US Army Corps of Engineers®
Chicago District

BUBBLY CREEK, SOUTH BRANCH OF THE CHICAGO RIVER, ILLINOIS FEASIBILITY STUDY

APPENDIX A
HYDROLOGY & HYDRAULICS

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Bubbly Creek
Ecosystem Restoration Feasibility Study

APRIL 2015

BUBBLY CREEK, SOUTH BRANCH OF THE CHICAGO RIVER, ILLINOIS ECOSYSTEM RESTORATION FEASIBILITY STUDY

APPENDIX A – Hydrology & Hydraulics

April 2015

Executive Summary

The hydrology and hydraulics (H&H) appendix documents details the development of a hydraulic SWMM model for Bubbly Creek, the results of the SWMM and TNET modeling, the HEC-RAS modeling and evaluation of stages due to ecosystem restoration measures, the ERDC Hydrodynamic and Water Quality Modeling for Bubbly Creek scope of work, the ERDC hydrodynamic modeling of Bubbly Creek, and the ERDC water quality modeling of Bubbly Creek.

List of Attachments

Attachment 1:

Development of a Hydraulic SWMM Model for Bubbly Creek

Development of a Hydraulic SWMM Model for Bubbly Creek

Prepared for:
U.S. Army Corps of Engineers
Chicago District

AECOM

November 30, 2012

DEVELOPMENT OF A HYDRAULIC SWMM MODEL FOR BUBBLY CREEK
USACE CONTRACT NO. W912P6-05-D-0002, TASK ORDER 0012

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I. INTRODUCTION

I.A. Historical Background

The South Fork of the South Branch of the Chicago River, referred to as Bubbly Creek, is a 1.25 mile channel located entirely within the City of Chicago, Cook County, Illinois. The channel begins near Racine Avenue and 38th Street at the Racine Avenue Pump Station (RAPS) and flows north into the South Branch of the Chicago River near Ashland Avenue as shown below. Bubbly Creek acquired its name from the gas bubbles that form and rise to the surface from decomposition of organic matter deposited from the Chicago Stockyards during the period 1865 to 1971.

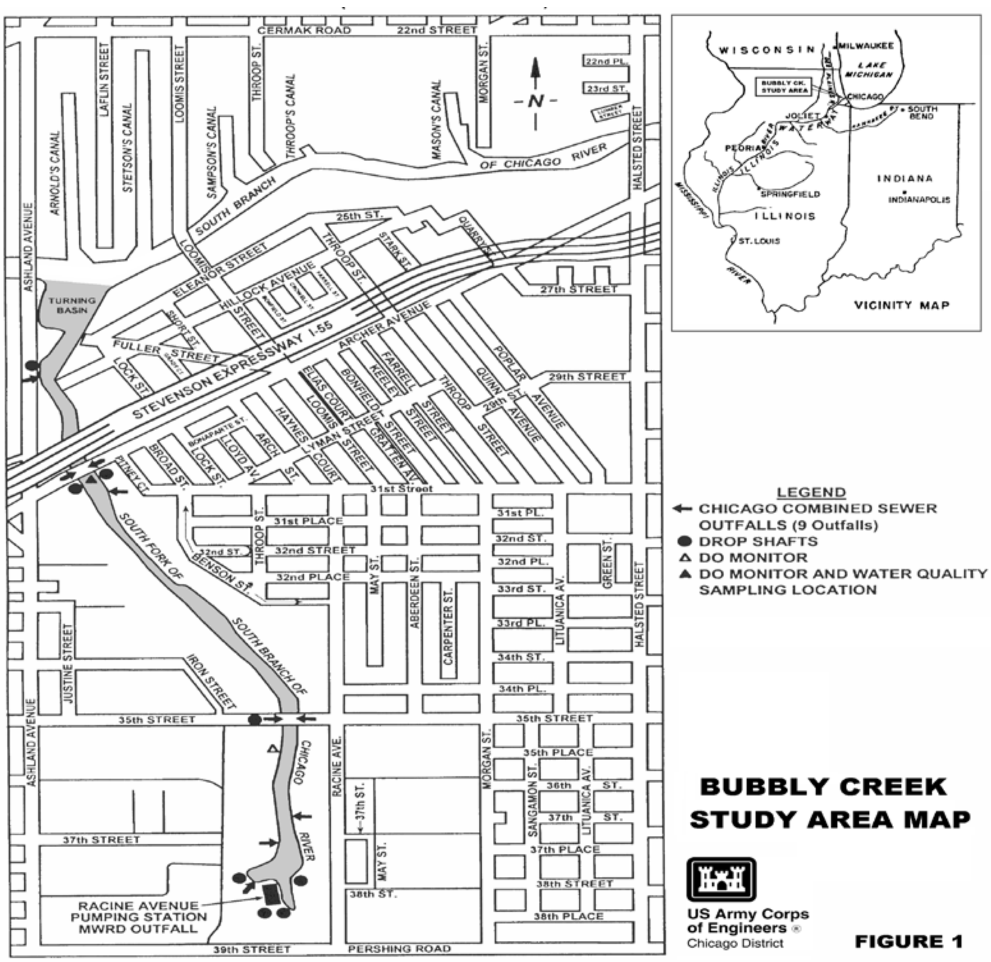


FIGURE 1

The South Fork of the South Branch of the Chicago River and its tributaries were once clear meandering creeks that slowly drained the vast marshland that occurred within its 5 square mile drainage area. In the early 1860's the Union Stock Yards were constructed along the banks of the South Fork and this small stream became an open sewer and disposal site for large quantities of blood, offal, hair, and other animal wastes from the meatpacking industry. Biochemical reactions caused by decomposing animal waste produces methane and hydrogen sulfide bubbles that constantly float to and break at the surface, for which the name "Bubbly Creek" is colloquially given. In 1923, the last tributary to Bubbly Creek, West Arm of the South Fork, was completely filled in as a remediation solution to the vast quantities of waste dumped in that channel. The Union Stockyards closed in 1971 after 105 years.

During the development of Chicago in the late 1800s and early 1900s, a vast sewer system was constructed to collect sanitary waste and storm runoff and convey it via massive underground combined sewers to the areas river system. A 30-square mile area of the central and south side of the City of Chicago originally drained to Bubbly Creek by gravity. Conditions in the channel degraded to a point where a bypass connection was constructed to pump fresh water from Lake Michigan to flush the system during dry weather. In 1939, the world's largest pump station, Racine Avenue Pumping Station (RAPS), was constructed and dry weather flows were diverted to the Stickney Water Reclamation Plant (SWRP) for treatment instead of directly discharging raw sewage to Bubbly Creek. Over the years, increases to treatment capacity at SWRP have reduced the amount of overflows which occur. The construction of the Tunnel and Reservoir Plan (TARP), which encompasses a system of deep tunnels and massive reservoirs used to store overflows, have drastically reduced the amount of combined sewer overflows (CSOs) to area rivers. Currently the tunnel portion of the project is complete, thus reducing the number of CSOs at RAPS. Unfortunately, even with the TARP project complete, overflow capacity will be required at RAPS in order to prevent local flooding and basement backup during large storm events.

Today, Bubbly Creek is a relatively straight 6,600-foot channel that originates at the RAPS and flows north to the confluence with the South Branch of the Chicago River during overflow events. The channel is mostly lined with vertical walls made of steel sheet pile, concrete, or wood and few areas of steep rocky soils. A mix of land uses are found along the banks of Bubbly Creek including industrial plants, trucking terminals, rail yards, and construction material yards which are giving way to new commercial and residential development. Channel depths vary from approximately 6-feet near RAPS to 14-feet at its mouth and channel widths vary between 120 to 200-feet wide. Due to hydrologic alterations, existing bottom sediments, combined sewer overflows, and lacking of riparian and in-stream habitats, Bubbly Creek remains a severely impaired ecosystem with vast opportunities for restoration.

I.B. Problem Identification

Below is a list of specific problems that contribute to the degradation of Bubbly Creek.

I.B.1. Stagnant Flow Conditions

During dry weather periods Bubbly Creek is stagnant, except for the occasional movement of water caused by a passing boat or slight surge from the South Branch. Following light to moderate rainstorms, flow in Bubbly Creek is not noticeably changed since most rainfall runoff is captured in the combined sewer system and conveyed for treatment and released downstream. Only small areas adjacent to the channel drain directly to Bubbly Creek and runoffs are too small to cause any changes in flow. Due to this short-circuiting affect on drainage, Bubbly Creek functions more like a lake system than a river system a majority of the time. During stagnant periods, severely degraded water quality in Bubbly Creek can be attributed to several factors including the biochemical interaction between the sediment and the water column, residual water quality from CSOs, and photosynthetic activity. Levels of dissolved oxygen (DO), which are good indicators of water quality impairment, typically plummet during stagnant periods and often reach zero.

Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) which operates RAPS conducted a demonstration project where dry weather flows were artificially introduced to Bubbly Creek by opening a gate at RAPS to allow water to enter and be pumped for treatment, thereby establishing a reverse flow in the creek when otherwise it would have been stagnant. MWRDGC performed this demonstration project for two summers with success in improving dry weather water quality at a cost of nearly 1.2 million dollars in added operating costs. It was determined that the creation of an artificial flow during dry weather flows can drastically improve water quality, but the method of artificial flow creation used in this project cannot be used as a long-term solution for the water quality improvements in Bubbly Creek since it requires additional treatment capacity that may not be available in wet weather and entails significant additional operating costs.

I.B.2. Combined Sewer Overflows

During excessively heavy rainfall events, the combined sewer system that drains surface water runoff and sanitary waste by gravity to RAPS can become overwhelmed. In order to prevent local flooding and basement backup within the sewershed, pumps at RAPS are turned on to discharge CSO to Bubbly Creek when the capacity of the sewer system is reached. When this occurs, the water level in the creek rises forcing the CSO to flow north toward the South Branch. At maximum overflow capacity, RAPS can discharge approximately 6,000 cubic feet per second, raising the upstream water level about 3 feet and increasing the channel water velocity to as much as 5 feet per second. During overflow events the water quality in the channel is severely degraded as CSO contains significant quantities of fresh sewage, street runoff solids, and some floatable materials. In addition to water quality degradation, riverine habitats are severely impacted due to high channel velocities caused by CSO discharges.

In the ten-year period between 1992 through 2001, overflow pumping to Bubbly Creek at RAPS had occurred 17 times per year on average. The highest was 27 times in 1993 and the lowest was 10 times in 1997. The duration of pumping varied from a few hours to a day or more, depending on the amount and duration of rainfall. The completion of the Tunnel and Reservoir Plan (TARP), which encompasses a system of deep tunnels and massive reservoirs used to store overflows, will reduce the frequency of overflows to Bubbly Creek. Unfortunately, the TARP project will not eliminate all CSOs, therefore pumping from RAPS will continue to occur when intense storms with large rainfall amounts hit the south side of Chicago.

I.B.3 Sediment Quality

The sediments within the Bubbly Creek channel contain remnants of animal wastes such as carcasses, hair, and offal from the meat processing plants that previously lined its banks, raw sewage once directly dumped into the channel, and solids contained in combined sewer overflows still released by RAPS and other CSO outfalls along the channel. The Illinois Environmental Protection Agency (IEPA), U.S. Environmental Protection Agency (USEPA), Metropolitan Water Reclamation District of Greater Chicago (MWRDGC), and USACE have all performed past sediment sampling and bulk chemistry analyses are consistent among these sampling events. The bulk of sediment information available was collected by the Chicago District in the spring of 2004. Thirteen core samples and five grab samples along the entire length of Bubbly Creek were sampled and analyzed. Sediment depths ranged between 5.5 and 16.8 feet and consisted primarily of sand and clay. Analytical results were compared to the U.S. Environmental Protection Agency (USEPA) toxicity characteristic leaching procedure (TCLP) and the Toxic Substances Control Act (TSCA). All samples collected and analyzed fell below these regulatory levels.

Sediment samples all showed elevated levels of polyaromatic hydrocarbons (PAHs) and heavy metals. Other detected contaminants included semi volatile organic compounds (SVOCs), volatile organic compounds (VOCs), polychlorinated biphenyls (PCBs), oil and grease, and nutrients. Biochemical reactions within the sediment caused by anaerobic organic decomposition produce methane and hydrogen sulfide bubbles that constantly float to the surface sometimes carrying clumps of sediment when made buoyant by entrapped gas bubbles. These clumps eventually sink when entrained gas vents to the atmosphere. Odors produced by the gases and the appearance of these clumps are aesthetically unpleasant.

I.B.4 Water Quality

In general, the water quality in the Chicago Waterway system is marginal, but constantly improving. Bubbly Creek is classified for *Secondary Use* by the Illinois Pollution Control Board (IPCB), which indicates the water is only suitable for limited contact activities such as boating and fishing. Bubbly Creek is also listed as an impaired stream by IEPA according the Section 303(d) of the Clean Water Act. The listed causes of impairment include high pH, low dissolved oxygen, and high total phosphorus with combined sewer overflows as the primary source of impairment. Stagnant flow conditions and the biochemical interaction with contaminated sediments also contribute to water quality degradation. Water quality is critical to maintaining high quality habitats needed to support diverse fish and wildlife populations. Poor water quality severely limits the aquatic habitat and communities within Bubbly Creek

I.B.5 Habitat and Biological Integrity

Currently, Bubbly Creek no longer provides a broad diversity of habitats nor the habitat quality necessary to maintain ecological functions and support healthy populations and communities of plants and animals. The health of the Bubbly Creek ecosystem has severely declined in response to a loss of habitat to support various life stages of aquatic and terrestrial biota and a reduction in habitat quality due to several factors. The lack of flow diversity caused stagnant flow conditions and high velocities from combined sewer overflows has resulted in severe habitat degradation. Poor sediment quality and the biochemical reactions from organic decomposition further degrade the aquatic habitat for fish and macroinvertebrates. Poor water quality caused by combined sewer overflows, hydrologic alterations, and reactions with underlying sediments also contribute to habitat degradation. The channel is absent of any aquatic vegetation, which provide critical habitat for fish, insects, and bird species.

An Index of Biotic Integrity (IBI) was used to assess the status and probable impacts to aquatic communities. The IBI may be viewed as a quantitative empirical model for rating the health of an aquatic ecosystem with a scale between 0 and 60. A fish survey was performed and six tolerant species were collected. Based on structural, compositional, and functional components of the fish community surveyed, Bubbly Creek received an IBI score of 10. This score corresponds to a very poor rating and is characterized as an imperiled aquatic ecosystem in which biotic integrity has been severely reduced. This rating coincides with the presence of only tolerant and non-native fishes.

I.B.6 Recreation

Limited recreational activities occur along Bubbly Creek. At the confluence with the South Branch of the Chicago River, the South Chicago Rowing Center has a small boat launch. Additionally, the City of Chicago constructed a canoe launch and pull-over and drop-off point for canoes at the Illinois and Michigan Canal Origins Park at the confluence of Bubbly Creek. Bank fishing is also common at the confluence of Bubbly Creek. Many developments that are being constructed along the Chicago River including Bridgeport Village, a single-family residential development area along a portion of the east bank of Bubbly Creek. Many of these developments are creating river walks to connect the waterways to residents. Due to the poor water quality and the lack of aquatic habitat and biological integrity, additional recreational opportunities are limited. Foul odors and unsightly floating debris also detract recreational users from Bubbly Creek.

I.C. Project Description

The Corps has undertaken a feasibility study to identify and evaluate alternatives for ecosystem restoration of Bubbly Creek. One aspect of this study is to develop an accurate simulation of combined sewer flows entering the RAPS wet well, the pumping operation of RAPS, the direct discharges to Bubbly Creek from RAPS, and combined sewer overflows (CSO) from outfalls north of RAPS. The specific goal of this task order is to develop a hydraulic sewer routing model using the Storm Water Management Model, SWMM. This model would simulate the combined sewer tributary areas that flow into RAPS as well and for the smaller areas that overflow directly to Bubbly Creek north of RAPS. The model will provide both combined sewer flow and water quality simulations for these areas as well as modeling the operations of RAPS including pumping to interceptors and TARP tunnels and combined sewer overflow pumping to Bubbly Creek.

II. DATA COLLECTION

Data for this project was collected from the Chicago District Corps of Engineers, City of Chicago Department of Water Management (CDWM), Metropolitan Water Reclamation District of Greater Chicago (MWRDGC), and CH2MHill (consultant working for CDWM). The data consisted of plans, sewer atlases, existing hydrologic and hydraulic model input files, operation plans, operational data, and meteorological data.

The Chicago District Corps of Engineers provided the existing HSPF/SCALP model input files used in Lake Michigan Diversion Accounting and CUP projects, the latest version of the TNET software and related input files, HSPF/SCALP subbasin maps, and Illinois State Water Survey and National Weather Service meteorological data. Preliminary operation plans for the operation of the MWRDGC TARP Mainstream tunnel and reservoir were provided.

CDWM and CH2MHill provided their existing Wallingford InfoWorks model network and input files as an export file in the SWMM5 format and the InfoWorks file format for the areas tributary to the Racine Avenue Pump Station (RAPS). Sewer atlases of this area were also provided.

MWRDGC provided plans of the Mainstream TARP drop shafts, tunnels, and connecting structures that served the area tributary to RAPS. MWRDGC also provided plans/atlases of their interceptors that are tributary to RAPS. A meeting with MWRDGC, Corps of Engineers, CDWM, and AECOM was held at the Stickney Wastewater Reclamation Plant to discuss how MWRDGC currently operates RAPS during dry and wet weather events. This meeting also included a tour of RAPS.

III. HSPF MODEL REVISION

III.A. Hydrology

The original HSPF/SCALP model used by the Corps in their Chicagoland Underflow Plan (CUP) and Lake Michigan Diversion Accounting projects represented the project area using 7 subareas. The scope of work for the current project was based on splitting these HSPF areas into smaller areas using the same IMPRO/OLFRO/SUBRO input parameters from the original subareas. At the project inception, the City of Chicago offered to provide the Corps with portions of their City-Wide InfoWorks model, which was in development at the time, for the CSO areas tributary to RAPS and Bubbly Creek. However, the level of detail in the InfoWorks model was much greater than originally planned for the SWMM5 model. The InfoWorks hydrology was based on approximately 4,000 subareas of approximately 20 acres in size. The scope of work for the SWMM5 model had assumed that there would be up to 60 subareas representing the hydrology.

In addition, the hydrology for the InfoWorks model was developed for single event design storms. The project scope of work was based on using continuous period modeling. The significantly greater detail and effort to convert and calibrate the InfoWorks for continuous simulation made using the InfoWorks model hydrology onerous for this project. Another reason for not using InfoWorks is that the hydrology for the rest of the CSO areas tributary to the Chicago Area Waterway System (CAWS), of which Bubbly Creek is part, is simulated using HSPF/SCALP for input to TNET. TNET was being used to generate the TARP tunnel and reservoir boundary conditions for this project as well. It was important to maintain the consistency of the hydrologic modeling between the various portions of the project. For these reasons, HSPF/SCALP was used to simulate the hydrology of the watershed.

After completing the SWMM5 hydraulic network (discussed in the following sections), inflow points along the sewer system were identified based on the sewer sizes of the tributary sewers at the nodes. There were 97 inflow points identified in the study area. The routines in InfoWorks were then used to identify the tributary area to each node. After InfoWorks identified the tributary area to each of the inflow points, the subbasins were reviewed and revised manually to correct errors in the automated routines.

The SWMM5 subbasin network was then overlaid on the original HSPF/SCALP SCA (sub-catchment area) boundaries and rain gage Thiessen polygons. This overlay was used to identify and/or revise the impervious/pervious percentages, rain gage weightings, dry weather flow populations, and other parameters for each of the 97 subbasins.

III.B. Water Quality

The HSPF model was revised to represent the runoff of fundamental water quality parameters, specifically water temperature, Dissolved Oxygen (DO), Biochemical Oxygen Demand (BOD), and Total Suspended Solids (TSS). The water quality output from HSPF was then used as input to the SWMM model. Values for each water quality block were generally set to the model’s default value or based on standard values found in the literature. Since calibration of the model was not possible, these values were not significantly adjusted. Since default values were generally used, they will not be discussed in this summary. To maintain consistency, the same methods were used to develop water quality loadings (i.e. the output time series) as were used to develop runoff hydrographs. Due to the structure of the HSPF and SCALP models, several steps are required to develop output. As a result, the majority of water quality code developed for this study relates to the management (i.e. conversion and weighting) of the output so that it maintains a format compatible with the SWMM model. The following summary briefly explains the steps used to manage the water quality output.

Currently the HSPF model is generating a runoff hydrograph for each SCA (i.e. SCALP) boundary as input to SWMM. Within the SWMM model, each SWMM subwatershed references the SCA in which its centroid is contained, and a ratio is applied to the SCA hydrograph based on the ratio of the SWMM area to the SCA area. The only exceptions are those SWMM subwatersheds draining directly to the South Fork of the South Branch of the Chicago River (Bubbly Creek). The purpose of applying the ratio in SWMM (as opposed to applying the ratio in HSPF and producing a hydrograph for each SWMM subwatershed in HSPF) is to limit the file size by reducing the number of hydrographs. Those subwatersheds draining directly to Bubbly Creek are developed similarly to the SCA boundaries, except the size of pervious and impervious areas are calculated based on the exact amount of area that overlaps each SCA (i.e. as opposed to using one SCA area in which the centroid is contained). Therefore, the final hydrographs for those subwatersheds draining directly to Bubbly Creek do not have a ratio applied in SWMM. As a result, the water quality output from the HSPF model includes BOD, TSS, DO, and Temperature time series files for each SCA boundary, as well as time series for each SWMM subwatershed draining directly to Bubbly Creek. Each constituent's time series represents the combination of impervious and pervious sources. As mentioned, the majority of code is used to perform the "weighting" of each water quality constituent to address land use and area. A summary of this process is discussed below.

The existing HSPF hydrology model for the Bubbly Creek watershed calculates the unit hydrograph for grass and impervious land use areas for each Thiessen rain gauge polygon area. This step produces two unit hydrographs, pervious and impervious, for each Thiessen area. A unit hydrograph is then synthesized for each SCA area by summing the product of the pervious and impervious SCA areas that overlap each Thiessen Polygon with the pervious and impervious Thiessen unit hydrographs. An SCA boundary may overlap one Thiessen polygon, or it may overlap multiple polygons.

The one or more SCA unit hydrographs for each land use are then summed to develop SCA pervious and impervious time series. Then a single time series is developed for each SCA boundary by summing the previous and impervious time series. The SCA hydrograph is read from SWMM and the ratio of SCA area to subwatershed is applied within SWMM.

IV. SWMM MODEL DEVELOPMENT

IV.A. Pipe Network

The scope of work initially called for developing a model of the CSO system tributary to RAPS that included all sewers that were 5-foot in diameter and larger. Sewers smaller than 5-foot in diameter were to be included as necessary for the CSO areas north of RAPS to adequately represent the collection system tributary to outfalls less than or equal to 5-foot diameter. Initially the pipe network was to be built from CDWM and MWRDGC sewer atlases and MWRDGC contract plans.

Shortly after the project began, the City of Chicago offered to make their InfoWorks model that was currently in development available to assist with the project. The pipe network in the InfoWorks model included pipes 42-inches in diameter and larger. The model also contained pipes smaller in diameter where necessary to adequately represent the hydraulics of the system. The InfoWorks model did not include the level of detail required at RAPS or at the drop shafts located along Bubbly Creek. Using the InfoWorks model to replace the SWMM model for the project did not meet several of the project requirements. Primarily, InfoWorks was not capable of reading/writing HEC-DSS hydrograph files for the HSPF inflows and the overflows to Bubbly Creek. Since InfoWorks is a proprietary program, the option to modify the source code to read/write HEC-DSS files was not an option.

