = -K_{den}\theta_{den}^{(T-20)}\frac{K_{dno}}{\left(K_{dno}+O2_w\right)}NO3_W$$
$$+K_{nit}\theta_{nit}^{(T-20)}\frac{O2_w}{\left(O2_w+K_{no}\right)}NH4_W - a_{nc}\left(1-P_{NH4}\right)\sum_{i=1}^{3}\left[\mu_{\max,i}F_{T,i}F_{N,i}F_{L,i}A_{W,i}\right]$$

(2.24)

where:

K_{den} = Denitrification rate constant [1/d]

θ_{den} = Temperature coefficient for denitrification

K_{dno} = Monod constant for denitrification [mg O$_2$/L]

$O2_w$ = Dissolved Oxygen concentration [mg O$_2$/L]

In the bottom sediment, the only process is denitrification. Nitrate is present in the sediment because of the diffusive transport from the overlying water column. The nitrate concentration in the sediment top layer is given by:

$$\frac{dNO3_B}{dt} = -K_{denB}\theta_{denB}^{(T-20)}NO3_B$$

(2.25)

2.2.10. Carbonaceous Biochemical Oxygen Demand (CBOD)

Carbonaceous Biochemical Oxygen Demand (CBOD) is used to define the magnitude of dissolved oxygen consumption by biodegradable organic material in the water under aerobic conditions. Practically, 5-day CBOD are used (expressed as CBOD$_5$). The CBOD$_5$ is affected by three factors – the denitrification process, settling, and die-off of the algae – as well as the in stream consumption of CBOD. The equation describing the decay of organic matter is as follows:

$$\frac{dBOD_W}{dt} = -K_{bod}\theta_{bod}^{(T-20)}\frac{O2_w}{\left(O2_w + K_{bodo}\right)}BOD_W +$$

$$\left[a_{oc}\sum_{i=1}^{3}\left[K_{die,i}A_{W,i}\right] - \frac{5}{4}\frac{32}{14}K_{den}\theta_{den}^{(T-20)}\frac{K_{dno}}{\left(O2_w + K_{dno}\right)}NO3_W\right]X_{conv} \quad (2.26)$$

where:

BOD $\quad=\quad$ Carbonaceous 5-day biochemical oxygen demand [mg O_2/L]

$K_{bod} \quad=\quad$ Oxidation rate constant for $CBOD_5$ [1/d]

$\theta_{bod} \quad=\quad$ Temperature coefficient for oxidation of CBOD

$K_{bodo} \quad=\quad$ Monod constant for oxidation of CBOD [mg O_2/L]

$a_{oc} \quad=\quad$ Oxygen to carbon ratio [mg O_2/mg C]

$X_{conv} \quad=\quad$ Conversion factor to calculate $CBOD_5$ for ultimate CBOD

$$X_{conv} = 1 - \exp\left(-5K_{bod}\right) \quad (2.27)$$

In the sediment, the settled algae and benthic organic matter are related to anaerobic degradation. In reality, the reaction mechanisms involved are very complicated. In the model, only the initial step in which the organic carbon is converted to reactive intermediates is included. This formulation is similar and consistent with the degradation of organic nitrogen and phosphorus within the sediment. The reactive intermediates, however, participate in further reactions. In the model the redox reactions oxidizing these intermediates are not included, but these reduced carbon products are expressed as negative oxygen equivalents that are transported across the sediment water interface. The equation describing organic carbon expressed as BOD_5 is given by:

$$\frac{dBOD_B}{dt} = \frac{a_{oc}K_{daB}\theta_{daB}^{(T-20)}A_B - \frac{5}{4}\frac{32}{14}K_{denB}\theta_{denB}^{(T-20)}NO3_B}{X_{conv}} - K_{bodB}\theta_{bodB}^{(T-20)}BOD_B \quad (2.28)$$

2.2.11. Dissolved Oxygen (DO)

Dissolved Oxygen (DO) is one of the most significant indexes to evaluate water quality in a water body. DO concentration in the water column is affected by two processes: 1) the deoxidation processes which decrease DO concentration, including degradation by degradable organic matters and respiration; and 2) the oxidation processes which increase DO concentration, such as diffusion of oxygen from the surrounding air, photosynthesis by hygrophytes etc. These two processes fluctuate and affect each other resulting in DO concentration changes in a water body. If a river is originally unpolluted, dissolved oxygen levels should be near saturation. However, when it is polluted by organic matter, DO is consumed and reduced to a low level, even close to zero. At this time, the decay of organic materials becomes a fermentation process under anaerobic conditions. The reduction of DO concentrations severely deteriorates water-quality and leads to destruction of the original ecological balance. Therefore, DO concentration is the important criterion which directly reflects contaminant degree. USEPA and local governments developed appropriate DO standards for different places. In this case, two DO standards for the CWS developed by the IEPA and MWRDGC will be discussed in Chapter 4-6.

In EUTROF 2, the DO concentrations depend on oxidation $CBOD_5$, reaeration, algal respiration, and nitrification in the water column. The equation is described as follows:

$$\frac{dO2_w}{dt} = K_{re}\theta_{re}^{(T-20)}\left(C_s - O2_w\right) - K_{BOD}\theta_{BOD}^{(T-20)}\frac{O2_w}{\left(O2_w + K_{bodo}\right)}\frac{BOD_w}{X_{conv}}$$

$$-\frac{64}{14}K_{nit}\theta_{nit}^{(T-20)}\frac{O2_w}{\left(O2_w K_{NO}\right)}NH4_w - \frac{32}{12}\sum_{i=1}^{3}\left[k_{res,i}\theta_{res,i}^{(T-20)}A_{w,i}\right] \tag{2.29}$$

$$+\sum_{i=1}^{3}\left[\mu_{max,i}F_{T,i}F_{N,i}F_{L,i}A_{w,i}\right]\left[\frac{32}{12}+\frac{48}{12}a_{nc}\left(1-P_{NH4}\right)NO3_w\right]-F_{XD}$$

where:

θ_{rea} =   Temperature coefficient for reaeration

K_{re} =   Reaeration-rate coefficient [s^2/m]

$$K_{re} = \frac{k_{mas}}{Z} \tag{2.30}$$

k_{mas} =   Mass transfer coefficient for oxygen given by the O'Connor-Dobbins

                   (1958) formula:

$$k_{mas} = k\frac{v^{0.5}}{Z^{1.5}} \tag{2.31}$$

k =   Constant in the O'Connor-Dobbins reaeration-rate coefficient formula.

                   (The default value equals 3.94)

C_s =   Oxygen saturation concentration [mg/L]

F_{XD} =   Diffuse exchange flux of oxygen from the water column into the

                   sediment bed (described in detail in section 2.2.12)

Production of oxygen results from primary production of algae. In the case where nitrate is used as a source for nitrogen for algae growth an additional oxygen production takes place, due to the reduction of nitrate during the assimilation process. The following equation is used to describe the sediment oxygen concentration:

$$\frac{dO2_B}{dt} = -K_{bodB}\theta_{bodB}^{(T-20)}\frac{BOD_B}{X_{conv}} \tag{2.32}$$

This negative concentration implies that the redox state in the sediment is reduced rather than oxidized. The computed negative concentration is considered to be the oxygen equivalence of the reduced intermediate products produced in the mineralization reaction mechanism.

2.2.12. Suspended Solids (SS)

Suspended solids is also considered as one of the important water-quality criteria. Generally, flow of water, resuspension, and sedimentation processes affects suspended solids concentration. In EUTROF 2, sedimentation is expressed as a first-order process. The following equation describes the suspended solids concentration in the water column (SS_w):

$$\frac{dSS_w}{dt} = \frac{v_{ss}}{Z} + \frac{F_{res}}{Z}$$ (2.33)

where:

v_{ss} = Settling velocity of suspended solids [m/d]

F_{res} = Suspended solids resuspension flux [m/d]

As the porosity and density of the sediment top layer are considered to be constant and only one fraction suspended solids is considered. The concentration of sediment is a constant and is given by:

$$SS_B = \rho \times (1 - POR) \times 1000$$ (2.34)

where:

ρ = Suspended solids density [kg/m^3]

POR = Sediment porosity

2.2.13. Sediment Model

The degradation of organic matter in the sediment can have an important effect on the concentration of oxygen and nutrients in the overlying water column. In reality, sediment activity related to other processes, such as degradation of organic matter, state and concentration of nutrients, occurrence of toxic conditions, is complicated. Therefore, in EUTROF 2, a simple method is used. For the description of the exchange fluxes a distinction must be made between dissolved constituents (like ammonia, nitrate, and oxygen) and constituents which can be associated with the suspended solids (like inorganic and organic phosphorus, organic nitrogen, and COBD$_5$). The organic phosphorus, organic nitrogen and CBOD$_5$ are considered to exist both in a dissolved and particulate forms. For a certain constituent, X, the following forms are distinguished:

$$DX_W = f_{dxw}TX_w \tag{2.35}$$

$$PX_W = (1 - f_{dxw})\frac{TX_w}{SS_w} \tag{2.36}$$

$$DX_B = f_{dxw}\frac{TX_w}{POR} \tag{2.37}$$

$$PX_B = (1 - f_{dxb})\frac{TX_B}{SS_B} \tag{2.38}$$

where:

TX = The total concentration of constituent X

DX = The dissolved portion of constituent X in mass per volume

PX = The particulate portion of constituent X as a fraction of the concentration of suspended sediments

f_{dx} =   Dissolved fraction of constituent X

POR   =   Porosity of the sediment top layer

      The subscripts W and B again denote the water column and the bottom sediments.

      The sediment layer is divided into an upper, active layer, and a lower, inactive layer. The total transport across the interface of the top layer and lower layer of the bottom sediments equals the sum of the fluxes. The following equations describe the concentration in the water column and the sediment top layer:

$$\frac{dX_W}{dt} = \frac{F_{XD} - F_{XS} + F_{XR}}{Z} + P_{XW} \tag{2.39}$$

$$\frac{dX_B}{dt} = \frac{F_{XD} - F_{XS} + F_{XR} + F_{XB}}{HB} + P_{XB} \tag{2.40}$$

where:

HB   =   Depth of the sediment top layer [m]

FXD   =   Diffuse exchange flux of oxygen from the water column into the sediment bed

FXS   =   Sedimentation flux of suspended solids in the sediment bed

F_{XR} =   Resuspension flux of solids and the particulate concentration in the sediment

F_{XB} =   Transport of sediment between top and lower sediment layer

      For these constituents, the separate fluxes (F_{XD}, F_{XS}, F_{XR}, and F_{XB}) can be described as follows:

The diffusive exchange flux (F$_{XD}$)

The dissolved fraction is subject to diffusive exchange. The difference between the concentration in the interstitial water (D$_{XB}$) and the water column (D$_{XW}$) is the driving force for mass transport.

$$F_{XD} = \frac{E_{diff}}{HB}(D_{XB} - D_{XW})$$ (2.41)

where:

E_{diff} =   Diffusive exchange rate constant [m^2/d]

The sedimentation flux (F$_{XS}$)

The flux of constituent X across the interface of the top and lower layers is equal to the sedimentation flux of suspended solids multiplied with the particulate constituent X concentration. The sedimentation flux also describes inclusion of pore water due to the formation of new sediment by sedimentation.

$$F_{XS} = F_{sed}PX_W + v_sDX_WPOR$$ (2.42)

where:

F_{sed} =   Sedimentation flux of suspended solids

v_s =   Benthic sediment settling velocity

The resuspension flux (F~XR~)

The resuspension of particulate X is given by the product of the resuspension flux of solids and the particulate concentration in the sediment and the release of pore water during resuspension.

$$F_{XR} = F_{res}PX_B + v_r DX_B POR \tag{2.43}$$

where:

v_r =   Benthic sediment resuspension velocity

Transport between top and lower sediment layers (F~XB~)

In EUTROF 2, the top layer depth of sediment is assumed to be a constant, so there is a transport of sediment between the top and lower sediment layers. If net sedimentation occurs, sediment is transported from the top to the lower layer. In case of net resuspension, the sediment top layer is replenished with sediment from the lower layer. In the model, there is an assumption that no diffusive exchange occurs between the two sediment layers. Therefore, the concentration in the top layer is only influenced by the quality of the lower layer if resuspension occurs. The following two equations describe the relation of transport between top and lower sediment layers:

$$F_{XB} = -v_{sd}TX_B \qquad \text{if } v_{sd} > 0 \tag{2.44}$$

$$F_{XB} = -v_{sd}TX_{LB} \qquad \text{if } v_{sd} < 0 \tag{2.45}$$

where:

v_{sd} =   Velocity by which the benthic surface is displaced

TX_{LB} =   The total concentration of constituent X in the lower sediment layer.

CHAPTER 3: DESCRIPTION OF CASE STUDY

3.1. Description of Chicago Waterway System

The Chicago Waterway System (CWS) starts from Lake Michigan at the Wilmette Pumping Station on the north and follows a path consisting of the North Shore Channel (NSC), lower portion of the North Branch Chicago River (NBCR), South Branch Chicago River (SBCR), the Chicago Ship and Sanitary Channel (CSSC). The Chicago River Main Stem flows into SBCR, and the Calumet-Sag Channel and Little Calumet flows into the CSSC composing the CWS. The CWS is a 76.3 mile branching network of navigable waterways controlled by hydraulic structures.

The North Shore Channel is a man-made channel 7.7 miles long. It starts from Wilmette and flows past Linden Street, Central Avenue, and Main Street, ending 1.36 miles downstream from the Devon Avenue in-stream aeration station. The North Side Water Reclamation Plant (NSWRP) divides the NSC into Upper and Lower parts. After the NSWRP, the NSC flows south until it reaches the junction with the North Branch Chicago River. The NBCR continues to flow south until it reaches wolf point where it connects to the Chicago River Main Stem and the SBCR. The discharge from the NBCR and the Main Stem flows southwest through the SBCR until Bubbly Creek Junction, which is the beginning of Chicago Sanitary and Ship Canal (CSSC), is reached. The CSSC is 31.3 miles long and flows downstream until it meets the Des Plaines River near Lockport. The Calumet-Sag Channel and Little Calumet River compose the Calumet River System which is another part of the CWS. These

channels flow from east to west until the Calumet-Sag Channel meets the CSSC at Sag Junction. The Little Calumet River has two segments: North and South. In this case, only Little Calumet River North which starts from the O'Brien Lock and Dam is considered in the proposed DO standards. A detailed schematic diagram of the CWS is shown in Figure 3.1.
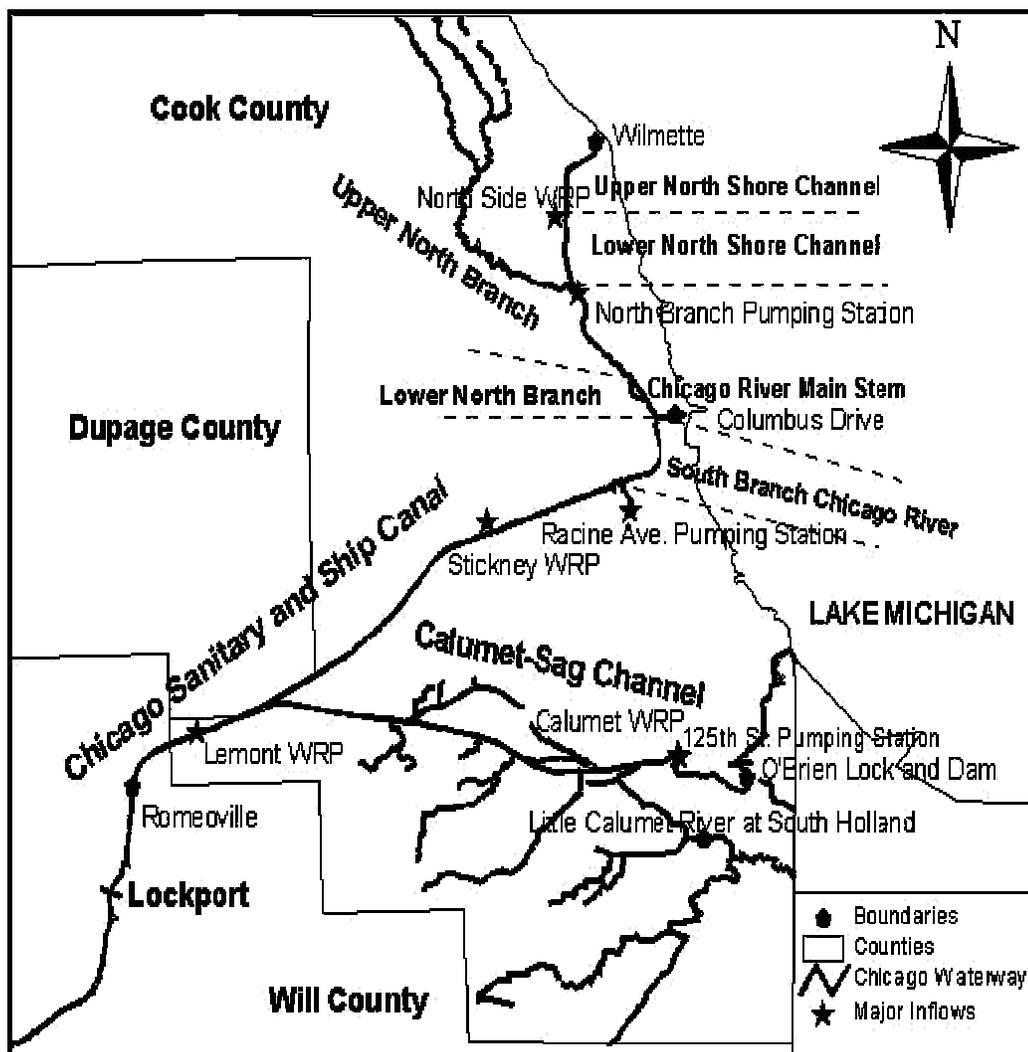
Figure 3. 1 Schematic diagram of the Calumet and the Chicago River Systems

From upstream to downstream on the CWS, generally upper reaches are narrower and shallower than lower reaches. The study area was divided into 17 reaches for water-quality simulation by CDM (CDM, 1992). These 17 reaches are shown in Figure 3.2. In this figure, 16 reaches can be found easily, but C17 (not shown in Figure 3.2) is the reach on the Little Calumet River (south) from the USGS South Holland gage to the confluence with the Calumet-Sag Channel. In this case, the Calumet River (south) was not included in the DO standard evaluation. Meanwhile, because C10 is out of the boundaries of this study, it is not marked in Figure 3.2, either. Bubbly Creek section (South Fork of the South Branch Chicago River) from the Racine Avenue Pumping Station to the CSSC, which was not considered in the previous QUAL2E model study, was added to the DUFLOW model for this case. Hence, totally 17 reaches are used in this simulation study.
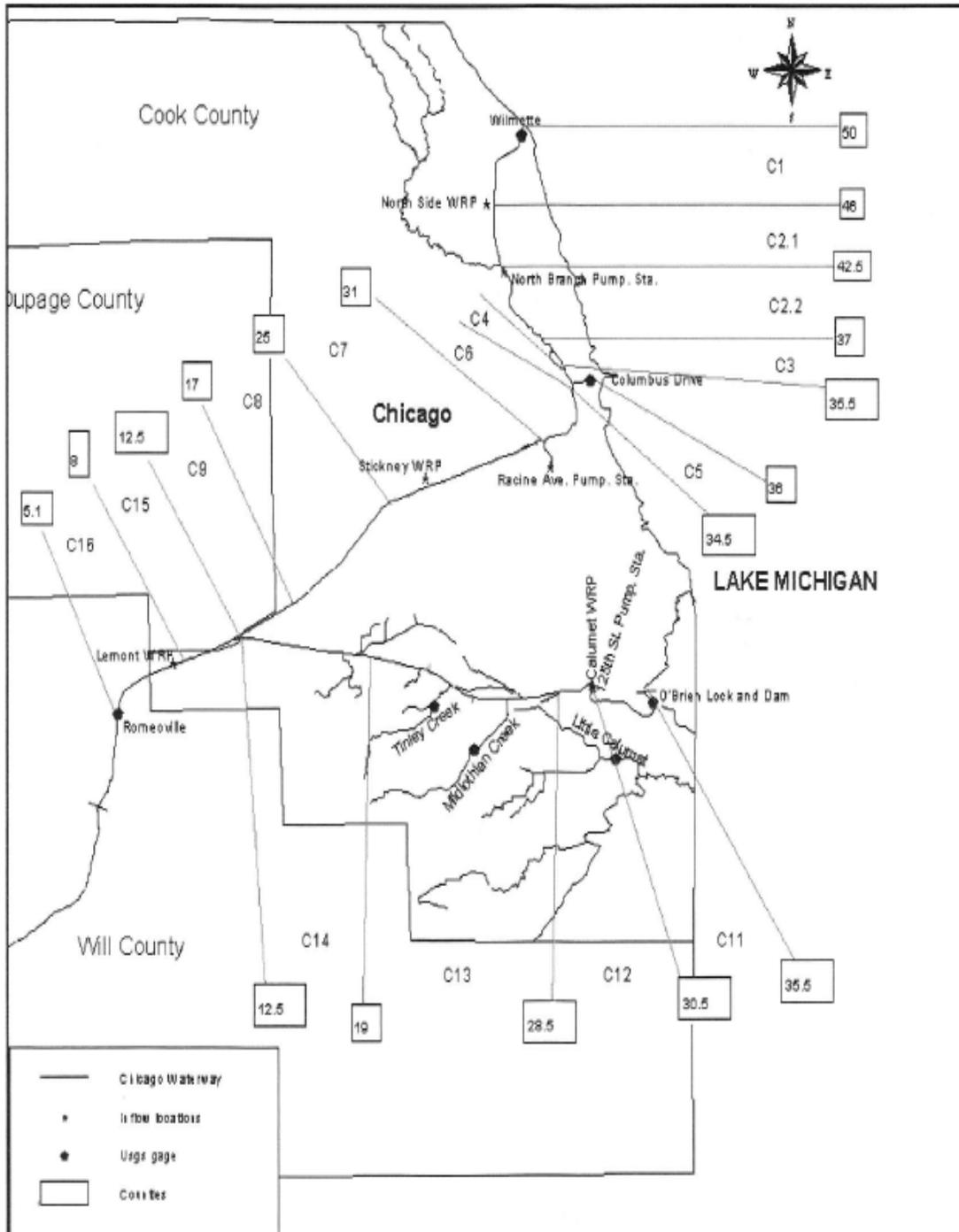
Figure 3. 2 Chicago Waterway System reaches. The numbers in boxes are the river miles from the Chicago Sanitary and Ship Canal at Lockport Lock and Dam (after Alp and Melching, 2006)

There are three primary locations where water is transferred from Lake Michigan to the CWS: the North Shore Channel at Maple Avenue (close to the Wilmette Pumping Station) is used as one of the inflow points in the model, the Chicago River Main Stem at Columbus Drive (close to Chicago River Controlling Works (CRCW)) is used as one of the inflow points in the model, and the Calumet River at the O'Brien Lock and Dam. The measured inflow data at three boundaries was provided by the USGS. Hydraulic data used in the model input were discussed in previous studies (Melching et al., in preparation, etc.) and hydraulic model verification was completed (Alp and Melching, 2006). The detailed description was discussed in those studies.

Hourly flow data used in the model comes from the MWRDGC for the treated effluent discharged to the CWS by four Water Reclamation Plants (WRPs): North Side WRP, Stickney WRP, Calumet WRP, and Lemont WRP (note: daily flow values are used at this WRP).

The CWS also receives CSO flows from three large pumping stations. The hourly flows for these three CSO pumping stations-North Branch, Racine Avenue, and 125th Street-were estimated according to measured pump operation records and capacities of operated pumps obtained from the MWRDGC.

In addition, there are nearly 240 gravity combined sewer overflows (CSOs) in the modeled parts of the CWS drainage area. However, because it is difficult to add all CSO locations in the model, only 28 representative CSO locations were defined in a previous study (Alp and Melching, 2006) and previous evaluations of possible water quality improvement strategies (CTE, 2006, 2007a-c) were based on this

representation. For each of these 28 representative CSOs flows were distributed based on the drainage areas. Figure 3.3 shows the locations of the 28 representative CSOs. However, the 28 representative CSO locations are still insufficient for subsequent assessment of needed water-quality improvements, thus, more gravity CSOs were added to the model of the CWS for this study. For example, on the NSC, with only four representative inflow points, the CSO flows overpowered the flows transferred as part of flow augmentation requiring higher amounts of transfer than might be needed if the flows were distributed as in reality (Melching et al., in preparation). Therefore, 19 gravity CSO locations are considered as CSO inflow points in the revised DUFLOW model used in this study and the flows were redistributed to these locations shown in Fig. 3.4.

Table 3. 1 Calculation of ungaged tributaries and watersheds

Stream Ungaged	Ratio with Midlothian*
Mill Creek West	0.55
Stony Creek West	1.086
Cal-Sag Watershed East	0.246
Navajo Creek	0.137
Stony Creek East	0.486
Ungaged Des Plaines Watershed	0.703
Calumet Union Ditch	1.168
Cal-Sag Watershed West	0.991

*The gaged Midlothian Creek drainage area is 12.6 mi^2, but these ratios are computed 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.

In the previous applications of the Marquette Model (e.g., Alp and Melching, 2006) the inflows from gravity CSOs were estimated as follows. During storm events, the measured and estimated (for ungaged tributaries) inflows were insufficient for simulated water-surface elevations at Romeoville to match the measured water-surface elevations when flow at Romeoville was the downstream boundary condition. If the simulated water-surface elevation is substantially below the observed value, the hydraulic model is artificially dewatering the CWS in order to match the observed flow at Romeoville indicating that the CWS is receiving insufficient inflow without considering the gravity CSOs. Thus, gravity CSO volume (starting with the volume imbalance between measured outflows at Romeoville and measured and estimated inflows) was added until reasonable water-surface elevations were simulated at Romeoville. This gravity CSO volume was added at the representative CSO inflow locations on a per area basis at the time of operation of the pumping stations.

Evaluations for events in 2001 and 2002 of simulated water-surface elevations in the CWS for the case of gravity CSO flows from the Corps models and pumping station flows from the operation records have yielded reasonable results throughout the CWS in comparison to the results for the original input to the Marquette Model (Alp and Melching, 2008). Hence simulated gravity CSO flows obtained from the Corps are used in the simulations to identify an integrated strategy for DO improvement in the CAWS. Detailed discussion of the Corps models (a combination of the Hydrological Simulation Program-Fortran, Special Contributing Area Loading Program, and Tunnel Network Model) is given in Espey et al. (2004).

The data from the USGS gage on the Little Calumet River (South) at South Holland provide a flow versus time upstream boundary condition for the water-quality model. Two tributaries- Tinley Creek (near Palos Park) and Midlothian Creek (at Oak Forest) are gaged by USGS- are considered as tributary flow to the Calumet-Sag Channel. The USGS gage on the Grand Calumet River at Hohman Avenue at Hammond, Ind. is a tributary to the Little Calumet River (North). Flow on the NBCR is measured just upstream of its confluence with the NSC at the USGS gage at Albany Avenue. The gaged flows at all 4 USGS gages are used as tributary inflows in the DUFLOW model of the CWS.

In the original hydraulic calibration (Shrestha and Melching, 2003), flows on Midlothian Creek were used to estimate flows on ungaged tributaries on an area-ratio basis. The drainage area ratio for the ungaged tributaries compared to the Midlothian Creek drainage area are listed in Table 3.1.