Because of the greater level of detail in the InfoWorks model, it was decided to use the InfoWorks pipe network information as the base data for the development of the SWMM5 model. InfoWorks has the capability to simplify the model based on criteria specified by the user. The InfoWorks routines were used to initially select pipe sizes at or above the 5-foot diameter size in the project criteria. This network was then reviewed for areas that required smaller pipe sizes to adequately model the CSO outfalls/drop shafts in the study area. After this base network was selected, an InfoWorks routine was used to eliminate nodes in runs of pipe of the same diameter. This network was then reviewed to identify areas where more detail was needed and areas where further simplification could be done. The final network simplification resulted in a system with 755 conduits, 727 manholes, and 66 outfalls. The 66 outfalls are not all outfalls to CAWS. In SWMM5 an outfall is a special node where water leaves the system. The outfalls consist of discharge points to CAWS, drop shaft inflows, TARP tunnel stub, and discharge points to other portions of the City sewer system and MWRDGC interceptors.

IV.B. Racine Avenue Pumping Station (RAPS) Operation

A set of operating rules were developed for the SWMM5 RAPS operation based on MWRDGC rules and discussion with the MWRDGC staff. The RAPS dry weather pumps that send flow to the Stickney Water Reclamation Plant (SWRP) were operated as shown in Table 1. The modeled total pump station dry weather capacity in SWMM5 was limited to 1,000 cfs to match the capacity used in the SCALP modeling for TARP and Lake Michigan Diversion Accounting.

Table 1 – Initial RAPS Pump Rules

Pump	Capacity (cfs)	Modeled Capacity (cfs)	Pump On Elevation (ft. CCD)	Pump Off Elevation (ft. CCD)
Dry Weather Pumps				
1	375	375	-27.0	-28.0
3	375	375	-26.5	-27.5
5	375	250	-25.0	-26.0
7	400	n/a	n/a	n/a
9	400	n/a	n/a	n/a
Wet Weather Pumps				
2	375	375	-19.0	-22.0
4	375	375	-18.6	-21.6
6	375	375	-18.2	-21.2
8	500	500	-17.8	-20.8
10	500	500	-17.4	-20.4
12	500	500	-17.0	-20.0
14	500	500	-16.6	-19.6
16	500	500	-16.2	-19.2
18	500	500	-15.8	-18.8

IV.C. Water Quality

IV.C.1. General Assumptions and Resources

The pollutant input structure to the SWMM model is similar to the hydraulic input structure and therefore carries the same assumptions. In the same manner as the hydraulic model, each of the 97 nodes identified as hydraulic time series input points were used to input water quality values for BOD, TSS, DO, and temperature. There are two components to the water quality model: surface water runoff and combined sewer base flows. Surface water quality was modeled using HSPF and incorporated in the SWMM through the direct input editor. Combined sewer base flow was modeled using the dry weather inputs editor and a series of time patterns. It should be noted here that SWMM does not have the capability to directly model temperature routing, so an indirect modeling approximation was used which is discussed in further detail below. The 20 second time step used in the hydraulic model was maintained for routing purposes, but reporting was changed to one hour increments.

IV.C.2. Model Input Preparation

IV.C.2.a. Surface inflow

HSPF was used to model surface water quality runoff in which mass flow time series for each pollutant and for each SCA area were produced as described in HSPF model section. These 20 SCA time series were input directly to each of the input nodes using the same conceptual manner as the hydraulic SWMM model and a mass conversion factor was applied to accommodate the concentration units (mg/L) defined in SWMM, converting from mass flow HSPF output. This conversion is necessary for SWMM to convert units of pollutant mass flow rate into concentration mass units per second, as indicated in Table 2. Additionally, the same SCA scale factors used for hydraulics were applied to the corresponding HSPF direct input time series.

Table 2. Direct Inflows Conversion Factors

WQ Pollutant	Inflow Model Type	HSPF Input Time Series Unit	SWMM Pollutant Model Unit	Conversion Factor
BOD	Mass	Lb/hr	Mg/L	125.9978
TSS	Mass	Lb/hr	Mg/L	125.9978
DO	Mass	Mg/hr	Mg/L	0.0002778
Temperature	Concentration	Deg F	Mg/L	1.0

Lb/hr are pounds per hour
Mg/hr are milligrams per hour
Mg/L are milligrams per liter
Deg F are degrees Fahrenheit

IV.C.2.b. Combined sewer base flow

Water quality of combined sewer base flows vary both spatially and temporally; therefore, pollutants were modeled by obtaining a representative average value for each pollutant and then creating time pattern variations around each value. Due to the lack of observed data in this specific system, in order to obtain a representative average value, public data from the receiving water reclamation plant (SWRP) was used.

Precipitation data and Tunnel and Reservoir Plan (TARP) return flow data were used to remove all non-base flow condition water quality data from the representative water reclamation plant data. These data were then summarized as a mean and monthly mean variation ratio for input to the SWMM dry weather inflows editor. This is discussed in more detail for each pollutant in the following sections and presented in Table 3. These values and time patterns were developed from data analysis and were checked against typical wastewater values outlined in “Wastewater Engineering,” Metcalf and Eddy, 2003.

As described below, certain periods of WRP data were obtained from MWRDGC published data. Attempting to use as much real data as possible, individual base flow time series for each subwatershed were developed for the entire period of record based on the WRP data; however, the large number of time series quickly exceeded memory capacity as SWMM has a limited capability to handle large time series. Therefore, the average value method was employed, focusing on more recent data and assuming that some pollutant loading accuracy would be lost as you temporally regress toward the beginning of the period of record. It should also be noted that nonrandom monthly variation was not found to occur within the TSS data set. As such, monthly time pattern ratios were not applied to TSS.

The goal of the modeling effort is to estimate the pollutant loading at CSOs and RAPS overflows to Bubbly Creek; it is therefore not the intent that pollutant concentrations upstream of the CSO and RAPS overflows are accurate to that specific area.

Table 3. Dry Weather Inflows Average Values and Time Pattern Variation

WQ Pollutant	Average Value	Time Pattern Type
BOD	416 Mg/L	Monthly and Daily
TSS	491 Mg/L	None
DO	2.22 Mg/L	Monthly
Temperature	60.2 Deg F	Monthly

Mg/L: milligrams per liter
Deg F: degrees Fahrenheit

IV.C.3. BOD

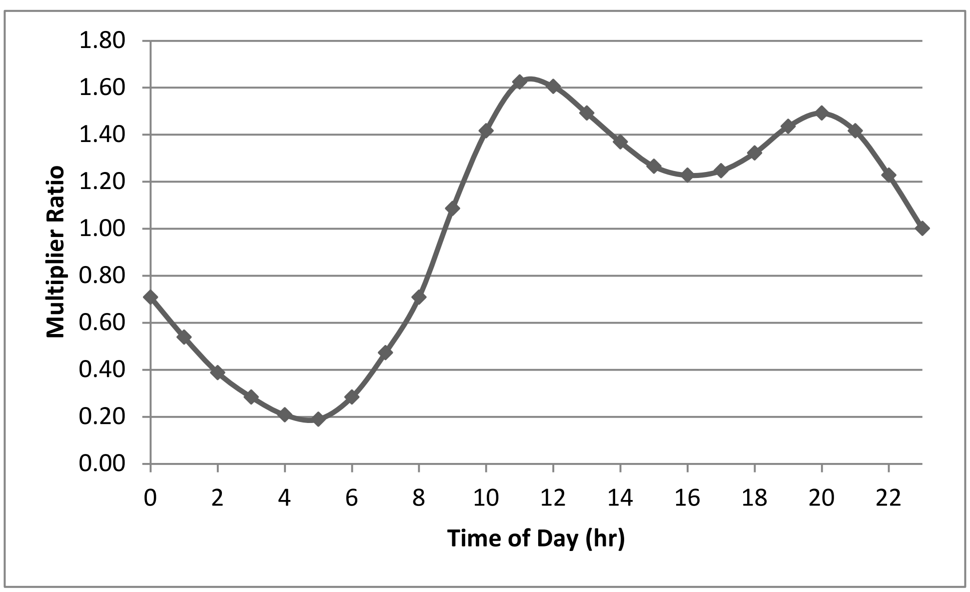
Daily BOD data were obtained from Stickney Water Reclamation Plant (SWRP), available from 1982 to 2008. Precipitation data and TARP data were available from 1990 to 2006. In order to remove effects of storm events within the data set for dry weather flow, BOD values with daily rain totals of 0.10 inches or more were excluded along with the following three days to allow sufficient time for the storm to flush through the system. After precipitation effects were removed, any days with TARP return flow greater than 10 MGD were also excluded. These remaining data were then analyzed, and it was determined that for the most accurate representation of current conditions only data from 2003 to 2007 would be used to calculate overall mean (Table 2) and monthly means. The monthly means were converted to a multiplier ratio with the average value for SWMM dry weather time pattern input as indicated in Table 4. This enabled SWMM to incorporate seasonal variation around the mean.

Table 4. Monthly Baseflow Time Pattern Ratios

Month	BOD	DO	Temperature
January	1.27	2.18	0.82
February	1.04	3.04	0.79
March	0.99	2.13	0.82
April	1.04	1.15	0.90
May	0.82	0.61	1.01
June	1.08	0.37	1.11
July	0.77	0.27	1.18
August	0.92	0.23	1.22
September	1.24	0.24	1.18
October	0.75	0.24	1.11
November	0.81	0.35	0.97
December	1.00	1.19	0.89

Typical hourly variation of BOD in domestic wastewater was extracted from “Wastewater Engineering,” Metcalf and Eddy, 2003, and applied as an hourly time pattern as shown in Figure 1.

Figure 1. Hourly Base Flow Time Pattern Ratios for BOD



IV.C.4. TSS

Daily suspended solids (SS) data were obtained from SWRP, available from 1982 to 2008. Precipitation data and TARP data were available from 1990 to 2006. In order to remove effects of storm events within the data set for dry weather flows, SS values with daily rain totals of 0.10 inches or more were excluded along with the following three days to allow sufficient time for the storm to flush through the system. After precipitation effects were removed, any days with TARP return flow greater than 10 MGD were also excluded. These remaining data were then analyzed, and it was determined that for the most accurate representation of current conditions only data from 2003 to 2007 would be used to calculate overall mean listed in Table 2. Nonrandom monthly variation was not found to occur within the data set and so monthly time pattern ratios were not applied to TSS.

IV.C.5. DO

Daily DO data were obtained from North Side Water Reclamation Plant (NSWRP), available for 2007 and 2008. This was the nearest location to the Bubbly Creek tributary area and period in which raw DO data were regularly collected and published. Precipitation data were also available from NSWRP during that time period. In order to remove effects of storm events within the data set for dry weather flow, DO values with daily rain totals of 0.10 inches or more were excluded along with the following three days to allow sufficient time for the storm to flush through the system. Return flows from TARP do not go to NSWRP, so these flow data were ignored. These remaining data were analyzed and used to calculate overall mean (Table 2) and monthly means. The monthly means were converted to a multiplier ratio with the average value for SWMM dry weather time pattern input as indicated in Table 3.

A first order decay factor was applied to this parameter. This factor was chosen during informal calibration of DO model results at RAPS with observed data provided by United States Army Corps of Engineers (USACE). A factor of 3 was chosen, which falls toward the high end of the range outlined in SDFSDF, 2009. The time of concentration from the upper most reaches to the TARP structures is approximately 5 hours, which is a fairly short time for the decay factor to act on DO concentration. This factor is required to bring down the relatively high HSPF surface input to observed data ranges.

IV.C.6. Temperature

Daily temperature data were obtained from SWRP, available for 2007 and 2008. This was the only period in which raw temperature data were regularly collected and published at SWRP. Precipitation and TARP data were also available from SWRP during that time period. In order to remove effects of storm events within the data set for dry weather flow, temperature values with daily rain totals of 0.10 inches or more were excluded along with the following three days to allow sufficient time for the storm to flush through the system. After precipitation effects were removed, any days with TARP return flow greater than 10 MGD were also excluded. These remaining data were analyzed and used to calculate overall mean (Table 2) and monthly means. The monthly means were converted to a multiplier ratio with the average value for SWMM dry weather time pattern input as indicated in Table 3.

As indicated earlier, SWMM does not directly model temperature routing. In order to approximate temperature at the outfalls, a flow weighted approach was used. Therefore, surface flow and base flow were modeled as concentrations. This combines the inflows at each node, but does not consider heat transfer to/from the underground conveyance system. A decay factor was not able to be applied to act as a heat loss mechanism because depending on weather/seasonal conditions, heat may be lost or gained from the soil surrounding the buried pipe network. It should be noted that due to the previously mentioned short time of concentration, the effect of these interactions is reduced.

IV.D. Software Modifications

The proposed software modifications were aimed at including the ability to read and write DSS files within the USEPA SWMM5 program. After examining the SWMM5 source code it was determined that the interface was developed in Borland's Delphi 7 and the computational engine was written in Microsoft C++ 2005. Borland's Delphi 7 software was no longer available. The software had been sold to another company. With the interface development software no longer commercially available, it was not possible to add the ability to read and write DSS files to the interface. The SWMM5 computational engine was examined to determine the feasibility of adding the ability to read and write DSS files in the computational engine. This had several disadvantages including that the program modifications would be hard coded into the program and would have to be rewritten/recompiled every time a change was needed to a DSS file location, name, etc. The modification would have to be added to the SWMM5 computational engine every time a new version was released by the USEPA. These disadvantages and the inability to modify the interface resulted in the decision to not modify the SWMM5 source code for the computational engine and interface.

The ability was still needed to read and write DSS files to allow the input of HSPF inflow hydrographs/pollutographs and the output of flow and stage hydrographs/pollutographs. SWMM5 has an option in the SWMM5 interface for User-written Tools. The development of a SWMM5 tool for reading and writing DSS files was examined as an alternative to modifying the computational engine or interface. Several preliminary tools were developed for SWMM5. One was a batch command that ran HEC-DSSVue to export DSS data to a file readable by SWMM5. An attempt was made to develop a separate program to read and write the DSS file flows and stages into a file readable by SWMM5. The common problems these attempts had were the inability to write an interface file that was readable by SWMM5 and any text files created were extremely large. The text files would have involved editing the information into the SWMM5 input file, but the size of the files would require a file editor that could handle extremely large text files. Neither of these options met the needs of the project to read and write DSS inputs in SWMM without additional editing.

During these SWMM5 modification attempts, discussions led to the possibility of modifying PC-SWMM from Computational Hydraulics. PC-SWMM has a proprietary interface to the USEPA SWMM5 program. PC-SWMM uses the standard SWMM5 computational engine for the hydraulic calculations and can import/export the standard USEPA SWMM5 input files. AECOM contracted with Computational Hydraulics to add the ability to read/write DSS files to PC-SWMM for flow, stage, and pollutants. This ability to read/write DSS files is now incorporated into their commercial release of the software. Therefore, AECOM was directed by the USACE to use PC-SWMM for the modeling in this study.

V. TNET MODEL

The Period-of-Record (POR) TNET model for the TARP tunnels included three scenarios:

1. The TARP tunnels with no reservoir
2. The TARP tunnels with a reservoir at the downstream terminus, and the inflow gates simulated as fully open
3. The TARP tunnels with a reservoir and a portion of the inflow gates simulated to close prior to the pressurization of the tunnels, and re-opened after the tunnel is fully pressurized.

The TNET model computes the overflow rate at each dropshaft during the simulation. The overflow can be caused by two different phenomena, and are summarized and included in the simulation output. The first type of overflow can be described as flow rejection, which is due to the dropshaft being simulated as closed during a period when the interceptors are delivering runoff toward the TARP system. This type is very dropshaft-specific, and controlled by the TNET input that relates the dropshaft opening percentage as a function of the hydraulic grade line in the tunnel. The second type of simulated overflow is simply the flow that is resurfacing from the tunnel while portions of the tunnel are pressurized up to local ground level.

The primary intent of simulating scenarios 2 and 3 was to determine the general increase in theoretical overflows produced by throttling the inflows (in scenario 3) to reduce the threat or frequency of surge events in the tunnel. A review of the simulation results indicates that during some events and in certain locations, the throttling of inflows actually reduces some of the overflow quantities. The explanation for this result is most likely that the throttled operation (scenario 3) produces fewer tunnel pressurization periods over the POR. While it is likely that some dropshafts will be routinely throttled during the pressurization process, the quantity and frequency of overflows at any particular dropshaft will be strongly affected by the inclusion/exclusion of that dropshaft in the throttling scheme.

Another note regarding the model results is the lack of the ability to perform simulations that include tunnel operations based on forecasted events, which results in the model overestimating the overflow quantities that occur on the receding limb of small rainfall events. It is difficult to assess the degree of conservativeness in the model results, but it does have some effect.

VI. SWMM MODEL CALIBRATION

VI.A. Hydraulic Calibration

The SWMM5 model was calibrated using the six year period, Water Year 2000 (October 1, 1999) through Water Year 2005 (September 30, 2005). This period was selected due to the availability of reported pumping volumes to Bubbly Creek at RAPS. The period was broken in to two periods. The first period (WY 2000 to WY 2002) was used for calibration and the second period (WY 2003 to WY 2005) was used for verification.

The hydrologic input data (temperature, rainfall, etc.) was provided by the Corps of Engineers. This data is used by the Corps for the Lake Michigan Diversion Accounting project. The observed RAPS pumping volumes to Bubbly Creek were obtained from MWRDGC. The HSPF model was run for the period January 1, 1999 to September 30, 2005 to generate the inflow hydrographs for SWMM5.

The results of the initial run were compared to the historic observed overflows. There were 47 observed events in WY 2000 to WY 2002. The SWMM5 model predicted 51 events during this period. The SWMM5 model predicted 13 events where overflows did not occur and missed 9 events where overflows occurred. The observed pumping volume is very low compared to the SWMM5 computed volume. The SWMM5 model over predicted the overflow volumes in 15 events and under predicted the overflow volumes in 45 events. For the WY 2003 to WY 2005 period, there were 56 observed overflow events. The SWMM5 model predicted 20 events where overflows did not occur and missed 6 events where overflows did occur. The SWMM5 model over predicted the overflow volumes in 22 events and under predicted the overflow volume in 34 events.

VI.A.1. Calibration Run No. 1

For the first calibration run, the maximum dry weather flow pumping from RAPS to Stickney WRP was decreased from 1,000 cfs to 750 cfs. In the SWMM5 model, only Pump Nos. 1 and 3 were included in the simulation. The wet weather pump operation was unchanged. The tunnel level at which the gates at Drop Shafts 26, 27, 28, and 29 closed was changed from 50% full to 70% full. The aim of these changes is to increase the volume of the pumped overflow at RAPS. The reduction in dry weather flow pumping to

Stickney WRP is based on discussions with MWRDGC operators that the maximum flow pumped to Stickney WRP is adjusted based on the other inflows to Stickney WRP. During some storm events, the pumping from RAPS to Stickney WRP may be stopped and all inflows to RAPS will be pumped to Bubbly Creek.

These changes increased the SWMM5 computed overflow volumes and a better match with the observed volumes. However, the number of predicted events increased when there was no observed event increased from 22 to 31.

VI.A.2. Calibration Run No. 2

For this run, changes were made to the operation of the dry weather flow pumps. The goal of these changes was to decrease the number of mispredicted events while increasing the total volume pumped to Bubbly Creek. The changes made are as follows:

1. When Wet Weather Pump No. 2 turns ON, Dry Weather Pump No. 5 turns OFF.
2. When Wet Weather Pump No. 4 turns ON, Dry Weather Pump No. 3 turns OFF.
3. When Wet Weather Pump No. 6 turns ON, Dry Weather Pump No. 1 turns OFF.
4. Total dry weather pumping capacity was set back to 1,000 cfs as in the initial run.

This sequencing of the dry weather and wet weather pump operations resulted in a better match in total volume and reduced the number of mispredicted events where SWMM 5 computed overflow events and there were no observed overflows. The largest discrepancy in predicted versus observed overflow volumes was in the events where the observed overflow volume was greater than 900 MG. The SWMM5 model under predicted the overflow volumes for these large events.

VI.A.3. Calibration Run No. 3

For this run changes were made in the dry weather pumping capacity and the order the wet weather pumps turn on. The off elevation for the wet weather pumps was lowered to allow the pumps to run longer. The changes made in this run are as follows:

1. Maximum dry weather pumping rate was increased to 1,125 cfs.
2. The order of operation of the wet weather pumps was changed from 2, 4, 6, 8, 10, 12, 14, 16, and 18 to 2, 8, 10, 12, 14, 16, 18, 4, and 6.
3. When Wet Weather Pump No. 2 turns ON, Dry Weather Pump No.5 turns OFF.
4. When Wet Weather Pump No. 8 turns ON, Dry Weather Pump No. 3 turns OFF.
5. When Wet Weather Pump No. 10 turns ON, Dry Weather Pump No. 1 turn OFF.
6. The wet weather pump OFF elevations were lowered by 3.5 feet.

These changes resulted in better pumped volume comparison for both periods. The increased dry weather pumping capacity did not result in a significant change in the number of mispredicted events where SWMM5 computed an overflow and there was not an observed overflow.

After discussions on possible pump station operation rule changes and/or hydrology changes, it was decided to see how the SWMM5 model performed for small, medium, and large storm events. This was based on the many very small predicted overflow volumes when there were no observed events and that the large events (>900 MG) were under predicted in the SWMM5 model. The results of these comparisons for the small, medium, and large events are shown in Tables 5, 6, and 7. These comparisons are for events where there was an observed overflow.

Table 5 – Comparison of Calibration Run No. 3 SWMM5 Model Run to Historic Observed Pumped Overflows for Small Storms (<250 MG)

	Observed Volume (MG)	SWMM5 Volume (MG)	Difference (MG) SWMM5 - Observed
Total Volume	4160.6	4695.4	534.8
Average Volume	154.1	173.9	19.8

Table 6 – Comparison of Calibration Run No. 3 SWMM5 Model Run to Historic Observed Pumped Overflows for Medium Events (250 MG < x < 600MG)

	Observed Volume (MG)	SWMM5 Volume (MG)	Difference (MG) SWMM5 - Observed
Total Volume	10660.7	9593.9	-1066.7
Average Volume	380.7	342.6	-38.1

Table 7 – Comparison of Calibration Run No. 3 SWMM5 Model Run to Historic Observed Pumped Overflows for Large Events (> 600 MG)

	Observed Volume (MG)	SWMM5 Volume (MG)	Difference (MG) SWMM5 - Observed
Total Volume	10264.8	5430.5	-4834.3
Average Volume	1140.5	603.4	-537.1

For the small events, SWMM5 slightly over predicted the overflow volume. The number of over predicted events is 14 and the number of under predicted events is 13. The error in the total volume for the small events is +13%.

For the medium events, SWMM5 is slightly under predicting the overflow volume. The number of over predicted events is 13 and the number of under predicted events is 15. The error in the total volume for the medium events is -10%.

There are only 9 large events in the six year period. SWMM5 under predicts all 9 events fairly significantly. The error in the total volume is -47%. The reasons for this large discrepancy are most likely due to operational decisions during the large events. RAPS has the ability by changing a few gate positions to bring flow from areas tributary to the interceptor between RAPS and Stickney WRP and pump that water to Bubbly Creek. The operators at Stickney WRP will reposition the gates to bring this additional flow to RAPS to protect the Stickney WRP and give priority to areas that are not protected by a pump station to minimize basement flooding in the CSO area. The area tributary to the interceptor between RAPS and Stickney WRP is not part of the HSPF and SWMM5 models prepared for this study. It was not possible to include rule variations based on the Stickney WRP inflows since that tributary area is not part of the SWMM5 model.

Based on the reasonable prediction of small and medium sized storms, this calibration run was accepted as a good representation of the hydrology and hydraulics of the CSO area tributary to Bubbly Creek. Table 8 shows the final RAPS pump operating rules assumed for this study.