Figure 3. 3 28 Representative combined sewer overflow (CSO) Locations used in earlier DUFLOW simulation studies (after Alp and Melching, 2006).
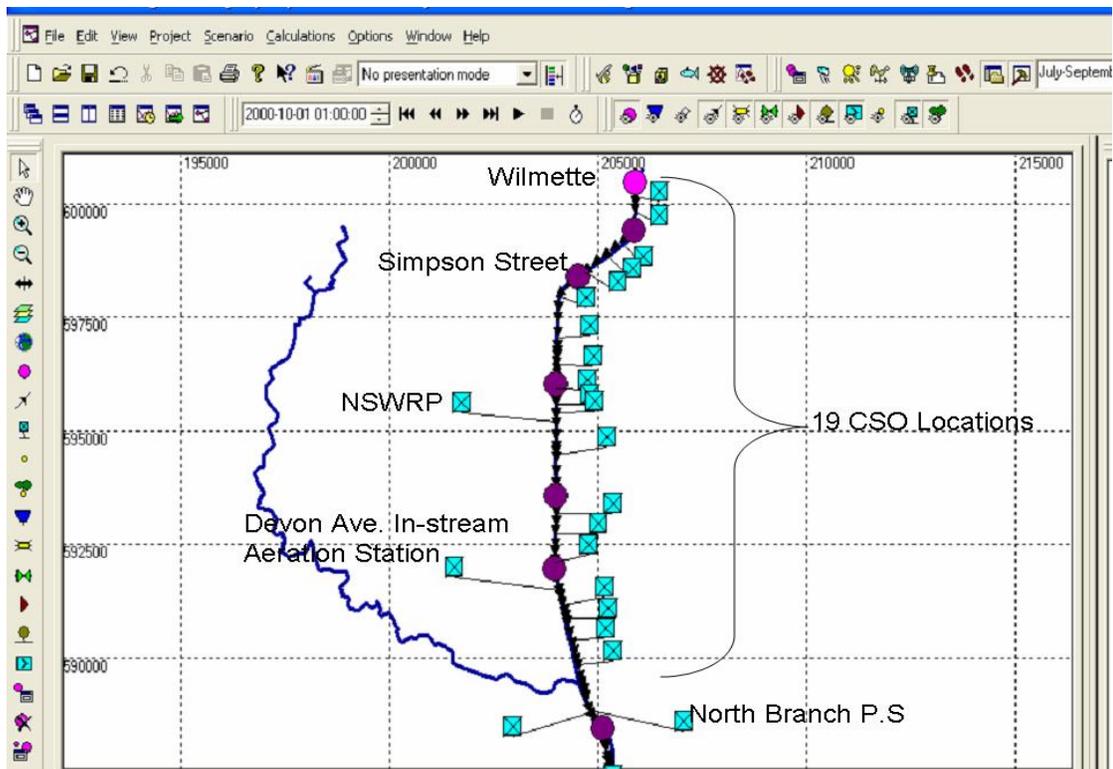
Figure 3. 4 Location of the 19 gravity CSOs on the North Shore Channel in the improved DUFLOW model

In order to improve water quality of the CWS, some improvement methods include: 1) transfer of aerated effluent from the NSWRP to the upstream end of the NSC, 2) transfer of aerated or unaerated flow from the SBCR to the upstream end of the South Fork of the SBCR (commonly known as Bubbly Creek) and supplemental aeration of Bubbly Creek, and 3) addition of supplemental aeration along NBCR, SBCR, CSSC, and Calumet-Sag Channel.

In particular, the goals of this study are to provide modeling support in the development of integrated strategies to meet proposed DO concentrations for at least 90% (Chapter 4) and 100% (Chapter 6) of the time for both the 2001 and 2003 WYs in accordance with IEPA's proposed DO standards. In addition, the MWRDGC developed a proposed set of dissolved oxygen (DO) standards for the CWS that

includes allowable hours for non-compliance because of wet weather. An integrated

strategy of the MWRDGC's DO standards also is developed using the DUFLOW

model (Chapter 5).

3.2. Proposed DO Standards for CWS

Water quality standards are defined for designated aquatic life use of water,

protection of public health, and restoring the quality of water consistent with the

requirements of Clean Water Act. For the CWS, expected water uses include public

water supply, recreation, fishing, and wild life protection. In this thesis, two sets of

proposed DO standards are considered, namely those developed by the IEPA and

presented to the Illinois Pollution Control Board and those developed by the

MWRDGC.

IEPA proposed DO standards

As a result of a Use Attainability Analysis of the CWS the IEPA identified

two aquatic life use classes for the CWS: Chicago Area Waterway System Aquatic

Life Use A waters (CAWS A) and Chicago Area Waterway System and Brandon

Pool Aquatic Life Use B waters (CAWS B) (IEPA, 2007). Figure 3.5 and Table 3.2

show detailed DO standards and the extent of the different waters in the CWS.

Table 3. 2 The DO standards for the aquatic life use designations proposed for the CWS

Designation	DO Standards (mg/L)		
	March-July Hourly minimum	August-February	
		Hourly Minimum	7-day average of daily minima
CAW Aquatic life Use A Waters	5.0	3.5	4.0
CAW and Brandon Pool Aquatic life Use B Waters	3.5	3.5	4.0

In this case, 90% and 100% compliance scenarios are developed in the Chapter 4 and 6, respectively, to determine the locations in the CWS that currently do not meet the listed proposed DO standards 90% and 100% of the time for both WYs 2001 and 2003, and to determine the integrated strategies to comply with those standards.

MWRDGC proposed DO standards

On the basis of historically measured DO concentrations in the various reaches, the total number of hours in the year of periods with DO concentrations less than the DO standard during wet weather periods was developed by the MWRDGC. The District's DO standards are listed in the following Table 3.3. Comparing the two sets of standards, the specific requirements developed by the MWRDGC is not high as those of IEPA. In this case, The DUFLOW model for the 2001 and 2003 Water Years was used to evaluate scenarios for achieving DO concentrations that meet the proposed standards at all locations in the CWS.

Table 3. 3 The proposed DO standards for the CWS developed by the MWRDGC

Waterways	DO standards (mg/L)	Maximum hours of Non-Compliance
North Shore Channel	4.0	600
North Branch Chicago River (upper)	4.0	88
North Branch Chicago River (lower)	3.5	200
Chicago River	3.5	88
South Branch Chicago River	3.5	88
Chicago Sanitary Ship Canal	3.5	500
Little Calumet River North	4.0	320
Little Calumet River South*	3.5	102
Calumet-Sag Channel	3.5	300

*Little Calumet River South was not evaluated in this thesis

Due to some missing effluent quality data from the NSWRP (affecting the NSC through CSSC) for January-April 2003, only October through December 2002 and May through September 2003 were evaluated for the 2003 Water Year. The whole 2003 Water Year was evaluated along the Calumet-Sag Channel and Little Calumet River North which are not affected by the NSWRP loads.

North Side WRP

Upper North Shore Channel

Lower North Shore Channel

North Branch Chicago River

Chicago River Lock & Controlling Works

South Branch Chicago River

Chicago River

Stickney WRP

South Fork South Branch Chicago River (Bubbly Creek)

Calumet River

Chicago Sanitary & Ship Canal

Lake Calumet Connecting Channel

Lake Calumet

Calumet WRP

Grand Calumet River

Calumet-Sag Channel

Little Calumet River

O'Brien Lock & Controlling Works

Lockport Controlling Works

Lockport Powerhouse & Lock

0   7,000   14,000 Feet

0   2.5   5 Miles

Designation	March - July	August - February	
	Daily Minimum (mg/l)	Daily Minimum (mg/l)	7-day Mean of Daily Minima (mg/l)
CAWS Aquatic Life Use A Waters	5.0	3.5	4.0
CAWS and Brandon Pool Aquatic Life Use B Waters	none	3.5	4.0

Chicago Area Waterway Aquatic Life Use Designations

Development of an Integrated Strategy to Meet Dissolved Oxygen
Water Quality Standards for the Chicago Area Waterway System (CAWS)
Expectations Workshop, January 9, 2008

CTE | AECOM

Figure 3. 5 Chicago Area Waterway Aquatic Life Use Designations

3.3. Flow Balance of CWS

Hydraulic model verification for the period of October 1, 2000 -September 30, 2001 (Water Year 2001) and October 1, 2002 -September 30, 2003 (Water Year 2003) was done by Melching et al. (in preparation). As previously stated, the inflow to the CWS consists of flows from tributaries, WRPs, pumping stations, gravity CSOs, and Lake Michigan through the controlling structures. Outflow from the CWS is measured at Romeoville and estimated at the Lockport Controlling Works. In previous studies, flow at Romeoville was studied as the downstream boundary condition for the model, but in this study hourly stage at the Lockport Controlling Works was used as the downstream boundary condition. Due to various reasons, there are some missing data from inflow locations. To deal with this problem, the missing data have been estimated by various mathematical and statistical methods described particularly in Shrestha and Melching (2003) and Melching et al. (in preparation).

3.4. Water-quality Input Data of CWS

Calibration and verification of the DUFLOW water quality model were done for the selected periods of October 1, 2000 to September 30, 2001 and October 1, 2002 to September 30, 2003 (Melching et al., in preparation). The water quality of the CWS is affected by the operation of four Sidestream Elevated Pool Aeration (SEPA) stations and two in-stream aeration stations (Devon Avenue aeration station and Webster Avenue aeration station). The CWS also receives pollutant loads from four WRPs, nearly 240 gravity CSOs (condensed to 43 representative locations to facilitate the modeling), three CSO pumping stations, direct diversions from Lake

Michigan, and eleven tributary streams or drainage areas. Assumptions used to consider the effects of the aeration stations on water quality and to determine the various pollutant loadings are discussed in this section, as are the constituent concentrations for the various inflows to the CWS.

3.4.1. SEPA Stations

The concept of the SEPA stations which applied an artificial aeration, was developed by the MWRDGC beginning in 1984. In early studies, DO concentrations in the CWS historically have been low in accordance with substantial pollutant loading and low in-stream velocities. The SEPA stations involve pumping a portion of the water from the stream into the elevated pool. Water is then aerated by flowing over a cascade or waterfall, and the aerated water is returned to the stream. In this case, totally five SEPA stations are present in the Calumet River System. They are distributed on the Calumet-Sag Channel, Little Calumet River (North), and Calumet River. Four of five SEPA stations are located in the study area for the water-quality modeling. Figure 3.6 and Table 3.4 demonstrate their locations and river miles.
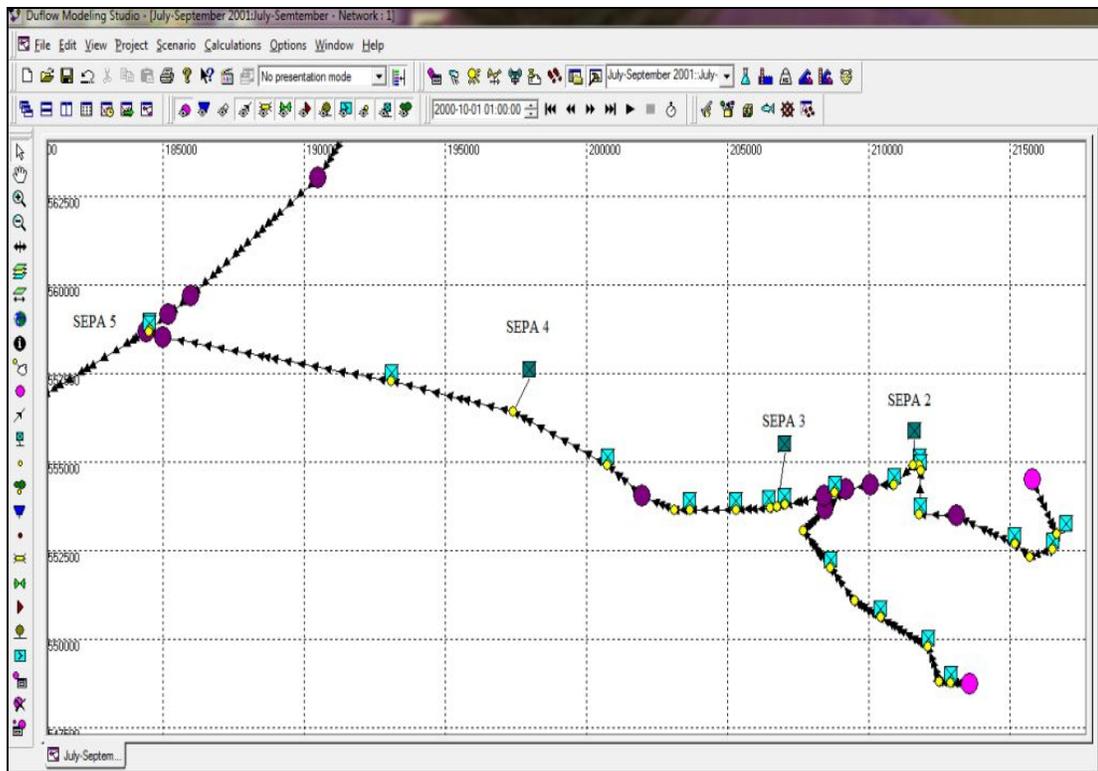
Figure 3. 6 SEPA station locations in the modeled portion of the Chicago River System

Table 3. 4 Locations and river miles of the SEPA stations in the modeled portion of the Chicago River System

SEPA Station No.	Location	River Mile from Lockport
2	127th Street	30.3
3	Blue Island	27
4	Worth (Harlem Avenue)	20.7
5	Sag Junction	12.3

In 1999, the efficiency of the SEPA stations in improving DO concentration along river was examined, and then the calculation method of DO loads from the SEPA stations was introduced in 2000 (Butts et al., 1999 and 2000). This calculation procedure is also used for estimating the oxygen loads from the SEPA stations as follows:

$$\text{Oxygen Load of SEPA} = Q_p \times \alpha \times (C_{sat} - C_{upstream}) \tag{3.1}$$

where:

Load	=	Oxygen load from the SEPA stations [g/s]
Q_p	=	Flow through the SEPA station [m^3/s] (equals the number of operated pumps $\times$ pump capacity)
C_{sat}	=	Saturation concentration of DO [mg/L] (determined from continuous in-stream temperature data)
$C_{upstream}$	=	DO concentration upstream of the SEPA station from continuous in-stream monitoring data or from simulations when evaluating scenarios [mg/L]
α	=	Fraction of saturation achieved (which is a function of the number of operated pumps) described by Butts et al. (1999).

It is worth noting that temperature is one of the key variables, since it affects reaction kinetics and the DO saturation concentrations. Measured water temperature from monitoring locations was input to the model at a one-hour time step.

All the calculated DO loads were input in the DUFLOW water quality simulation directly. For the 90% compliance scenario for IEPA's proposal and MWRDGC proposal, the actual number of operating pumps was used for calculation of the DO loads. However, in order to meet 100% compliance with the IEPA standards, the number of operating pumps in use was assumed to be three (maximum operation). Because the number of SEPA station pumps in operation affects downstream DO concentrations, a summary comparison of the input loads from the SEPA stations for WYs 2001 and 2003 is presented in Tables 3.5 and 3.6, respectively.

Table 3. 5 Characteristics of hourly DO load (g/s) from the SEPA stations for 2001 WY

SEPA No.	Mean		Max		Min		SD	
	Actual pumps used	Assumed 3 pumps used	Actual pumps used	Assumed 3 pumps used	Actual pumps used	Assumed 3 pumps used	Actual pumps used	Assumed 3 pumps used
2	4.03	26.80	11.59	61.11	0.00	11.70	4.05	10.79
3	3.53	21.62	31.51	48.92	0.00	8.54	5.23	5.13
4	7.01	21.41	25.93	42.27	0.00	8.32	6.38	4.51
5	5.22	32.63	24.99	66.30	0.00	10.98	5.29	12.42

Table 3. 6 Characteristics of hourly DO load (g/s) from the SEPA stations for 2003 WY

SEPA No.	Mean		Max		Min		SD	
	Actual pumps used	Assumed 3 pumps used	Actual pumps used	Assumed 3 pumps used	Actual pumps used	Assumed 3 pumps used	Actual pumps used	Assumed 3 pumps used
2	4.04	24.10	26.42	46.91	0.00	9.12	5.25	6.08
3	5.20	19.71	25.51	54.50	0.00	0.00	6.03	5.32
4	4.85	21.54	40.41	49.04	0.00	1.52	6.77	5.16
5	5.85	33.39	56.75	63.98	0.00	4.88	9.91	8.99

3.4.2. In-stream Aeration Stations

There are two diffused aeration stations located in the study area. Due to low DO problems in the past, they were built in 1979 and 1980, respectively. The first one, called the Devon Avenue Aeration Station, is located on the NSC, while another one, called the Webster Avenue Aeration Station, is on the NBCR. The efficiency of DO transfer for the Devon Avenue facility was studied by Polls et al. (1982), then the same DO diffusion process was applied for Webster Avenue facility by Alp and Melching (2004).

DO load from the two diffused aeration stations is calculated based on the DO transfer efficiency of the stations. The actual number of operated blowers was monitored by the MWRDGC. It was used to determine the percentage DO increase from upstream to downstream of the aeration station as per Polls et al. (1982). Unfortunately, only the total number of operating hours per day was provided by the MWRDGC. Because on-and-off times of blowers are unknown, blower operation hours were carefully determined using intervals where increases and decreases in DO concentrations were observed downstream of the aeration stations. The Addison Street and Division Street continuous DO station measurements were used for downstream of the Devon Avenue and Webster Avenue aeration stations, respectively. Discharge and DO concentration upstream of Devon Avenue were calculated using a mass balance approach. The NSWRP and NSC at Main Street continuous DO concentration and discharges were used to calculate DO and discharge upstream of the Devon Avenue aeration station, while the Fullerton Avenue continuous DO monitor was used to estimate Webster Avenue aeration station conditions Alp and Melching (2004) for the model calibration, and simulated upstream values are used when evaluating the integrated strategies for DO improvement. Equation 3.2 describes the calculation of the hourly DO load for the model input.

$$\text{In-stream Aeration Station DO Load} = \%\,DO_{increase} \times DO_{upstream} \times \frac{Q}{100} \qquad (3.2)$$

where:

DO Load   =   Oxygen load from in-stream aeration stations [g/s]

%DO_{increase}   =   Percent DO increase from upstream to downstream of the

aeration station (it is determined by regression equations

between upstream percentage of DO saturation and

downstream DO absorption for a given number of operating

blowers (Polls et al., 1982))

$DO_{upstream}$ =   Measured or simulated DO concentration upstream of the

aeration station [mg/L]

Q =   Discharge at the aeration station [m^3/s]

In this study, to develop an integrated strategy to meet IEPA's proposed DO

standards for both 2001 and 2003 WY, the actual hours and number of operating

blowers at the two in-stream stations was used in the calculation when 90%

compliance scenario needed to be meet; while assumed maximum capacity of

operating blower (3 blowers) was applied for computing SEPA station operations for

the 100% compliance scenario. In terms of the specific DO standards proposed by the

MWRDGC,   the Devon Avenue aeration station was to be operated for additional

106 hours at the maximum capacity (3 blowers on; 64 hours changed from 0 to 3

blowers on, 30 hours changed from 1 to 3 blowers on, and 12 hours changed from 2

to 3 blowers on) in the 2001 Water Year to achieve the desired level of compliance in

the NBCR. No change from the actual operations of the Webster Avenue aeration

station was required for either water year, and no change from the actual operations

of the Devon Avenue aeration station was required for the 2003 Water Year.

3.4.3. Water Reclamation Plants (WRPs)

There are four water reclamation plants (WRPs) whose effluent affects the

water quality as point sources in the CWS: the NSWRP, Stickney WRP, Calumet

WRP, and Lemont WRP. They greatly contributed loads to the entire system. From the measured records at the facilities, daily average concentrations were used in the model. Figures 3.7- 3.14 show daily measured water quality concentrations of the four WRPs for WYs 2001 and 2003, separately. In these figures, the constituents are as follows:

DO   =   Dissolved Oxygen

$CBOD_5$ =   5-day carbonaceous biochemical oxygen demand

TSS   =   Total suspended solids

TKN   =   Total Kjeldahl nitrogen as nitrogen

NH4-N   =   Ammonium as nitrogen

Org-N   =   Organic nitrogen as nitrogen

NO_3-N   =   Nitrate as nitrogen

P-TOT   =   Total phosphorus
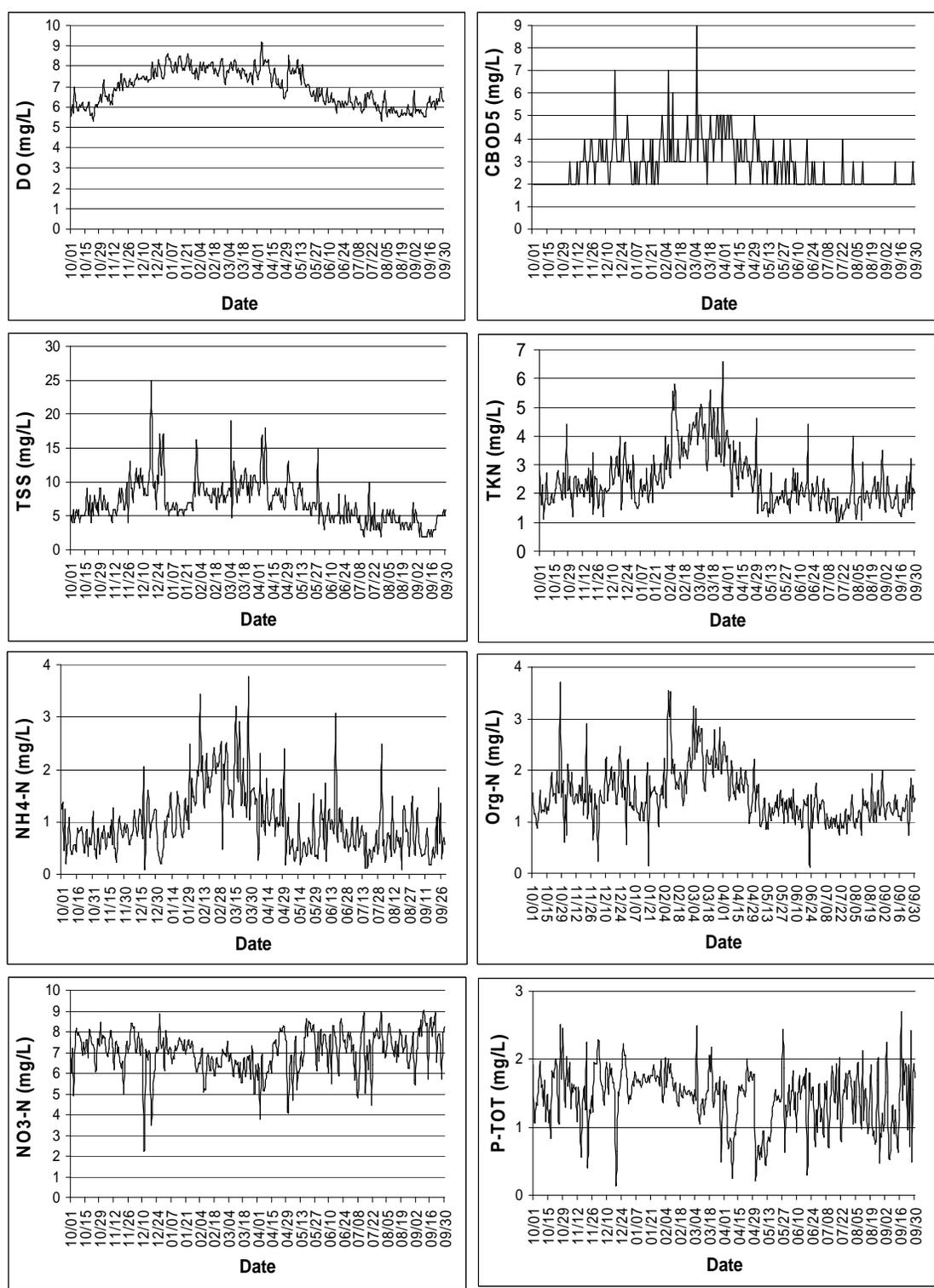
Figure 3. 7 North Side WRP daily effluent measured constituent concentrations for Water Year 2001
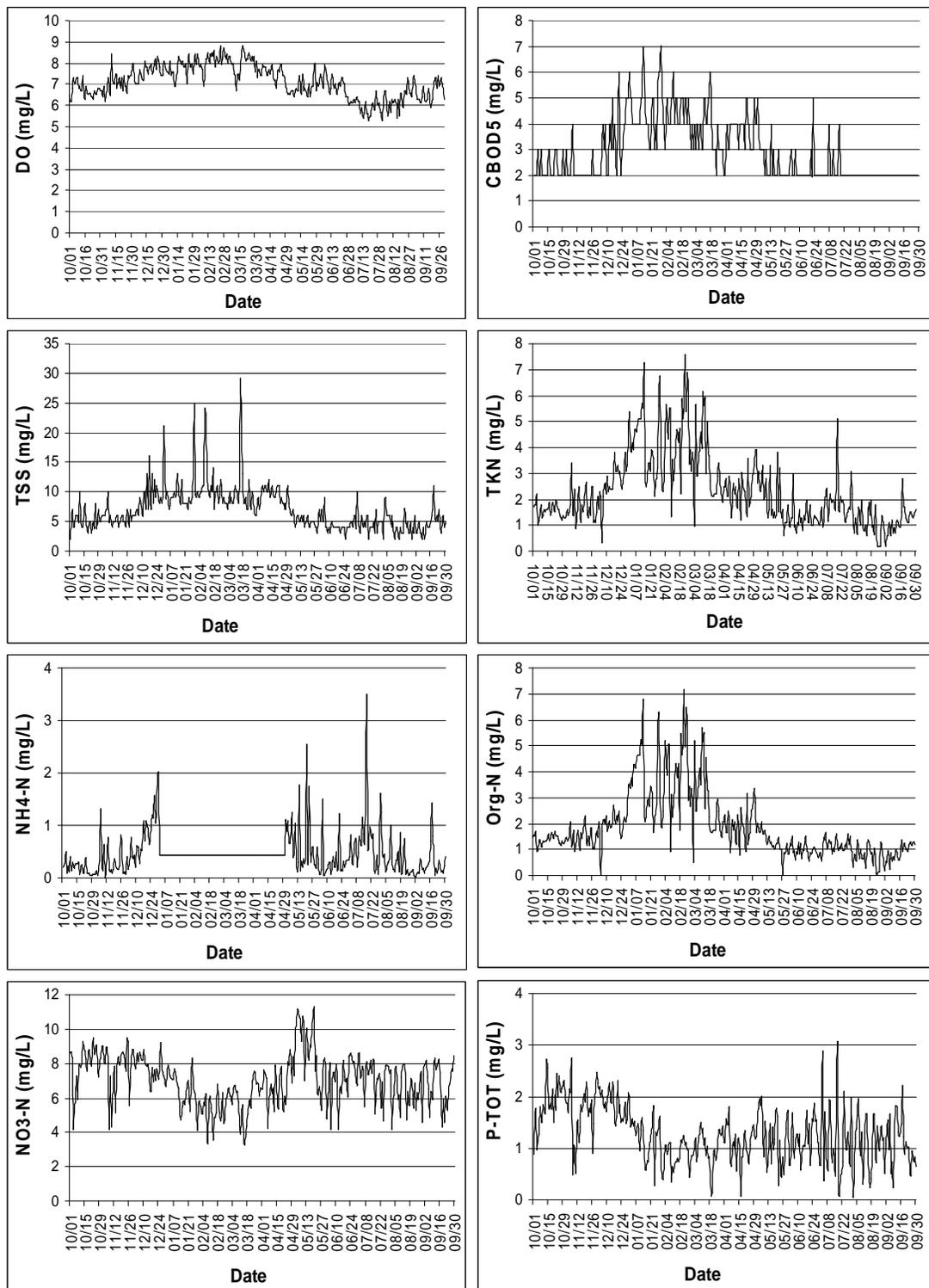
Figure 3. 8 North Side WRP daily effluent measured constituent concentrations for Water Year 2003 (Note: the straight line in the NH4-N (ammonium) concentration from January 1 – April 30 indicates the missing data at the NSWRP)
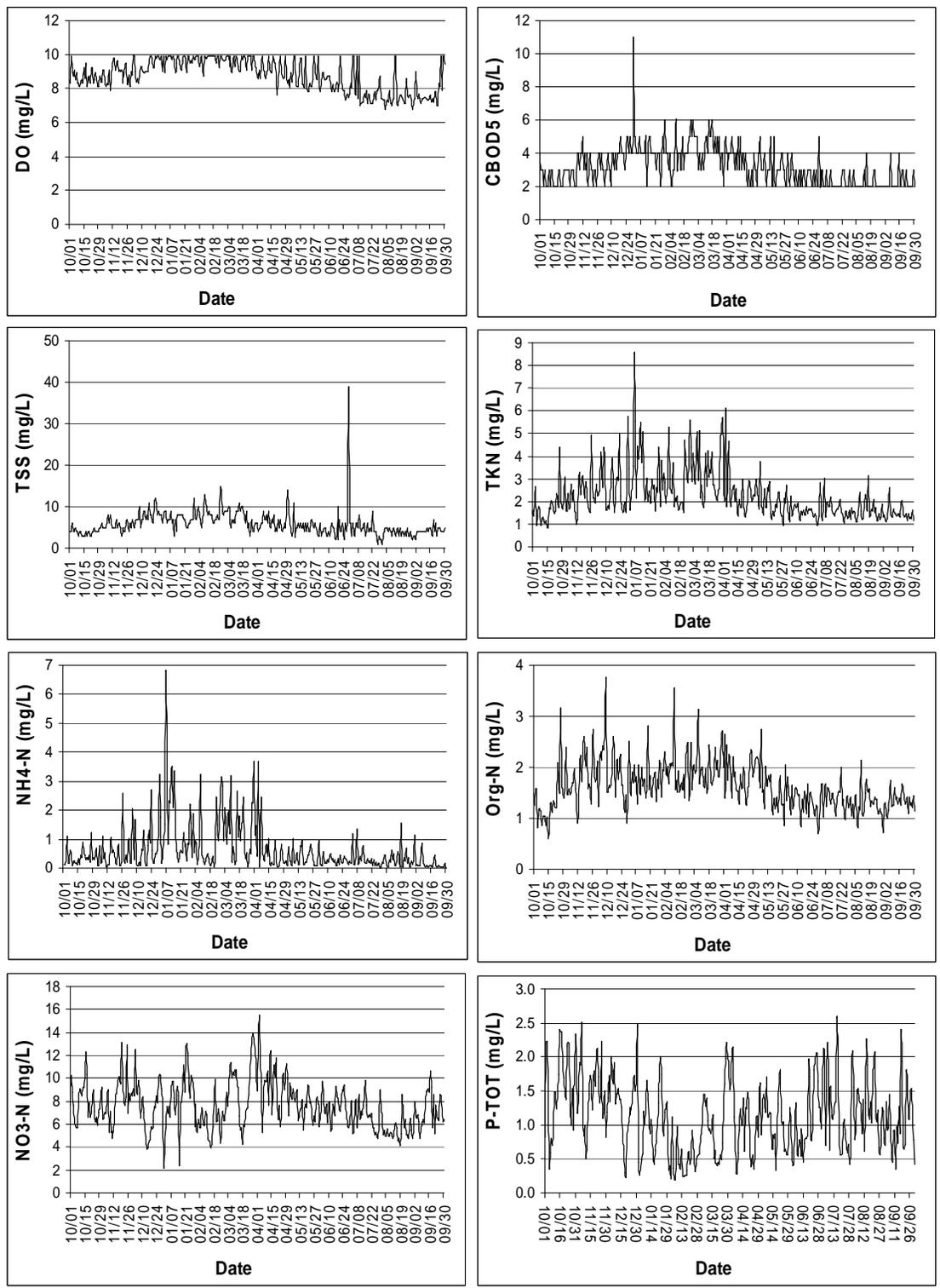
Figure 3. 9 Stickney WPR daily effluent measured constituent concentrations for Water Year 2001
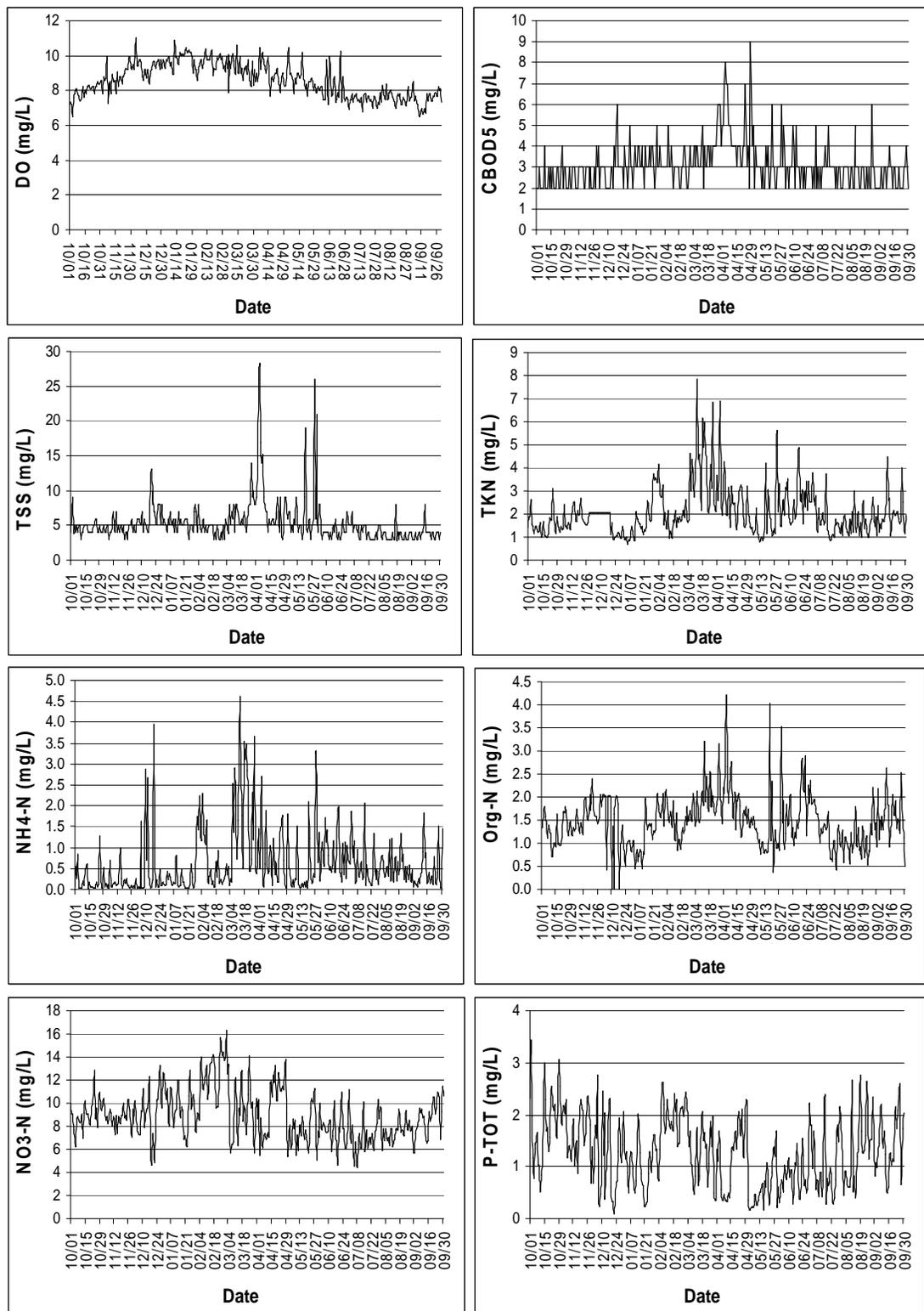
Figure 3. 10 Stickney WRP daily effluent measured constituent concentrations for Water Year 2003
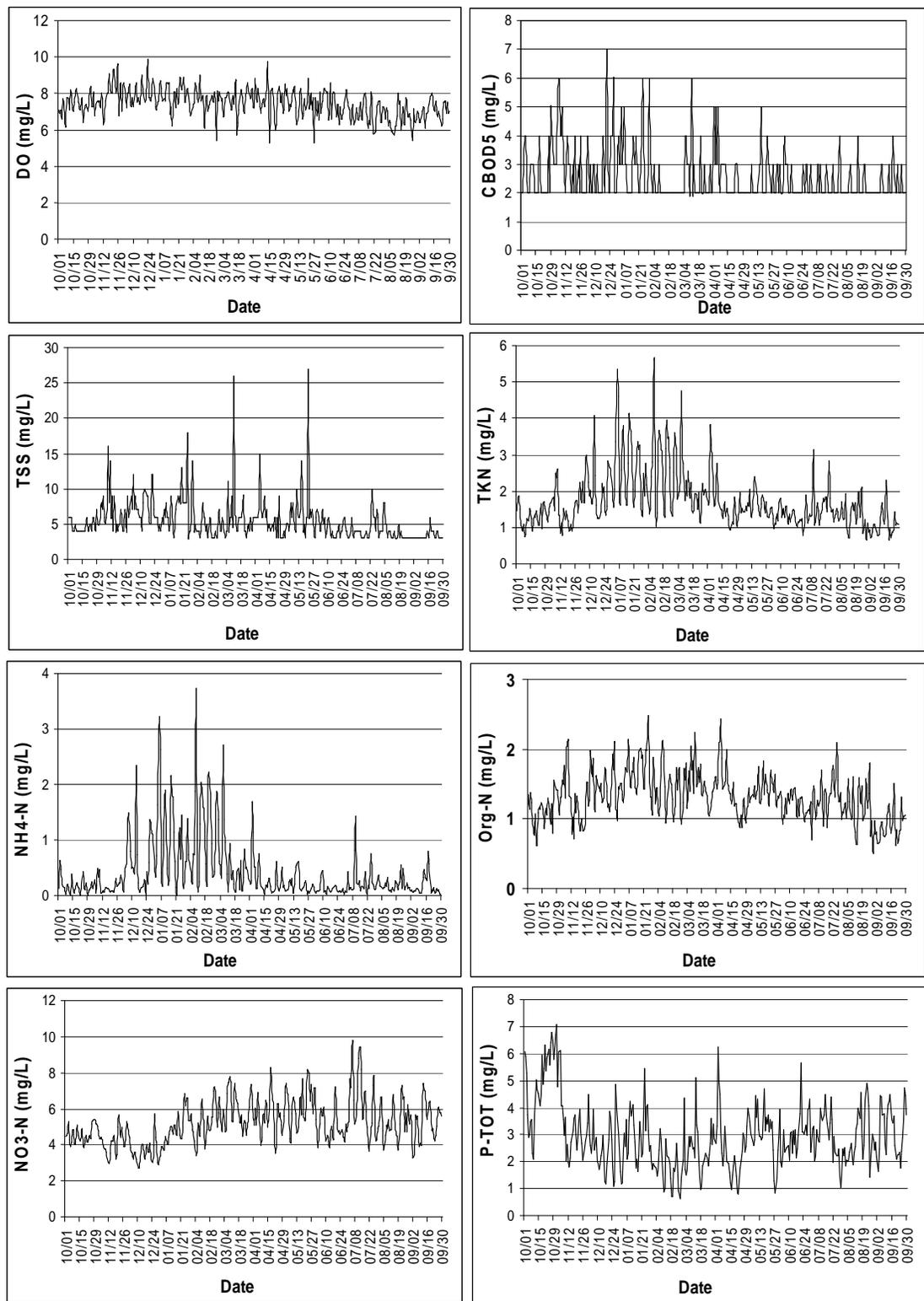
Figure 3. 11 Calumet WRP effluent measured constituent concentrations for Water Year 2001
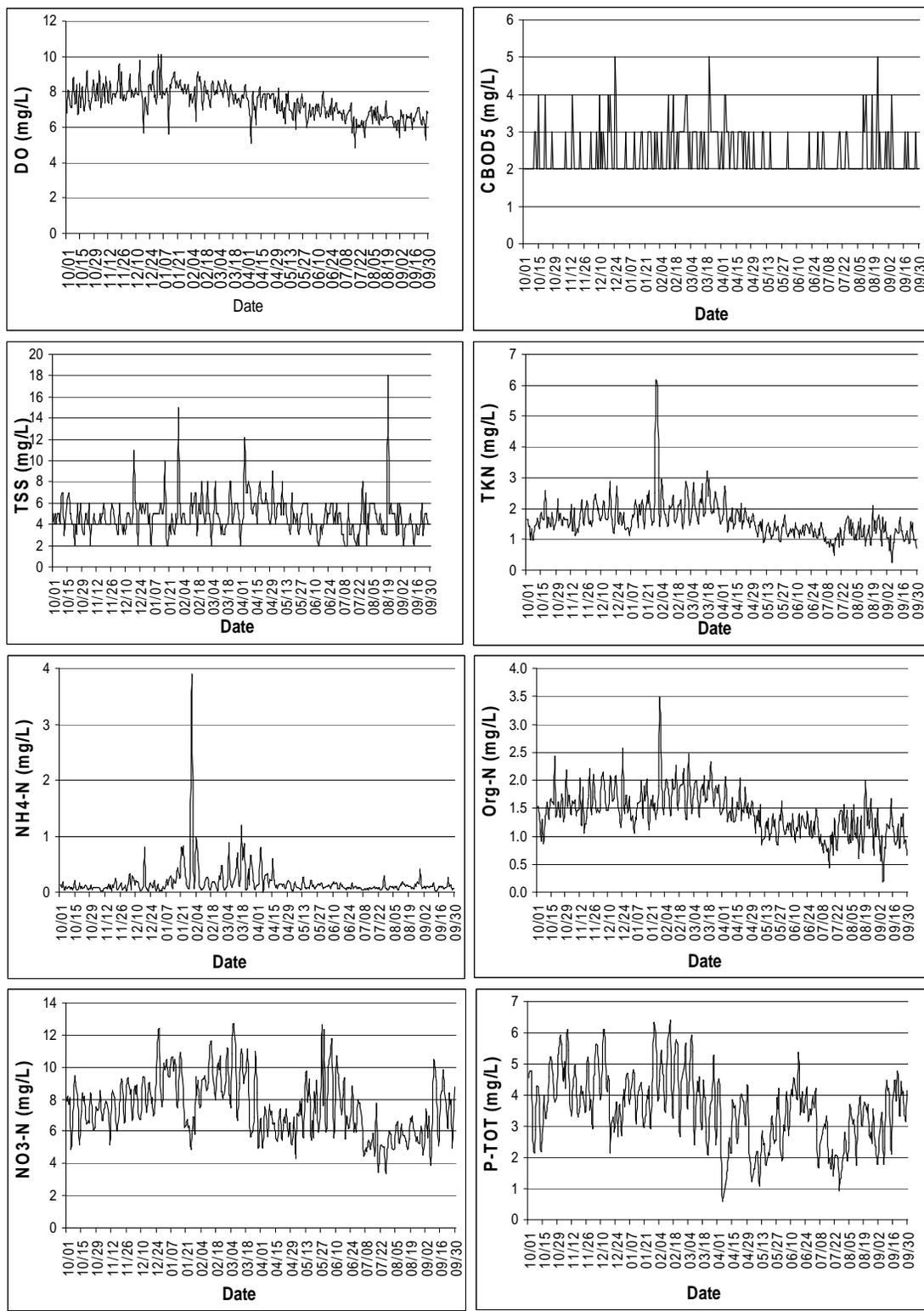
Figure 3. 12 Calumet WRP effluent measured constituent concentrations for Water Year 2003
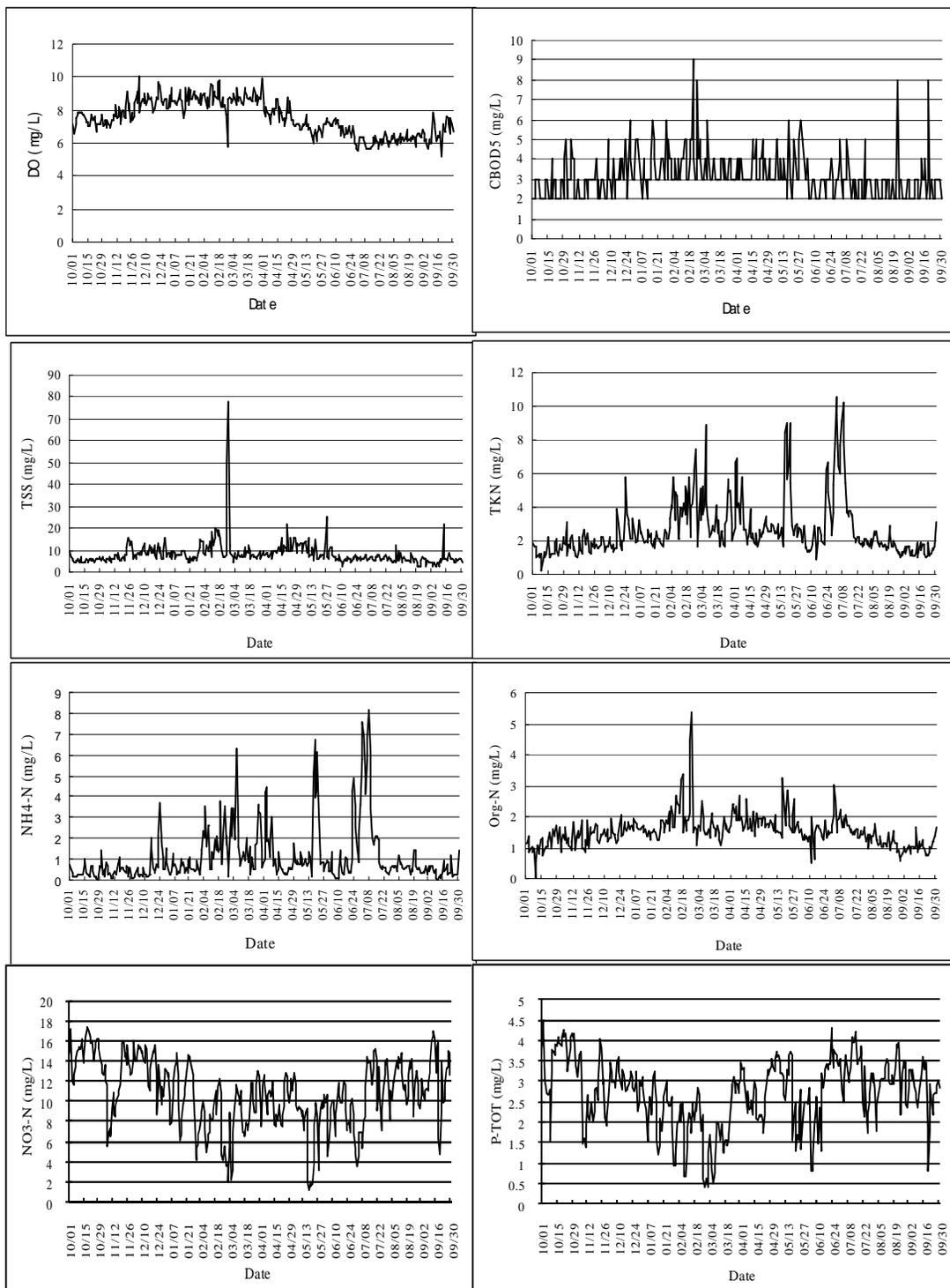
Figure 3. 13 Lemont WRP effluent measured constituent concentrations for Water Year 2001
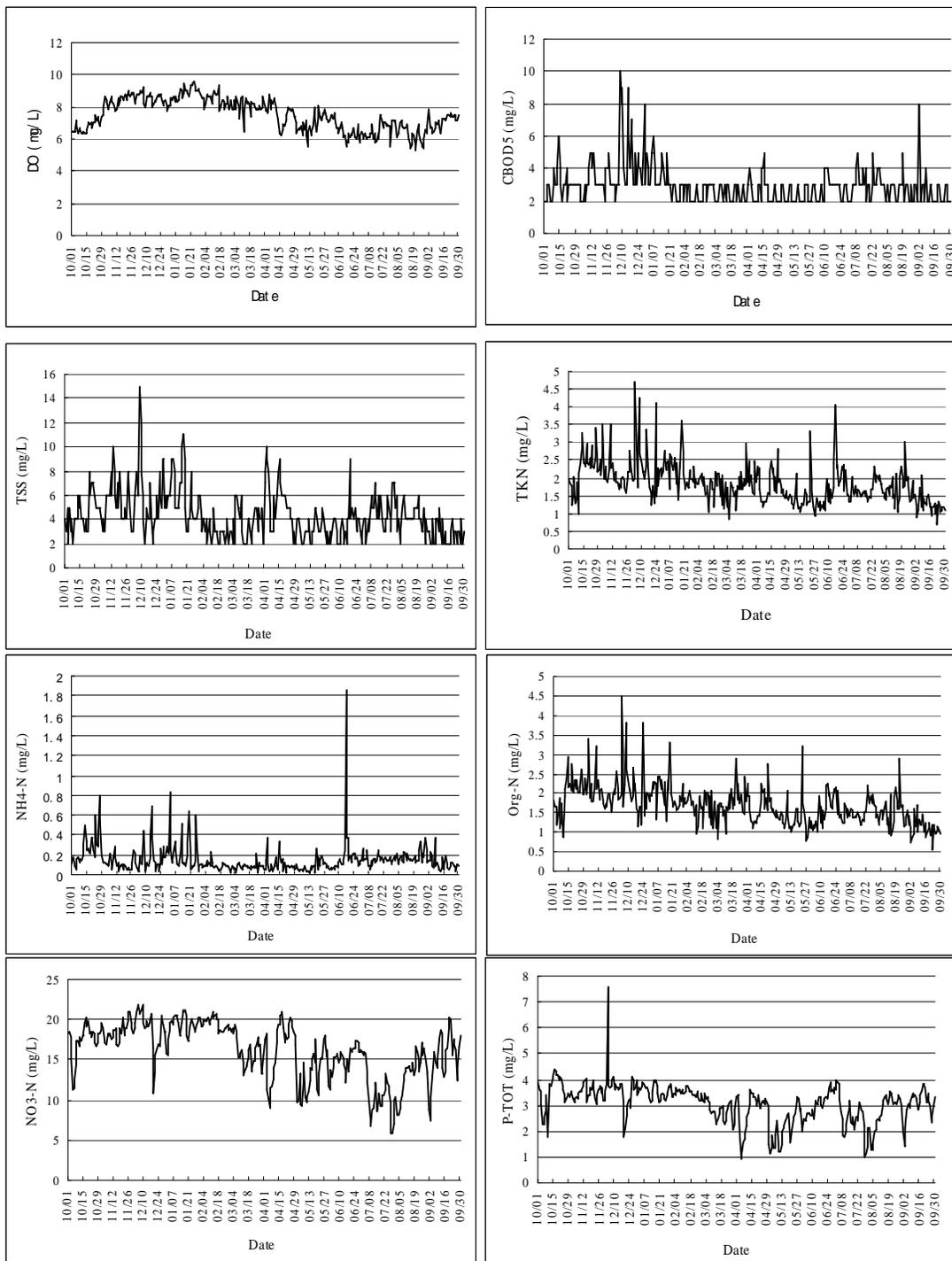
Figure 3. 14 Lemont WRP effluent measured constituent concentrations for Water Year 2003

In the model, inorganic phosphorus and organic phosphorus are the input constituents. Organic phosphorus (P_{org}) in water is related to suspended solids (TSS) and can be estimated by the following equation (eq. 3.3) for model input. Total phosphorous can be measured easily, then the calculation of inorganic phosphorous (P_{inorg}) concentration can be described as the difference between total and organic phosphorous (eq. 3.4).

$$P_{org} = 0.7 \times 0.025 \times TSS \tag{3.3}$$

$$P_{inorg} = P_{total} - P_{org} \tag{3.4}$$

Among these four WRPs, based on the requirements, the North Side WRP and Calumet WRP were applied for flow augmentation to the Wilmette Pumping Station and O'Brien Lock and Dam, respectively, in order to meet different proposed DO standards along the CWS.

3.4.4. Boundaries and Tributaries

Boundaries

In the CWS, there are three upstream boundaries in the water-quality model: 1) at Maple Avenue on the NSC (near the Wilmette Pumping Station); 2) at Columbus Drive on the Chicago River Main Stem (near the CRCW); and 3) at O'Brien Lock and Dam. Measured concentrations of DO, $CBOD_5$, ammonia, nitrate, etc. at Columbus Drive were used in the model. Because flow augmentations were introduced at Wilmette and O'Brien Lock and Dam, water-quality inputs at these two locations needed to be re-calculated hourly on the basis of mass balance of the

transferred effluent and recorded flows instead of using their monthly average measured concentrations.

Tributaries

Totally the pollution loads of 11 tributaries affect the water quality in the CWS. Only three of them (Little Calumet River, Grand Calumet River, and NBCR) were sampled for water-quality constituent concentrations as part of the MWRDGC monthly waterway sampling program. A limited amount of event mean concentration data are available on the Little Calumet River (South) at Ashland Avenue (8 events) and the North Branch Chicago River at Albany Avenue (9 events) in the summer and fall of 2001 (Alp and Melching, 2006). These data were felt to be insufficient to describe storm flows for all events and all tributaries for WYs 2001 and 2003. Thus, in order to be consistent throughout the simulation periods of WYs 2001 and 2003 and use the same kinetic parameters, long-term average in-stream concentrations were used for both wet and dry periods (Melching et al,. in preparation). A detailed description of water quality calculation for the Little Calumet River at South Holland can be found in Alp and Melching (2006). Model input data is listed in Table 3.7, where NO2+NO3-N represents nitrite plus nitrate as nitrogen and Sol-P represents soluble phosphorus. These water-quality constituent concentrations also are used for the unsampled tributary streams on the south side of the CWS.

Table 3. 7 Little Calumet River at South Holland water-quality concentrations

CBOD$_5$ (mg/L)	TSS (mg/L)	DO (mg/L)	TKN (mg/L)	NH$_4$-N (mg/L)	Org-N (mg/L)	P-Tot (mg/L)	NO$_2$+NO$_3$-N (mg/L)	Sol-P (mg/L)
3.15	36.15	*	1.47	0.28	1.18	1.40	5.07	0.97

* Monthly average DO concentrations measured between 2000 and 2004 are used

Concentrations measured between 1990 and 2004 at the Grand Calumet River at Burnham Avenue were used for the concentrations at the Grand Calumet River at Hohman Avenue gage, and are listed in Table 3.8. Average concentrations (2000-2004) for the North Branch Chicago River at Albany Avenue are listed in Table 3.9.

Table 3. 8 Grand Calumet River at Hohman Avenue water-quality concentrations

CBOD$_5$ (mg/L)	TSS (mg/L)	DO (mg/L)	TKN (mg/L)	NH$_4$-N (mg/L)	Org-N (mg/L)	P-Tot (mg/L)	NO$_2$+NO$_3$-N (mg/L)	Sol-P (mg/L)
6.69	34.97	*	4.33	2.01	2.32	0.74	7.73	0.22

* For DO measured hourly concentrations from the Grand Calumet River at Torrence Avenue station were assigned to the inflows on the Grand Calumet River at Hohman Avenue

Table 3. 9 North Branch Chicago River at Albany Avenue water-quality concentrations

CBOD$_5$ (mg/L)	TSS (mg/L)	DO (mg/L)	TKN (mg/L)	NH$_4$-N (mg/L)	Org-N (mg/L)	P-Tot (mg/L)	NO$_2$+NO$_3$-N (mg/L)	Sol-P (mg/L)
4.79	21.41	*	1.38	0.28	1.10	0.93	4.20	0.81

* Monthly average DO concentrations measured between 2000-2004 are used

3.4.5. Combined Sewer Overflows (CSOs)

In the CWS, although nearly 240 gravity CSO locations discharge to the study area, 43 combined and representative CSO locations were selected in the model plus the three CSO pumping stations (PS)-North Branch PS, Racine Avenue PS, and 125th Street PS. For the three CSO pumping stations, average constituent concentrations were calculated based on available historic event mean concentrations measured by

the MWRDGC for each pumping station. Mean water-quality constituent concentrations for the North Branch PS, Racine Avenue PS, and 125th Street PS were applied in the model as listed in Table 3.10. For evaluating IEPA's proposed DO standards, there is a flow transfer from the end of SBCR to the Racine Avenue PS. Thus, a set of new water-quality constituents was calculated by mass balance for model input at the Racine Avenue PS. The detailed approach is explained in Chapter 4.