Table 8 – RAPS Pump Operation Rules

Pump	Capacity (cfs)	Modeled Capacity (cfs)	Pump On Elevation (ft. CCD)	Pump Off Elevation (ft. CCD)
Dry Weather Pumps				
1	375	375	-27.0	-28.0*
3	375	375	-26.5	-27.5**
5	375	375	-25.0	-26.0***
7	400	n/a	n/a	n/a
9	400	n/a	n/a	n/a
Wet Weather Pumps				
2	375	375	-19.0	-25.5
8	500	500	-18.6	-25.1
10	500	500	-18.2	-24.7
12	500	500	-17.8	-24.3
14	500	500	-17.4	-23.9
16	500	500	-17.0	-23.5
18	500	500	-16.6	-23.1
4	375	375	-16.2	-22.7
6	375	375	-15.8	-22.3

* Pump 1 does not operate when Pump 2 is on.

** Pump 3 does not operate when Pump 8 is on.

*** Pump 5 does not operate when Pump 10 is on.

VI.B. Water Quality Comparison

Given that water quality observed data were not thought to be readily available for the areas tributary to Bubbly Creek at project inception, calibration of the water quality portion of the model was not included in the scope of the project. However, the project approach included the option for a general comparison with any reported data that became available, if applicable.

VI.B.1. Available Observed Data

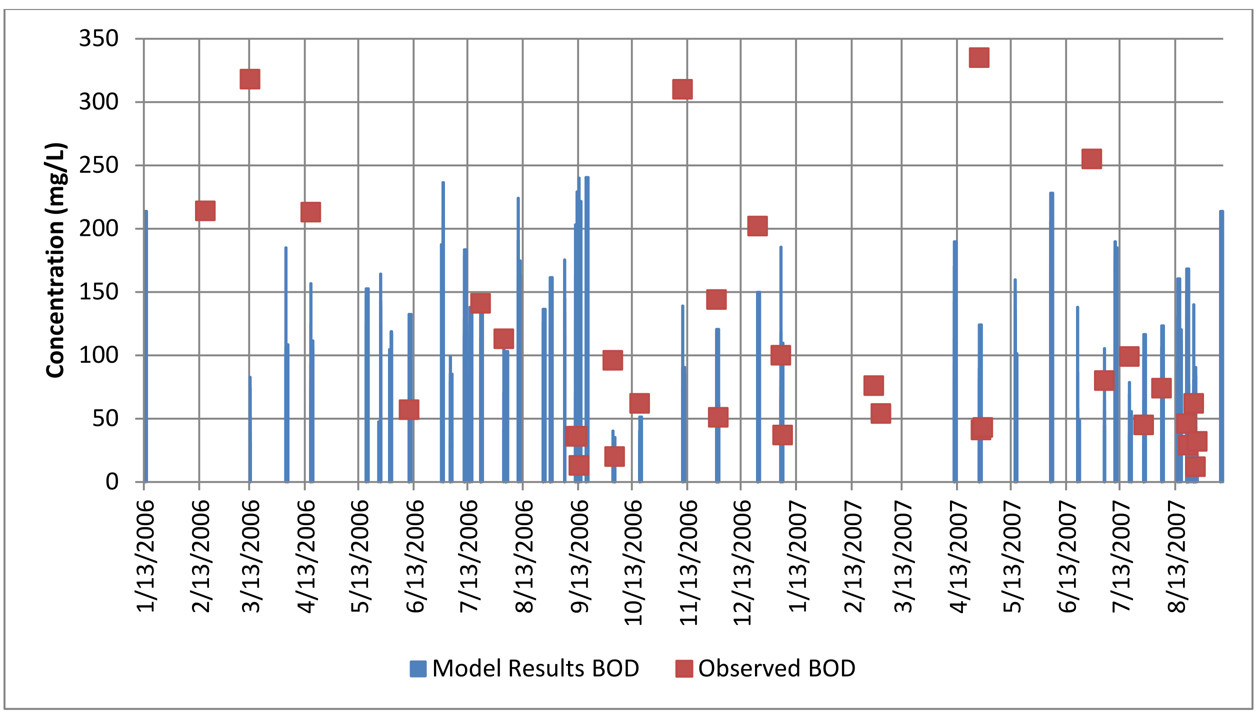
During the project, USACE provided available observed data collected by the MWRDGC at the wet weather pump discharges at RAPS, including data for BOD, TSS and DO. No temperature data were available. These samples from the pumping station have been taken since 2002. Data from 2002 to 2005 was not detailed enough to be compared to hourly model output. Data from 2006 to 2007 appeared to be instantaneous grab samples at specific times during wet weather pumping. It was concluded that the grab sample data could be compared more reasonably with the hourly model output. As such, 2006-2007 data were selected for the comparison.

The observed data were plotted with the model output to present an overall visual comparison. The model output was plotted on an hourly basis and the observed instantaneous grab sample data plotted on the same graph. Although this is not an exact comparison of data, observed data could be expected to fall within the range of model predicted peaks.

VI.B.2. BOD

From Figure 2, observed data generally fall within the model predicted output range, except for some peaks from 2006-2007. Predicted results appear to be appropriate for average conditions, but under-predicting occasional peak periods.

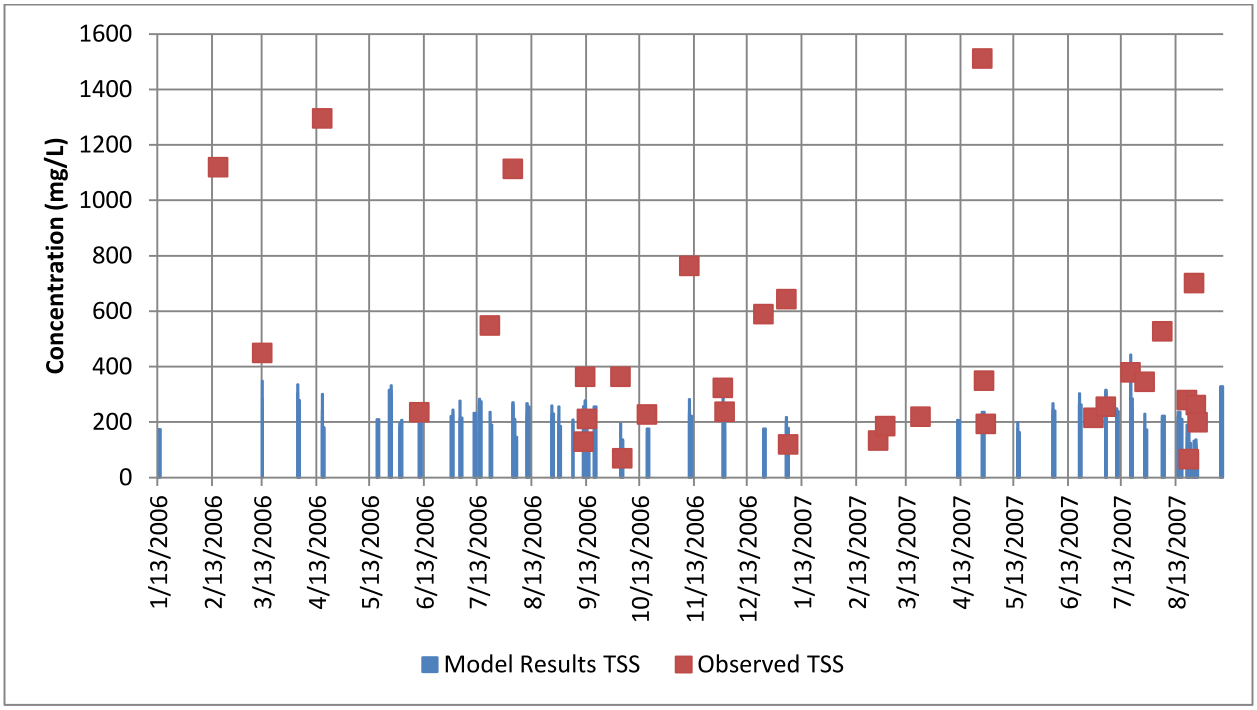
Figure 2. Observed Instantaneous BOD vs Hourly Model BOD Results from 2006 to 2007 at RAPS



VI.B.3. TSS

From Figure 3, observed TSS values are often greater than model predictions from 2006 to 2007. Further evaluation determined that the model will not be able to predict the extreme variability in the observed data set. In a pump station wet well, sediment may settle and accumulate, then be resuspended when pumps are turned on. This may cause TSS to not be uniform throughout the water column and grab samples to be highly variable. Therefore, grab samples may not be a good comparison with model results. The fairly consistent output of the model indicates that results may be more accurate over a longer period of time, rather than in individual overflow analysis.

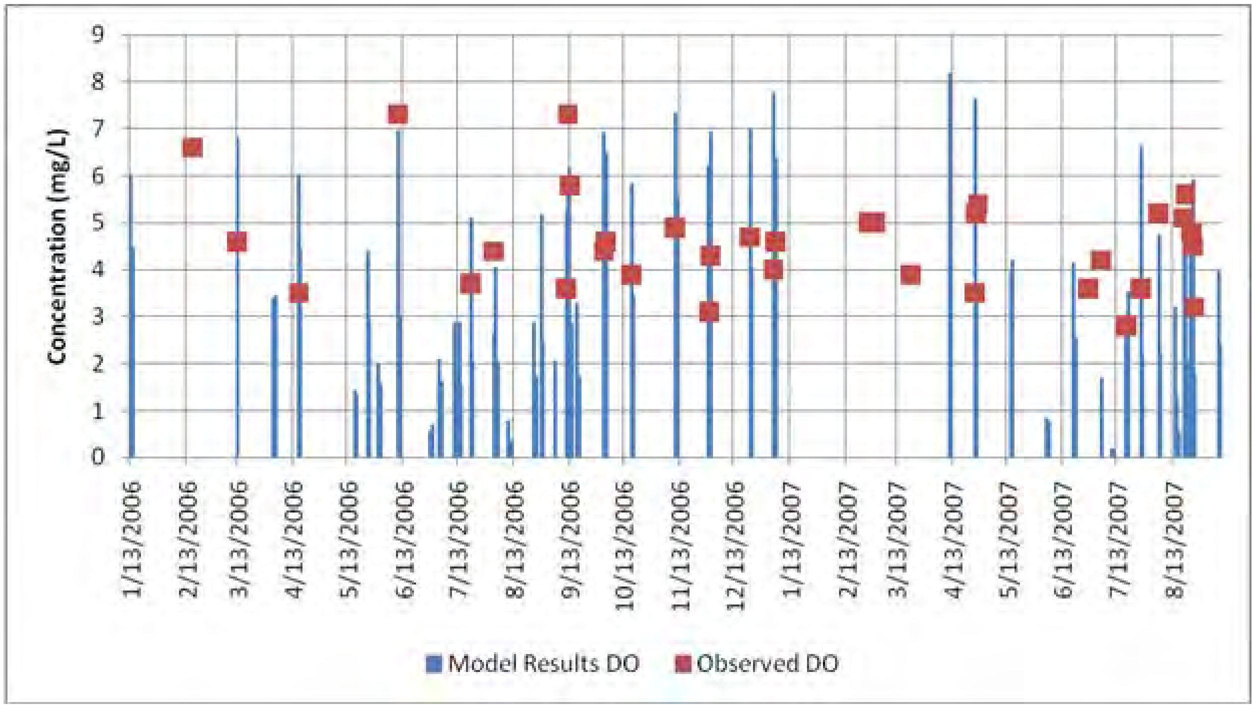
Figure 3. Observed Instantaneous TSS vs Hourly TSS model Result from 2006 to 2007 at RAPS



VI.B.4. DO

Figure 4 shows that observed DO data fall within the predicted range the majority of the time, with the exception of a few peak readings.

Figure 4. Observed Instantaneous DO vs Hourly DO Model Result from 2006 to 2007 at RAPS



VII. PERIOD OF RECORD ANALYSES

VII.A. Hydraulic Results

The period of record is January 1949 through September 2007. The period was broken up into nine six-year periods and one four-year period. This was done to shorten computational time by being able to run multiple periods simultaneously and keep the file sizes manageable.

VII.A.1. Existing Conditions - 2006

A summary of the overflows for the calendar year 2006 was prepared as a sample of the number of overflows that occur at the various locations along Bubbly Creek. Figure 5 shows the location of the CSO discharges along Bubbly Creek. Table 9 lists a summary of the number of events and volumes during calendar year 2006 at each of the overflow points along Bubbly Creek. Figure 6 shows the overflow hydrograph at RAPS.

Figure 5 – Bubbly Creek Study Area Map

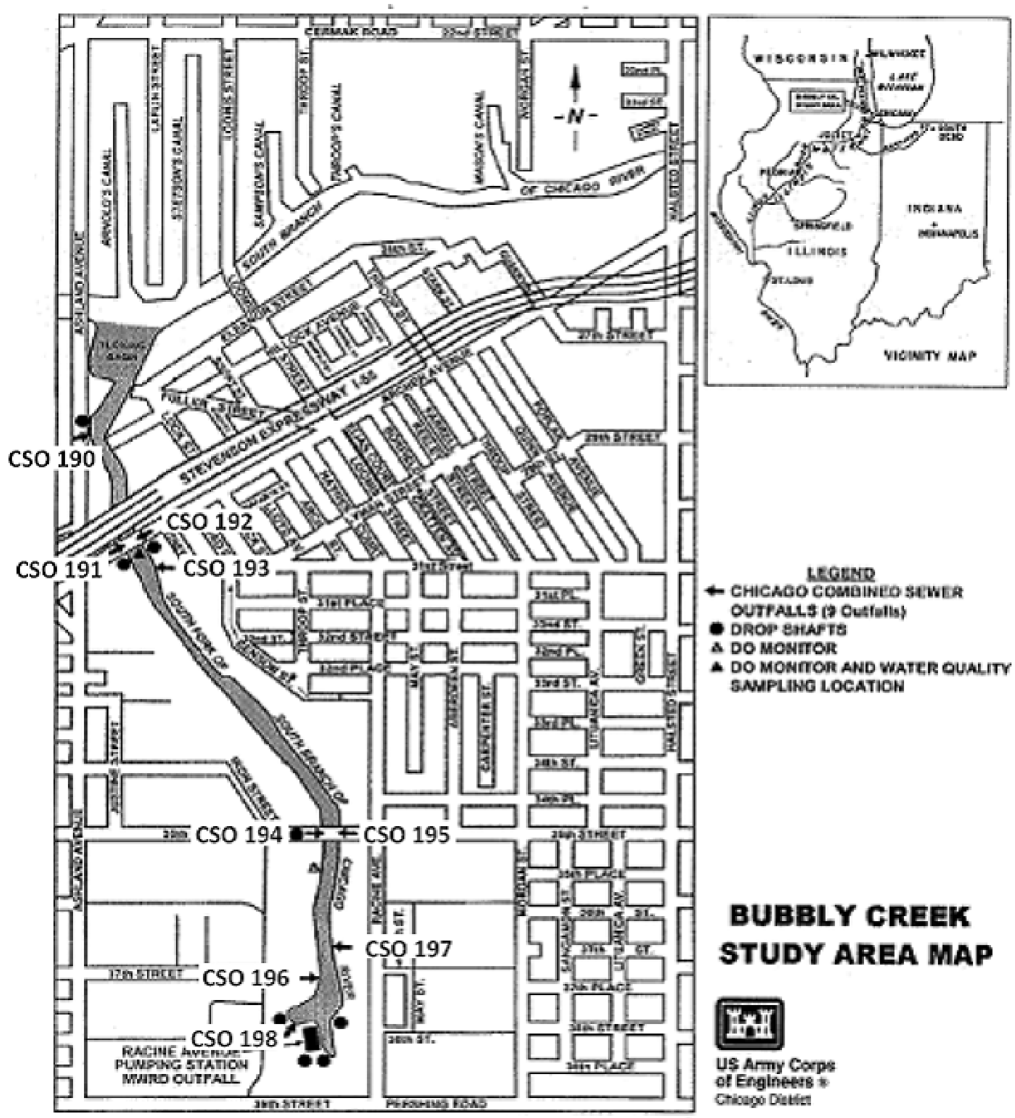
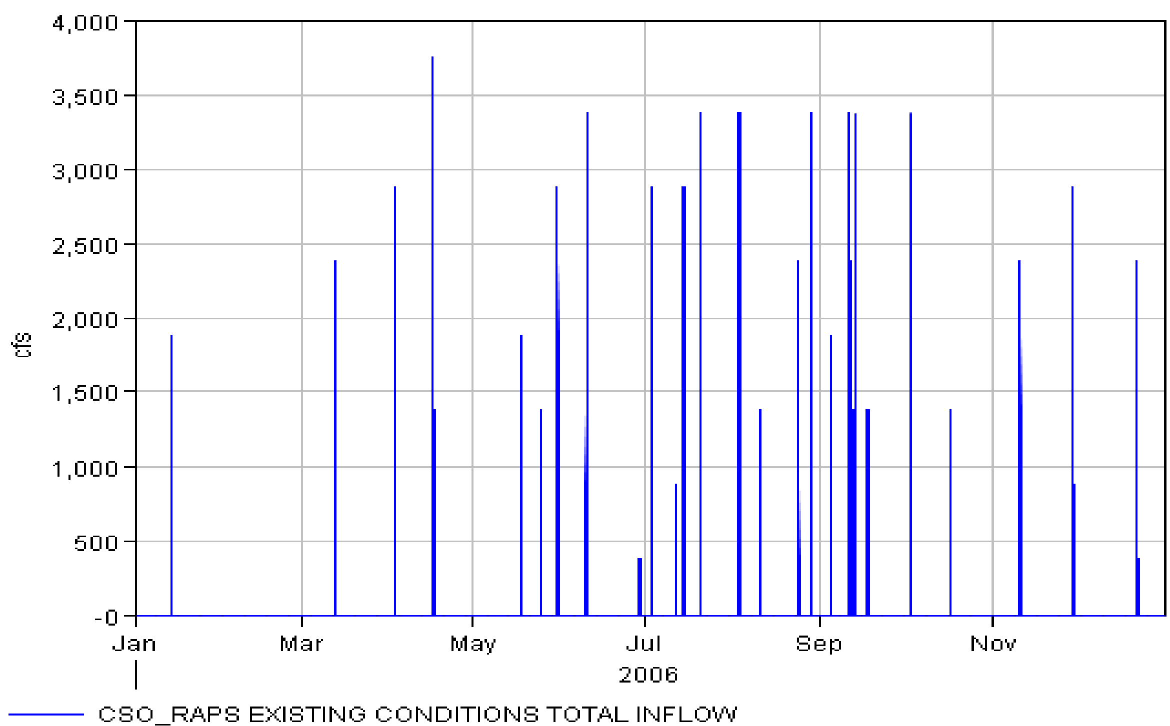


Table 9 – Summary of Overflows for Calendar Year 2006 (Existing Conditions)

Location	Volume (MG)	No. of Events
RAPS	8961.4	26
CSO-190	14.2	16
CSO-191	0.4	6
CSO-192	0.7	2
CSO-193	0.8	5
CSO-194	239.8	14
CSO-195	0.1	3
CSO-196	5.2	16
CSO-197	0.1	2
CSO-198	0.0	0

Figure 6 – RAPS Overflow Hydrograph for Calendar Year 2006 (Existing Conditions)



VII.A.2. Future Conditions with Drop Shaft Gates Open - 2006

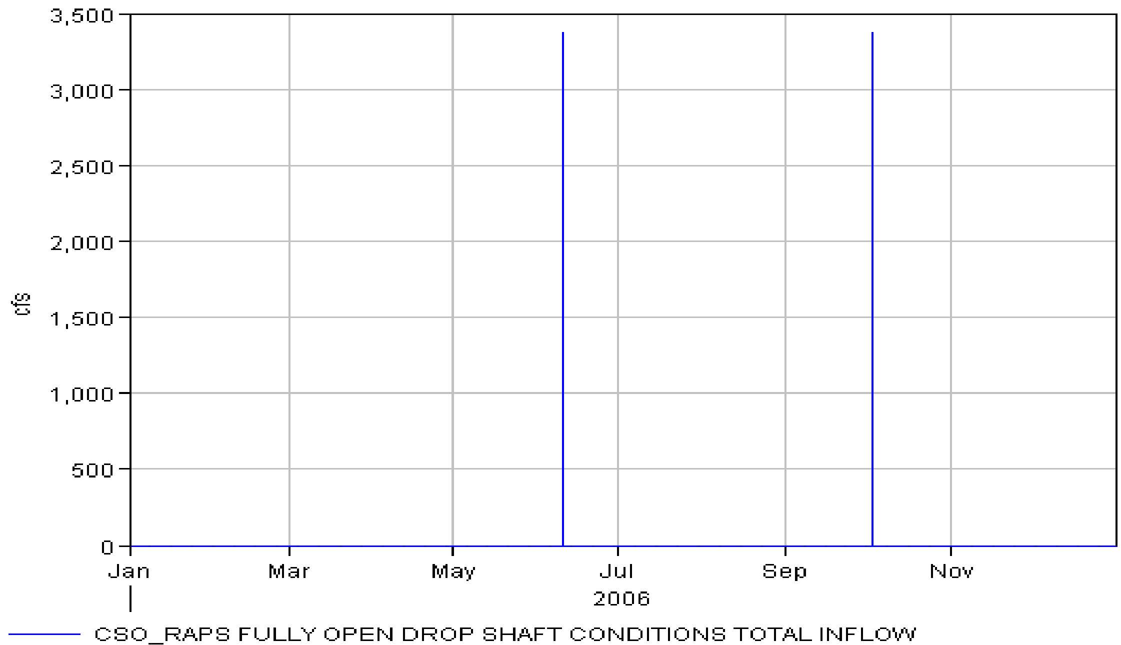
The fully open model simulates an operating plan scenario where tunnel dropshaft gates are open at the onset of a storm and remain open until the tunnel nears capacity at which point they are closed. Specifically at RAPS, the model initiates with open gates at DS-27, 28, 29 to the tunnel at the start of a storm. The gates are closed and stormwater pumps turned on when the tunnel level nears local ground level (540 ft NGVD, or -40 ft CCD).

Similar to the existing conditions, a summary of the overflows for the calendar year 2006 was prepared as a sample of the number of overflows that occur at the various locations along Bubbly Creek. Table 10 lists a summary of the number of events and volumes during calendar year 2006 at each of the overflow points along Bubbly Creek shown on Figure 5. Figure 7 shows the overflow hydrograph at RAPS.

Table 10 – Summary of Overflows for Calendar Year 2006 (Fully Open Condition)

Location	Volume (MG)	No. of Events
RAPS	712.0	2
CSO-190	0.6	2
CSO-191	11.1	2
CSO-192	0.7	2
CSO-193	0.0	0
CSO-194	1.5	2
CSO-195	0.2	3
CSO-196	5.0	13
CSO-197	0.03	3
CSO-198	0.0	0

Figure 7 – RAPS Overflow Hydrograph for Calendar Year 2006 (Fully Open Condition)



VII.A.3. Future Conditions with Drop Shaft Gates Partially Open - 2006

The partially open model simulates an operating plan scenario where tunnel dropshaft gates are open at the onset of a storm, but then some are closed during tunnel filling to allow a controlled pressurization of the tunnel system. Once pressurized, the dropshaft gates are then opened and allow flows to the tunnel. As the tunnel reaches capacity, the gates are then closed again. Specifically at RAPS, the model initiates with open gates at DS-27, 28, 29 to the tunnel at the start of a storm. The gates are closed when the South Fork Tunnel reaches 40% full (345.5 ft NGVD29, or -234 ft CCD). When the gates are closed, the stormwater pumps at RAPS are in services and discharge to Bubbly Creek until the tunnel is fully pressurized (420 ft NGVD, or -159.5 ft CCD). Once the tunnel is pressurized, the gates are opened again and the stormwater pumps turned off. The gates are closed again and stormwater pumps turned on when the tunnel level nears local ground level (540 ft NGVD, or -40 ft CCD).

Similar to the existing conditions and fully open gate condition, a summary of the overflows for the calendar year 2006 was prepared as a sample of the number of overflows that occur at the various locations along Bubbly Creek. Table 11 lists a summary of the number of events and volumes during calendar year 2006 at each of the overflow points along Bubbly Creek shown on Figure 5. Figure 8 shows the overflow hydrograph at RAPS.