Table 3. 10 The mean values of the event mean concentrations for pumping stations discharging to the Chicago Waterway System

	Constituent	Average (mg/L)
North Branch Pumping Station	DO	4.0
	$CBOD_5$	35.4
	NH_4-N	2.9
	NO_3-N	0.7
	Org-N	6.1
	Org-P	1.0
	In-P	0.4
	TSS	102
Racine Avenue Pumping Station	DO	6.9
	$CBOD_5$	51.2
	NH_4-N	1.6
	NO_3-N	0.8
	Org-N	4.1
	Org-P	0.2
	In-P	0.7
	TSS	825
125th Street Pumping Station	DO	4.3
	$CBOD_5$	25.7
	NH_4-N	1.0
	NO_3-N	1.8
	Org-N	3.6
	Org-P	0.4
	In-P	1.3
	TSS	76

CHAPTER 4: SIMULATION RESULTS TO MEET IEPA PROPOSED DO
STANDARDS 90% OF THE TIME

4.1. Introduction

In this chapter, simulation scenarios involving NSC and Bubbly Creek flow augmentation are presented. Two baseline simulations for the 2001 and 2003 WYs are first considered. Both of these two baseline simulations considered using actual blower operations at the Devon Avenue and Webster Avenue in-stream aeration stations and actual pump operations at the four SEPA stations. The first step in developing the 90% compliance scenario is to determine the locations in the CWS that currently do not meet the proposed dissolved oxygen (DO) standards 90% of the time based on baseline simulations and measured data for WYs 2001 and 2003.

4.2. Statement of The Problem

Figures 4.1 and 4.2 show the percentage compliance with the proposed DO standards achieved by the measured and simulated DO concentrations for the 2001 and 2003 WYs, respectively, along the NSC, NBCR, SBCR, and CSSC. Figures 4.3 and 4.4 show the percentage compliance with the proposed DO standards achieved by the measured and simulated DO concentrations for WYs 2001 and 2003, respectively, along the Little Calumet River (North) and Calumet-Sag Channel. (Note: the last point which was not marked in the Figures 4.1 and 4.2 is Linden Street (River Mile 49.8 from Lockport and the river mile of Lockport is 291 from Grafton at the mouth of the Illinois River). Figure 4.5 shows the percentage compliance with the proposed DO standards achieved by the simulated DO concentrations for WYs 2001 and 2003

Figure 4. 2 Simulated and measured compliance with the IEPA proposed DO standards for WY 2003 along the NSC, NBCR, SBCR, and CSSC

Figure 4. 3 Simulated and measured compliance with the IEPA proposed DO standards for WY 2001 along the Little Calumet River (North) and Calumet-Sag Channel
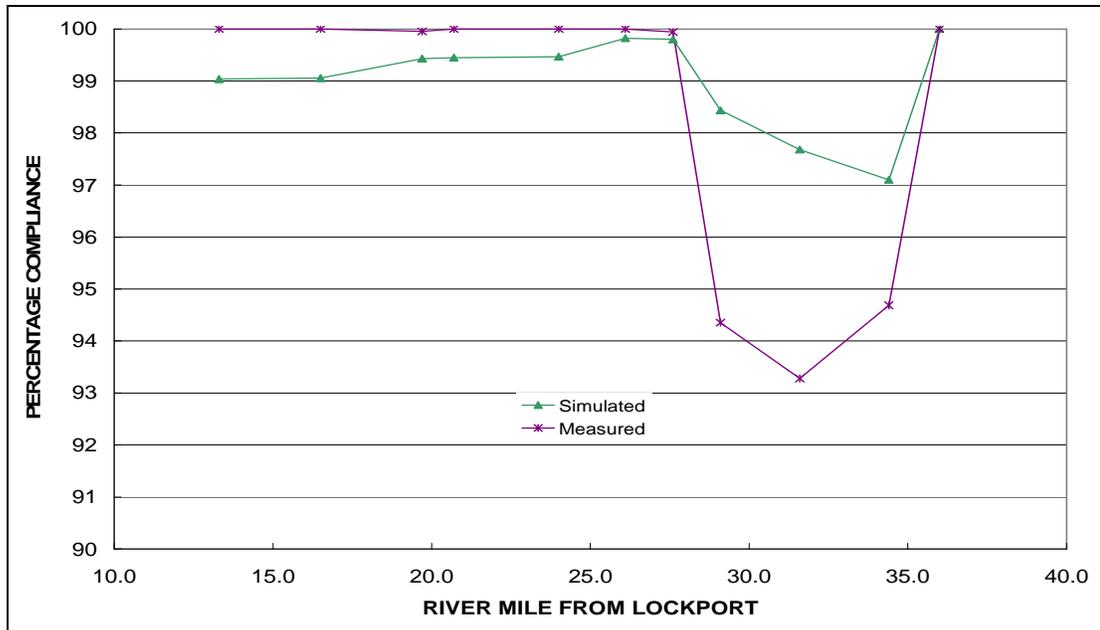
Figure 4. 4 Simulated and measured compliance with the IEPA proposed DO standards for WY 2003 along the Little Calumet River (North) and Calumet-Sag Channel
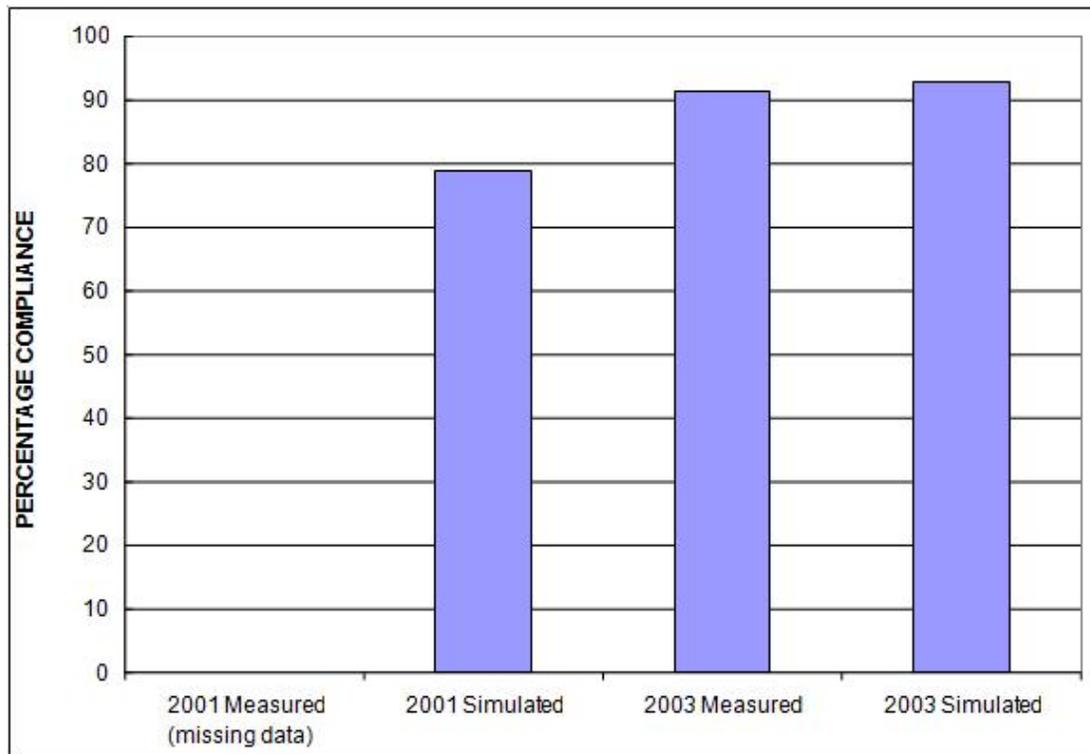
Figure 4. 5 Measured and simulated compliance with the IEPA proposed DO standards on Bubbly Creek at Interstate 55.

Table 4. 1 River miles of key locations in the Chicago River System

	Linden Street	Simpson Street	Main Street	Fullerton Avenue	Jackson Boulevard	Cicero Avenue
River Mile	49.8	48.5	46.5	37.9	34	26.3

Two aspects in the foregoing figures require further consideration: i) the measured DO concentrations at Fullerton Avenue (on the NBCR) do not meet the proposed DO standards 90% of the time (86.7% compliance for both WYs 2001 and 2003), whereas the simulated DO concentrations meet the proposed DO standards 90% of the time; ii) similarly, the measured percentage compliance is far smaller than the simulated percentage compliance for Main Street and Simpson Street on the upper NSC for WY 2001 while the simulated percentage compliance is lower than the measured percentage compliance for WY 2003. Three factors can affect the differences in the percentage compliance between the simulated and measured DO concentrations.

1)	Missing measured data—the simulations yield DO concentrations for every hour in the WY under consideration, whereas at each measurement location some data are missing throughout the year. If data were missing during a period of compliance, the compliance computed for the year would be lower than the actual compliance. Table 4.2 lists the percentage of missing data for each DO monitoring location in the CWS. Note the large percentages of missing data in WY 2001 in the Little Calumet River (North) and Calumet-Sag Channel is because these monitors were installed in July 2001.

2)	Model error relative to the measured DO concentrations.

3)   Error in the measured DO concentrations relative to the true cross sectional average DO concentrations.

Each listed location which does not meet compliance 90% of the time is discussed in the following sections.

Table 4. 2 Percentage of missing data for WYs 2001 and 2003 for the DO monitoring locations in the CWS

Location	Waterway	2001	2003
Linden Street	North Shore Channel	34.84	2.02
Simpson Street	North Shore Channel	7.00	24.13
Main Street	North Shore Channel	6.43	4.89
Addison Street	North Branch Chicago River	2.01	5.24
Fullerton Avenue	North Branch Chicago River	3.92	7.60
Division Street	North Branch Chicago River	2.00	1.99
Kinzie Street	North Branch Chicago River	0.07	0.02
Chicago River Controlling Works	Chicago River	4.02	2.28
Michigan Avenue	Chicago River	36.05	4.57
Clark Street	Chicago River	0.09	1.96
Jackson Boulevard	South Branch Chicago River	2.18	0.01
Interstate 55	Bubbly Creek	100.0	5.78
Cicero Avenue	Chicago Sanitary and Ship Canal	0.35	11.65
Baltimore and Ohio Railroad	Chicago Sanitary and Ship Canal	3.21	8.34
River Mile 11.6	Chicago Sanitary and Ship Canal	4.83	5.65
Romeoville	Chicago Sanitary and Ship Canal	3.32	3.90
130th Street	Calumet River	78.93	14.89
Conrail Railroad	Little Calumet River (north)	79.54	19.19
Central and Wisconsin Railroad	Little Calumet River (north)	77.66	1.63
Halsted Avenue	Little Calumet River (north)	77.68	1.96
Division Street	Calumet-Sag Channel	77.66	1.93
Kedzie Street	Calumet-Sag Channel	77.67	3.87
Cicero Avenue	Calumet-Sag Channel	79.59	1.94
River Mile 20.7	Calumet-Sag Channel	81.50	10.32
Southwest Highway	Calumet-Sag Channel	85.33	8.00
104th Avenue	Calumet-Sag Channel	80.23	12.05
Route 83	Calumet-Sag Channel	4.04	21.12

4.3. Flow Augmentation for the Upper North Shore Channel

The first step of improving DO concentrations in the NSC is to transfer 30 MGD of aerated effluent from the NSWRP to the Wilmette Pumping Station. Figure 4.6 shows the percentage compliance with the proposed DO standards on the Upper North Shore Channel (UNSC) at Main Street as a function of the transferred amount of aerated effluent from the North Side Water Reclamation Plant to the upstream end of the NSC at Wilmette.   From Figure 4.6, it can be seen that transfer of 29 MGD is needed to achieve at least 90% compliance at Main Street for both WYs 2001 and 2003. Further, a transfer of 30 MGD is needed to achieve at least 90% compliance throughout the entire UNSC. This transfer of 30 MGD is far smaller than the 90 MGD needed to achieve 90% compliance with a DO standard of 5 mg/L at Main Street reported in Alp and Melching (2006) or the 100 MGD needed to achieve 90% compliance with a DO standard of 5 mg/L throughout the UNSC (CTE, 2007 c). This large difference results from the fact that in the proposed DO standards 5 mg/L does not need to be met in the critical periods, such as August, September, and October, compared to the case evaluated by Alp and Melching (2006) and CTE (2007). Therefore, a 30 MGD transfer of aerated flow was implemented considering for both water years.
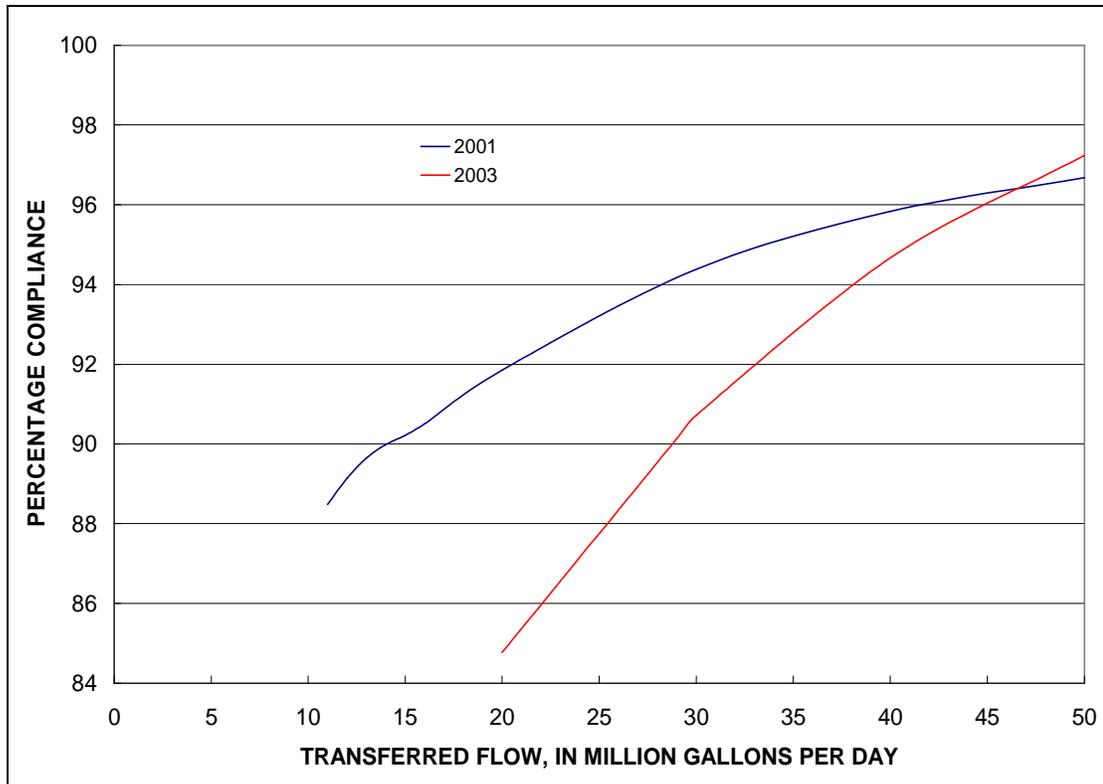
Figure 4. 6 Percentage compliance with the IEPA proposed DO standards at Main Street on the Upper NSC as a function of the transfer of aerated effluent from the NSWRP to the upstream end of the NSC.

The simulated results with a 30 MGD transfer of aerated flow for Linden Street, Simpson Street, and Main Street show the obvious improvement of DO concentrations in the upper NSC for both WYs 2001 and 2003 (see Figures 4.7-4.8). It can be seen that 30 MGD flow augmentation with aerated effluent can achieve compliance 90% of the time during dry and wet years (see Table 6.1).
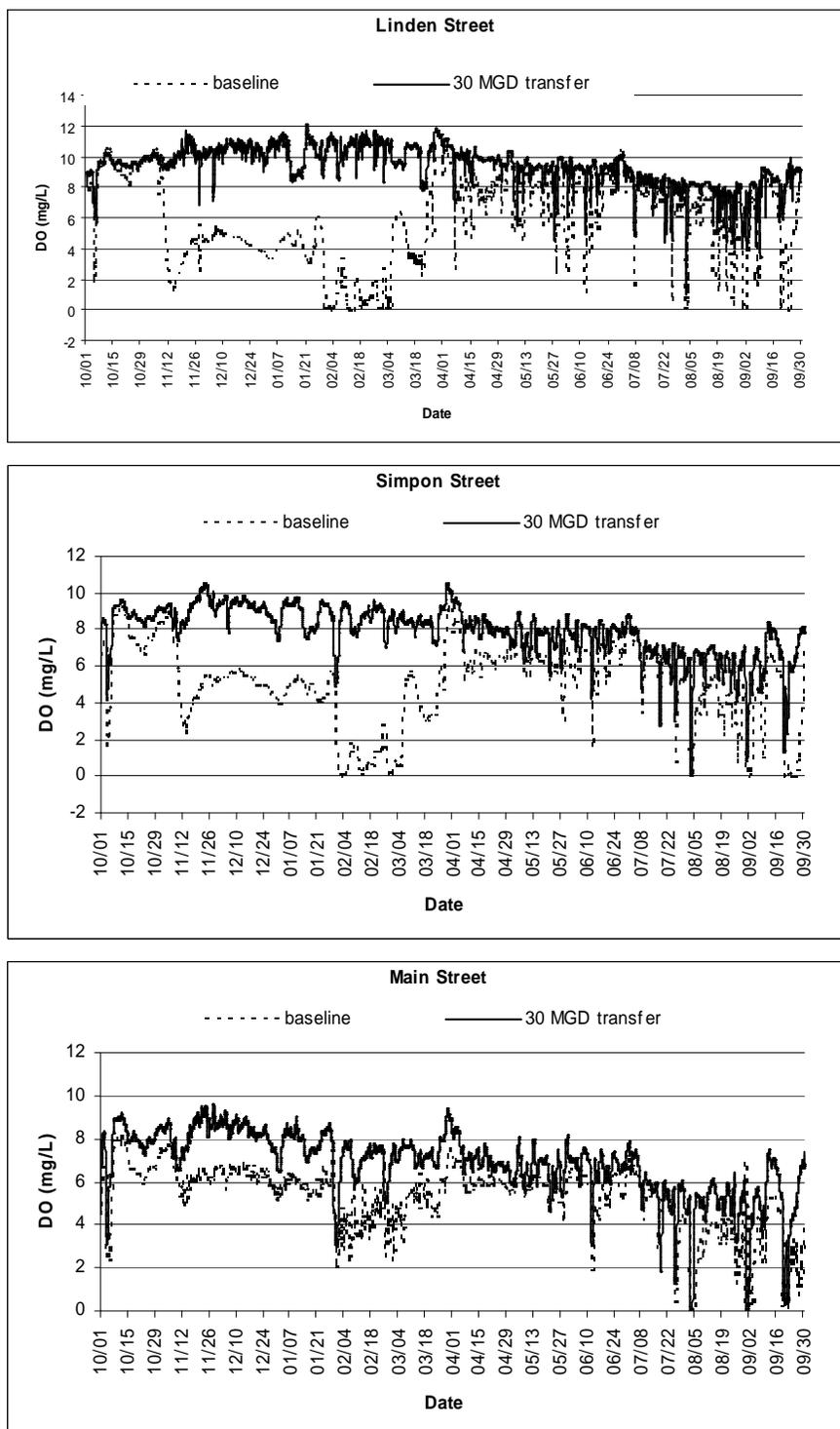
Figure 4. 7 Simulated hourly DO concentrations at Linden Street, Simpson Street, and Main Street on the NSC for a 30 MGD transfer of aerated effluent from the NSWRP to the upstream end of the NSC compared with baseline simulated concentrations for WY 2001

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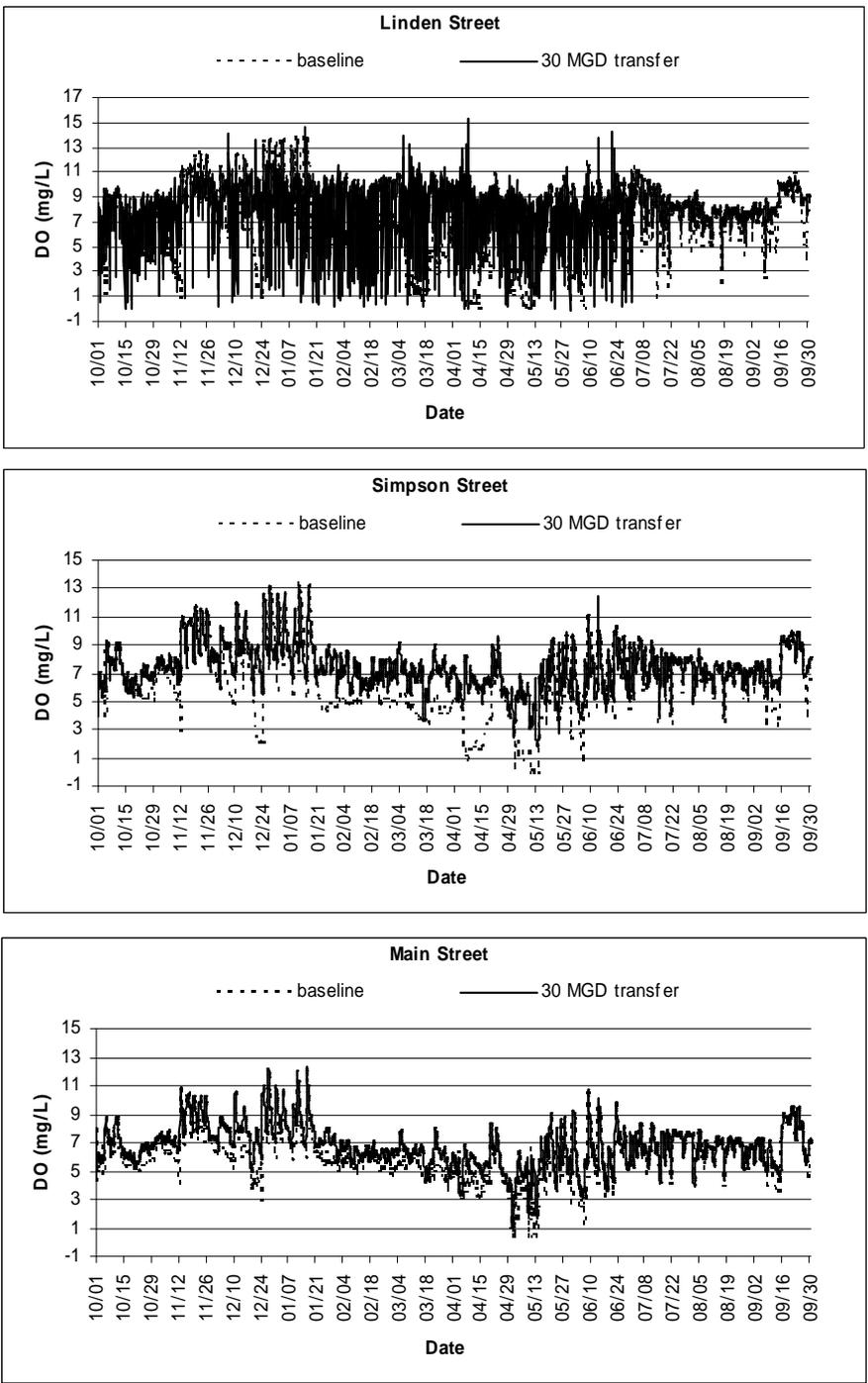
Figure 4. 8 Simulated DO concentrations at Linden Street, Simpson Street, and Main Street on the NSC for a 30 MGD transfer of aerated effluent from the NSWRP to the upstream end of the NSC compared with baseline simulated concentrations for WY 2003

4.4. Flow Augmentation for Bubbly Creek

In order to increase DO concentrations to meet IEPA's standards 90% of the time on Bubbly Creek, flow augmentation from the SBCR to the upstream end of Bubbly Creek (Racine Avenue Pumping Station) should be introduced. The withdrawal point for flow augmentation for Bubbly Creek is the intersection of the SBCR and Throop Street. This point is slightly upstream of the junction of Bubbly Creek and the SBCR (approximately 0.4 miles).

When considering this flow transfer, the maximum amount of the transfer is limited to a flow that will not scour the bottom sediments in Bubbly Creek. The sediment quality in Bubbly Creek is considered to be very poor and resuspension of these sediments would substantially degrade water quality in Bubbly Creek and the CSSC. The two-dimensional (2-D) and three-dimensional (3-D) modeling of water quality in Bubbly Creek being done by the University of Illinois at Urbana-Champaign (UIUC) and related measurements of sediment mobility may eventually define a best estimate of the true upper bound on flow transfer for Bubbly Creek. However, in this study the best available information was used to set the maximum flow transfer. On the basis of preliminary runs of the 2-D model, Motta et al. (2009) suggested that for a recirculation discharge of 50 MGD sediment resuspension from the bed is avoided. In 2003, the MWRDGC conducted a series of field tests of creating flow in Bubbly Creek by drawing water from the creek into the Racine Avenue Pumping Station and sending it to the Stickney WRP for treatment. In these experiments, Bubbly Creek flow would be maintained at 38 MGD for six days or 75 MGD for five days during each demonstration event (Sopcek, 2004). Since sediment

resuspension was not reported as a product of these demonstration events, 75 MGD

has been set as the maximum flow transfer in the simulations evaluated in this case.

Two sets of simulations considering diversion of a portion of SBCR flow to the

upstream end of Bubbly Creek are done: aerated flow transfer and unaerated flow

transfer.

4.4.1. Flow Transfer with Aeration

In this section, simulation scenarios of Bubbly Creek flow augmentation with

aeration are presented. As was done by Alp and Melching (2006), flow was

withdrawn from the SBCR at Throop Street, aerated to saturation, and inserted at the

upstream end of Bubbly Creek. In order to compute the saturated DO concentration,

the water temperature at Throop Street was determined by linear interpolation from

the hourly temperature data at Jackson Boulevard and Cicero Avenue (the nearest

upstream and downstream, respectively, monitoring stations for the time periods

under consideration). The concentrations of all other constituents in the transferred

flow were the computed values for Throop Street assuming an aerated flow transfer

of 30 MGD on the upper NSC and the actual operations of the Devon Avenue and

Webster Avenue in-stream aeration stations.

Different amounts of aerated flow transfer for WYs 2001 and 2003 were

applied to determine the optimal amount of flow augmentation. Figures 4.9 and 4.10

show the percentage compliance along Bubbly Creek for various aerated flow transfer

amounts for WYs 2001 and 2003, respectively. Note that I-55 and 36th St. represent

the locations of the Interstate-55 and 36th Street DO monitors. Results of the various

aerated flow augmentation simulations from the figures show that the aerated flow transfers improve DO conditions in Bubbly Creek. As can be seen from the figures, an aerated flow transfer of 10 MGD achieves at least 90% compliance along all of Bubbly Creek for both WYs 2001 and 2003. Further this transfer raises the compliance at Cicero Avenue to 91.6% for WY 2001.