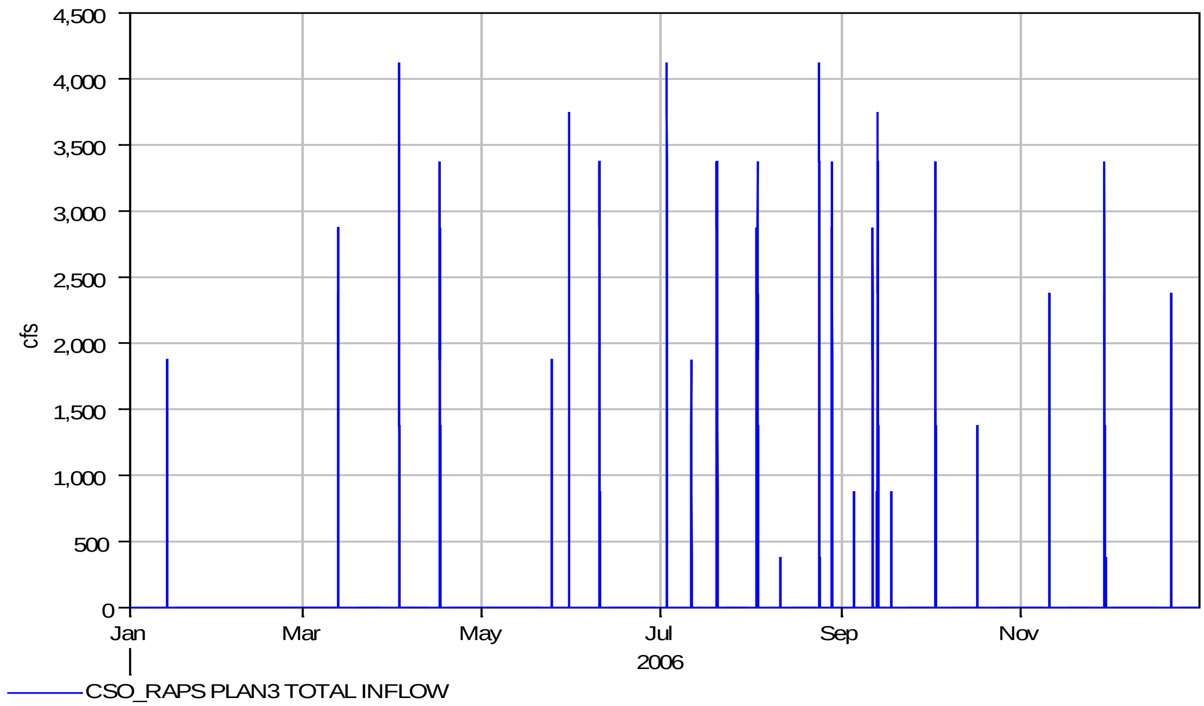
While it appears that the closing of dropshaft gates (partially open condition) results in CSO frequencies and volumes that are similar to the existing condition (without reservoir), there are several reasons why the modeled results are overly conservative and probably not representative of actual future operations. Through various transient model studies it was determined that there is a surge potential at various points in the tunnel system for large storm events. The surging is primarily a result of limited tunnel conveyance. Two particular areas showed a higher surge potential, one of the areas being RAPS. While it is widely anticipated that the dropshaft gates would not require closing for the smaller more frequent events, the modeling nevertheless included all storm events as having dropshaft gates closed during the pressurization phase. Additionally, the initial draft operation plan calls for 70 percent of the tunnel system dropshaft gates to be closed for large storm events, and through familiarity of real world operations it was believed that these constraints could be relaxed such that the gates could be throttled in a partially or fully open condition during the pressurization phase. In the modeling for the partially open condition the other 70 percent of dropshaft gates were included as being closed during the pressurization phase, even for smaller more frequent events. This adds to the modeled CSOs at RAPS since it increases the time it takes to pressurize the tunnel. For these reasons it is unlikely that the CSOs would show such a dramatic shift as to reflect the results of having no reservoir. The modeled results should more resemble the fully open condition if the smaller more frequent events were not included in the partially open condition modeling, and if the other 70 percent of dropshaft gates were only included for the very large storms.

The primary intent of simulating this condition was to determine the general increase in theoretical overflows produced by throttling the inflows to reduce the threat or frequency of surge events in the tunnel. Similar to the simulation results of the TNET model for the TARP tunnels, in certain locations (CSO 191 and 196 being the points with the largest overflow rates) the throttling of inflows actually reduces some of the POR overflow quantities at those locations. The explanation for this result is most likely that the partially open operation plan produces fewer tunnel pressurization periods over the POR. While it is likely that some dropshafts will be routinely throttled during the pressurization process, the quantity and frequency of overflows at any particular dropshaft will be strongly affected by the inclusion/exclusion of that dropshaft in the throttling scheme.

Table 11 – Summary of Overflows for Calendar Year 2006 (Partially Open Condition)

Location	Volume (MG)	No. of Events
RAPS	7,661.8	23
CSO-190	0.4	1
CSO-191	8.3	1
CSO-192	0.6	2
CSO-193	0.3	1
CSO-194	5.2	1
CSO-195	0.01	2
CSO-196	5.5	12
CSO-197	0.004	2
CSO-198	0.0	0

Figure 8 – RAPS Overflow Hydrograph for Calendar Year 2006 (Partially Open Condition)



VII.A.4. Discharge Results for Full Period of Record

The discharge results for the full 57-3/4 year period of record (January 1949-September 2007) are shown below for the three different conditions. Results are expressed as average annual volumes. As described in the preceding section, modeling results for the CSOs for the partially open condition are vastly overestimated due to the inclusion of small storms and the other 70 percent of dropshafts in the dropshaft gate closure scheme.

Table 12 – Summary of Overflows for January 1949 – September 2007

Average Annual Volume

Location	Existing Condition MG	Future Condition Fully Open Dropshafts MG	Future Condition Partially Open Dropshafts MG
RAPS	4,928.6	734.6	4690.8
CSO-190	10.0	0.8	0.9
CSO-191	0.3	13.7	10.1
CSO-192	0.7	0.7	0.7
CSO-193	0.6	0.1	0.1
CSO-194	194.6	2.5	2.8
CSO-195	0.05	0.1	0.04
CSO-196	3.0	3.0	2.9
CSO-197	0.03	0.04	0.03
CSO-198	0.00	0.00	0.00

VII.B. Water Quality Results

The time intervals in the period of record were split into 3-year spans so that system memory would not be overwhelmed. Results were then exported to DSS files and recombined to the full 58-year period of record for CSOs 190 to 198 and RAPS. The periods of record are too lengthy to obtain meaningful information from a graphical figure in this document; however, the DSS files can be used in this manner. Resulting model predictions for the most recent 5 water years of the period of record are tabulated below.

Existing Conditions

VII.B.1.a. BOD

For water years 2003 to 2007, BOD concentrations ranged from 0 to 480.8 mg/L (2006, CSO 196) across the ten CSOs with a maximum loading of nearly 2,600 tons to Bubbly Creek from RAPS in 2006 (See Table 13).

Table 13. BOD Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	4,655,617.8	5,172,935.1	2,940,009.4	3,977,106.3	2,630,742.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	228.2	240.5	221.9	194.1	240.2
CSO 190	Ann Load (lbs)	209.4	72.2	23.8	174.3	62.9
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	93.5	75.8	52.6	79.6	42.5
CSO 191	Ann Load (lbs)	116.0	56.4	4.1	210.9	30.4
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	16.0	12.7	12.8	36.2	11.5
CSO 192	Ann Load (lbs)	66.5	28.6	56.2	26.1	2.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	9.9	15.4	15.4	12.5	14.9
CSO 193	Ann Load (lbs)	132.8	38.5	0.0	82.7	33.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	93.4	52.4	0.0	96.3	50.9
CSO 194	Ann Load (lbs)	9,716.4	15,342.5	8,802.2	7,534.5	5,563.6
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	64.6	82.0	93.4	66.9	86.5
CSO 195	Ann Load (lbs)	3.0	1.1	0.0	8.7	0.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	80.5	42.0	0.0	35.5	8.9
CSO 196	Ann Load (lbs)	561.9	509.0	332.5	383.7	468.9
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	80.1	480.8	96.8	65.3	82.0
CSO 197	Ann Load (lbs)	0.6	2.3	0.0	6.5	4.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	102.2	156.0	0.0	182.2	88.3
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.1.b. TSS

For water years 2003 to 2007, TSS concentrations ranged from 0 to 579.1 mg/L (2004, CSO 196) across the ten CSOs with a maximum loading of over 6,500 tons to Bubbly Creek from RAPS in 2006 (See Table 14).

Table 14. TSS Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	12,199,891.2	13,088,588.6	6,599,624.1	11,794,234.7	8,461,470.7
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	443.0	347.9	465.1	409.6	354.6
CSO 190	Ann Load (lbs)	2,520.8	1,821.4	453.7	7,008.4	1,799.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	248.3	218.4	214.3	377.0	340.0
CSO 191	Ann Load (lbs)	1,909.4	1,999.6	52.1	3,800.2	1,338.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	230.3	433.2	141.4	376.3	342.6
CSO 192	Ann Load (lbs)	2,622.5	705.4	2,971.2	1,681.5	64.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	552.7	257.8	515.4	470.2	481.1
CSO 193	Ann Load (lbs)	599.3	255.1	0.0	631.6	300.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	211.1	189.5	0.0	363.6	225.4
CSO 194	Ann Load (lbs)	115,908.0	134,390.5	104,248.2	105,889.7	101,851.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	522.1	464.0	511.7	578.6	492.6
CSO 195	Ann Load (lbs)	4.8	6.7	0.0	37.7	2.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	142.5	254.8	0.0	259.6	345.8
CSO 196	Ann Load (lbs)	10,933.8	9,067.5	9,368.1	12,491.8	10,963.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	546.7	461.3	515.1	579.1	496.7
CSO 197	Ann Load (lbs)	1.3	5.1	0.0	34.7	53.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	254.2	281.4	0.0	395.8	337.5
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.1.c. DO

For water years 2003 to 2007, DO concentrations ranged from 0 to 34.24 mg/L (2007, CSO 190) across the ten CSOs with a maximum loading of over 152 tons to Bubbly Creek from RAPS in 2007. Extreme peaks of DO occurred in the simulation results. This is a result of the way HSPF handles period average and instantaneous time steps. There is no way to correct this issue in HSPF and will result in a few unreasonable time step concentrations. However, these unreasonable concentrations occur during low flow periods and should not significantly affect mass loading over larger periods of time, e.g. an annual load (See Table 15).

Table 15. DO Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	304,107.2	254,523.9	137,345.6	251,818.6	140,620.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	8.2	7.9	7.4	8.8	6.8
CSO 190	Ann Load (lbs)	341.6	116.8	29.2	281.6	89.9
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	34.2	15.7	14.4	11.5	10.8
CSO 191	Ann Load (lbs)	161.2	75.6	3.5	144.0	55.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.1	14.3	9.6	10.7	9.4
CSO 192	Ann Load (lbs)	99.6	0.0	0.0	25.4	0.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	7.6	0.3	0.7	7.2	7.6
CSO 193	Ann Load (lbs)	93.2	14.0	0.0	33.0	15.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	9.5	7.9	0.0	10.6	9.5
CSO 194	Ann Load (lbs)	3,772.9	3,048.5	1,279.6	2,624.0	1,562.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.3	14.6	10.5	11.2	9.4
CSO 195	Ann Load (lbs)	0.3	0.3	0.0	1.7	0.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.0	8.4	0.0	10.2	8.1
CSO 196	Ann Load (lbs)	433.8	121.9	47.6	240.7	95.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.1	14.5	10.7	11.0	9.1
CSO 197	Ann Load (lbs)	0.1	0.2	0.0	0.6	1.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	8.7	8.0	0.0	9.4	7.5
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0

	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.1.d. Temperature

Due to the concentration method of modeling temperature as describe above, temperature values less than 32 Deg F were predicted. Only minimum values greater than 32 Deg F were tabulated below. A better idea of the temperature predictions can be found by looking at the DSS hourly data time periods of interest (See Table 16).

Table 16. Temperature Minimum and Maximum in Degrees Fahrenheit, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Min (Deg F)	38.5	37.8	41.0	39.3	42.5
	Max (Deg F)	70.0	69.3	61.6	64.2	60.1
CSO 190	Min (Deg F)	33.4	32.3	32.9	41.6	44.2
	Max (Deg F)	71.6	70.6	44.1	69.8	58.3
CSO 191	Min (Deg F)	41.9	33.1	43.8	41.2	47.7
	Max (Deg F)	70.5	68.3	43.9	66.0	52.3
CSO 192	Min (Deg F)	33.5	33.3	33.3	56.6	35.4
	Max (Deg F)	64.4	51.3	54.8	64.8	61.9
CSO 193	Min (Deg F)	40.9	42.6	0.0	41.4	48.2
	Max (Deg F)	71.3	68.1	0.0	65.8	51.3
CSO 194	Min (Deg F)	32.9	33.1	32.0	32.1	32.0
	Max (Deg F)	71.4	70.1	68.2	66.5	67.0
CSO 195	Min (Deg F)	42.2	51.1	0.0	48.2	50.9
	Max (Deg F)	70.5	65.7	0.0	65.3	51.1
CSO 196	Min (Deg F)	33.1	32.8	32.3	33.2	32.8
	Max (Deg F)	70.5	70.6	66.9	66.3	68.8
CSO 197	Min (Deg F)	42.6	45.9	0.0	49.0	51.9
	Max (Deg F)	60.2	69.2	0.0	66.9	64.5
CSO 198	Min (Deg F)	0.0	0.0	0.0	0.0	0.0
	Max (Deg F)	0.0	0.0	0.0	0.0	0.0

VII.B.2. Future Conditions with Drop Shaft Gates Open

VII.B.2.a. BOD

For water years 2003 to 2007, BOD concentrations ranged from 0 to 409.4 mg/L (2006, CSO 197) across the ten CSOs with a maximum loading of over 250 tons to Bubbly Creek from RAPS in 2007 (See Table 17).**Table 17. BOD Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007**

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	504,804.2	90,165.2	227,310.4	412,824.4	236,607.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	240.5	131.5	143.1	127.1	81.1
CSO 190	Ann Load (lbs)	2.5	6.3	0.0	0.1	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	14.9	25.5	0.0	6.9	0.0
CSO 191	Ann Load (lbs)	12,370.7	3,271.6	0.0	3,695.9	2,062.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	174.8	97.2	0.0	81.0	142.8
CSO 192	Ann Load (lbs)	80.7	26.3	50.2	23.8	0.4
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	9.0	15.8	14.7	12.1	14.3
CSO 193	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0
CSO 194	Ann Load (lbs)	1,849.7	36.0	515.7	7.2	1.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	165.6	17.0	74.6	48.7	141.8
CSO 195	Ann Load (lbs)	5.5	1.7	24.0	56.8	48.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	377.5	317.9	317.9	97.6	135.9
CSO 196	Ann Load (lbs)	500.9	426.4	279.3	331.0	445.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	52.5	59.2	83.7	56.5	65.7
CSO 197	Ann Load (lbs)	1.0	1.0	2.0	8.5	15.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	318.6	409.4	282.0	361.0	153.2
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.2.b. TSS

For water years 2003 to 2007, TSS concentrations ranged from 0 to 578.2 mg/L (2004, CSO 196) across the ten CSOs with a maximum loading of over 820 tons to Bubbly Creek from RAPS in 2007 (See Table 17).

Table 17. TSS Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	1,649,549.6	735,254.4	657,420.0	1,617,099.0	922,361.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	314.6	219.4	260.9	321.5	281.5
CSO 190	Ann Load (lbs)	3.9	219.2	0.0	0.6	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	102.9	128.7	0.0	45.5	0.0
CSO 191	Ann Load (lbs)	16,430.3	6,903.1	0.0	18,460.3	9,400.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	246.0	198.9	0.0	373.1	257.4
CSO 192	Ann Load (lbs)	2463.5	643.8	2665.7	1536.4	7.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	552.4	371.4	515.5	470.4	411.2
CSO 193	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0
CSO 194	Ann Load (lbs)	2,226.3	736.9	1,224.4	35.4	3.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	252.7	173.6	177.1	164.6	255.5
CSO 195	Ann Load (lbs)	10.9	8.8	22.2	175.6	95.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	341.8	269.6	288.8	444.8	190.2
CSO 196	Ann Load (lbs)	11,611.8	8,189.5	8,755.9	11,447.7	10,056.6
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	548.3	508.9	515.0	578.2	496.5
CSO 197	Ann Load (lbs)	2.2	2.9	2.3	21.1	22.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	322.0	418.8	297.0	437.6	385.9
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.2.c. DO

For water years 2003 to 2007, DO concentrations ranged from 0 to 10.4 mg/L (2004, CSO 195) across the ten CSOs with a maximum loading of nearly 25 tons to Bubbly Creek from RAPS in 2007. See the DO section under the existing scenario for a description on potential unreasonably high DO concentration peaks (See Table 18).

Table 18. DO annual loading, minimum concentration, and maximum concentration 2003 to 2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	48,813.0	23,300.7	11,295.0	29,886.8	14,978.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	6.5	7.3	4.9	6.2	4.5
CSO 190	Ann Load (lbs)	2.1	16.6	0.0	0.1	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	8.8	9.1	0.0	8.9	0.0
CSO 191	Ann Load (lbs)	25.3	23.3	0.0	8.7	6.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	2.4	1.0	0.0	1.8	0.2
CSO 192	Ann Load (lbs)	91.5	0.0	0.0	23.2	0.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	7.7	6.3	0.7	9.0	4.7
CSO 193	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0
CSO 194	Ann Load (lbs)	1.8	36.2	2.6	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	6.7	8.5	0.4	1.2	0.2
CSO 195	Ann Load (lbs)	0.4	0.3	0.5	1.1	5.4
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	9.5	6.1	9.1	10.4	9.2
CSO 196	Ann Load (lbs)	381.4	119.1	27.7	250.2	75.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.4	14.3	11.5	11.2	9.1
CSO 197	Ann Load (lbs)	0.1	0.1	0.0	0.4	0.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	9.4	6.9	8.7	9.9	8.7
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.2.d. Temperature

Due to the concentration method of modeling temperature as describe above, temperature values less than 32 Deg F were predicted. Only minimum values greater than 32 Deg F were tabulated below. A better idea of the temperature predictions can be found by looking at the DSS hourly data time periods of interest (See Table 19).

Table 19. Temperature Minimum and Maximum in Degrees Fahrenheit, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Min (Deg F)	54.5	51.6	41.5	46.0	52.3
	Max (Deg F)	68.1	53.0	60.1	58.3	59.7
CSO 190	Min (Deg F)	58.2	51.6	0.0	61.0	0.0
	Max (Deg F)	62.5	54.8	0.0	61.0	0.0
CSO 191	Min (Deg F)	54.3	54.2	0.0	49.5	51.8
	Max (Deg F)	59.0	54.4	0.0	57.5	58.8
CSO 192	Min (Deg F)	33.2	33.5	33.2	57.3	36.7
	Max (Deg F)	64.5	60.3	53.6	64.8	57.9
CSO 193	Min (Deg F)	0.0	0.0	0.0	0.0	0.0
	Max (Deg F)	0.0	0.0	0.0	0.0	0.0
CSO 194	Min (Deg F)	53.4	51.8	33.7	32.3	50.6
	Max (Deg F)	59.3	52.7	33.7	32.3	58.4
CSO 195	Min (Deg F)	42.2	55.4	46.3	48.5	45.3
	Max (Deg F)	70.0	65.3	62.2	59.4	63.3
CSO 196	Min (Deg F)	32.3	32.0	33.5	33.0	32.2
	Max (Deg F)	70.8	70.5	69.3	66.4	69.1
CSO 197	Min (Deg F)	42.6	51.4	44.2	41.7	46.7
	Max (Deg F)	69.5	70.4	60.0	63.2	66.0
CSO 198	Min (Deg F)	0.0	0.0	0.0	0.0	0.0
	Max (Deg F)	0.0	0.0	0.0	0.0	0.0

VII.B.3. Future Conditions with Drop Shaft Gates Partially Open

VII.B.3.a. BOD

For water years 2003 to 2007, BOD concentrations ranged from 0 to 338.8 mg/L (2007, CSO 197) across the ten CSOs with a maximum loading of over 1,940 tons to Bubbly Creek from RAPS in 2006 (See Table 20).

Table 20. BOD Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load(lbs)	1,888,251.3	3,883,594.5	2,385,289.2	3,869,724.9	2,361,069.7
	Min (mg/L)	8.8	8.9	26.5	12.7	16.2
	Max (mg/L)	248.3	248.3	209.2	203.8	219.5
CSO 190	Ann Load(lbs)	23.1	0.0	0.0	3.2	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.5	0.0	0.0	5.4	0.0
CSO 191	Ann Load(lbs)	11,402.8	0.0	0.0	3,757.2	887.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	199.8	0.0	0.0	74.6	49.1
CSO 192	Ann Load(lbs)	20.4	30.5	52.3	27.3	0.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	12.1	15.7	15.3	12.5	15.3
CSO 193	Ann Load(lbs)	7.6	0.0	0.0	1.5	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	26.4	0.0	0.0	5.1	0.0
CSO 194	Ann Load(lbs)	298.0	0.0	0.0	30.0	257.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	167.6	0.0	0.0	65.2	105.7
CSO 195	Ann Load(lbs)	0.0	11.4	0.9	0.0	1.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	221.8	270.9	29.6	0.0	56.2
CSO 196	Ann Load(lbs)	101.8	374.8	322.7	420.1	452.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	83.1	83.1	57.4	55.1	47.9
CSO 197	Ann Load(lbs)	0.0	5.5	0.2	1.0	0.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	333.8	329.4	74.5	187.9	187.8
CSO 198	Ann Load(lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.3.b. TSS

For water years 2003 to 2007, TSS concentrations ranged from 0 mg/L to 580.3 (2004, CSO 196) across the ten CSOs with a maximum loading of over 5,730 tons to Bubbly Creek from RAPS in 2004 (See Table 21).

Table 21. TSS Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	5,206,481.5	10,501,398	5,769,882.8	11,466,538	7,897,458.8
	Min (mg/L)	45.9	61.5	142.6	118.2	92.9
	Max (mg/L)	454.1	343.9	464.9	410.4	369.2
CSO 190	Ann Load (lbs)	99.3	0.0	0.0	23.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	21.9	0.0	0.0	36.4	0.0
CSO 191	Ann Load (lbs)	13,316.1	0.0	0.0	19,529.3	3,988.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	232.3	0.0	0.0	387.2	220.8
CSO 192	Ann Load (lbs)	109.9	754.4	2,778.5	1,762.7	6.5
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	552.3	356.8	515.5	470.5	411.3
CSO 193	Ann Load (lbs)	14.8	0.0	0.0	18.8	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	42.9	0.0	0.0	61.5	0.0
CSO 194	Ann Load (lbs)	809.9	0.0	0.0	379.5	751.2
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	194.9	0.0	0.0	218.2	217.1
CSO 195	Ann Load (lbs)	0.0	40.8	6.1	0.0	5.9
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	228.2	332.9	210.3	0.0	185.6
CSO 196	Ann Load (lbs)	905.9	6,652.3	9,621.9	13,146.8	9,848.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	548.2	463.7	515.1	580.3	495.9
CSO 197	Ann Load (lbs)	0.1	9.6	0.7	4.5	0.4
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	308.0	396.8	222.2	321.1	194.5
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.3.c. DO

For water years 2003 to 2007, DO concentrations ranged from 0 mg/L to 14.5 (2006, CSO 196) across the ten CSOs with a maximum loading of nearly 124 tons to Bubbly Creek from RAPS in 2004. See the DO section under the existing scenario for a description on potential unreasonably high DO concentration peaks (See Table 22).

Table 22. DO Annual Loading, Minimum Concentration, and Maximum Concentration, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Ann Load (lbs)	160,044.8	230,855.2	129,895.0	247,844.0	135,846.7
	Min (mg/L)	0.5	0.1	0.3	0.4	0.4
	Max (mg/L)	8.2	7.1	7.5	8.9	6.9
CSO 190	Ann Load (lbs)	39.7	0.0	0.0	14.8	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	8.8	0.0	0.0	9.8	0.0
CSO 191	Ann Load (lbs)	12.1	0.0	0.0	4.1	0.8
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.6	0.0	0.0	0.2	0.0
CSO 192	Ann Load (lbs)	34.3	0.1	0.0	26.6	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	7.7	6.0	0.7	7.1	3.9
CSO 193	Ann Load (lbs)	3.5	0.0	0.0	3.3	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	7.5	0.0	0.0	9.4	0.0
CSO 194	Ann Load (lbs)	194.5	0.0	0.0	55.5	0.1
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	6.3	0.0	0.0	9.3	0.0
CSO 195	Ann Load (lbs)	0.0	1.4	0.2	0.0	0.3
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	3.9	8.2	7.8	0.0	9.5
CSO 196	Ann Load (lbs)	95.2	74.8	41.0	271.7	69.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	10.1	14.5	9.7	10.7	9.1
CSO 197	Ann Load (lbs)	0.0	0.3	0.0	0.1	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	7.4	6.6	7.5	6.5	7.6
CSO 198	Ann Load (lbs)	0.0	0.0	0.0	0.0	0.0
	Min (mg/L)	0.0	0.0	0.0	0.0	0.0
	Max (mg/L)	0.0	0.0	0.0	0.0	0.0

VII.B.3.d. Temperature

Due to the concentration method of modeling temperature as describe above, temperature values less than 32 Deg F were predicted. Only minimum values greater than 32 Deg F were tabulated below. A better idea of the temperature predictions can be found by looking at the DSS hourly data time periods of interest (See Table 23).

Table 23. Temperature Minimum and Maximum in Degrees Fahrenheit, 2003-2007

Overflow		2007	2006	2005	2004	2003
RAPS	Min (Deg F)	38.7	38.0	41.3	39.6	41.9
	Max (Deg F)	69.3	66.8	61.0	64.2	60.4
CSO 190	Min (Deg F)	58.2	0.0	0.0	57.4	0.0
	Max (Deg F)	59.3	0.0	0.0	60.5	0.0
CSO 191	Min (Deg F)	52.7	0.0	0.0	57.0	49.6
	Max (Deg F)	58.8	0.0	0.0	57.0	49.6
CSO 192	Min (Deg F)	33.3	33.3	33.2	59.9	38.2
	Max (Deg F)	64.5	58.1	54.6	64.8	54.8
CSO 193	Min (Deg F)	59.0	0.0	0.0	57.8	0.0
	Max (Deg F)	59.2	0.0	0.0	58.7	0.0
CSO 194	Min (Deg F)	41.6	0.0	0.0	32.2	40.2
	Max (Deg F)	59.3	0.0	0.0	57.9	48.3
CSO 195	Min (Deg F)	65.8	50.9	55.1	0.0	44.5
	Max (Deg F)	66.1	68.3	55.2	0.0	66.8
CSO 196	Min (Deg F)	32.4	32.1	32.3	32.2	33.0
	Max (Deg F)	71.3	70.2	69.3	66.8	69.1
CSO 197	Min (Deg F)	43.3	44.0	53.7	42.9	51.4
	Max (Deg F)	68.9	69.4	56.5	65.4	70.0
CSO 198	Min (Deg F)	0.0	0.0	0.0	0.0	0.0
	Max (Deg F)	0.0	0.0	0.0	0.0	0.0

VIII. Summary

In summary, comparing the three simulation scenarios (existing, gates fully open, and gates partially open) over the entire period of record, the fully open scenario produces the least total overflow volume and number of overflow events over all the overflow locations as shown on Figure 5 and listed in tables 9 through 11. The fully open operation reduces overflow volume by 85 percent when compared to the total overflow volume for the existing condition over the entire period of record. Notably, the partially open scenario produces only slightly better overflow results than the existing condition with total overflow volume reduced by only 9 percent. The results for the partially open scenario are extremely conservative and overestimate the CSO discharges due to the inclusion of smaller more frequent storms and the additional 70 percent of dropshafts in the dropshaft gate closure scheme.

The water quality results under the existing conditions scenario are within reasonable ranges. A detailed calibration of the water quality model was not possible due to insufficient data; however, an informal comparison was made with some observed samples of BOD, DO, and TSS at RAPS. BOD and DO appear to be in agreement with this data, whereas TSS appears to be under predicting. This may be due to the extremely variable nature of the observed data set and the difficulty of predicting this parameter within the wet well of a pumping station. TSS will be more accurate as a mass loading over a longer period of time rather than in comparison to a grab or 24-hour composite sample from the RAPS wet well. Due to the limitations of SWMM, a heat load was not calculated; however, surface water temperature was modeled and conservatively routed through the pipe network (combing with dry weather flow temperatures) to approximate the temperature of the overflows. Water quality results are dependent on hydraulics as a transport vehicle for pollutant load. Any inaccuracies in hydraulics will also be expressed in the water quality results.

Attachment 2:

HEC-RAS Modeling of Bubbly Creek

HEC-RAS Modeling of Bubbly Creek

1. A HEC-RAS model was developed for the purpose of determining stage impacts on Bubbly Creek resulting from ecosystem restoration features, primarily placing substrate on the channel bottom and the addition of vegetation. The model includes 40 cross sections that define the 1.25 mile reach extending from the Racine Avenue Pump Station (RAPS) north to the confluence with the South Branch of the Chicago River. An aerial view of the model is shown in Figure 1.

BUBBLY CREEK CROSS-SECTIONS

U.S. Army Corps of Engineers
Chicago District

Date: 7/24/2013

N
W E
S

1 in = 650 feet

Coordinate System: NAD 1983 StatePlane Illinois East FIPS 1201 Feet
Projection: Transverse Mercator
Datum: North American 1983

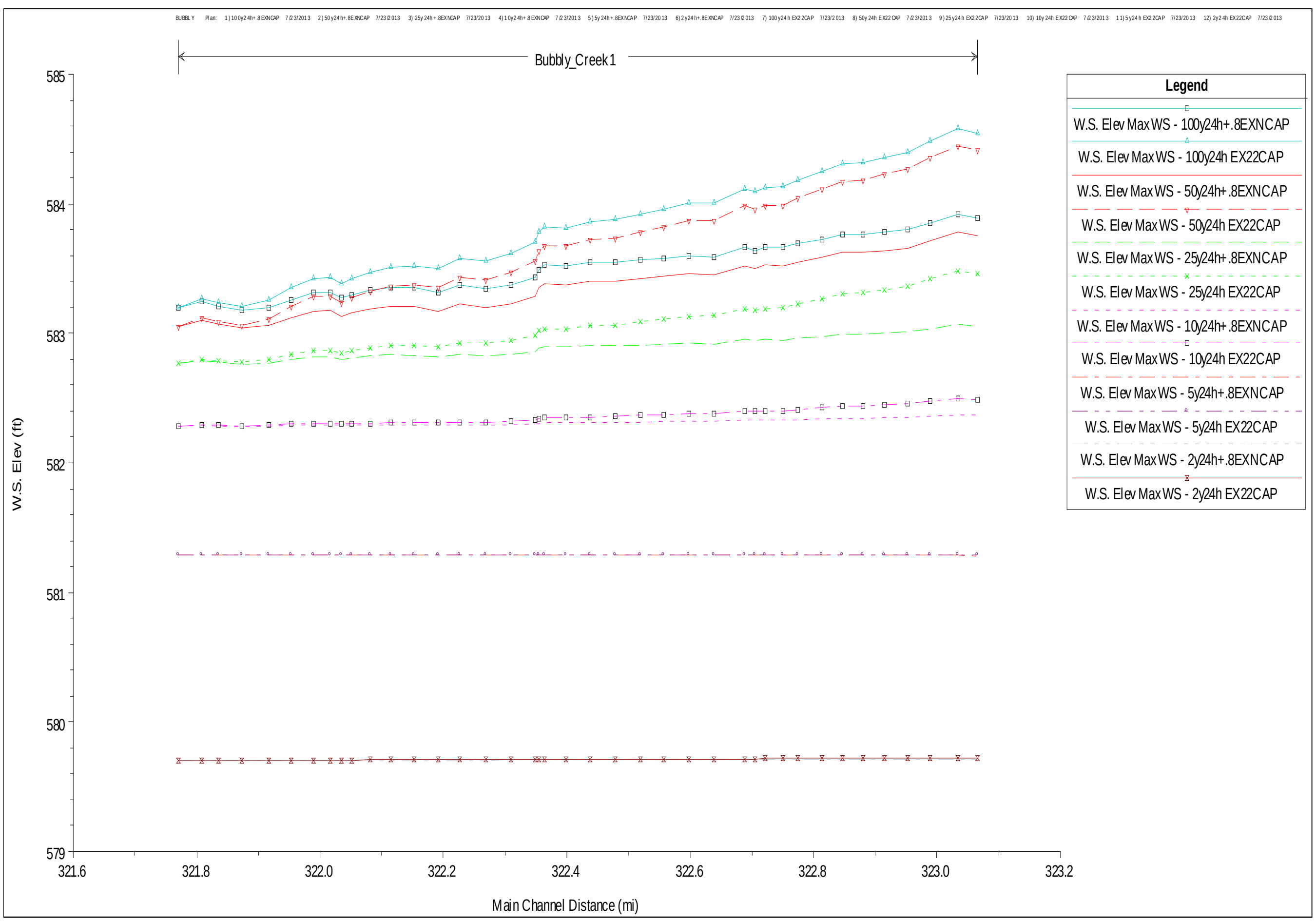
Path: C:\Users\h6thnjb\Documents\Bubbly Creek\bubblycreekSECTIONS.mxd

2. The HEC-RAS model utilizes the modeled stage at the confluence with the South Branch of the Chicago River derived from the Great Lakes and Mississippi River Interbasin Study (GLMRIS). Modeled events include the 24-hour 100-year, 50-year, 25-year, 10-year, 5-year and 2-year storm events. Also included was the 12-hour 100-year event since this event resulted in a slightly higher stage along the creek than the 24-hour storm duration. All modeled scenarios used a Lake Michigan elevation of +0.8 CCD which represents the long term average. Flows at RAPS for the 100-year event were conservatively increased to 6,000 cfs from a historic maximum of approximately 5,200 cfs which is the maximum recorded discharge since RAPS was put into service in 1939. Additionally, the flow data used at the other outfall locations are conservatively estimated as well since the discharges do not account for submergence of the outfalls. Figure 2 shows the maximum water surface elevation (NAVD88 datum) for the existing conditions for the various storm events with RAPS being at the right extent of the plot and the confluence at the far left.

Figure 2 – Maximum Water Surface Elevations for Existing Conditions

3. The impact of a 22-inch uniform substrate layer was modeled along with an increase in roughness (Manning's N value) to account for the initial implementation of aquatic vegetation. The existing conditions N-value used was 0.025. The N-value accounting for the smooth well graded substrate stone that ranges from 0.8 to 2 inches (depending on the need for dredging) was also 0.025. The N-value was adjusted upward to 0.03 (bank to bank) to account for vegetation shortly after the time of planting. The effects of the substrate and vegetation in the first few years prior to full migration and growth of the vegetation can be seen in Figure 3. The maximum water surface elevations shown are for the no-reservoir condition which also represents the future condition with the reservoir full at the start of an event. Stage differences between the current condition and channel bottom with new substrate are shown to decrease with decreasing storm events, with the highest difference occurring at the upstream end (RAPS) and decreasing to zero at the confluence. The 100-year event shows a maximum difference of 7.8 inches at RAPS. Those differences near RAPS decrease to 1.4 inches for the 10-year event and become negligible for the 5 and 2 year events. It is important to note that the stage increases due to the 22-inch substrate without vegetation and the stage increases due to the vegetation alone are similar in magnitude. It is also significant that the channel stays within bank for the 100-year event, even when assuming the reservoir is full and not available at the start of an event.

Figure 3 – Comparison of Maximum Water Surface Elevations with and without 22-inch vegetated channel bottom with new substrate for the No Reservoir/Full Reservoir Condition



4. Despite the channel staying within its banks for the 100 year event with the substrate and after substrate settlement, the 9 outfalls along the creek are submerged to one degree or another. The stage increases due to the substrate must be evaluated to determine the potential impacts to basement flooding and to determine if mitigation measures are necessary. Consequently, it was decided to utilize the City of Chicago's InfoWorks model to evaluate the impacts on basement flooding for the worst case scenario when the reservoir is full at the start of an event and the best case scenario when the stage 2 reservoir is on-line and empty at the start of an event. It was also decided that the 12-hour 100-year storm would be evaluated instead of the 24 hour duration storm since it resulted in higher stages than the 24-hour 100-year storm. Additionally, the 3-hour 100-year storm was to be evaluated since it was determined that the maximum discharge for most of the outfalls occurs for the 3-hour storm duration. Results from that basement flooding impact study are expected to be available by the end of August 2013.

5. The substrate is expected to settle 12 inches over an 9 year period with 60 percent of the settlement occurring over the first 3 years. As a result, the modeling further adjusted the N-value to account for full migration and growth of the vegetation for the condition with new substrate after the 12-inch settlement. The N-value that was used was 0.035. Although the n-value for the type of vegetation that is being planted ranges from 0.035 to 0.05 the lower value was used to account for the fact that the vegetation will not fully establish itself in all locations and that some locations will have no growth or limited vegetation density. Figure 4 shows the comparison of the maximum water surface elevations along the creek for the existing bathymetry, the 22-inch substrate (0.030 n-value) and the substrate after settlement (0.035 n-value). All cases assume that the reservoir is not available at the start of an event.

Figure 4 – Comparison of Maximum Water Surface Elevations for Existing Channel Bathymetry, 22-inch Vegetated Substrate (0.03 n-value), and Vegetated Substrate After Settlement (0.035 n-value) for the 12-hour and 3-hour 100-year Storm Events with Reservoir Unavailable

6. Figure 4 shows that the maximum stages resulting from the substrate after settlement are slightly higher than those prior to settlement. Maximum water surface elevations are approximately 0.5 inches higher for the substrate after the 12-inch settlement. This is the result of the higher n-value that accounts for the full migration and growth of the vegetation.

7. The differences in the maximum water surface elevations for the future condition (with stage 2 reservoir on-line and empty at start of storm) are miniscule due to a significant portion of the CSOs being captured by the McCook reservoir and the greatly reduced CSO discharges to Bubbly Creek from RAPS and the 9 other outfalls. Figure 5 and Figure 6 show these differences for the 12-hour 100-year storm and the 3-hour 100-year storm. Note that in these cases the maximum stages are slightly lower for the substrate after settlement.

Figure 5 – Comparison of Maximum Water Surface Elevations for the Future With Reservoir Condition for the 12-hour 100-year Storm.

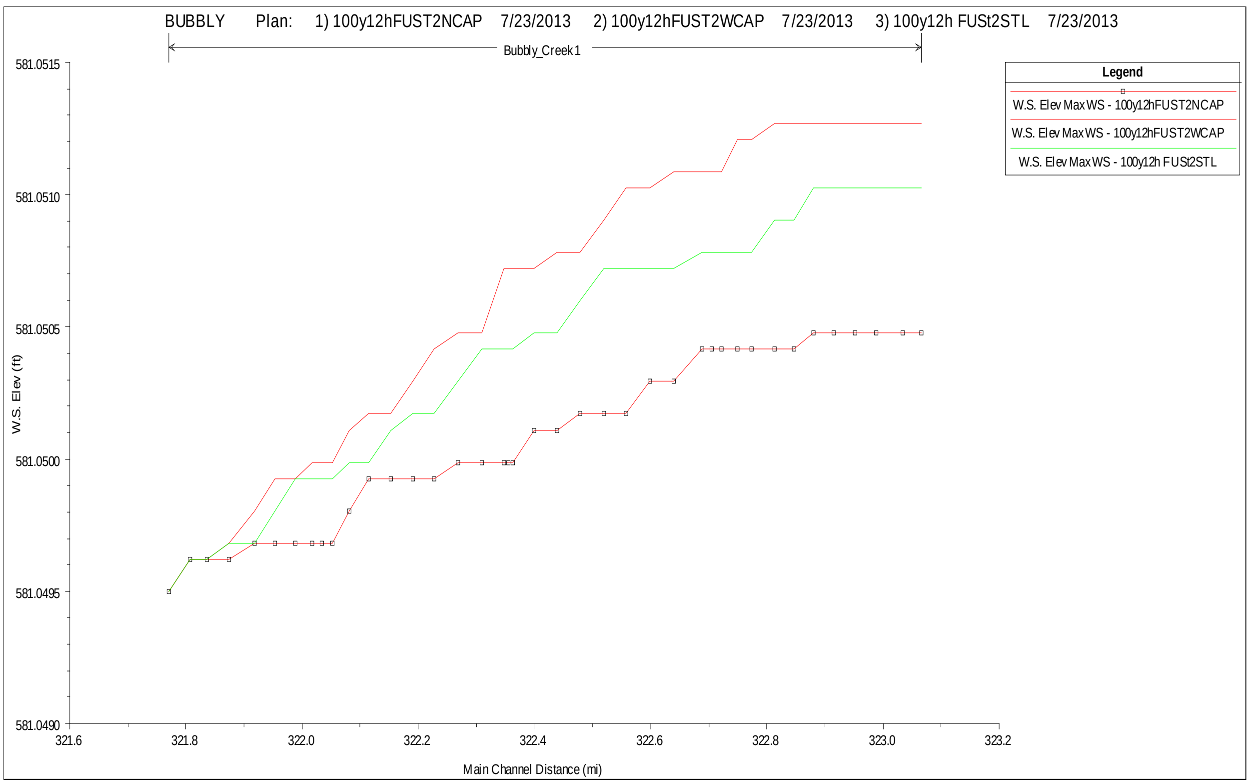
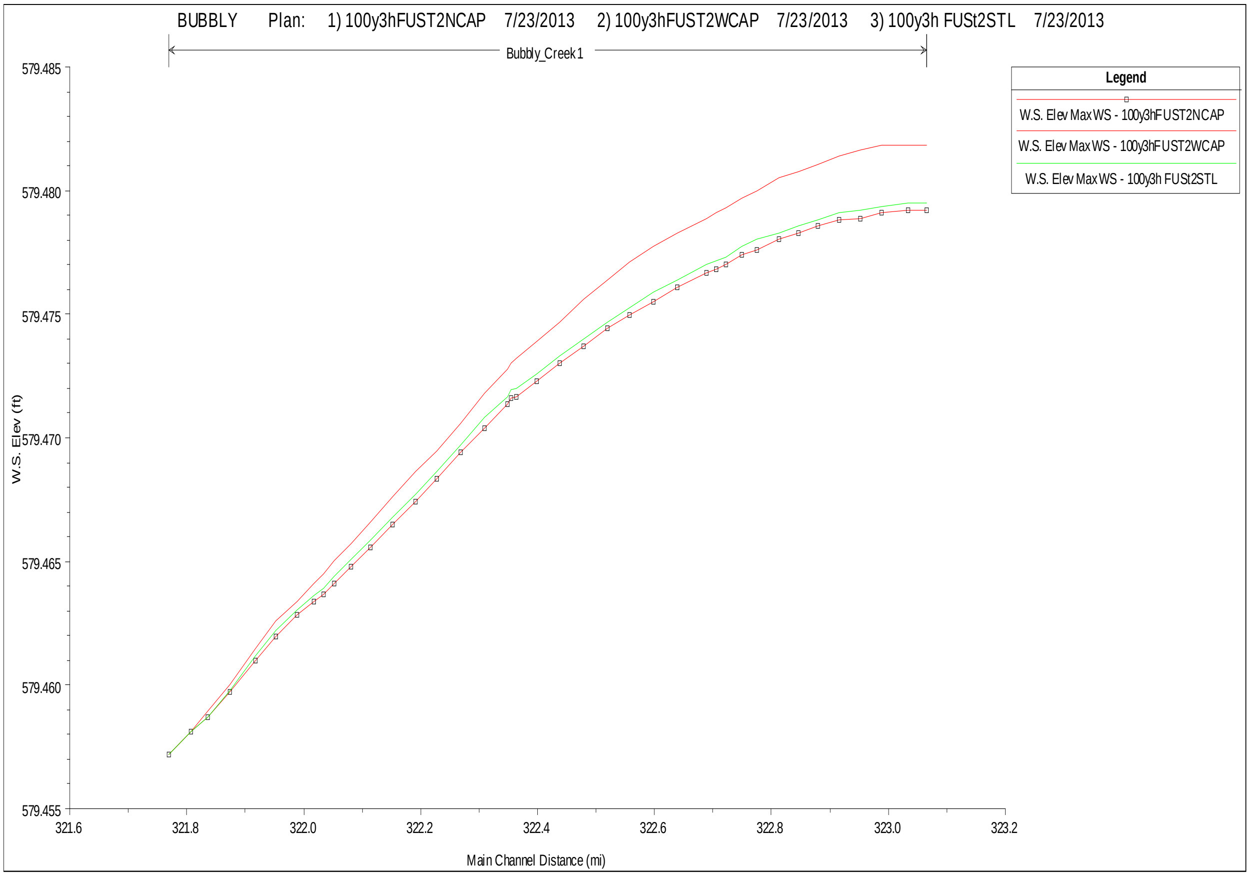
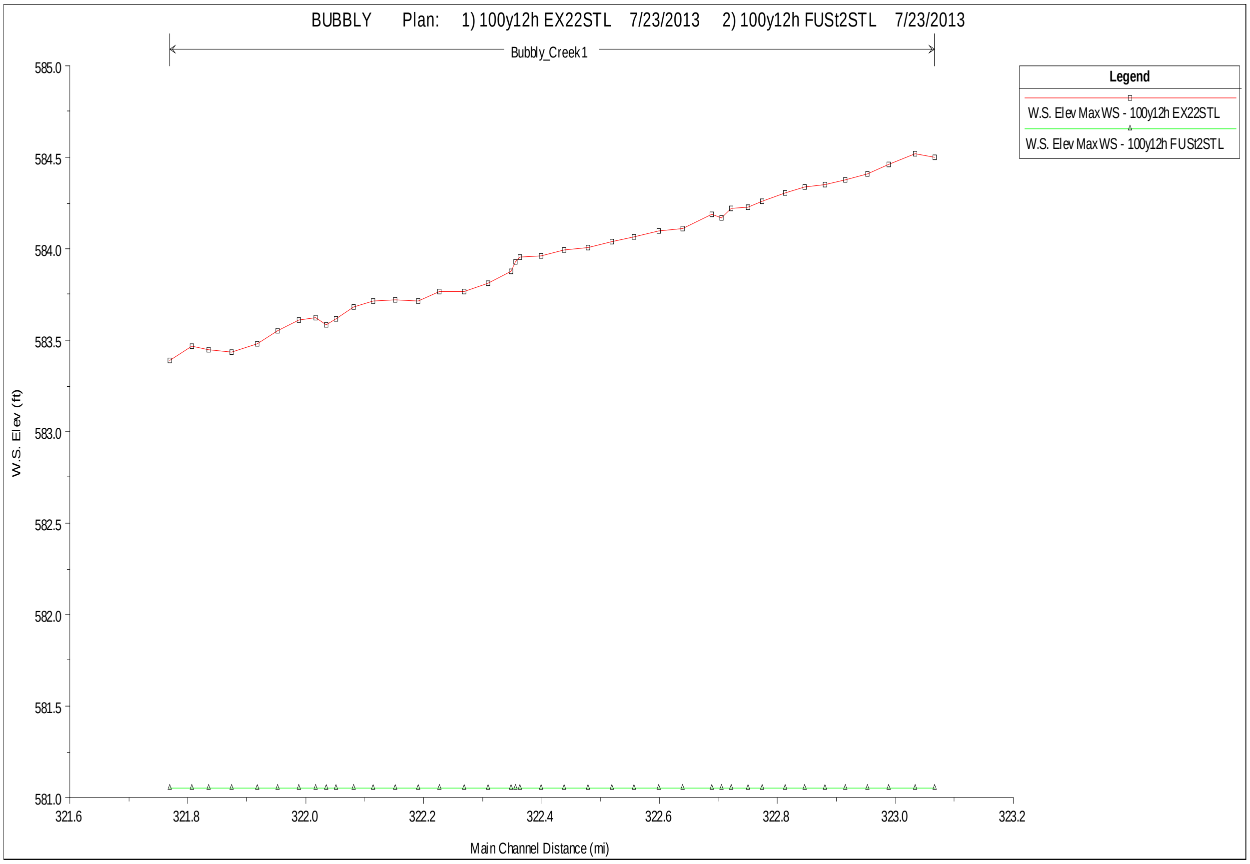