It also can be seen in Fig. 4.9 that for WY 2001 a transfer of 10 MGD of aerated flow to the upstream end of Bubbly Creek yields a minimum percentage compliance of 90.13% (at 36th Street) whereas a transfer of 75 MGD of aerated flow yields a minimum compliance of 92.83% (at the junction with the CSSC). In order to achieve 90% compliance, it can be seen that 90.13% compliance supplemental aeration would required for about 36 days, whereas for 92.83% compliance supplemental aeration would be needed for about 26 days. Thus, a transfer of 7.5 times more flow would only reduce the time that supplemental aeration is needed by 10 days. It seems that these 10 days can more effectively be raised to full compliance via supplemental aeration. Thus, for the 100% compliance scenario in Chapter 6 a transfer of 10 MGD of aerated flow is applied.
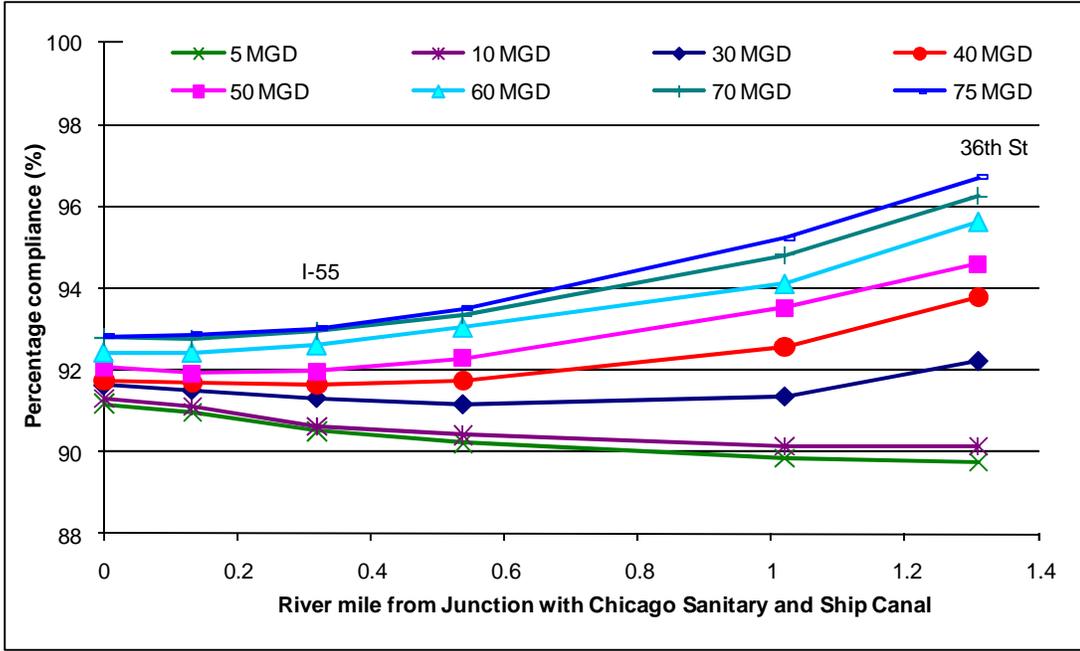
Figure 4. 9 Simulated compliance with the IEPA proposed DO standards for WY 2001 along Bubbly Creek for different amounts (in million gallons per day, MGD) of aerated flow transfer
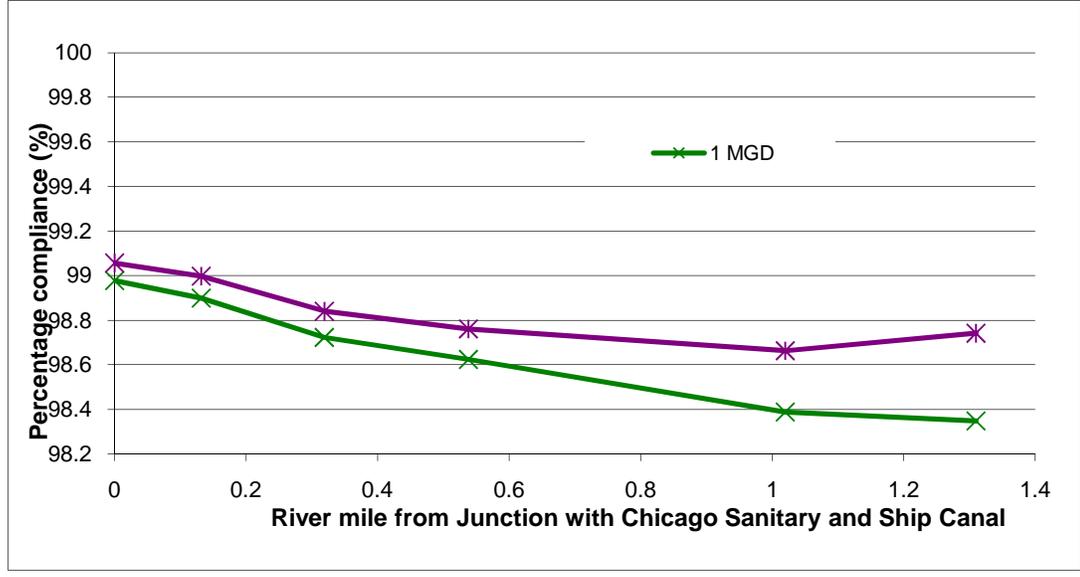
Figure 4. 10 Simulated compliance with the IEPA proposed DO standards for WY 2003 along Bubbly Creek for different amounts (in million gallons per day, MGD) of aerated flow transfer

4.4.2. Flow Transfer without Aeration

In this section, simulations of scenarios of Bubbly Creek flow augmentation without aeration are presented. For the evaluation of unaerated flow transfer, the simulated concentrations of all water-quality constituents, including DO, at Throop Street were used for the transferred flows. The concentrations of all constituents were computed assuming an aerated flow transfer of 30 MGD on the upper NSC and the existing operations of the Devon Avenue and Webster Avenue in-stream aeration stations.

Figures 4.11 and 4.12 show the percentage compliance along Bubbly Creek for different amounts of unaerated flow transfer for WYs 2001 and 2003, respectively. For WY 2003, just the transfer of 30 MGD of aerated flow on the upper NSC results in greater than 90% compliance with the proposed DO standard throughout Bubbly Creek. Whereas, for WY 2001, a transfer of 70 MGD of unaerated flow from Throop Street to the upstream end of Bubbly Creek results in 90% compliance with the proposed DO standard throughout Bubbly Creek. Further the transfer of 70 MGD raises the compliance at Cicero Avenue to 92.5% for WY 2001.
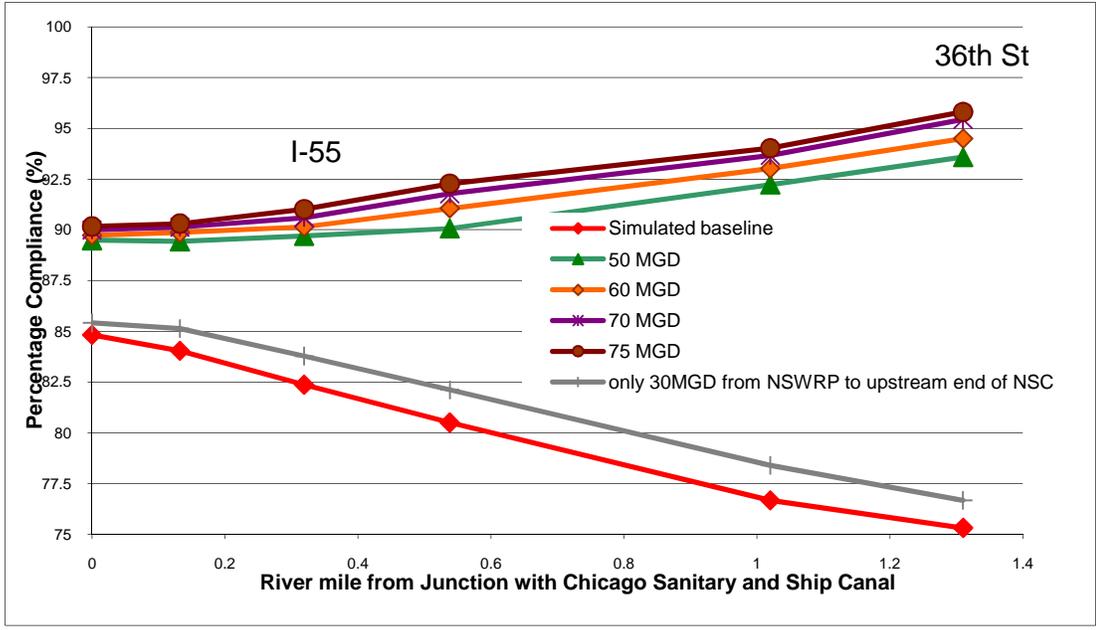
Figure 4. 11 Simulated compliance with the IEPA proposed DO standards for WY 2001 along Bubbly Creek for different amounts (in million gallons per day, MGD) of unaerated flow transfer
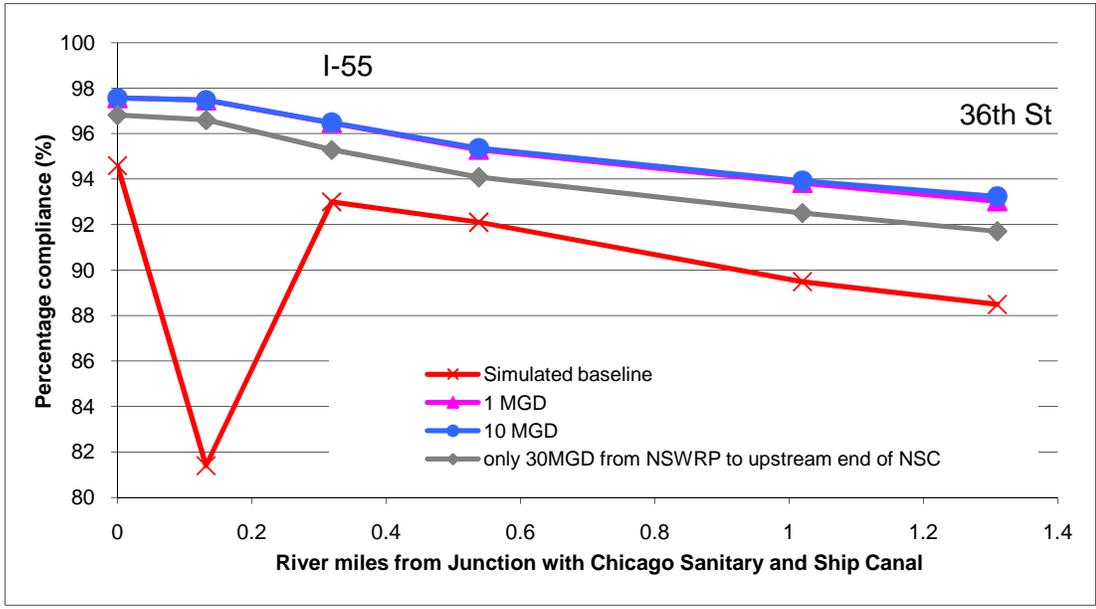
Figure 4. 12 Simulated compliance with the IEPA proposed DO standards for WY 2003 along Bubbly Creek for different amounts (in million gallons per day, MGD) of unaerated flow transfer

In addition, the DO concentration for a 10 MGD flow transfer with aeration and a 70 MGD flow transfer without aeration at I-55 for 2001 WY is shown in Figure 4.13. It can be seen that flow augmentation with aeration can improve DO concentration more effectively, compared to unaerated flow transfer on Bubbly Creek.
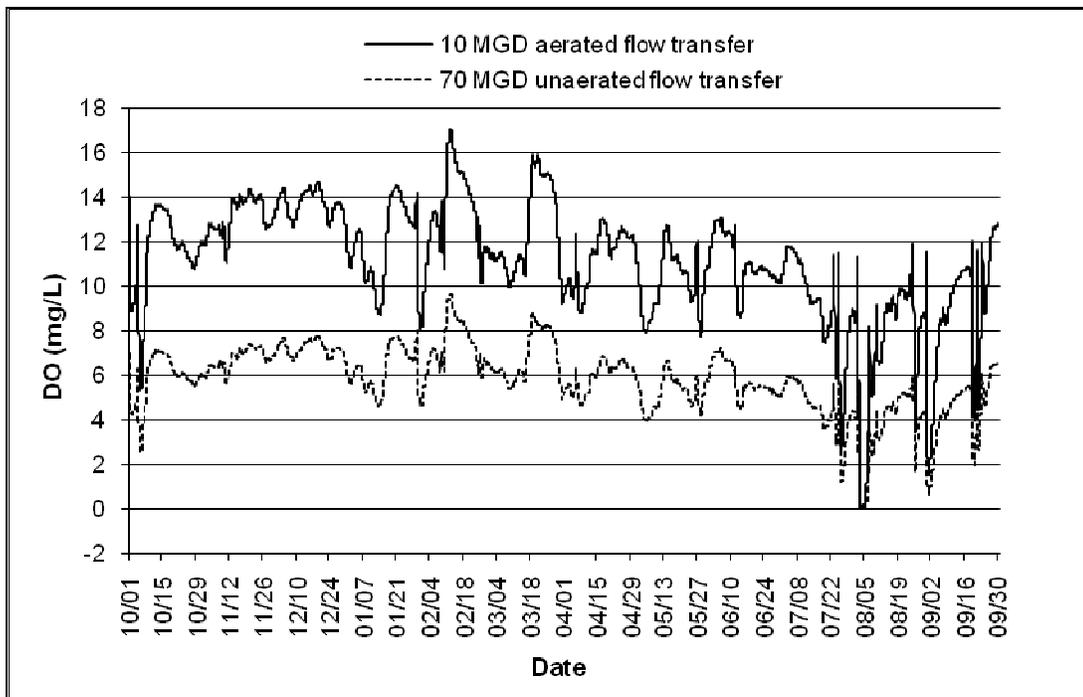
Figure 4. 13 Comparison of flow augmentation effectiveness with and without aeration along Bubbly Creek for WY 2001 at I-55

4.5. Analysis of Conditions at Fullerton Avenue

Fullerton Avenue is located on the NBCR. Figures 4.1 and 4.2 show a compliance problem at Fullerton Avenue for WYs 2001 and 2003 for the measured DO concentrations, whereas the simulated DO concentrations do meet the proposed DO standards 90% of the time. In order to determine the reasons for this result,

missing data of WYs 2001 and 2003 and measured data for 2005-2007 calendar years

were analyzed.

Table 4.2 indicates that 3.92% and 7.60% of the possible DO measurements

are missing for WYs 2001 and 2003, respectively. In each Water Year, the simulated

DO concentrations in the periods of missing data were less than the proposed DO

standards for 95 hours or 1.1 percent of the entire year. Thus, if the true DO

concentrations were similar to the simulated concentrations, DO concentrations at

Fullerton Avenue would meet the proposed DO standards more than 90% of the time

(86.7% + (7.6%-1.1%) = 93.2%) for WY 2003. Whereas the DO concentrations for

WY 2001 would meet the proposed DO standards slightly less than 90% of the time

(86.7% + (3.9%-1.1%) = 89.5%).

Figure 4.14 presents the measured percentage compliance with the proposed

DO standards for calendar years 2005-2007 along the NSC, NBCR, SBCR, and

CSSC. For 2006 and 2007, measured DO concentrations met the proposed DO

standards more than 90% of the time at Fullerton Avenue and also for each of these

years the amount of missing data was less than for other years with no data missing in

2007 and 3.86% of the data missing for 2006. For 2005, the percentage compliance

with the proposed DO standards was 85.3%, but also 10.16% of the possible data

values were missing. Thus, the low percentage compliance with the proposed DO

standards at Fullerton Avenue for measured DO in WYs 2001 and 2003 appears to be

the result of missing data. The conclusion that 90% compliance with the proposed

DO standards is achieved at Fullerton Avenue determined on the basis of the

simulated DO concentrations, therefore, is accepted as reasonable, and no remedial

measures will be applied to the NBCR to meet 90% compliance at Fullerton Avenue.
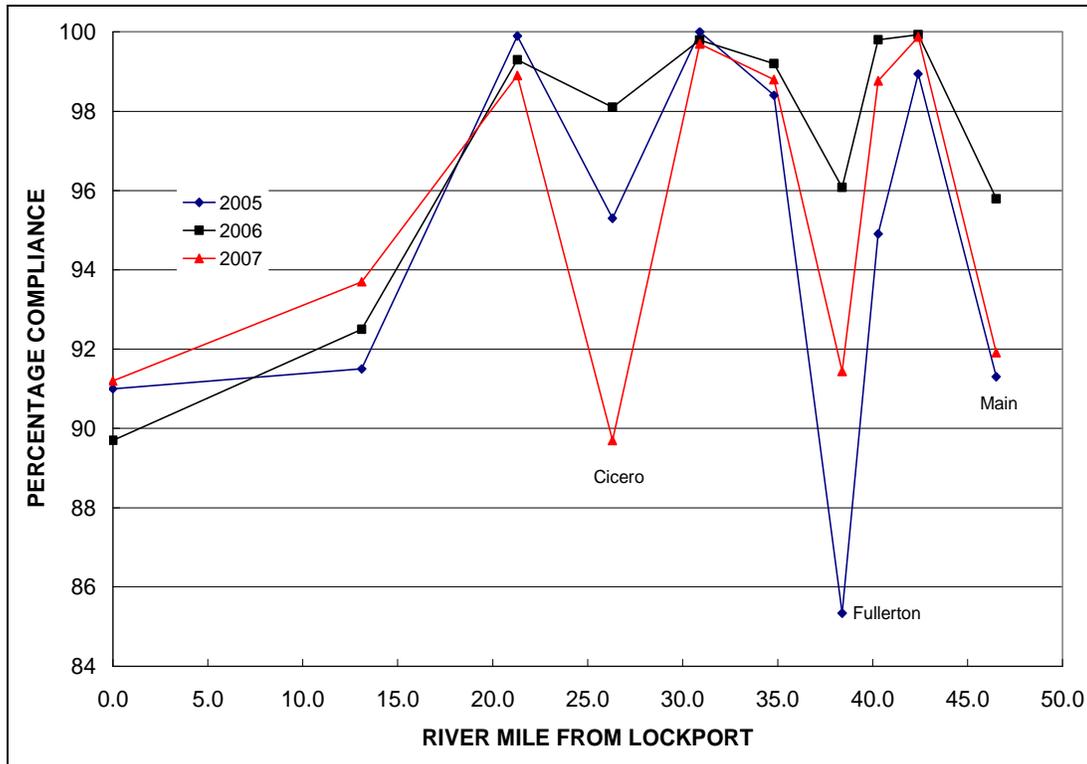
Figure 4. 14 Measured percentage compliance with the IEPA proposed DO standards for calendar years 2005-2007 along the NSC, NBCR, SBCR, and CSSC

CHAPTER 5: SIMULATION RESULTS TO MEET MWRDGC PROPOSED DO
STANDARDS

The MWRDGC has developed a proposed set of dissolved oxygen (DO)

standards for the CWS that includes an allowance for non-compliance during wet

weather periods. The total number of hours in a year of periods with DO

concentrations less than the DO standard was determined on the basis of historically

measured DO concentrations in the various reaches. Detailed allowable maximum

hours of non-compliance with the DO standards are listed in Table 3.3. The first step

in developing the compliance scenario is to determine the locations and hours in the

CWS that currently do not meet the proposed DO standards based on the baseline

simulations for both WYs 2001 and 2003. The development of an integrated strategy

to meet the MWRDGC's proposed DO standards is presented in this chapter.

5.1. Supplementary Aeration Stations

The DUFLOW model for the 2001 Water Year was used to evaluate scenarios

for achieving DO concentrations that meet the MWRDGC proposed DO standards at

all locations in the CWS. Previous baseline simulations (October 1, 2000-September

30, 2001 and October 1, 2002-September 30, 2003) were selected to determine the

locations of the new aeration stations. The purpose of the new aeration stations are to

maintain the total number of hours in the periods with DO concentrations less than

the allowable DO standards to values less than the maximum number of hours

specified in Table 3.3 for each waterway. In this case, new aeration stations were

added to the river network wherever needed starting upstream and moving

downstream. This means, when the total number of non-compliance hours of simulated DO concentrations are above the allowable non-compliance hours at a location, a new aeration station was added at that place. The maximum DO load of all new aeration stations was chosen as 80 g/s and operation hours were based on the number of hours which exceeded maximum allowable non-compliance hours. Simulation results for Water Years 2001 and 2003 are given in the following sections.

5.1.1. October 1, 2000-September 30, 2001 (Water Year 2001)

From the WY 2001 baseline simulation only the SBCR and CSSC waterways needed to be fixed. In order to achieve compliance, the following approaches were applied in the model:

1)   Flow augmentation of 24 MGD of aerated flow from the North Side WRP to the Wilmette Pumping Station on the NSC.

2)   The Devon Avenue in-stream aeration station was to be operated for additional 106 hours at the maximum capacity (3 blowers on; 64 hours changed from 0 to 3 blowers on, 30 hours changed from 1 to 3 blowers on, and 12 hours changed from 2 to 3 blowers on) instead of actual blower operations in the baseline simulation.

3)   The Webster Avenue in-stream aeration station was operated as per its actual number of working blowers and operation hours.

4)   The first aeration station was added between Canal Street and 18th Street on the SBCR (1.5 miles downstream from Jackson Boulevard) with 80 g/s DO loads and operation hours were 950 hours (operation hours were defined as the sum of the hours exceeding the allowable hours of non-compliance with the station

starting 6-hours earlier than the occurrence of each DO concentration problem to account for the flow travel time from the aeration station to the points of low DO concentrations).

5)   The second aeration station was added at Throop Street on the SBCR with 80 g/s DO loads and operation hours were 202 hours (the same method as for the first new aeration station was used to determine the operation hours).

Table 5. 1 Number of hours that DO concentrations are less than the proposed target concentrations at different locations for WY 2001

Location	Allowable hours of non-compliance	Hours of non-compliance with 24 MGD transfer from NSWRP to Wilmette and Devon Avenue operations adjustment	Hours of non-compliance with the 1st new aeration station on the SBCR	Hours of non-compliance with the 2nd new aeration station on the SBCR
Halsted Street	88	477	68	62
Throop Street	88	866	202	65
Bubbly Creek Junction	500	1062	418	306
Cicero Avenue	500	676	418	353

Note: the 1st aeration station is located at 1.5 miles downstream from Jackson Boulevard and the 2nd aeration station is located at Throop Street both on the SBCR.

Simulation results are given in Table 5.1. It can be seen that the addition of the two new aeration stations results in drastic increase in DO for WY 2001. For example, at Throop Street the DO concentration is less than 3.5 mg/L for only 65 hours (0.74% of the entire year). Plots of DO concentrations for the baseline and the two new aeration stations simulations are shown in Figure 5.1, and the locations of the new added aeration stations in the model are shown in Figure 5.2. Comparing the