Figure 6 – Comparison of Maximum Water Surface Elevations for the Future With Reservoir Condition for the 3-hour 100-year Storm



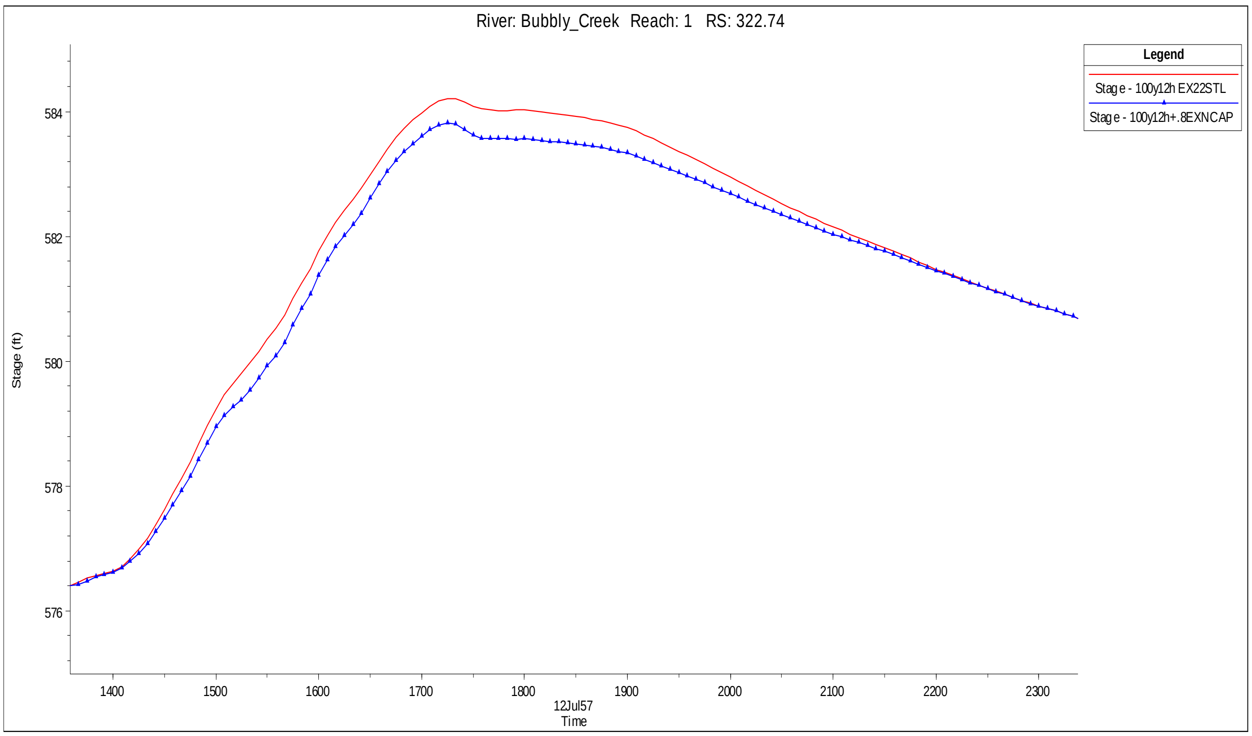
8. The impact of the McCook stage 2 reservoir is shown in Figure 7. Maximum water surface elevations are reduced by as much as 3.5 feet due to the reservoir.

Figure 7 – Comparison of Maximum Water Surface Elevations for the Exisiting (No Reservoir/Full Reservoir) and Future (With Stage 2 Reservoir) Conditions for the 12-hour 100-year storm



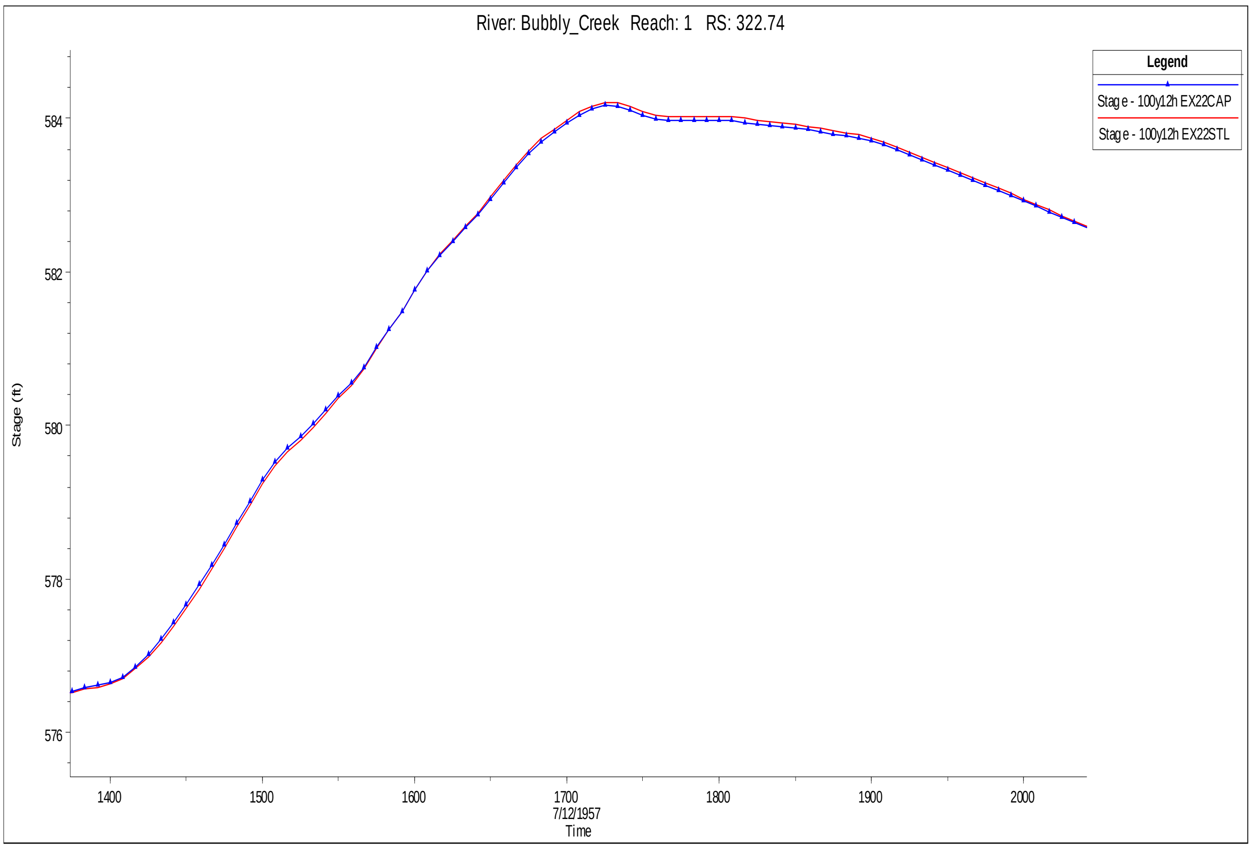
9. Under the InfoWorks modeling that is being conducted stage hydrographs at the various outfalls will be evaluated to determine the impacts of the increased stages on the basement flooding due to increased submergence of the outfalls. Figure 8 shows the stage hydrographs for the existing channel bottom and substrate after settlement at station 322.74, located at the CSO-194 outfall at 35th street, for the existing no-reservoir/full reservoir condition for the 12-hour 100-year storm. Stages increase by as much as 5.4 inches at this location due to the substrate and vegetation. Part of the basement flooding study is to determine if mitigation measures such as pumping and/or dredging would be required to mitigate for any increases in basement flooding.

Figure 8 – Stage Hydrographs Comparing the Existing Channel Bottom to the Substrate after Settlement at 35th Street (station 322.74) for the No Reservoir/Full Reservoir Condition for the 12-hour 100-year Storm.



10. A comparison of stage hydrographs for the 22-inch substrate with vegetation (0.030 n-value) and the substrate after 12-inch settlement (0.035 n-value) shows the impact of the increase in n-value. The stage hydrographs for the two cases are very similar, meaning that the slight increase in n-value after settlement was able to offset stage decreases due to settlement. Figure 9 shows the stage hydrographs for these two substrate conditions at the same location as Figure 8 for the existing no-reservoir/full reservoir condition for the 12-hour 100-year storm. The stage hydrographs track each other very closely with some stages for the 22-inch substrate being higher or lower than the substrate after 12-inch settlement, depending on the point in time of the hydrograph.

Figure – Stage Hydrographs for the 22-inch Substrate versus the Substrate after 12-inch Settlement at 35th Street (station 322.74) for the No Reservoir/Full Reservoir Condition for the 12-hour 100-year Storm.



Attachment 3:

Modeling of Base Condition for Bubbly Creek

Modeling of Base Condition for Bubbly Creek

Sung-Chan Kim and Ernest R. Smith
Coastal and Hydraulics Laboratory, Engineer Research and Development Center

1. Introduction

A base condition for Bubbly Creek was determined as the existing condition with the full capacity of Racine Avenue Pumping Station (RAPS) and the maximum combined sewer overflows (CSOs) in the system. CH3D (Curvilinear Hydrodynamic in three-Dimensional) model and GTRAN (Gridded Transport) model were sequentially applied to assess the spatial and temporal variations of hydrodynamic conditions and bed stresses. These predicted bed stresses provided sediment transport characteristics and thus the stable sediment characteristics required for new sediment substrate.

2. Grid Setup and Implementation of RAPS and CSO Flows

Existing Bubbly Creek and adjacent Chicago Sanitary and Shipping Canal CH3D model grids were used (Fig 1). The model grid was built to represent Bubbly Creek (BC) and the portion of Chicago Sanitary and Ship Canal (CSSC) to Lake Michigan. The total length of CSSC in the model is about 31 km. BC is connected to the CSSC at about 8 km from the eastern boundary at the Lock. The horizontal resolution is typically 100 m long and 20 m wide from the eastern boundary to upstream of the BC entrance (about 10 km from the eastern boundary) and then gradually elongated to 700 m long near the western boundary. Inside BC, typical grid resolution is 15 m long and 5 m wide. The detailed coverage of the grid is shown in Appendix A. The dimension of computational nodes in the horizontal plane is 169 by 185. The maximum number of layers in the vertical direction with 1-m increments is set as 8, corresponding to 8 m of mean depth. The model bathymetry was updated for Bubbly Creek with the most recent update bathymetry provided by Chicago District (Fig 2). Total number of active water cells is 7631 and the number of flow faces is 18788.

There are 3 boundaries for the whole system: Eastern boundary is set at Chicago River at Columbus Drive, Northern boundary is at NBCR at Grand Avenue, and the Western boundary is set at CSSC Lemont. For the western and northern boundaries, flow conditions were extracted from USGS stream gages and at the eastern boundary, water levels from the USGS gage was used. Figure 3 shows the flows at the two boundaries. Additional flow boundary conditions were imposed for the flows from Racine Avenue Pumping Station (RAPS) and CSO flows (Fig3). A total of 8 CSO locations were identified within Bubbly Creek.

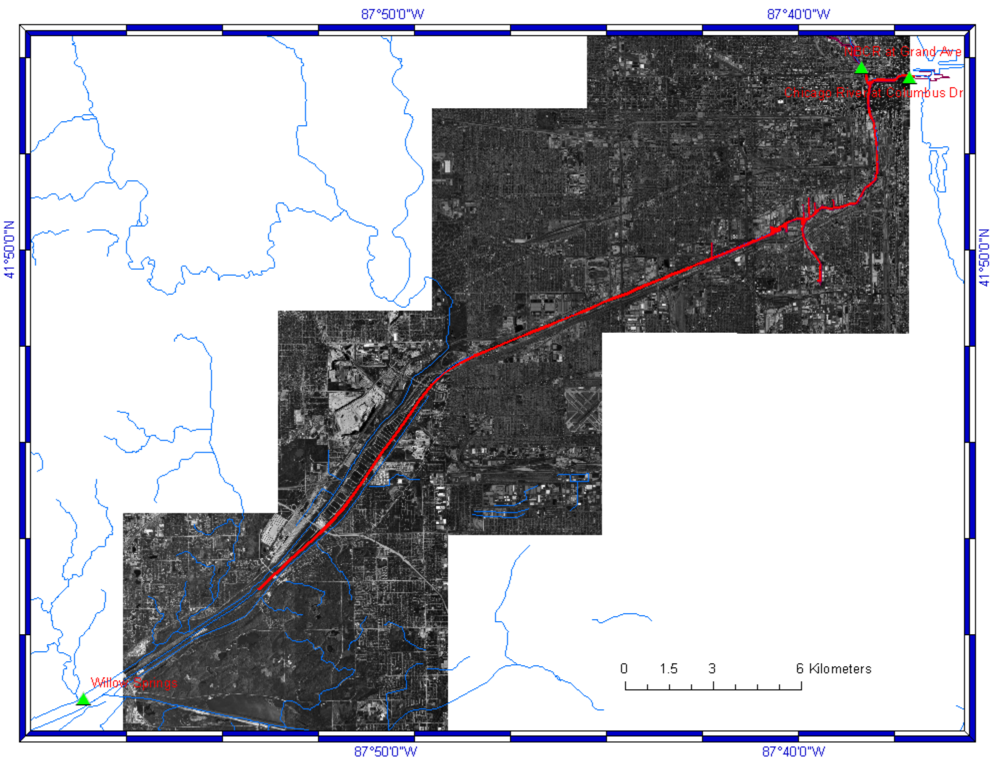


Figure 1. Model grid with 3 boundaries.

Figure 2. Bathymetry data (TIN file) for Bubbly Creek.

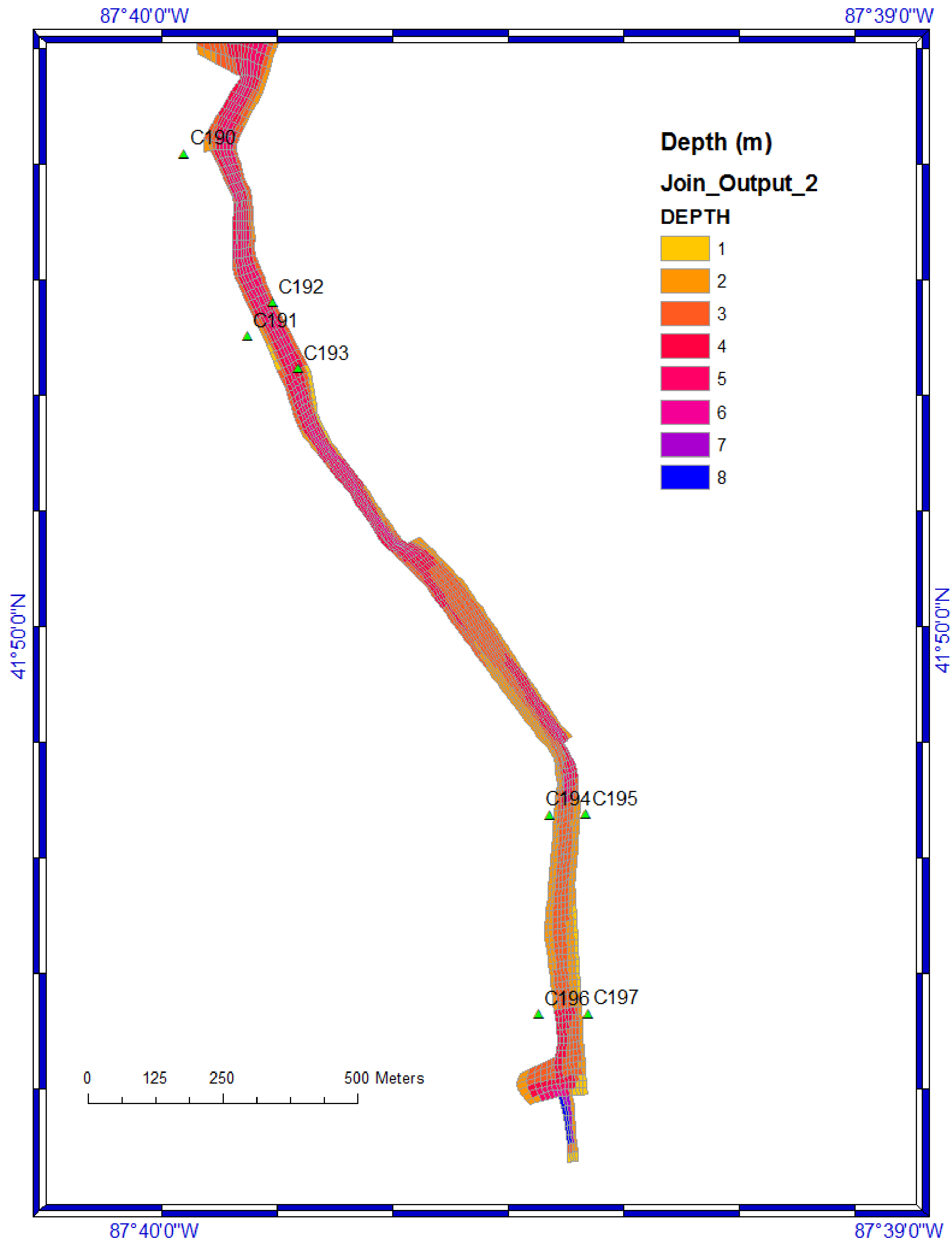


Figure 3. Depths interpolated to model grid for Bubbly Creek. Also shown are 8 CSO locations.

3. Simulation Period and Implementation of Full Capacity Flows

October 2, 2006 was marked by a hydraulic event. Figure 4 shows water levels recorded from the USGS gage at Columbus Drive and stream flow measurements from USGS stream gages at Grand Avenue (Northern Branch) and Lemont (Chicago Sanitary and Shipping Canal) as well as RAPS flows. The simulation period was the 15 days between 10/1/2006 and 10/15/2006. The spin up for the run was 5 days between 9/26/2006 and 9/30/2006.

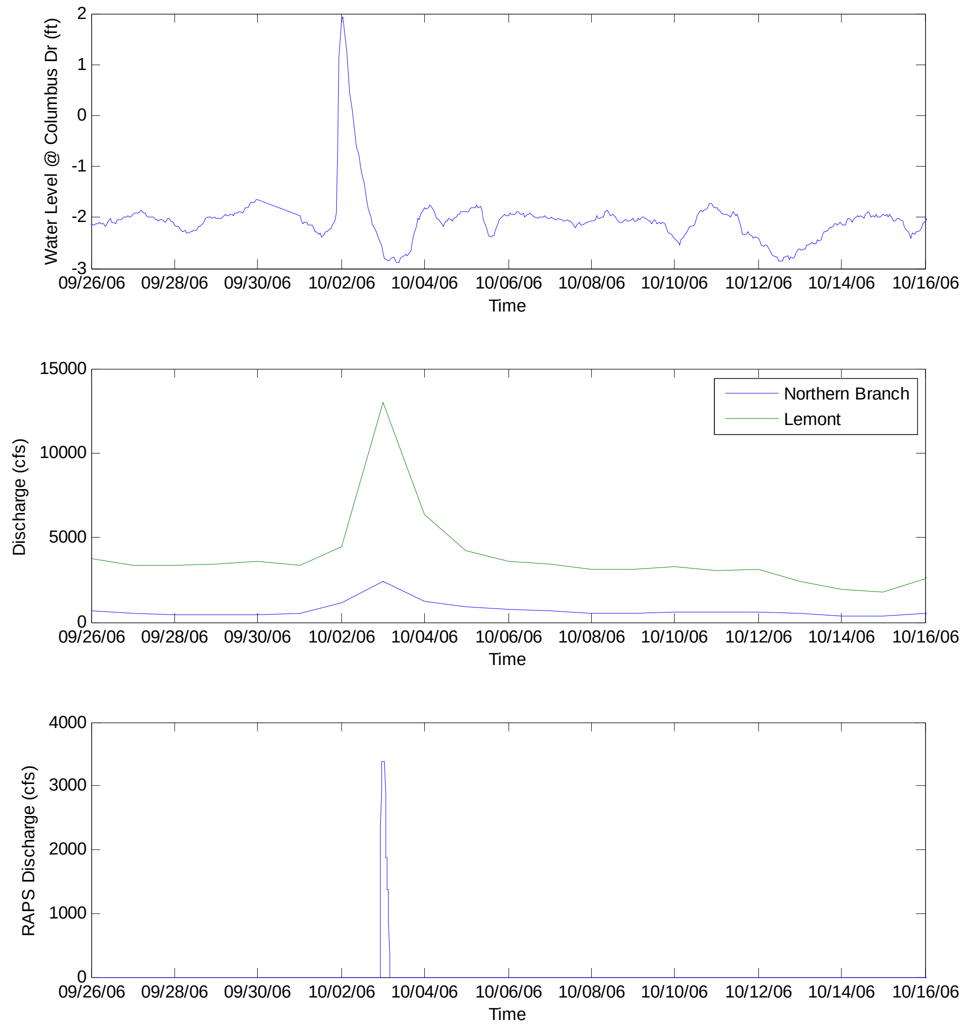


Figure 4. Hydrodynamic forcing conditions for the simulation period.

The RAPS flow data was multiplied by a factor to implement the full capacity of RAPS (6000 cfs at maximum). A similar manipulation was done for the CSOs to reach the maximum flows (Table 1) during the event.

Table 1. CSO locations and maximum flows

CSO Outfall	Maximum Flow (cfs)
190	126
191	8
192	78
193	133
194	541
195	84
196	82
197	31

4. CH3D Model Results

The high flow event during the later part of 10/2/2006 showed high bottom currents throughout Bubbly Creek. Figure 5 shows the time series of water elevations of the creek and bottom currents at several selected stations in the creek. Also shown are a line crossing just south of the 35th street Bridge and selected stations along the creek to show bottom current variations. High water levels precede the flow event at the beginning of 10/2 and the flow event was short-lived (4 hours). The water elevation disturbances during this event were visible but not significant (Fig 5a). Bottom currents were high just south of 35th street Bridge (Fig 5b), and higher at flanks (above 200 cm/s) than the channel (about 150 cm/s). Considerable variations by a few factors were also shown along the creek (Fig 5c).

Figure 6 shows distribution of the maximum bottom currents in Bubbly Creek during the simulation time period (10/1/2006 to 10/15/2006). Figure 7 shows the cumulative distribution of the maximum bottom current in the creek. Median velocity is about 95 cm/s, the 95 percentile is about 140 cm/s, and the 99 percentile is about 160 cm/s.

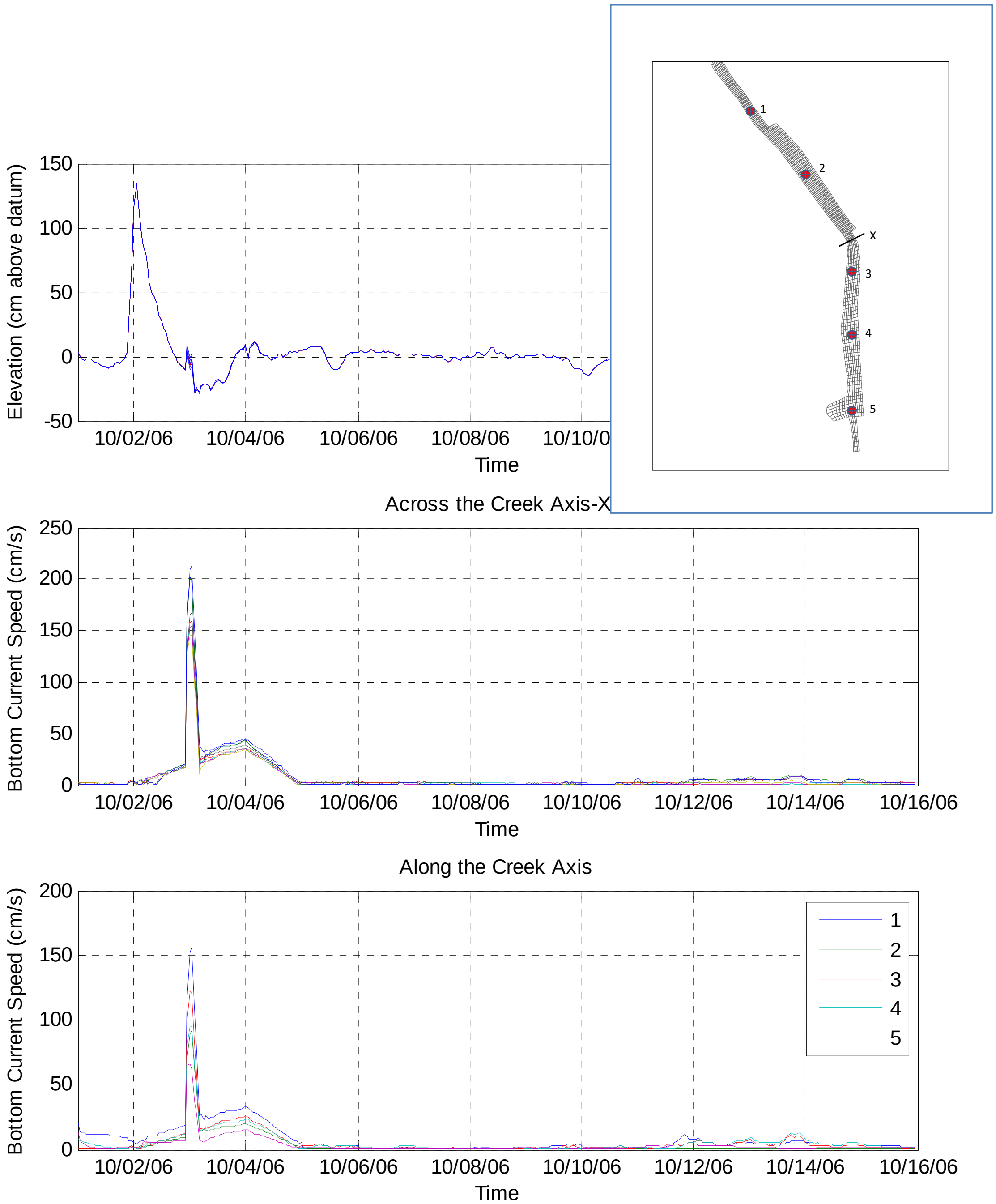


Figure 5. Time series plots: (a) elevation; (b) bottom current across channel; and (c) bottom currents along channel.

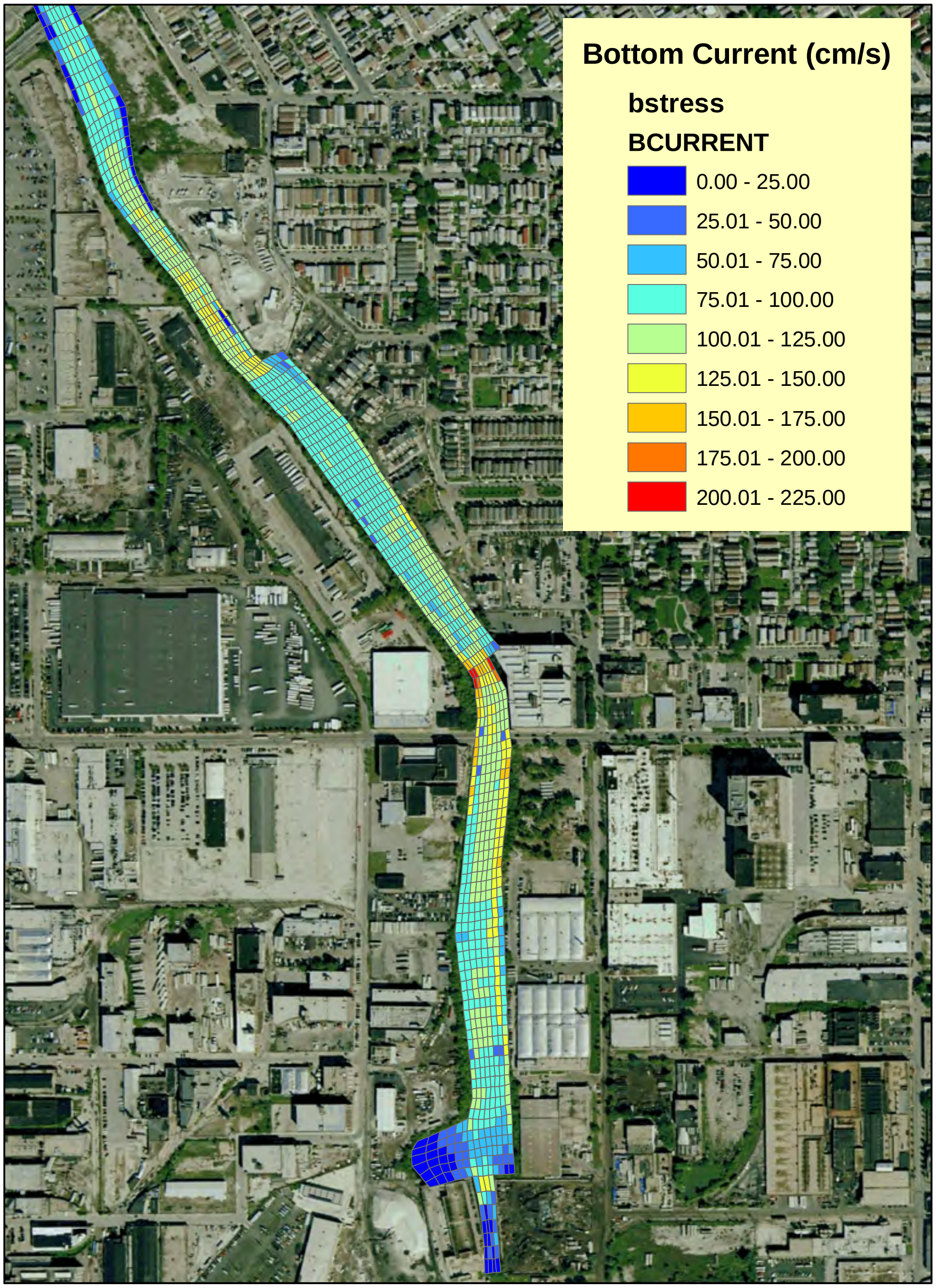


Figure 6. Maximum bottom currents for 10/1/2006 - 10/15/2006 with full capacity RAPS flow and the maximum CSO flows.

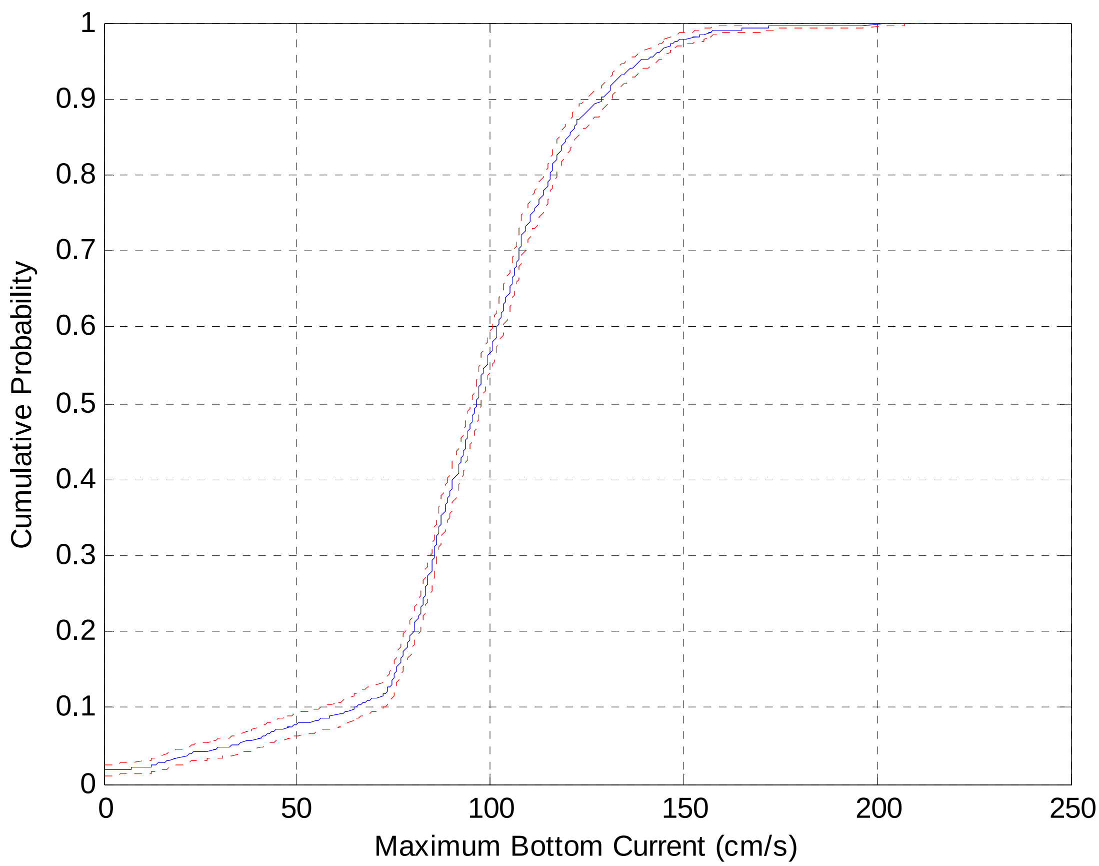


Figure 7. Estimated empirical cumulative distribution function of bottom current in Bubbly Creek with 95 percent confidence interval.

5. GTRAN Results

The GTRAN model is typically applied to predict potential transport magnitudes and pathways for a given grain size through calculation of the bottom shear stress. GTRAN calculates sediment transport at grid points through a collection of sediment transport methods from input hydrodynamics and sediment bed conditions. Transport occurs if the bed stress exceeds the critical shear stress for the grain size (d_{50}). For the Bubbly Creek application, it was desired to determine the sediment size at which no transport occurs for the high flow event simulated by CH3D. Therefore, the maximum velocity at each grid point over the time series was used to determine bed stress at each location. Bed stresses calculated from the maximum velocities were taken as the critical shear stress for incipient motion to determine the minimum substrate grain size at which transport would not occur.

For Bubbly Creek, GTRAN requires location, depth, and depth-averaged velocities for each time step. Depth-averaged velocities were obtained from the CH3D grid nodes. Substrate sizes were determined for two conditions: a condition in which the new substrate was placed at the existing bed depth, implying dredging of the creek, and a condition in which the new substrate was placed on top of the existing bed.

Figure 8 shows the distribution necessary d_{50}-sizes to remain stable over the creek at the existing bed depth. Sizes were sorted into ranges denoted by color. Stable d_{50}'s range from less than 1 mm to 20 mm.

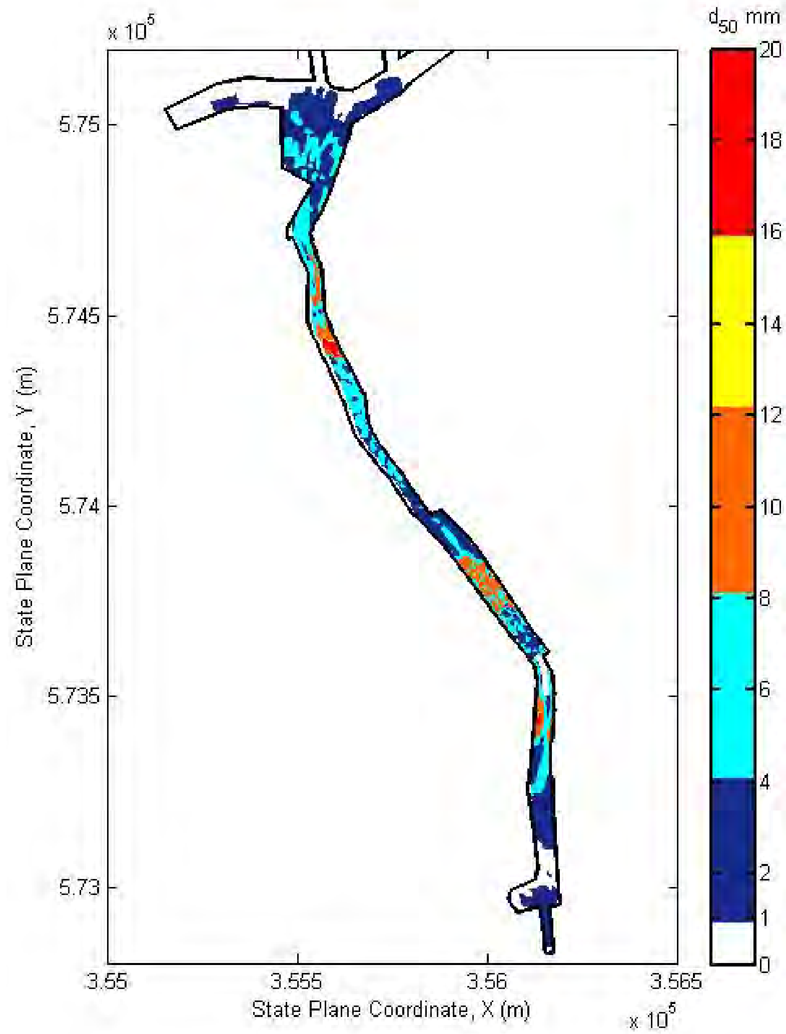


Figure 8. Required stone size required at existing depth.

The second scenario examined consisted of placing new substrate on top of the existing bed. It was determined that substrate with a thickness of up to 55 cm (22 inches) was required for the stone sizes to be placed. The addition of the new substrate reduces the depth in creek, which would increase currents. Through continuity, the currents resulting from CH3D were adjusted by reducing the depth of the entire creek by 55 cm. Figure 9 shows the results of the GTRAN simulation with the new substrate in place. The maximum d_{50} increased to 50 mm as a result of the shallower depth.

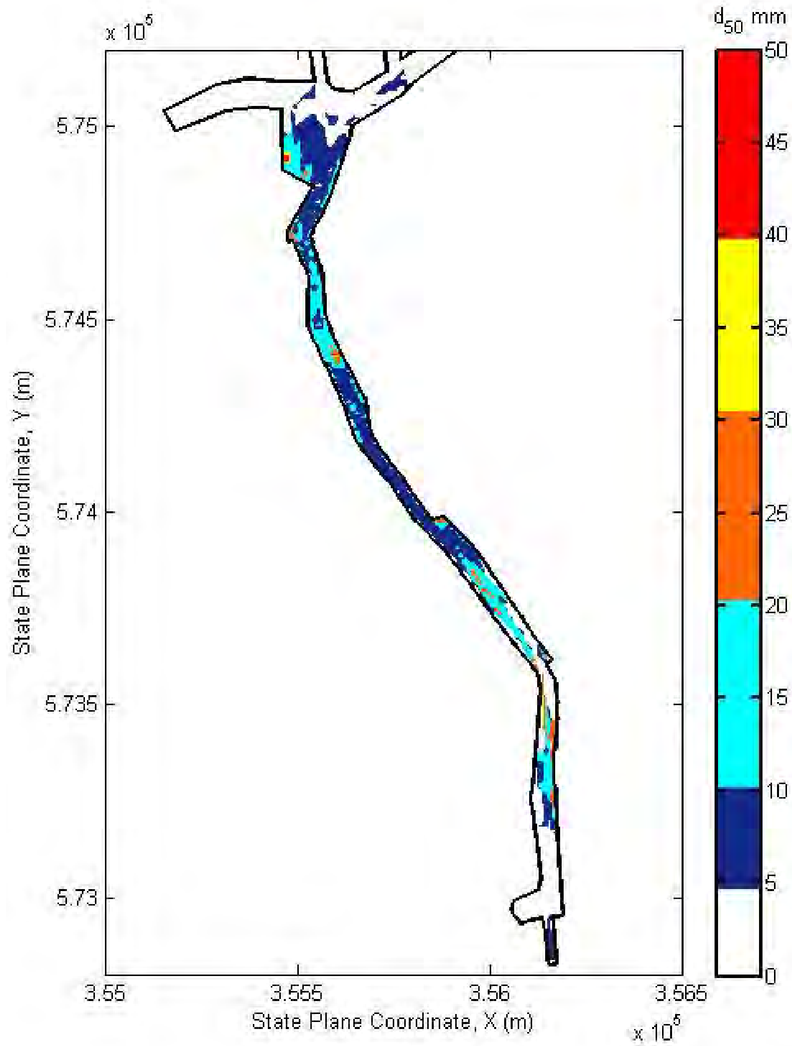


Figure 9. Required stone size required with 55-cm substrate layer installed.

Depths are shallow in the turning basin at the entrance to Bubbly Creek and there was concern that the 55-cm substrate layer would emerge from portions of the turning basin. Figure 10 shows a 10-mm stone size is necessary for substrate at the existing bed depth. However, the required d_{50} increases to 50 mm if the 55-cm thick new substrate layer were in place (Figure 11). Figures 12 and 13 show maximum velocities in the turning basin for the depth with the existing bed and with the added 55-cm substrate layer, respectively. Velocities with the existing bed reach ~1.2 m/s, but increase to 2.6 m/s with the new substrate in place. Because of the existing shallow depth and relatively low velocities, a less thick substrate layer was proposed for the turning basin. Figure 14 indicates that if the added substrate thickness is reduced to 46 cm that the required d_{50} in the basin is reduced to 30 mm. A thin substrate layer of just 20 cm would likely reduce the required D50 to about 15 mm.

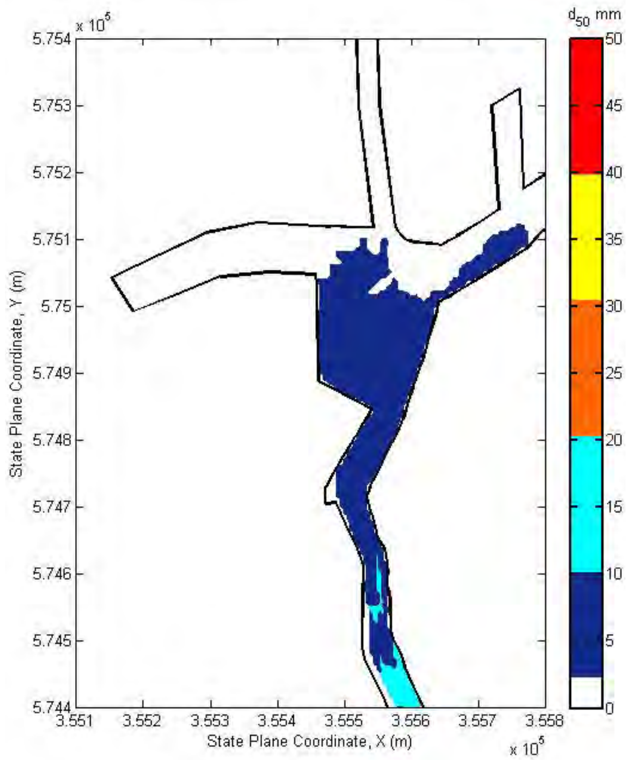


Figure 10. Required stone size required in the turning basin at existing depth.

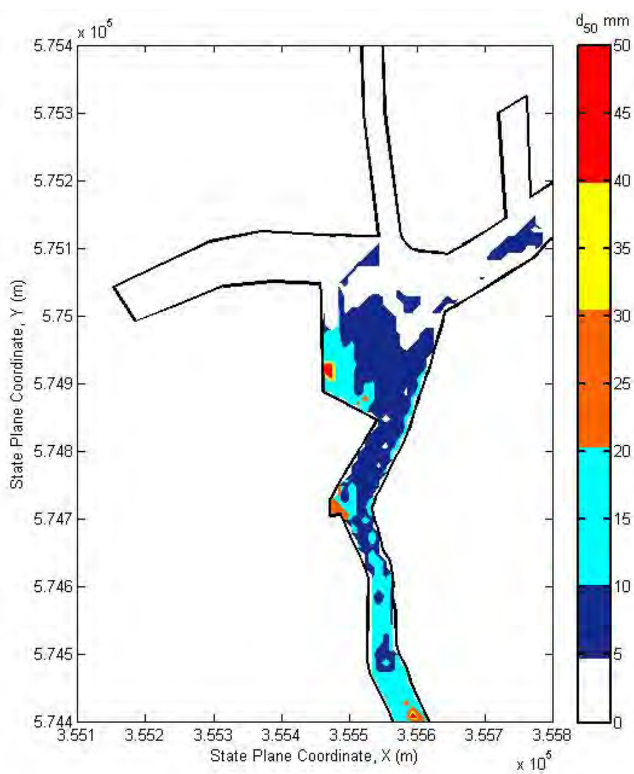


Figure 11. Required stone size required in the turning basin with 55-cm substrate layer installed.

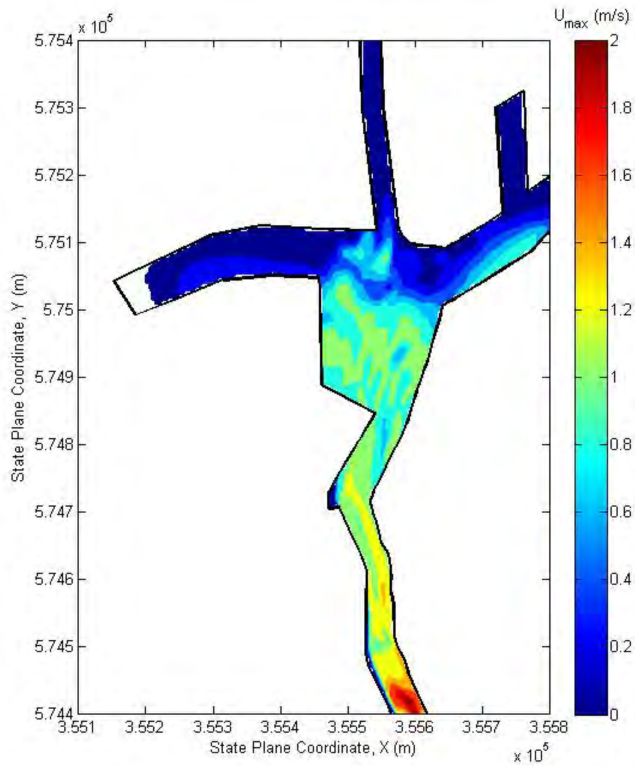


Figure 12. Maximum velocities in the turning basin at existing depth.

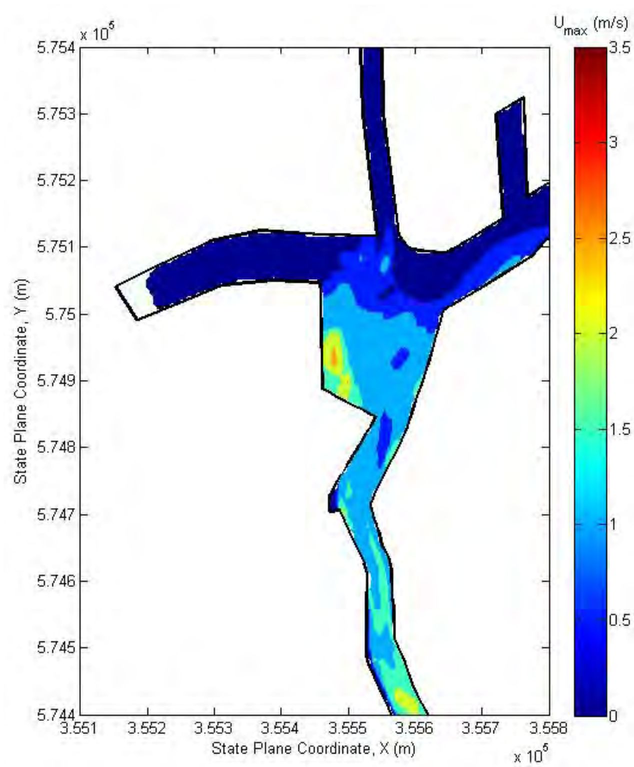


Figure 13. Maximum velocities in the turning basin with 55-cm substrate layer installed.