two simulations- baseline and 2 new supplemental aeration stations- the approach of integrating flow augmentation on the NSC, adjusted operating hours at Devon Avenue, and new added aeration stations on the SBCR is an effective method to improve DO concentrations in order to achieve the MWRDGC's proposed DO standards.
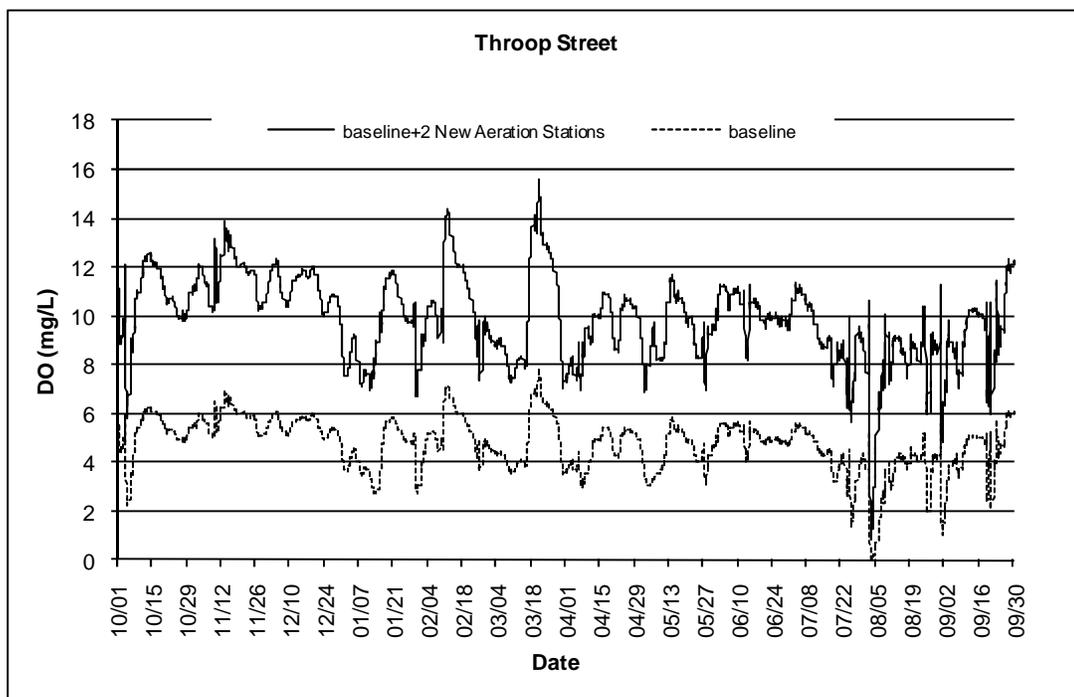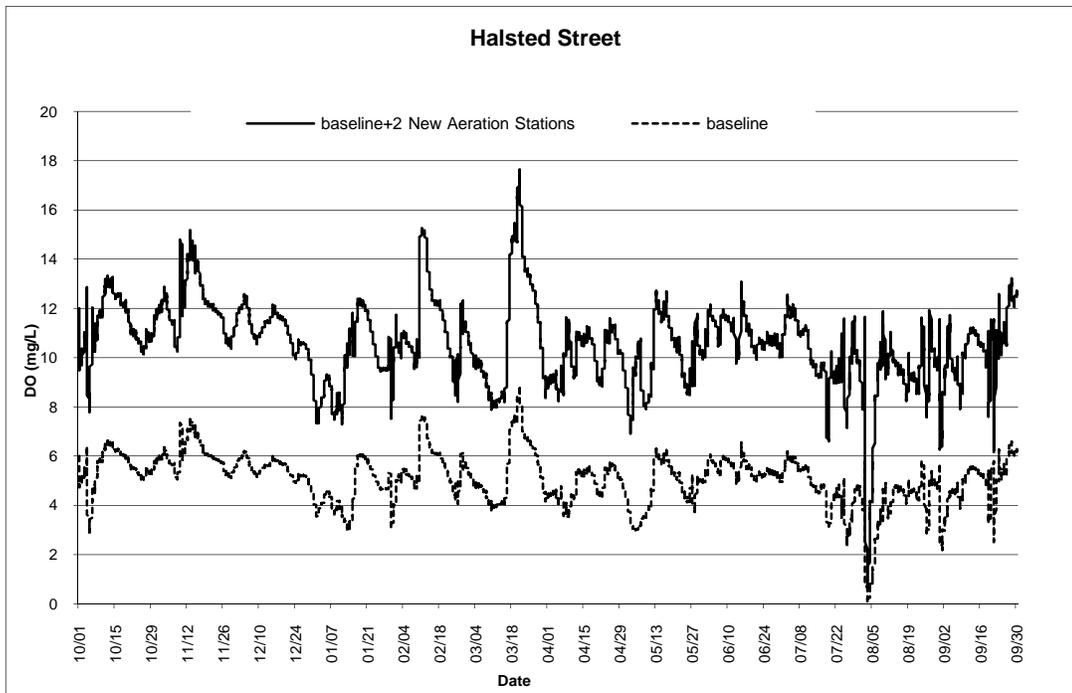
Figure 5. 1 Dissolved Oxygen (DO) concentrations for the baseline and the 2 new aeration stations simulations for Water Year 2001
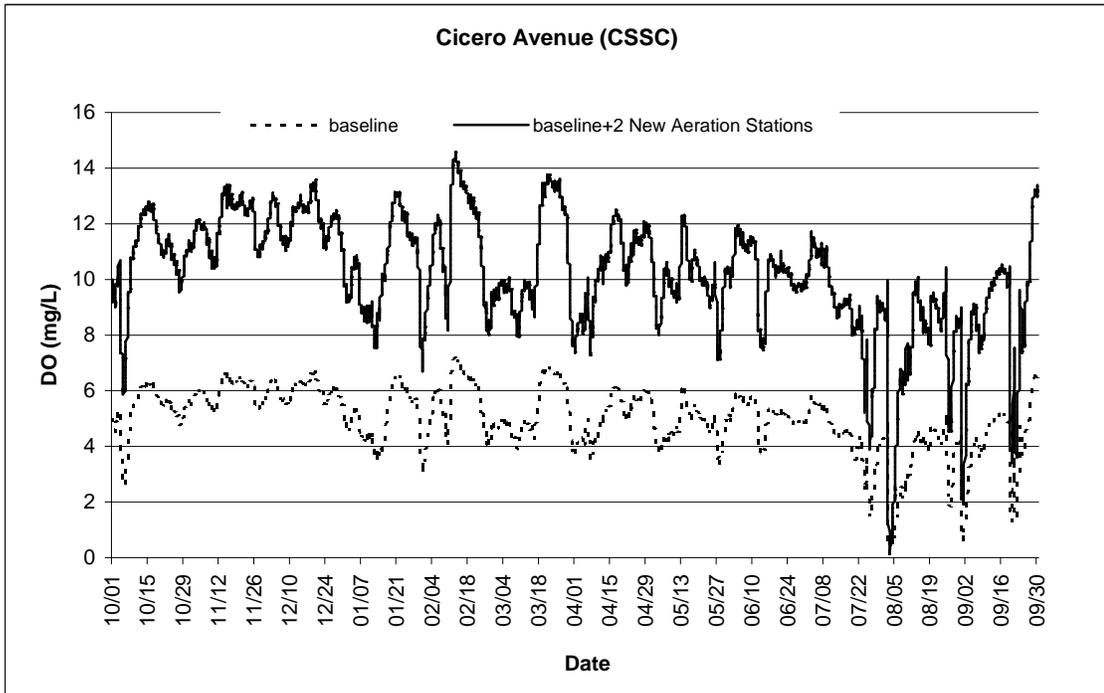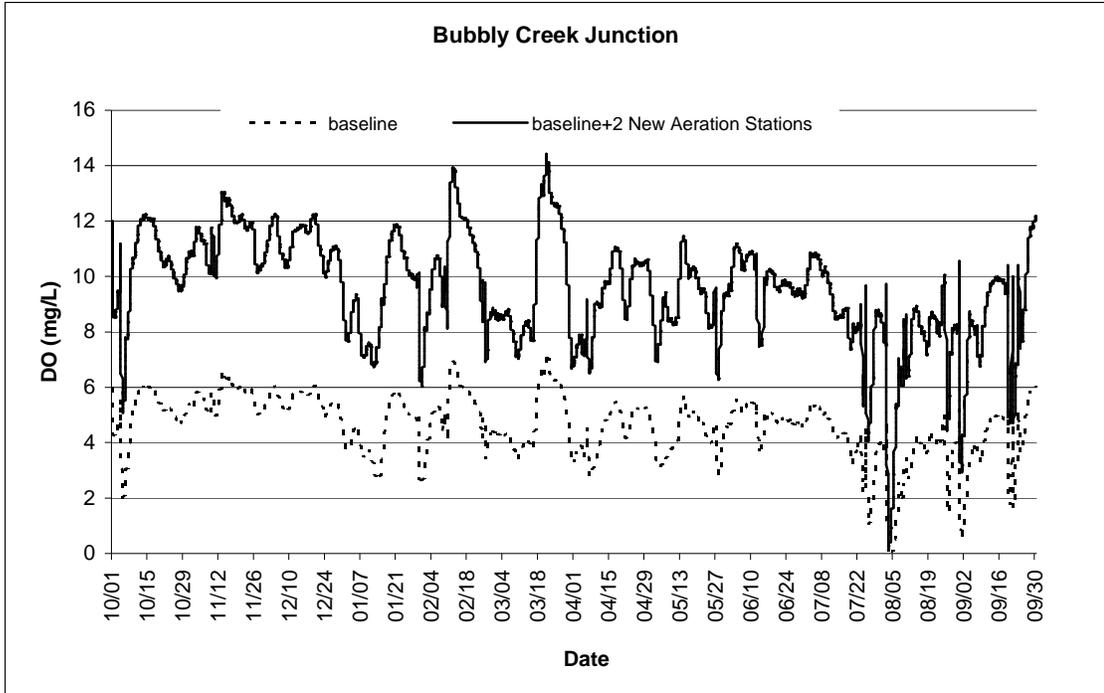
Figure 5. 2 (continued) Dissolved Oxygen (DO) concentrations for the baseline and the 2 new aeration stations simulations for Water Year 2001
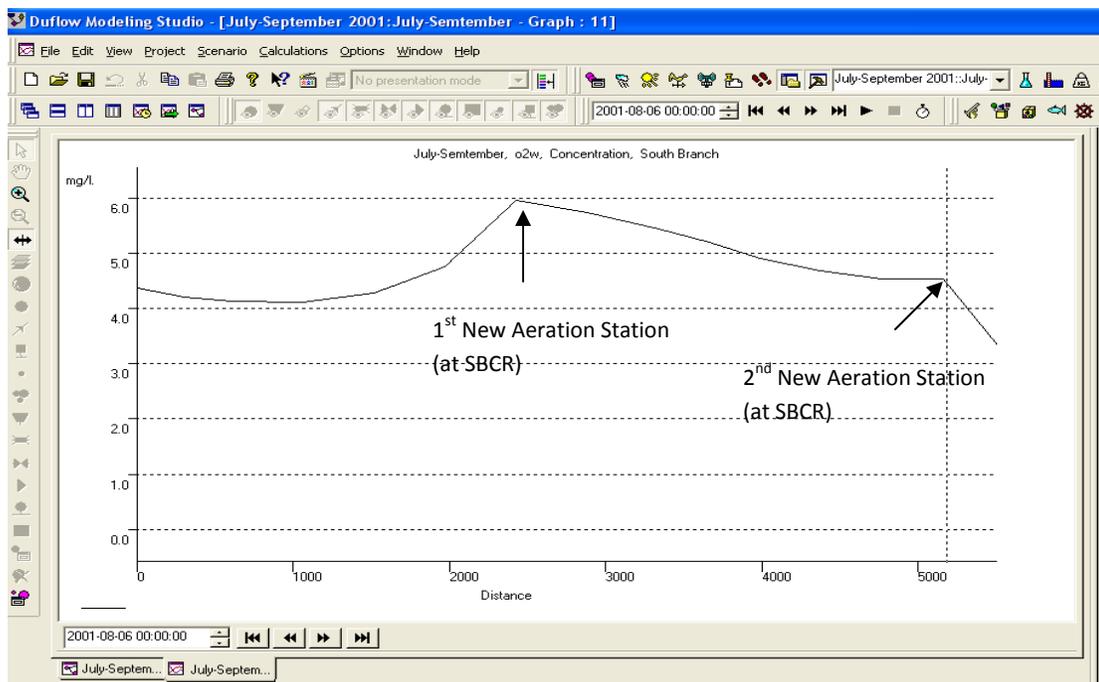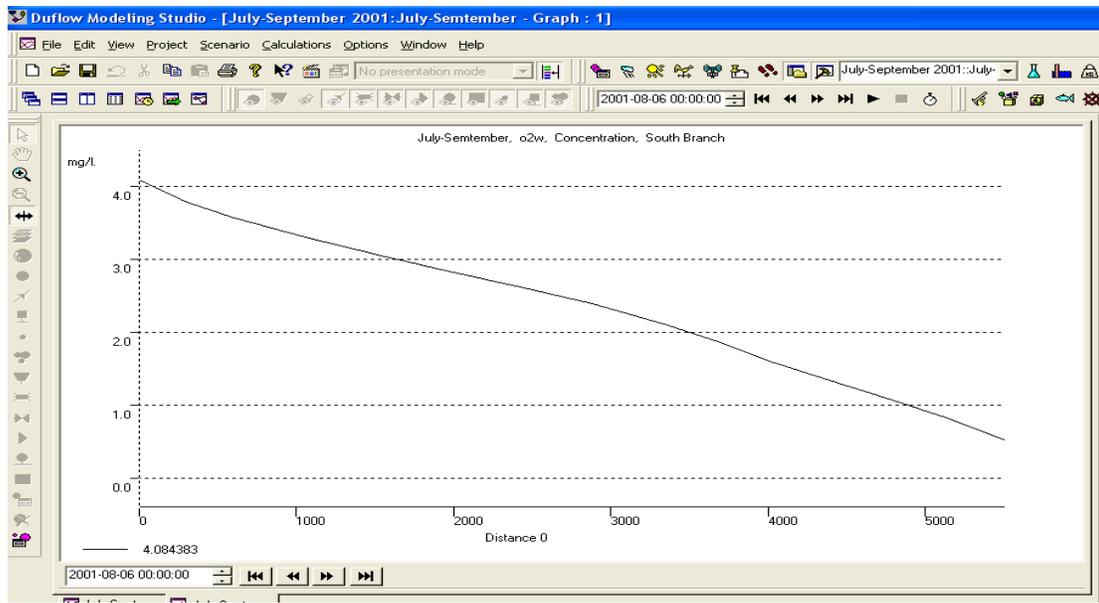
Figure 5. 3 Identification of new aeration station locations on the South Branch Chicago River (SBCR) for WY 2001, where the upper and lower show the DO concentration along the SBCR without and with supplemental aeration, respectively, for midnight on August 6, 2001

5.1.2. October 1, 2002-September 30, 2003 (Water Year 2003)

Similarly, from the 2003 baseline simulation, the SBCR and CSSC waterways also needed additional aeration. On the basis of actual need and further cost analysis, only one new aeration station was needed to achieve the compliance for WY 2003. The approach described below is slightly different from WY 2001.

1)   Flow augmentation of 24 MGD of aerated flow from the North Side WRP to the Wilmette Pumping Station on the NSC.

2)   No changes from the actual operations of both Devon Avenue and Webster Avenue in-stream aeration stations were required.

3)   An aeration station was added at Throop Street on the SBCR with 80 g/s DO loads and 186 operation hours (the same location as the second new aeration station of WY 2001).

It is important to remember that because of the missing effluent ammonia data for the NSWRP, only October through December 2002 and May through September 2003 were evaluated along NSC, NBCR, SBCR, and CSSC for the 2003 WY.

Simulation results are shown in Table 5.2. It can be seen that only one new aeration station is needed on the SBCR to achieve the proposed standards for WY 2003. On the SBCR, only Throop Street (186 hours) cannot meet the required maximum hours (88 hours) of DO concentrations less than 3.5 mg/L after NSC flow augmentation on the NSC. However, when a new aeration station is added at Throop Street, 100% compliance can be achieved at this location. Plots of DO concentrations for the baseline and the new aeration station simulations are shown in Figure 5.3.

Table 5. 2 Number of hours that DO concentrations are less than the proposed target concentrations at different locations for WY 2003

Location	Allowable hours of non-compliance	Hours of non-compliance with 24 MGD transfer from NSWRP to Wilmette and Devon Avenue operations adjustment	Hours of non-compliance with new aeration station on the SBCR
Halsted Street	88	48	24
Throop Street	88	186	0
Bubbly Creek Junction	500	329	159
Cicero Avenue	500	317	240

Note: this new aeration station is located at Throop Street on the SBCR.

The location of the new added aeration station in the model is shown in Figure 5.4. Like the simulations for WY 2001, water quality conditions after adding the aeration station and NSC flow augmentation on the SBCR and the beginning of CSSC are better than water quality of baseline simulation. Especially, at Throop Street, where DO concentrations are greater than 3.5 mg/L 100% of the evaluated time. The approach of integrating flow augmentation at NSC and new added aeration stations is an effective strategy to improve DO concentrations. Thus, it is reasonable and reliable to apply integrated flow augmentation and supplemental aeration station to achieve MWRDGC's proposed DO standards.
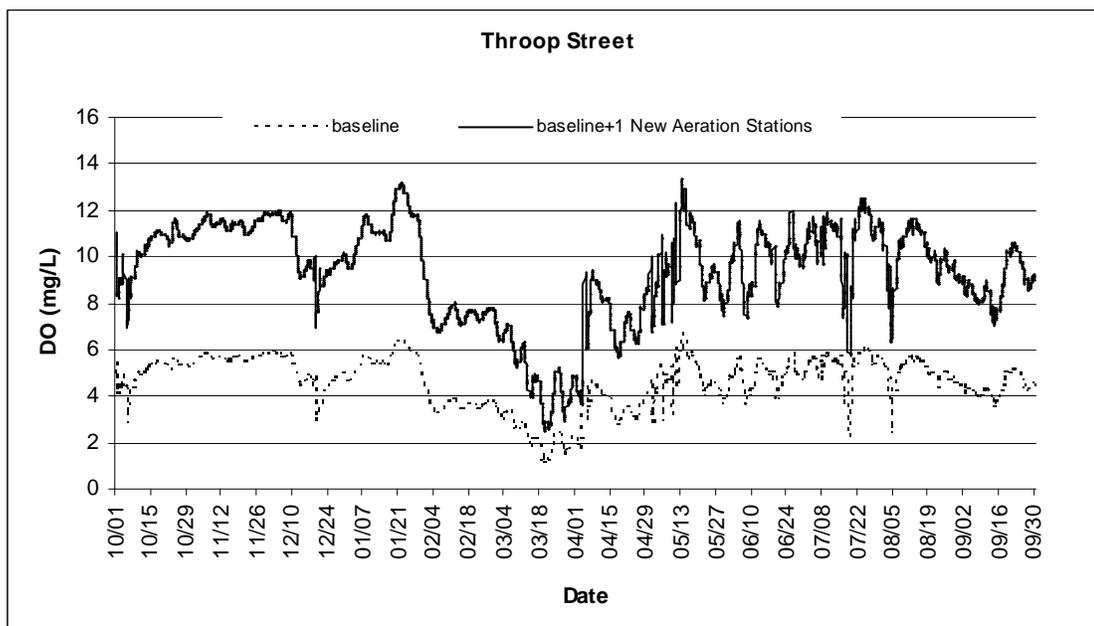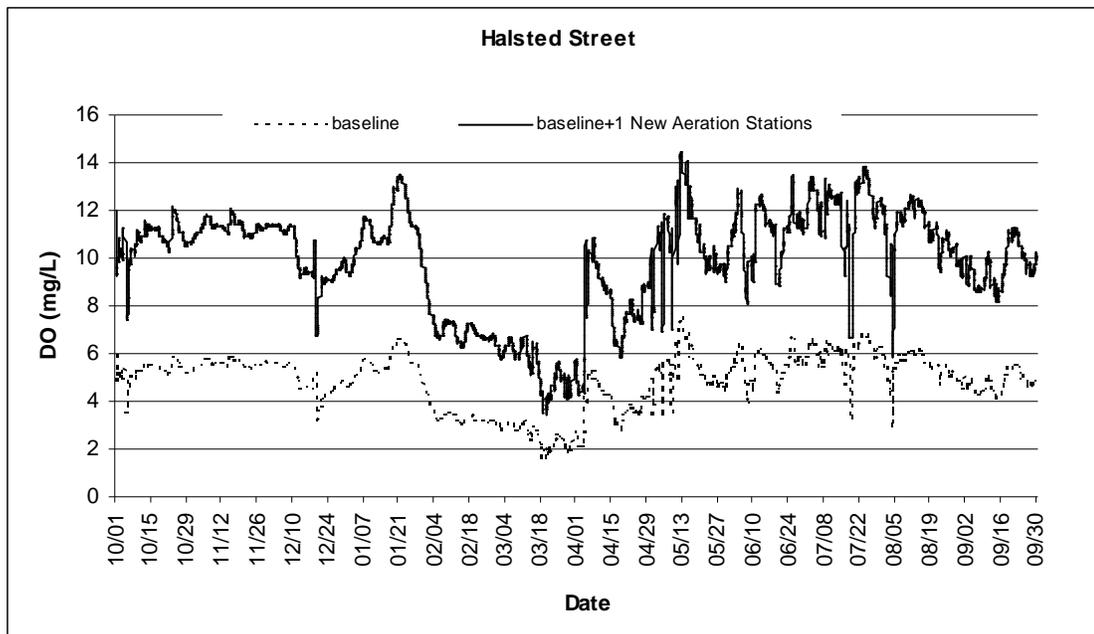
Figure 5. 4 Dissolved Oxygen (DO) concentrations for the baseline and the new aeration station simulations for Water Year 2003
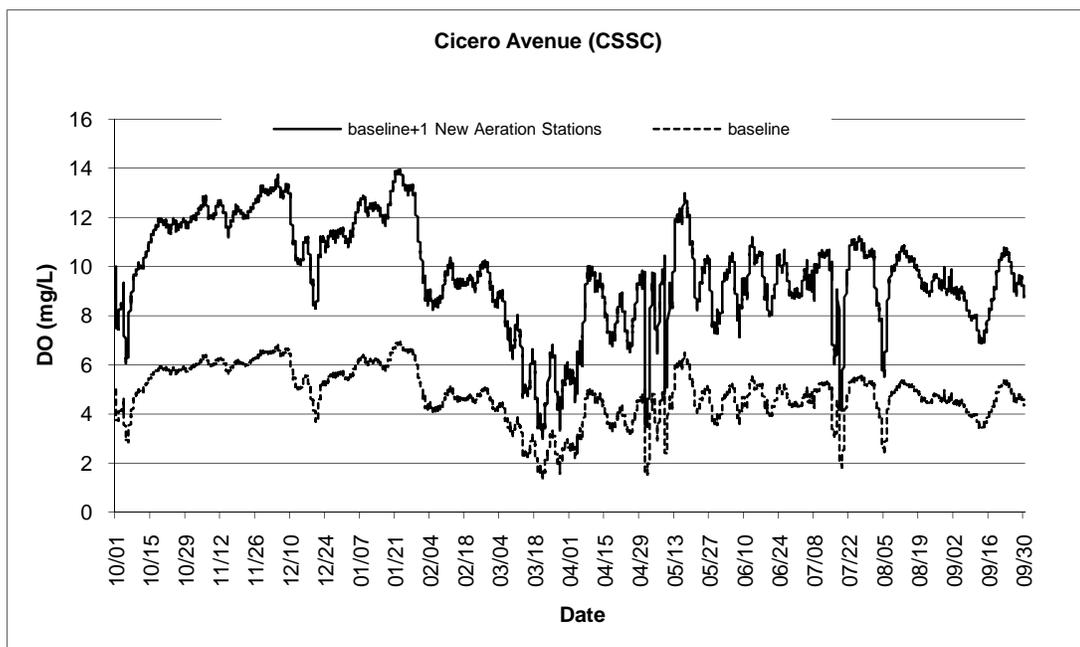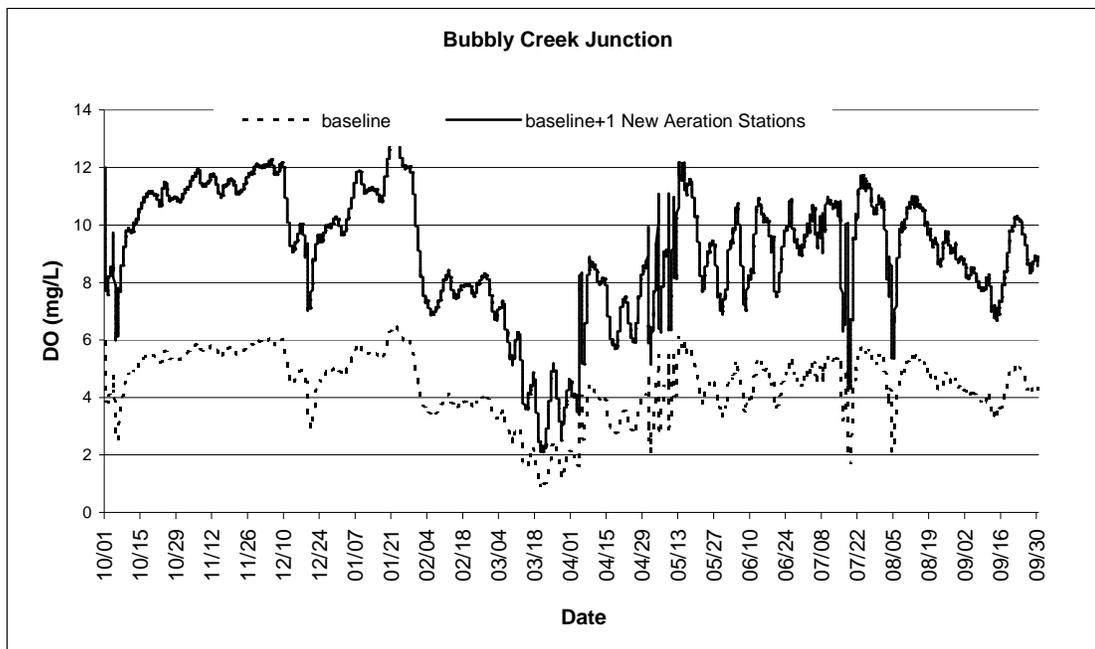
Figure 5. 5 (continued) Dissolved Oxygen (DO) concentrations for the baseline and the new aeration station simulations for Water Year 2003
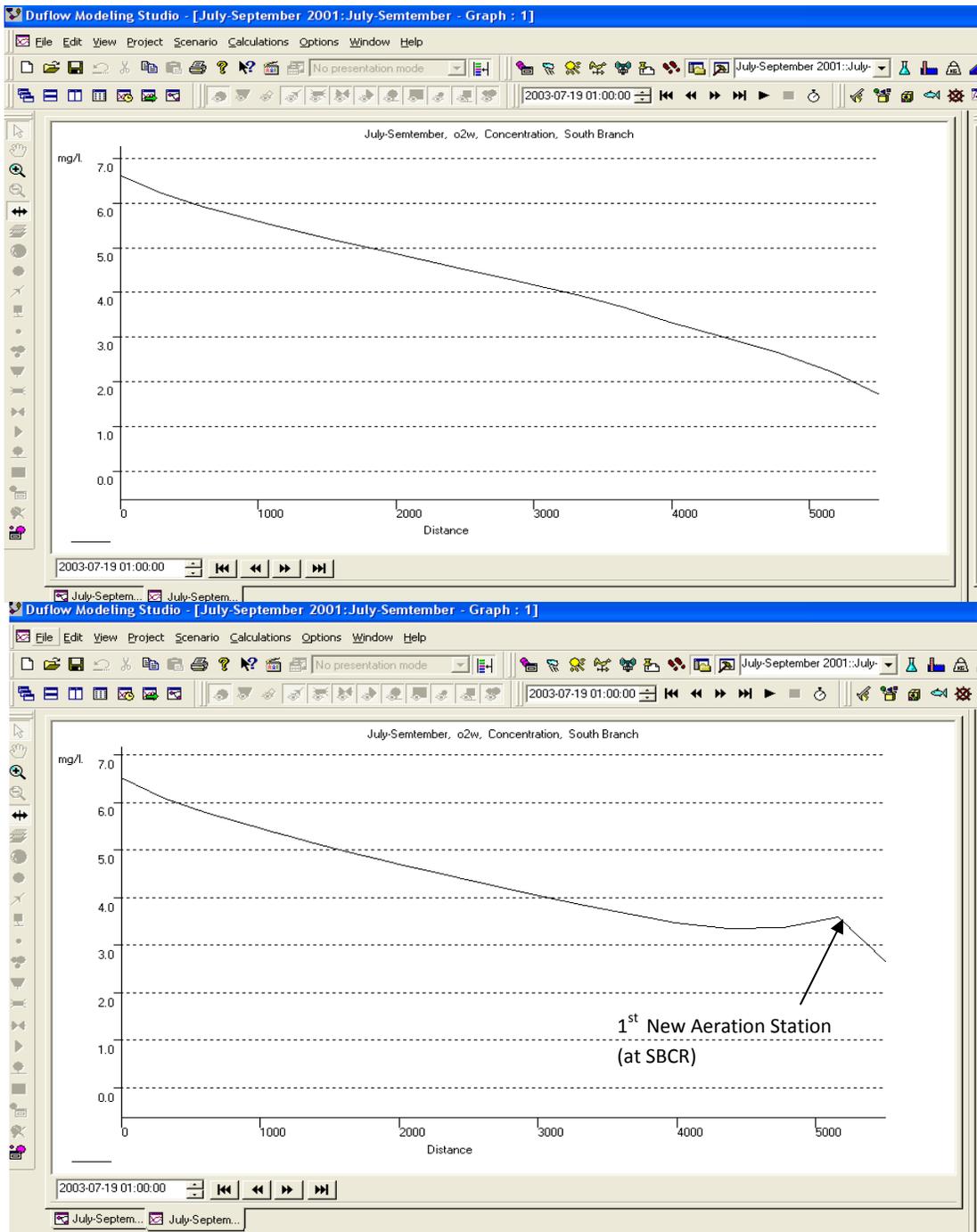
Figure 5. 6 Identification of the new aeration station new aeration station locations on the South Branch Chicago River (SBCR) for WY 2003, where the upper and lower show the DO concentration along the SBCR without and with supplemental aeration, respectively, for 1 a.m. on July 19, 2003

5.2. The Cost Estimate for the Integrated Strategy

Based on the facilities required in the simulation of WYs 2001 and 2003, the MWRDGC requested that AECOM-CTE determine the order of magnitude capital and annual costs for the facilities required to meet the proposed DO standards.

Basis of cost estimate

The AECOM-CTE (2009) estimate is an order of magnitude cost estimate and is based upon a variety of assumptions. This order of magnitude cost estimate is roughly equivalent to a level 5 cost estimate according to the cost estimate classification system recommended by the Association for the Advancement of Cost Engineering (AACE) and has an approximate accuracy range of -30% to +50%.

Assumptions

The following are the assumptions and simplifications utilized to prepare the order of magnitude cost estimate for the facilities required to achieve compliance with the MWRDGC's proposed DO standards:

1)   Only one aeration technology-supplemental aeration using ceramic disc diffusers in the waterway with an on-shore blower facility- was utilized.

2)   Only one aerated flow augmentation technology- U-tube aeration of pumped flow- was utilized.

3)   The number, location, and sizing of the aeration stations and hours of operation of the stations for the cost estimate are based upon DUFLOW model results provided by this thesis MWRDGC .

4)   Inflation corrected unit costs derived from previous studies conducted by AECOM-CTE for the IEPA's Use Attainability Analysis (UAA) study form the basis for the cost estimate. Present worth was based upon a 20 year life with a present worth factor of 19.42, 3% interest rate and 3% inflation rate.

5)   It was assumed that vacant land is available and can be purchased with minimal demolition costs. However, given the size of the aeration stations, this may not be possible.

6)   The annual hours of operation for the proposed facilities as well as the "additional hours" of annual operation of the existing Devon and Webster Avenue stations was determined by this thesis and provided to AECOM-CTE. It is noted that "additional hours" are those annual hours of operation needed to operate the existing stations over and above the normal operating hours now used to meet existing the Illinois Pollution Control Board (IPCB) DO standards. Costs for electricity and the required labor to operate and maintain these stations for the additional annual hours were included in the order of magnitude cost estimate. The unit electricity cost in June 2008 dollars was 0.0750 $/kWh.

Order of magnitude cost estimate

Based on the model simulation and cost assumptions previously described, AECOM-CTE estimates the order of magnitude capital costs to meet the MWRDGC's proposed DO standards to be $50,410,000. Total annual operating costs are estimated to be $523,000. The total present worth is estimated at $60,434,000.

On the basis of the simulation results used for the cost estimate, the operation of the aeration stations is relatively infrequent. Achieving compliance with the MWRDGC's proposed DO standards will require a complex waterway DO monitoring network during the infrequent times of operation. Providing and maintaining the monitoring network and automated system and the infrequent use of the aeration stations would be a significant challenge and the costs for this approach have not been included here.

Similar cost estimate for the integrated strategies given in Chapter 4 and 6 currently are being prepared by AECOM-CTE.

CHAPTER 6: SIMULATION RESULTS TO MEET IEPA PROPOSED DO
STANDARDS 100% OF THE TIME

In Chapter 4, the condition of compliance 90% of the time with IEPA's proposed DO standards is discussed. It is not difficult to achieve 90% compliance by integrating flow augmentation and supplemental aeration stations. Ninety percent compliance is an interesting planning cast, but the IEPA proposal requires compliance 100% of the time for both WYs 2001 and 2003. In this chapter, evaluating the integrated strategies including the combination of flow augmentation at three locations and more supplemental aeration stations for the entire CWS is discussed. It is remember that only October through December 2002 and May through September 2003 for WY 2003 is evaluated along NSC, NBCR, SBCR, and CSSC because of missing effluent ammonia data for the NSWRP, whereas the entire period of WY 2001 (October 1, 2000-September 30, 2001) is considered.

6.1. Description of Flow Augmentation

In previous chapters and studies (Alp and Melching, 2006), flow augmentation was applied at two locations: 1) from the North Side Water Reclamation Plant to the upstream end of the North Shore Channel (Wilmette Pumping Station); 2) from Throop Street on the SBCR to the upstream end of Bubbly Creek (Racine Avenue Pumping Station). In this case, additional flow transfer from the Calumet Water Reclamation Plant to the O'Brien Lock and Dam was evaluated. The first set of simulations evaluated the fixed amounts of aerated flow transfer at the three foregoing locations on the basis of the conditions of the baseline simulations.

Actual blower operations of the existing Devon Avenue and Webster Avenue in-stream aeration stations were applied, and each SEPA station was operated at full capacity (3 pumps) for the four SEPA stations in the modeled portion of the CWS.

6.1.1. Flow Augmentation from North Side WRP to Upstream End of North Shore Channel

North Side WRP daily effluent temperature was used to compute the DO saturation in the transferred flows for the NSC. It was found that flow transfer from the North Side WRP to the Wilmette Pumping Station is an effective way to improve DO concentrations on the NSC. Table 6.1 lists the percentage compliance with DO concentration standards of 5.0 mg/L (March-July) and 3.5 mg/L (August-February) at different locations on the Upper NSC for WYs 2001 and 2003 for various amounts of aerated flow transfer.

Table 6. 1 The percentage of time that DO concentrations are greater than or equal to the target concentrations at different locations on the UNSC for various transfers of aerated NSWRP effluent to the Wilmette Pumping Station

	Linden Street		Simpson Street		Main Street	
Scenario	2001 WY	2003 WY	2001 WY	2003 WY	2001 WY	2003 WY
Baseline	73.89	80.31	72.85	77.99	76.44	80.99
30MGD	99.66	94.36	97.74	95.38	94.38	90.72
40MGD	99.94	96.05	98.46	98.25	95.83	94.67
50MGD	99.97	97.58	99.00	99.36	96.68	97.24

As shown in Table 6.1, at Main Street at least 94.6% of the time for both WYs 2001 and 2003 with a transfer of 40 MGD of aerated effluent. That means, 473 hours

(approximately 20 days) cannot meet the proposed DO standards. At this point, 100% compliance can probably be more effectively achieved by adding aeration stations that would only operate as needed on these 20 days rather than by a continuously operating flow augmentation. Therefore, the 100% compliance scenario was developed combining an aerated flow transfer of 40 MGD with the placement and operation of in-stream aeration stations along the NSC.

6.1.2. Flow Augmentation for Bubbly Creek

In Chapter 4, it was shown that flow augmentation with aeration on the Bubbly Creek can yield higher DO concentrations than unaerated transfers for much lower flow rates, so a flow transfer with aeration was applied in this case. The water temperatures measured at Jackson Boulevard and Cicero Avenue were linearly interpolated to get water temperature at Throop Street, which was used to calculate the DO saturation in the transferred flows for Bubbly Creek. Eight (5, 10, 30, 40, 50, 60, 70, and 75 MGD) and two (1 and 10 MGD) different fixed amounts of aerated flow transfer have been evaluated for WYs 2001 and 2003, respectively. Figures 4.9 and 4.10 show the percentage compliance with DO concentrations greater than or equal to 3.5 mg/L at different locations along Bubbly Creek with various amounts of aerated flow transfers.

In Figure 4.9, there are two locations whose percentage compliances are less than 90% with a 5 MGD flow transfer. Therefore, 10 MGD is the minimum amount of transfer flow to be used to achieve 90% compliance of time for each point along Bubbly Creek for Water Year 2001. If using a 75 MGD flow transfer, although

percentage compliances of all locations along Bubbly Creek are greater than 90%, a transfer of 7.5 times more flow only slightly increases the percentage compliance. For Water Year 2003, the percentage compliance at each location exceeds 98% of time with a 10 MGD (even a 1 MGD) flow transfer. Hence, a 10 MGD aerated flow transfer from Throop Street to the upstream end of Bubbly Creek was selected for flow augmentation on Bubbly Creek. In addition, when a 10 MGD flow transfer is applied, there is still a minimum 90.13% percentage that occurs at 36th Street for 2001 WY. Thus, in order to achieve compliance 100% of the time, supplemental aeration stations were required.

6.1.3. Flow Augmentation from the Calumet WRP to the Little Calumet River

The Calumet WRP daily influent temperature was used to compute DO saturation in the transferred flows for the Calumet River at O'Brien Lock and Dam. Six (1, 10, 20, 30, 40, and 50 MGD) fixed amounts of aerated flow transfers were evaluated for WYs 2001 and 2003. Table 6.2 lists the percentage compliance with 5.0 mg/L (March-July) and 3.5 mg/L (August-February) standards for various amount of flow transfer for different locations on the Little Calumet River (north) and Calumet River for WYs 2001 and 2003.

Table 6. 2 The percentage of time that DO concentrations are greater than or equal to the target concentrations at different locations on the Little Calumet River (north) and Calumet River for various aerated transfers from the Calumet WRP to O'Brien Lock and Dam

	O'Brien Lock and Dam		Conrail Railroad		Indiana Avenue	
Scenario	2001 WY	2003 WY	2001 WY	2003 WY	2001 WY	2003 WY
1MGD	98.40	94.22	98.44	94.64	100	100
10MGD	98.40	94.23	98.57	94.75	100	100
20MGD	98.46	94.51	98.65	95.58	100	100
30MGD	98.58	95.18	98.68	95.66	100	100
40MGD	98.68	95.48	98.72	95.81	100	100
50MGD	98.78	97.08	98.86	97.26	100	100

Since the percentage compliance condition of WY 2003 is not as good for WY 2001, WY 2003 was selected as the critical year to determine the flow amount that maximizes the effectiveness of the flow transfer. By analyzing simulation results in Table 6.2, it can be found that an aerated flow transfer of 30 MGD yields a minimum percentage compliance of 95.18% with the proposed DO standards at the O'Brien Lock and Dam. That means, the proposed DO standards cannot be met for 18 days, approximately. Figures 6.1 and 6.2 more directly show the percentage compliance that DO concentrations are greater than or equal to IEPA proposed standards at different locations along the Little Calumet River (north) and Calumet River with various amounts of aerated flow transfer.
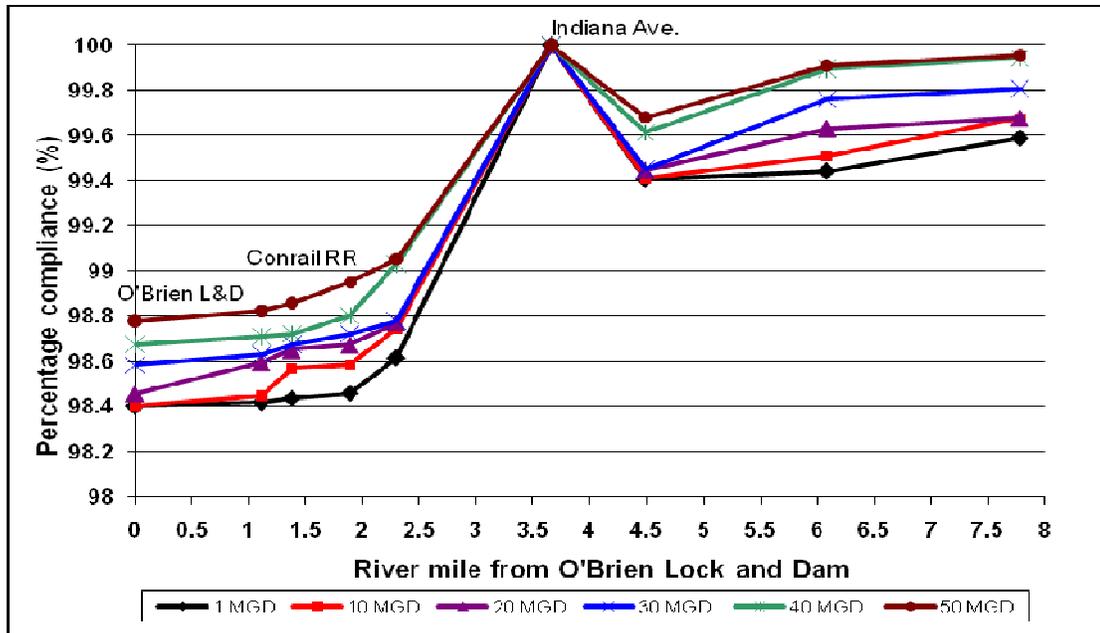
Figure 6. 1 The percentage of time that DO concentrations are greater than or equal to the target concentrations at different locations on the Little Calumet River (north) and Calumet River for various aerated transfers from the Calumet WRP to O'Brien Lock and Dam for WY 2001
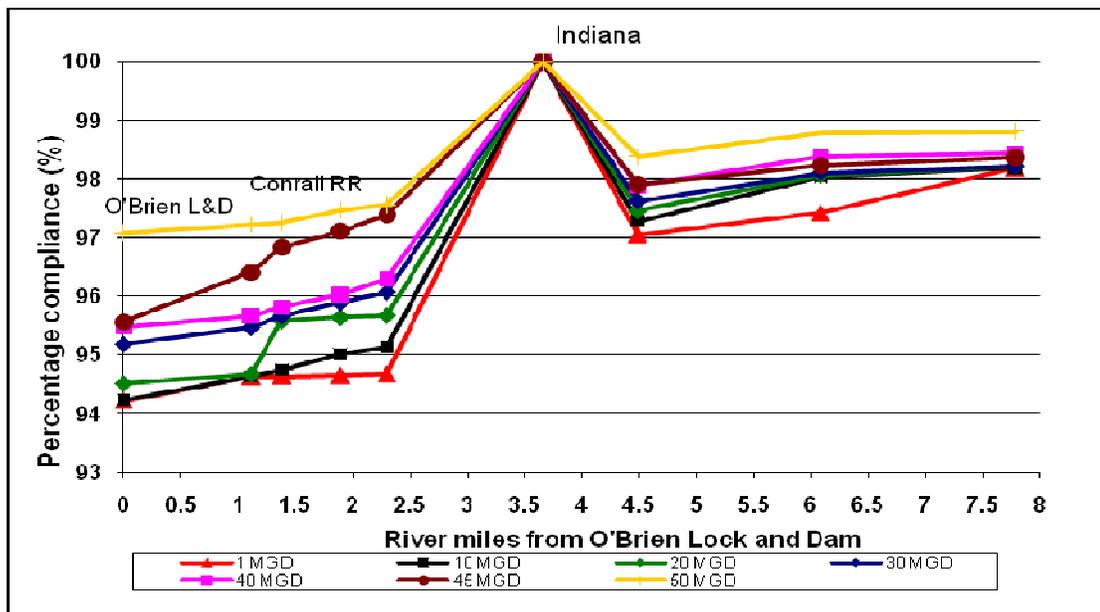
Figure 6. 2 The percentage of time that DO concentrations are greater than or equal to the target concentrations at different locations on the Little Calumet River (north) and Calumet River for various aerated transfers from the Calumet WRP to O'Brien Lock and Dam for WY 2003

Comparing to 30 and 50 MGD flow transfers, a transfer of 1.7 times more flow only increases the compliance time by 1.9% (166 hours) still leaving 256 hours of non-compliance. Hence, 100% compliance can probably more effectively be achieved by adding new aeration stations that only turn on-and-off as needed rather than by a continuously operating flow transfer. Therefore, the 100% compliance scenario was developed, combing an aerated flow transfer of 30 MGD with the operation of in-stream aeration stations along the Calumet River and Little Calumet River (north).

6.2. Description of supplementary aeration stations

In this section, the addition of new aeration stations to the flow transfer given in Section 6.1 is evaluated to achieve 100% compliance with the IEPA proposed DO standards for the entire waterway system. The purpose of adding the new aeration stations is to raise DO concentrations to or above 5 mg/L (March-July) and 3.5 mg/L (August-February) as required. Because the periods of January-April were not taken into account for 2003 WY, 3.5 mg/L and 5 mg/L were only considered for the periods of August-December and May-July were considered, respectively, in WY 2003. In this case, new aeration stations were added to the NSC, NBCR, SBCR, and CSSC wherever needed for WY 2001 first, since the condition of 2001 WY is worse than WY 2003 on these waterways, whereas for the Little Calumet River (north) and Cal-Sag Channel. WY 2003 was used to establish the locations of the new aeration stations because the condition of WY 2003 is worse than WY 2001 on these

waterways. This means that when the simulated DO concentration drops below 5 or 3.5 mg/L, as appropriate, at a location a new aeration station would be introduced.

6.2.1. October 1, 2000-September 30, 2001 (Water Year 2001)

Based on the WY 2001 baseline simulation results, each waterway in the CWS needed improvements in DO concentrations. In order to achieve 100% compliance, the following approaches were applied in the model:

1)   Flow augmentation of 40 MGD of aerated flow was introduced at the Wilmette Pumping Station from the North Side WRP, and then additional aerators were added along the NSC, NBCR, SBCR, and Chicago River Main Stem.

2)   Once 100% compliance at all locations upstream of Throop Street was reached, 10 MGD transfer of aerated flow from the Throop Street to the upstream end of Bubbly Creek was applied and new aeration stations were added to Bubbly Creek.

3)   When 100% compliance was achieved on Bubbly Creek and the SBCR, the procedure of adding aerators was moved down to the CSSC until 100% compliance was reached up to the Sag junction.

4)   A 30 MGD transfer of aerated effluent from the Calumet WRP to the O'Brien Lock and Dam was applied, meanwhile pump operations of four SEPA stations were adjusted to their maximum capacities (3 pumps operating for each SEPA station) and new aerators were added as needed until 100% compliance was achieved on the Calumet River, Little Calumet River (north) and the Cal-Sag Channel.

5)   In order to achieve compliance 100% of the time for the remainder of the CSSC, new aerators were added on the CSSC downstream from Sag junction, as needed.

It should be noted that the size and operation hours of the new aeration stations also needed to be determined, in addition to their locations. Oxygen loads of 80 g/s were tried to maintain the DO concentrations above 5 mg/L or 3.5 mg/L as appropriate, but in some cases, loads of 100 g/s were needed. As a new aeration station was added, the effect of the new aeration station was observed and another aeration station was added at the location where the DO concentration dropped below the proposed standards. This exercise was a trial and error practice and availability of space for construction of an aeration station was not considered during the simulation.

Simulation results showed 25 new supplementary aeration stations with different operation hours were needed to achieve the proposed DO standards of 3.5 mg/L and 5 mg/L for periods of August-February and March-July, respectively, for Aquatic Life Use A waters and of 3.5 mg/L throughout the year for Aquatic Life B waters for WY 2001. Descriptions of locations, oxygen loads, and operation hours of the proposed aeration stations are listed in Table 6.3.

Table 6. 3 Locations, operation hours and oxygen loads of the supplementary aeration stations in the Chicago Waterway System for 100% compliance with the DO standards proposed by IEPA

No.	Waterways	River Mile*	Operation Hours-2001	Operation Hours-2003	Max Loads (g/s)	Locations
1	NSC	340.8	134	233	80	0.20 mi downstream from Wilmette Pumping Station
2	NSC	339.66	214	0	80	0.54 mi downstream from Central Ave.
3	NSC	339.12	102	0	80	0.38 mi downstream from Simpson St.
4	NSC	338.53	113	84	80	0.97 mi downstream from Simpson St.
5	NSC	336.55	222	161	80	0.95 mi downstream from Main St.
6	NBCR	332.99	0	211	80	2.01 mi downstream from Devon Ave.
7	NBCR	331.82	102	30	80	0.78 mi downstream from Wilson Ave.
8	Main Stem	-	78	0	80	0.037 mi downstream from CRCW
9	SBCR	325.57	376	0	80	0.03 mi downstream from NBCR Junction
10	SBCR	324.09	84	0	80	1.51 mi downstream from NBCR Junction
11	SBCR	323.52	51	168	80	2.08 mi downstream from NBCR Junction
12	SBCR	321.9	150	183	80	Throop St.
13	Bubbly Creek	-	946	0	80	0.13 mi upstream from Bubbly Creek Junction
14	Bubbly Creek	-	253	0	80	0.72 mi upstream from Bubbly Creek Junction
15	Bubbly Creek	-	17	0	80	36th St.
16	CSSC	321.1	85	75	100	Damen Ave.
17	CSSC	320.6	46	0	80	Western Ave.
18	CSSC	319.82	99	0	80	0.78 mi downstream from Western Ave.
19	CSSC	318.26	100	55	90	2.34 mi downstream from Western Ave.
20	CSSC	317.21	92	0	80	0.09 mi downstream from Cicero Ave.
21	CSSC	308.6	78	31	80	3.7 mi downstream from the Baltimore and Ohio Railroad (B&O RR) Bridge
22	CSSC	305.04	37	0	80	0.94 mi upstream from Route #83
23	CSSC	296.74	52	21	80	0.54 mi upstream from Romeoville
24	LCRN	326.5	0	106	80	Grand Calumet River Junction
25	LCRN	320.32	0	93	80	0.22 mi upstream from Halsted St.
26	LCRN	320	129	106	80 (2001) 100 (2003)	0.35 mi upstream from the junction of Little Calumet River
27	Cal-Sag Channel	309.4	150	289	80	Mill Creek Junction
28	Cal-Sag Channel	304.57	62	165	80	0.27 mi upstream from Route #83

* : River miles for the CWS often are described relative to the confluence of the Illinois River with the Mississippi River at Grafton, IL., in this case the River Mile for Lockport is 291, and all of the values are based on the Lockport River Mile

- : no available river mile values

After large storms, low DO concentrations are observed for an extended period of time. By analyzing the detailed times of DO problem occurrence, the main critical periods in which the proposed DO standards would not be met were May, July, August, and September, especially in July and August.

Simulation results showed that four new aeration stations would be needed on the upper NSC, whereas one aeration station would be needed for the lower NSC because flow augmentation, NWSRP flows, and the Devon Avenue in-stream aeration station could not provide enough dissolved oxygen for the river system. Only one new aeration station was needed on the upper NBCR located upstream from the Webster Avenue in-stream aeration station. In accordance with the water quality conditions on the Chicago River Main Stem and SBCR, one and four new aeration stations would be needed to increase DO concentrations to or above 3.5 mg/L 100% of the time, respectively. In previous assessment by the Research and Development Department of the MWRDGC, three aeration stations would be needed for Bubbly Creek (CTE, 2007c). In this exercise, although the number of new aeration stations is the same, the locations are different. On the CSSC, eight aeration stations would be added to raise DO concentration above 3.5 mg/L at all locations. Since a transfer of 30 MGD of aerated flow was introduced from Calumet WRP to O'Brien Lock and Dam, the proposed DO standards would be met 100% of the time along the Little Calumet River (north) with only one new aeration station. Meanwhile, because the four SEPA stations were assumed to be operated at full capacity, two new aeration stations can provide sufficient dissolved oxygen to meet the proposed DO standards along the Cal-Sag Channel. Dissolved oxygen concentration profiles along the

waterway segments with the 25 new aeration stations operating are shown for

selected critical periods in Figures 6.3-6.6.
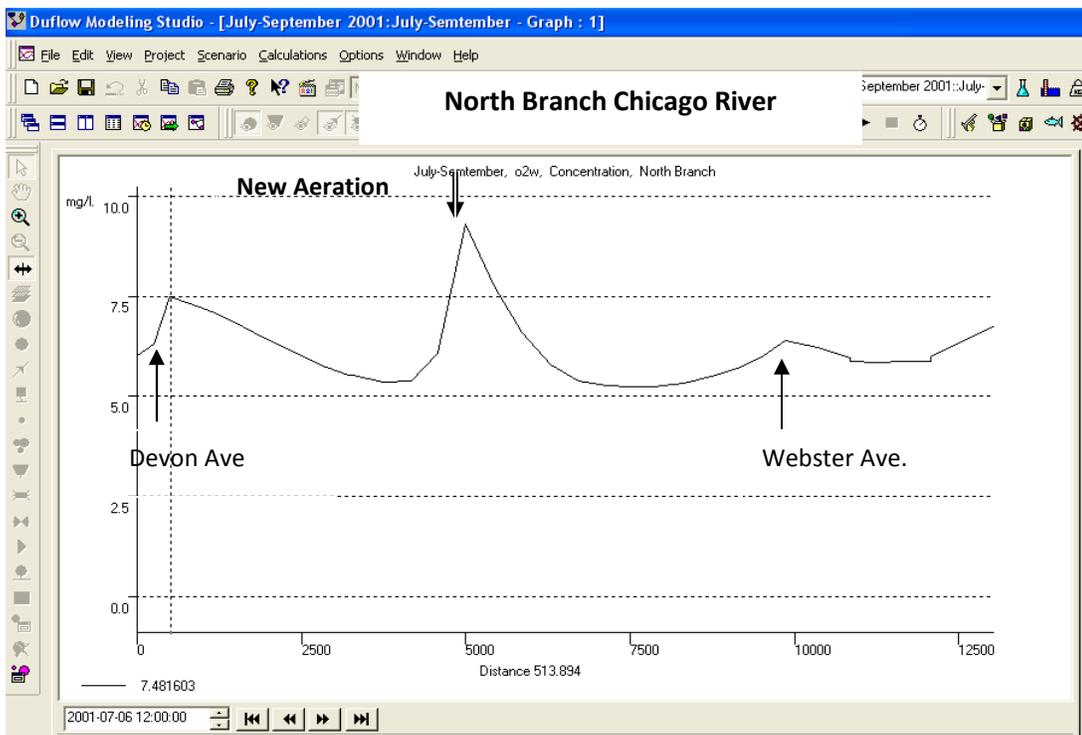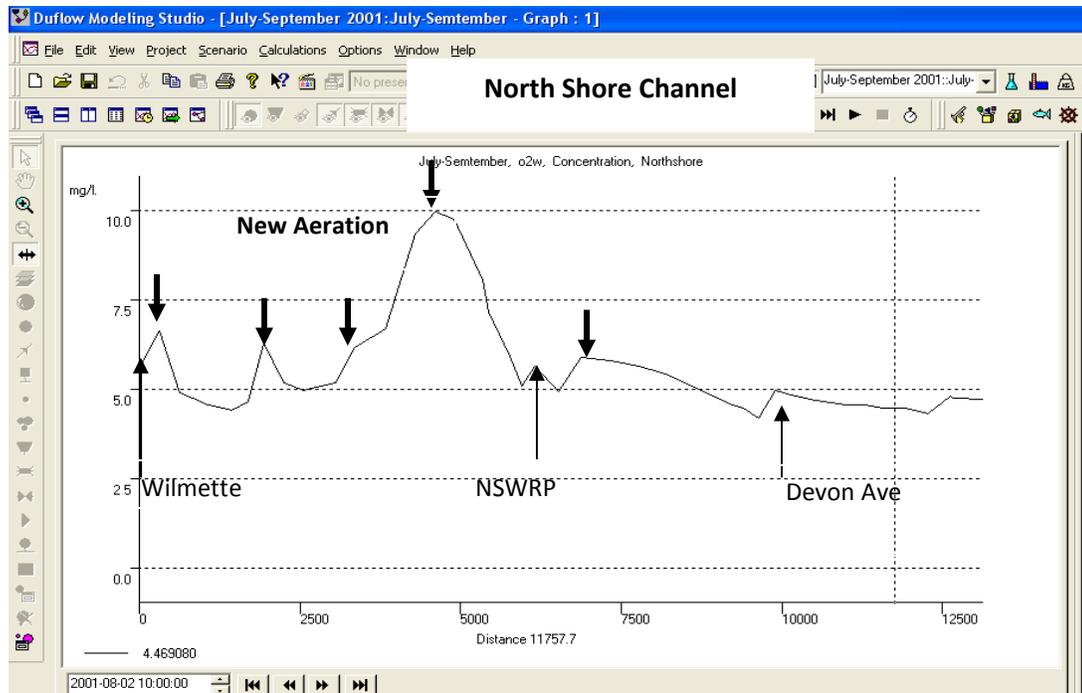
Figure 6. 3 Dissolved oxygen concentration profiles in the Chicago Waterway System for selected critical periods of August 2, 2001 (North Shore Channel) and July 6, 2001(North Branch Chicago River) where the downward arrows indicate locations of new aeration stations
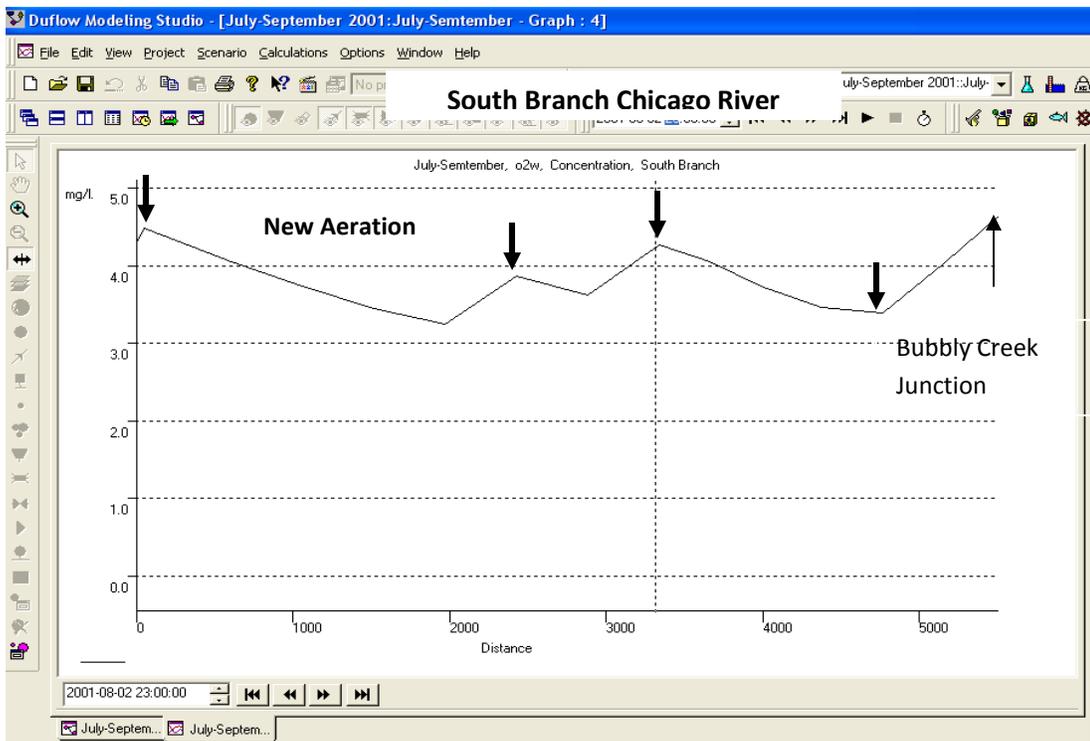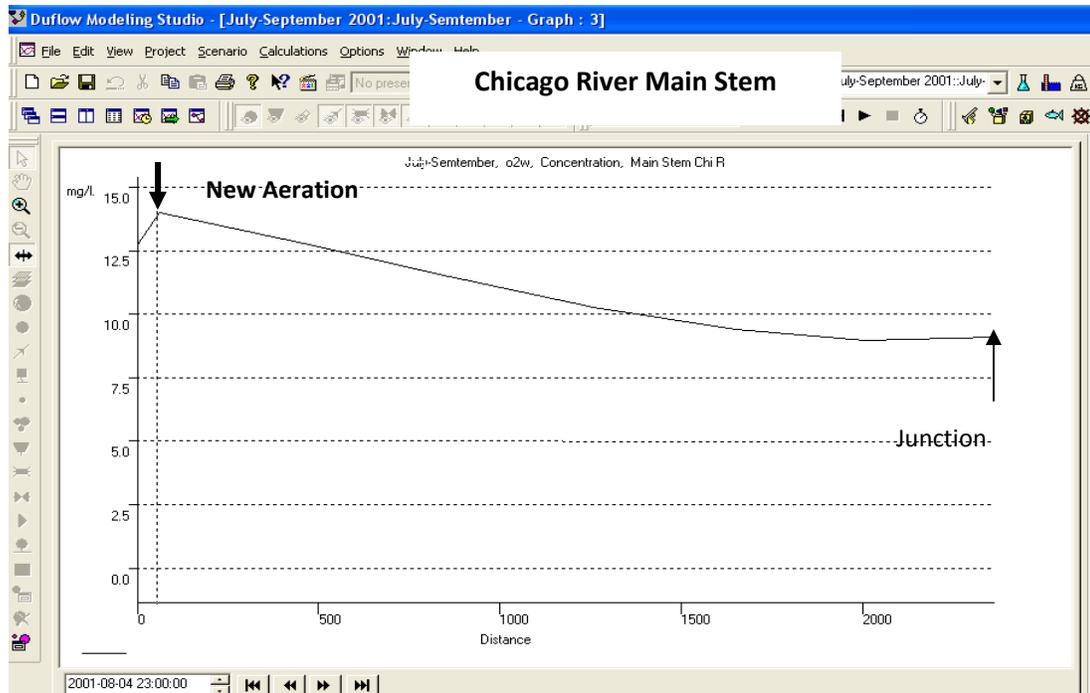
Figure 6. 4 Dissolved oxygen concentration profiles in the Chicago Waterway System for selected critical periods of August 4, 2001 (Chicago River Main Stem) and August 2, 2001(South Branch Chicago River) where the downward arrows indicate locations of new aeration stations
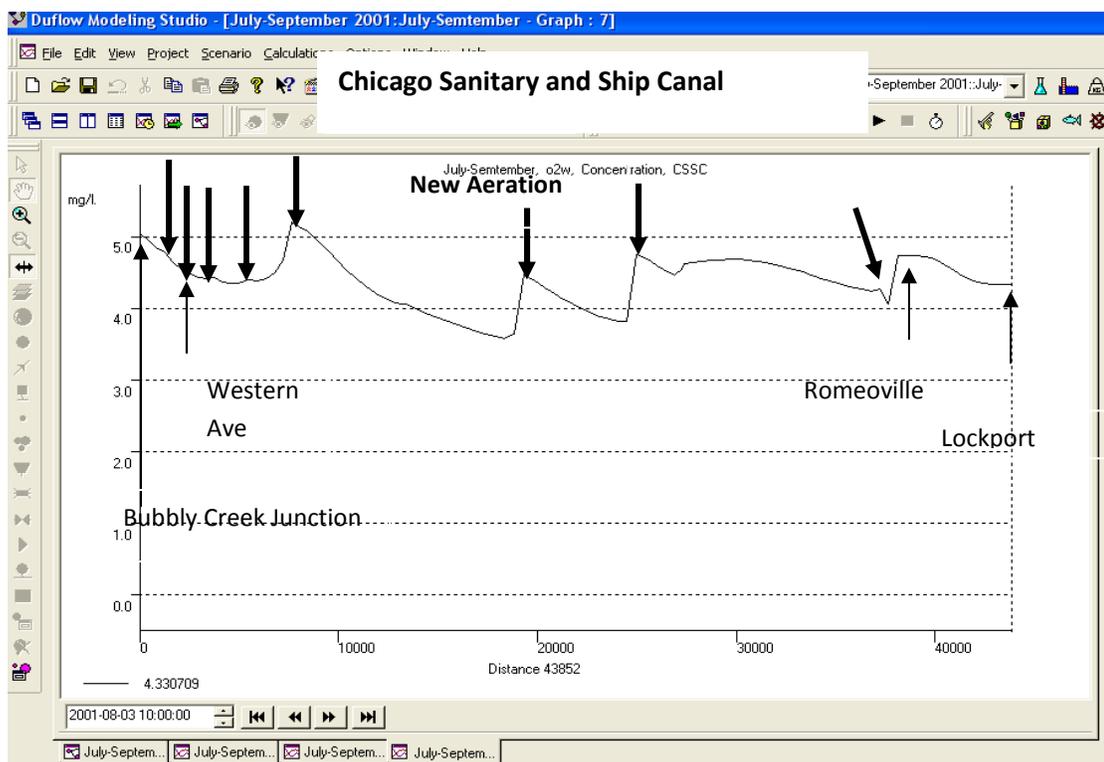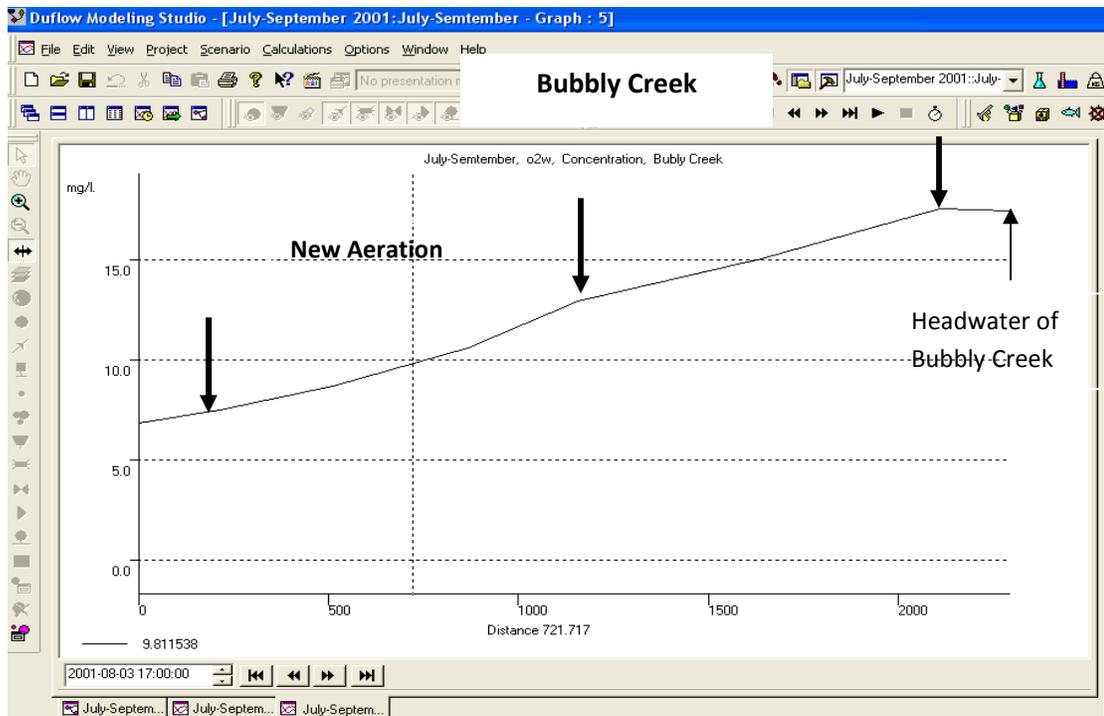
Figure 6. 5 Dissolved oxygen concentration profiles in the Chicago Waterway System for a selected critical period of August 3, 2001 (Bubbly Creek and Chicago Sanitary and Ship Canal) where the downward arrows indicate locations of new aeration stations
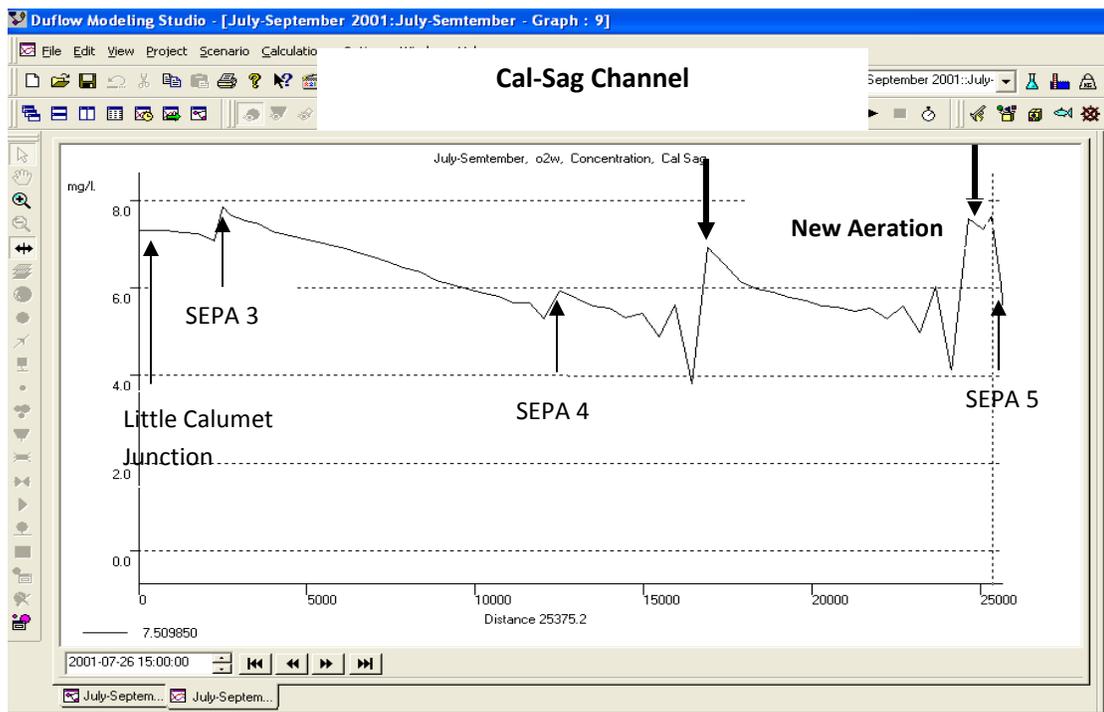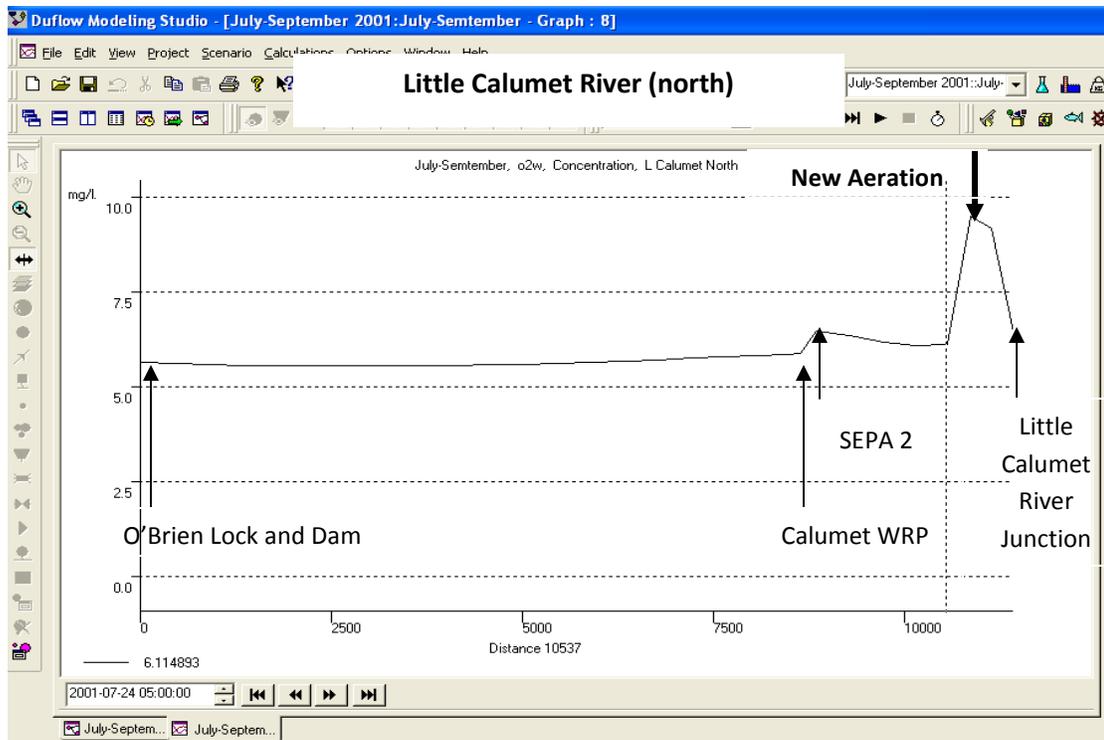
Figure 6. 6 Dissolved oxygen concentration profiles in the Chicago Waterway System for selected critical periods of July 24, 2001 (Little Calumet River north) and July 26, 2001(Cal-Sag Channel) where the downward arrows indicate locations of new aeration stations

The determination of the operation hours for each new aeration station is feasible. Actual DO problem hours and periods of CSO occurrence were taken into account. In addition, considering flow travel time most new added aeration stations need to turn on 12-hours in advance of the periods of low DO concentrations.

6.2.2. October 1, 2002-September 30, 2003 (Water Year 2003)

The baseline simulation results for WY 2003 are better than WY 2001. Especially, the result that no new aeration stations are needed on Bubbly Creek. A similar procedure as for WY 2001 was applied in the model, to achieve 100% compliance with the IEPA proposed DO standard. Considering the construction cost and space availability, the locations of the new aeration stations of both WYs 2001 and 2003 were given the same placement to the greatest extent.

For WY 2003, because for the periods listed in Table 6.4 the measured DO concentrations were missing at the Wilmette Pumping Station, the measured DO concentration needed to be estimated in order to calculate the DO mass balance, which is used as the new DO input at Wilmette during flow augmentation from NSWRP to the Wilmette Pumping Station. Table 6.4 lists the periods of missing data and the estimated DO concentrations.

Table 6. 4 The periods of missing data and DO concentration estimates

Date	Time	Number of missing hours	Estimated DO concentrations (mg/L)
05/12/2003	0:00-23:00	24	8.0
05/13/2003	0:00-13:00	24	8.0
05/30/2003	0:00-7:00	8	3.0
06/07/2003	0:00-9:00	10	5.0
06/30/2003	5:00-7:00	3	3.7

Simulation results showed 16 new supplementary aeration stations with operation hours different from those for WY 2001 were needed to achieve 100% compliance with the proposed DO standards of 3.5 mg/L and 5 mg/L for periods of August-February and March-July, respectively, for Aquatic Life Use A waters and of 3.5 mg/L throughout the year for Aquatic Life Use B waters. Only periods of August-December and May-July were considered on the NSC, NBCR, SBCR, and CSSC. The locations, oxygen loads, and operation hours of the proposed aeration stations are listed in Table 6.3.

As shown in the Table 6.3, two new aeration stations would be needed on the upper NSC (the same location as the first and fourth aeration stations in WY 2001), whereas one aeration station would be needed for the lower NSC. For WY 2003, only one new aeration station on the NBCR (as was needed for WY 2001) was not enough to meet the proposed DO standards. Thus, another new aeration station one was added on the NBCR. Its location is shown in Figure 6.7. The water quality conditions were excellent on the Chicago River Main Stem so no new aeration station was added and the number of new aeration stations needed dropped by one relative to WY 2001. Two new aeration stations were needed at the downstream end of the

SBCR in WY 2003 corresponding to the final two locations on the SBCR needed for WY 2001. For Bubbly Creek, no new aeration stations would be needed, because flow transfer on the upper NSC, and two in-stream aeration stations at Devon Avenue and Webster Avenue, and the seven new aeration stations upstream provided plenty of oxygen for the creek. Compared to WY 2001, the number of new aeration stations was halved on the CSSC, but the DO concentrations were still above 3.5 mg/L at all locations. However, a transfer of 30 MGD of aerated flow from the Calumet WRP to O'Brien Lock and Dam cannot provide enough oxygen to meet the proposed DO standards along the Little Calumet River (north), therefore two more new aeration stations (for a total of three new stations) were added rather than one aeration station needed for WY 2001. The locations of these two additional aeration stations are also shown in Figure 6.7. Similarly, because the four SEPA stations are assumed to operate at full capacity, only two new aeration stations would be needed along the Cal-Sag Channel. Therefore, 16 new aeration stations would be added to achieve full compliance with the IEPA proposed DO standard for WY 2003. Fourteen of them operated with a maximum oxygen load of 80 g/s, while 2 aeration stations need to operate with a 100 g/s maximum oxygen load, one on the CSSC and the other on the Little Calumet River (north). Like the simulations for WY 2001, most of the new aeration stations need to turn on 12-hours before the periods of low DO concentrations due to the travel time of flow, whereas the two aeration stations on the NSC needed to operate 24-hours in advance.

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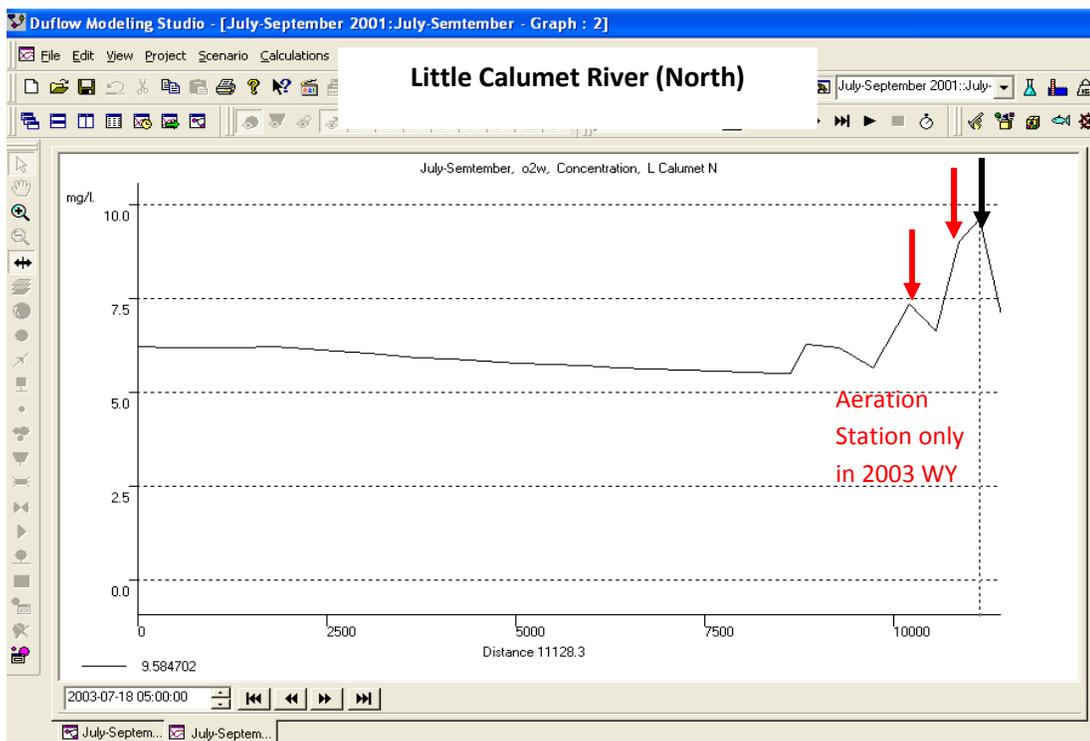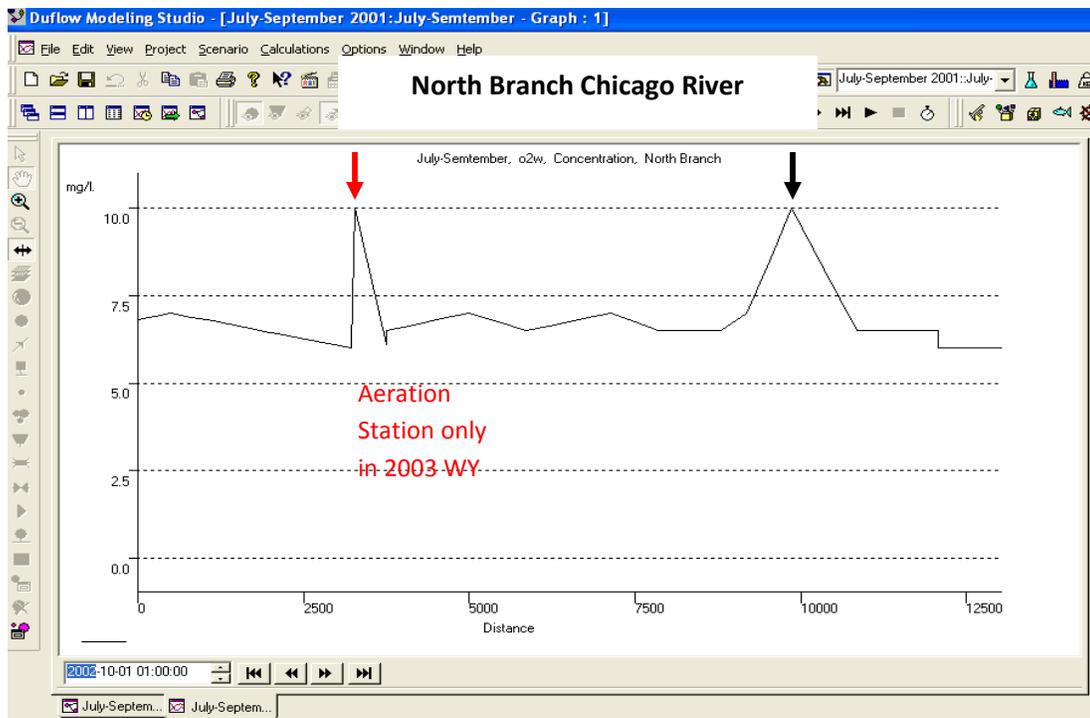
Figure 6. 7 Dissolved oxygen concentration profiles in the Chicago Waterway System for selected critical periods of October 1, 2003 (North Branch Chicago River) and July 18, 2003 (Little Calumet River north) where the downward arrows indicate locations of new aeration stations

On the basis of the analysis of the DUFLOW model for WYs 2001 and 2003, in total 28 new supplementary aeration stations with a maximum oxygen load of 80 or 100 g/s would be needed to achieve the IEPA proposed DO standards 100% compliance of the time for both wet and dry years.

CHAPTER 7: CONCLUSION AND RECOMMENDATIONS

Re-calibration of an unsteady water-quality model and hydraulic verification for the Chicago Waterway System (CWS) was completed for the periods of October 1, 2000-September 30, 2001(2001 WY) and October 1, 2002-September 30, 2003(2003 WY) after making some improvements to the previous model. The DUFLOW model of the CWS is able to simulate water quality under unsteady flow conditions, and can be used to assist water-quality management and planning decision making. The model was applied to evaluate the effectiveness of various integrated strategies to meet proposed DO standards for the CWS.

Two different types of proposed DO standards were evaluated for the 2001 and 2003 WYs: one developed by the IEPA and the other developed by the MWRDGC. First, 90% and 100% compliance with the IEPA proposed DO concentrations were evaluated. Then 100% compliance with the DO standards developed by the MWRDGC was evaluated.

From the baseline simulations, the NSC (Linden Street, Simpson Street, and Main Street), Bubbly Creek (for WY 2001 only), and Cicero Avenue on the CSSC (for WY 2001 only) did not meet the IEPA proposed DO standards 90% of the time. The combination of flow augmentation on the NSC, and on Bubbly Creek was developed to achieve 90% compliance for both years. Thirty MGD of aerated flow augmentation from the NSWRP to the upstream end of the NSC can achieve the compliance 90% of the time on the NSC in both WYs 2001 and 2003. Flow augmentation with and without aeration on Bubbly Creek also was evaluated. A 10 MGD transfer of aerated flow was sufficient to bring DO concentrations to target

levels on Bubbly Creek and the CSSC, whereas, a 70 MGD transfer of unaerated flow was required to meet the proposed DO standards 90% of the time on Bubbly Creek and the CSSC for WY 2001. Thus, using unaerated flow transfer on Bubbly Creek to achieve 90% compliance with the proposed DO standards is not an effective method.

In order to meet the MWRDGC's proposed standards, a method combing a 24 MGD transfer of aerated flow on the NSC with adjustment of the operating hours of the Devon Avenue in-stream aeration station and 2 new aeration stations on the SBCR can be an effective management alternative to increase DO concentrations to desired levels for 2001 WY, whereas only 24 MGD of aerated flow augmentation plus 1 new aeration station on the SBCR can meet the MWRDGC standards for WY 2003. A maximum oxygen load of 80 g/s is applied for three new aeration stations.

The most difficult condition to achieve is 100% compliance with the IEPA's proposed DO standards for WYs 2001 and 2003. First, aerated flow augmentation was applied on the NSC, Bubbly Creek, and the Little Calumet River (north), and then new aeration stations were added in the CWS. The size, locations, and operating hours of the supplementary aeration stations were determined. For WY 2001, it was determined that total of 25 new aeration stations along the CWS distributed as 5 new aeration stations on the NSC, 1 new aeration station on each of the NBCR, Chicago River Main Stem, and Little Calumet River (north); 4 new aeration stations on the SBCR; 3 new aeration stations on Bubbly Creek; 8 new aeration stations on the CSSC; and 2 new aeration stations on the Cal-Sag Channel, can achieve 100% compliance. For WY 2003, 16 new aeration stations were needed in the CWS distributed as 3 new aeration stations on the NSC; 2 new aeration stations for each of the NBCR, SBCR,

and Cal-Sag Channel; 4 new aeration stations on the CSSC; and 3 new aeration stations on the Little Calumet River (north). In addition, 2 of the new aeration stations needed maximum oxygen loads of 100 g/s for WY 2003 instead of 80 g/s for WY 2001. Because of different operation hours for each new aeration station and travel time issues between aeration stations and trouble spots, it is hard to decide the on-and-off time for the new aeration stations in real time. At the same time, it is possible that for another year a localized high load during a storm could result in violation of the DO standards even with 28 additional aeration stations and 6 existing aeration stations (in the modeled portion of the CWS). Thus, it is difficult to guaranty 100% compliance.

Therefore, considering feasibility, achieving the IEPA's DO standards 90% of the time is recommended rather than 100% of the time. Meanwhile, MWRDGC's proposed DO standards can be met easily.

Based on the model simulation and cost assumptions previously described, AECOM-CTE estimates the order of magnitude capital costs to meet the MWRDGC's proposed DO standards to be $50,410,000. Total annual operating costs are estimated to be $523,000. The total present worth is estimated at $60,434,000. The cost estimates of achieving the IEPA DO standards 90% and 100% of the time currently are being developed in process and cannot be included in this thesis.

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