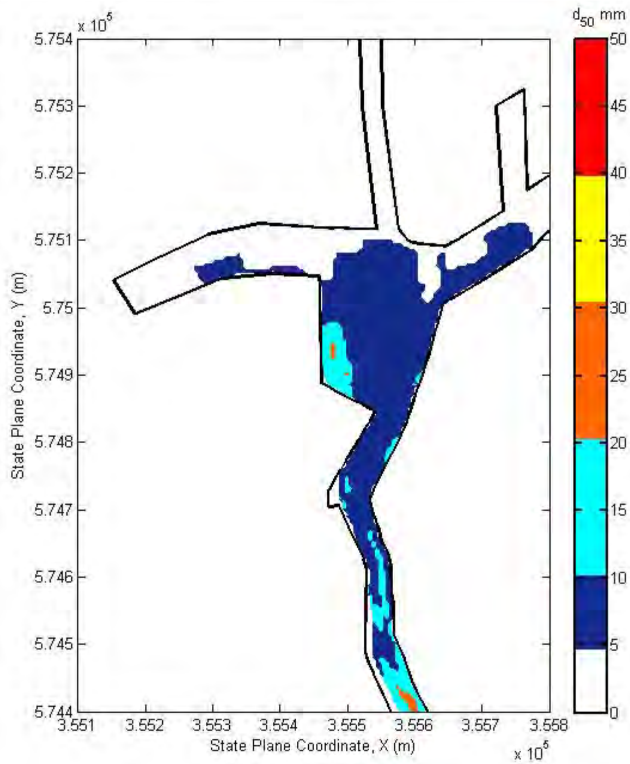


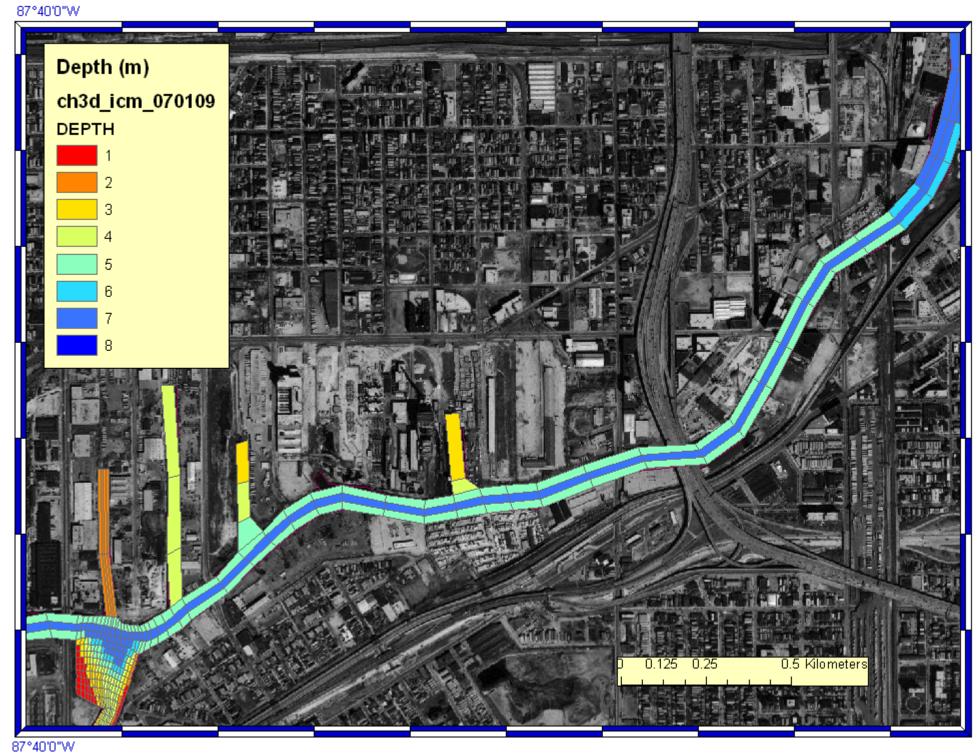
Figure 14. Required stone size required in the turning basin with 55-cm substrate layer installed.

6. Summary

The CH3D model was set up, calibrated and ran to determine current velocities and bottom shear stresses for a stable coarse-grained substrate with surfaces at approximately the existing bottom elevation and at approximately 22 inches above existing grade. The predicted bottom shear stresses were used in the GTRAN model to determine the stable D_{50} particle size for the substrate. At existing grade, the stable D_{50} particle sizes are predominantly below 10 mm, but can be as large as 20 mm in a few small areas of the channel. If substrate is added to the channel without dredging to maintain the channel cross-section and conveyance, the currents in the channel and therefore the bottom shear stresses will increase significantly, particularly in shallow areas. If a 22-inch substrate layer were placed above the existing grade, then the stable D_{50} particle sizes are predominantly below 20 mm, but can be as large as 50 mm in a few small areas of the channel.

The southwest section of the turning basin is very shallow; therefore, small changes in substrate thickness without dredging can have large changes in the stable D50 particle size. A 4-inch reduction in thickness reduces the required D50 from 50 mm to 30 mm. A thin substrate layer should be considered for the turning basin.

Appendix A. Figures to show the grid coverage.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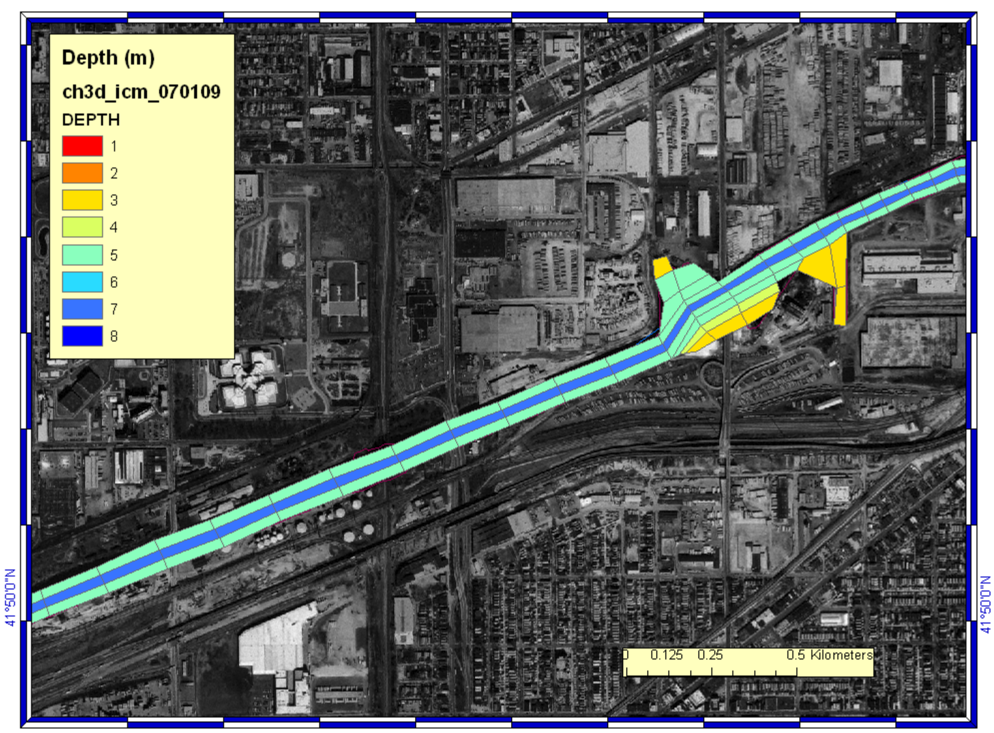
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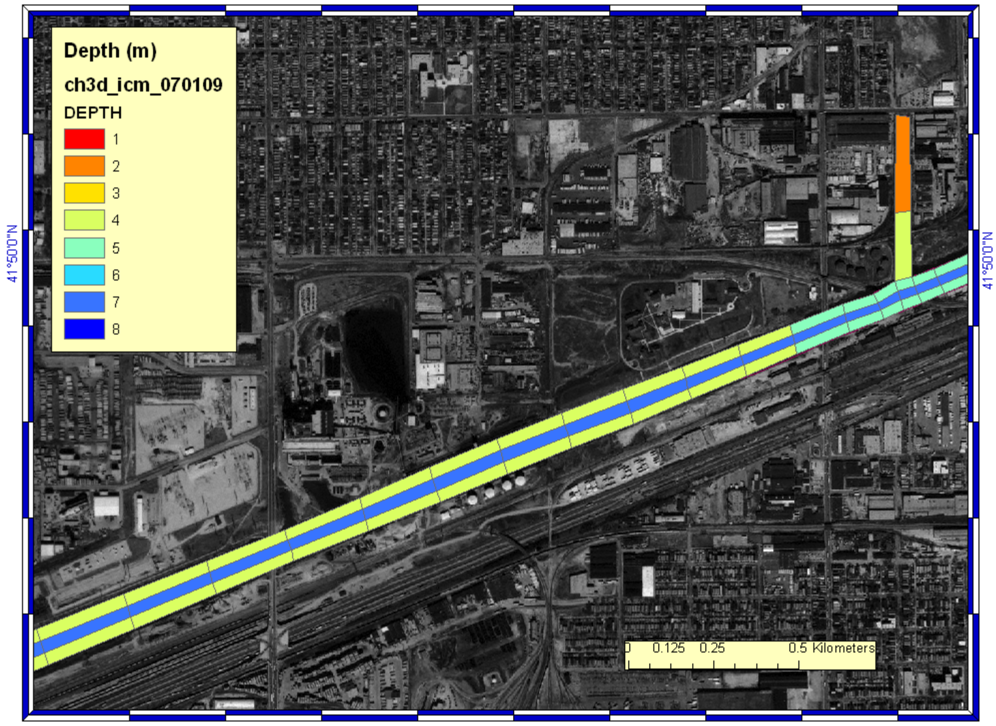
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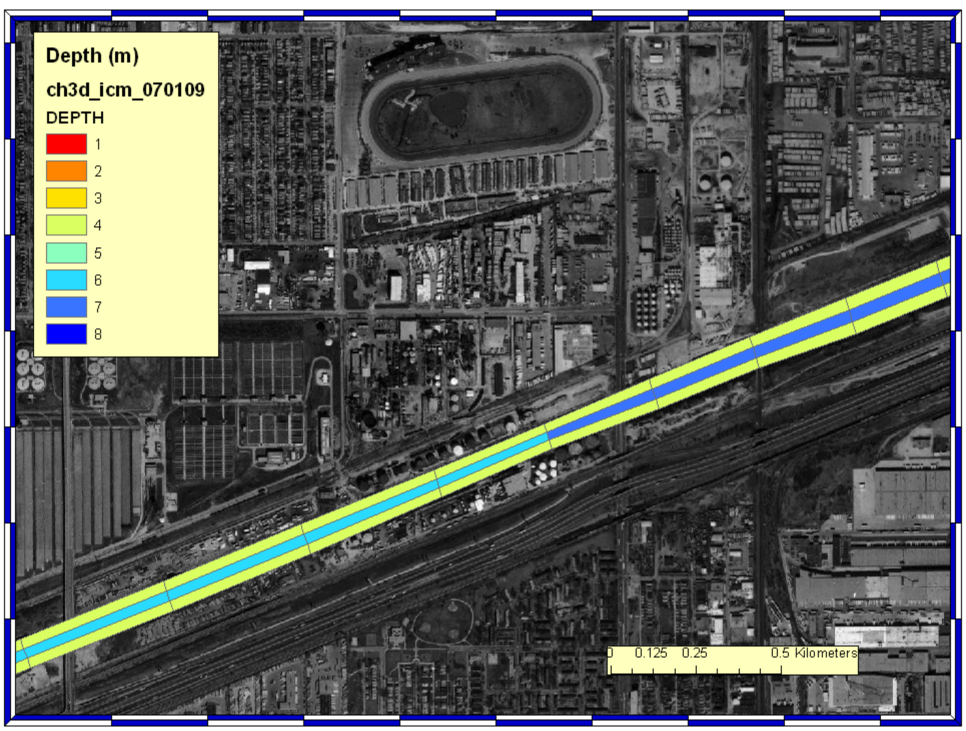
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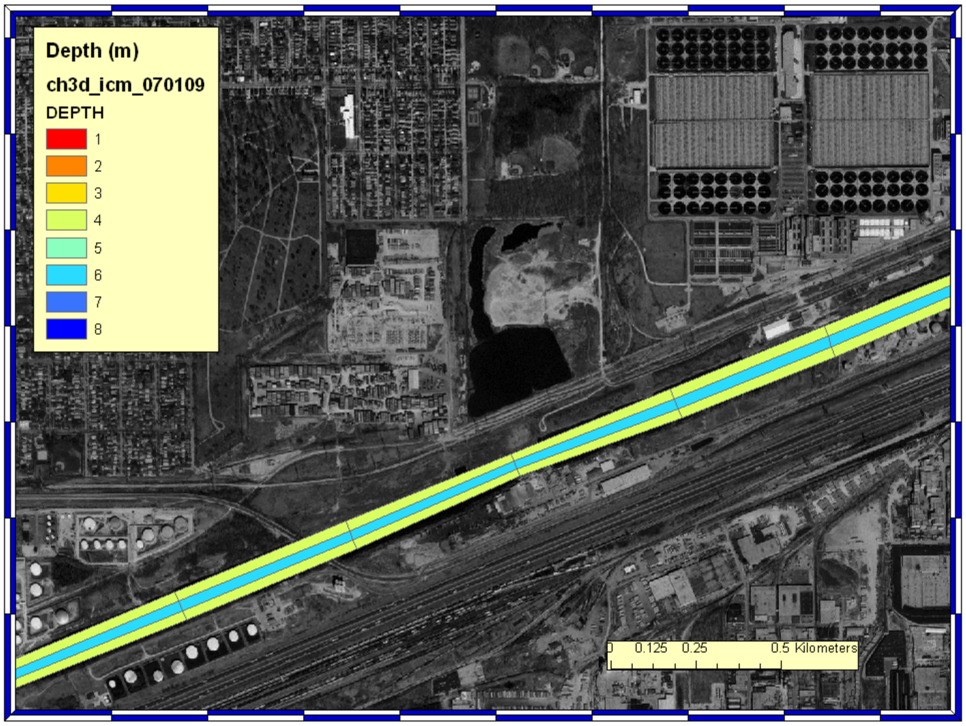
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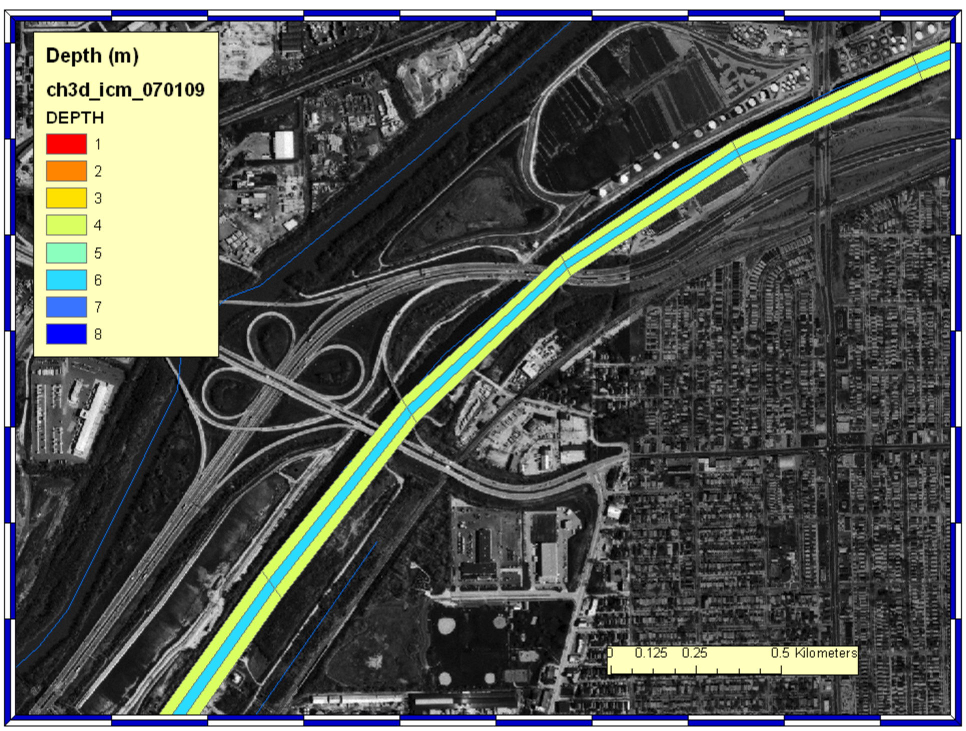
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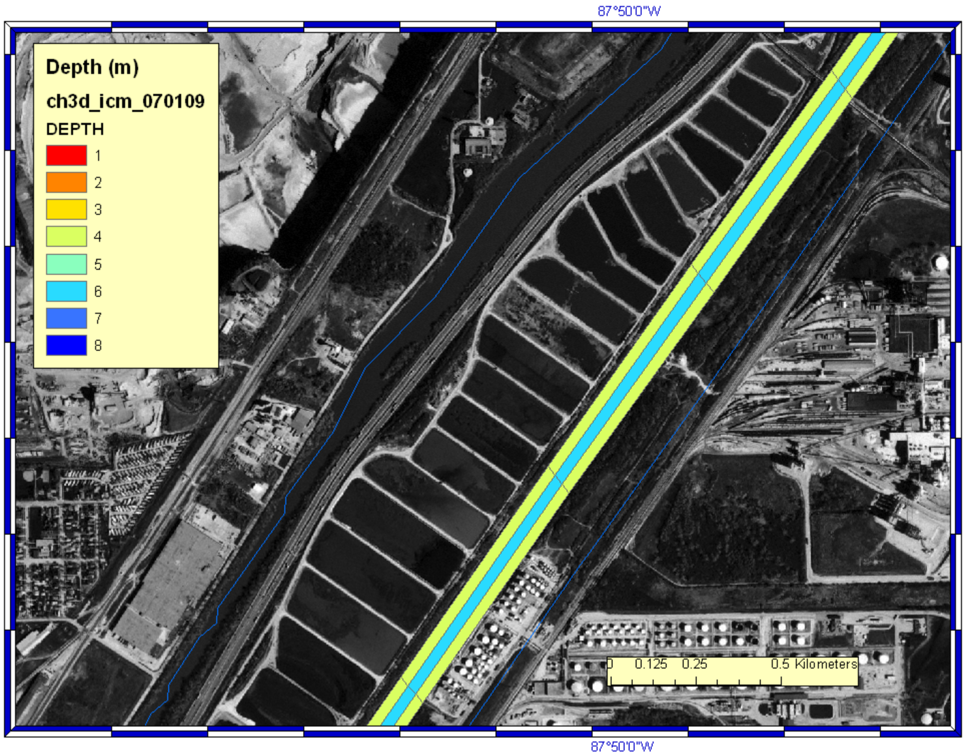
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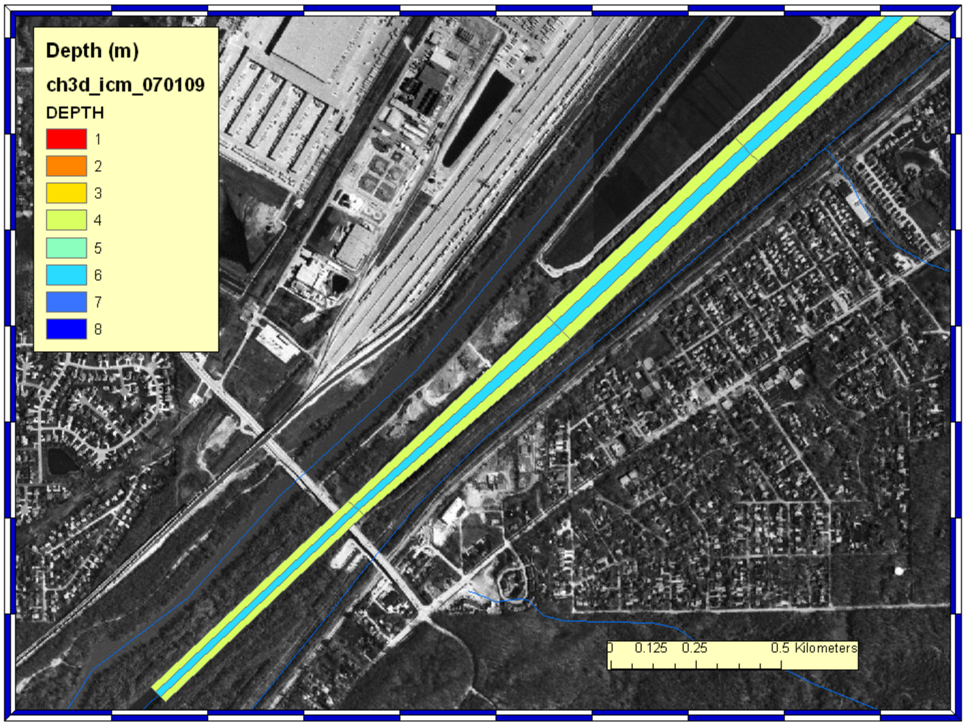
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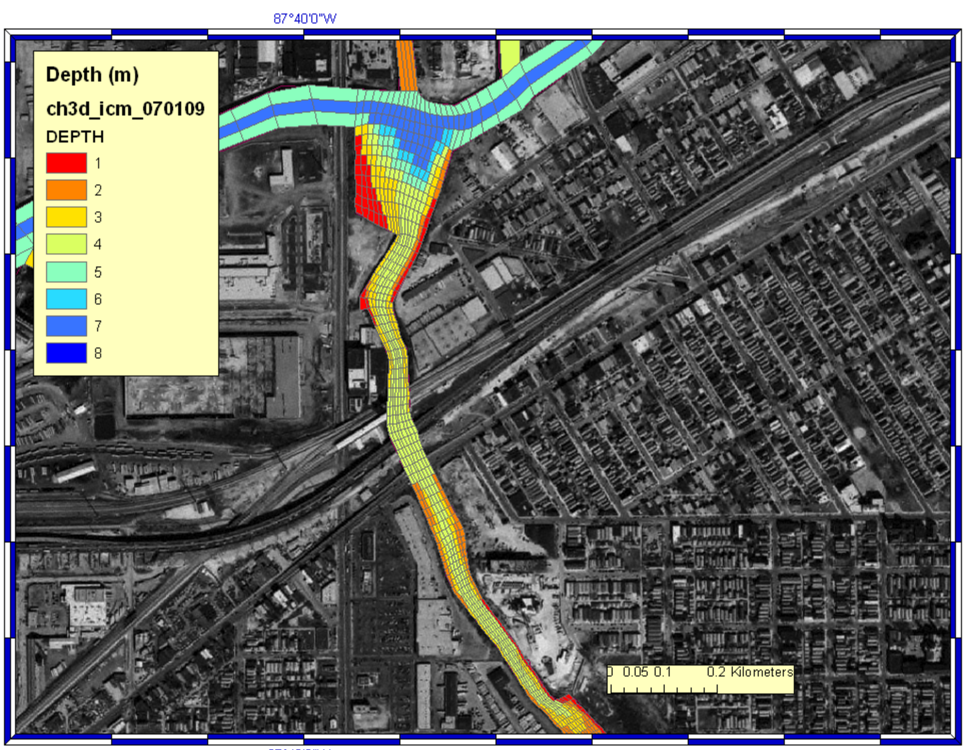
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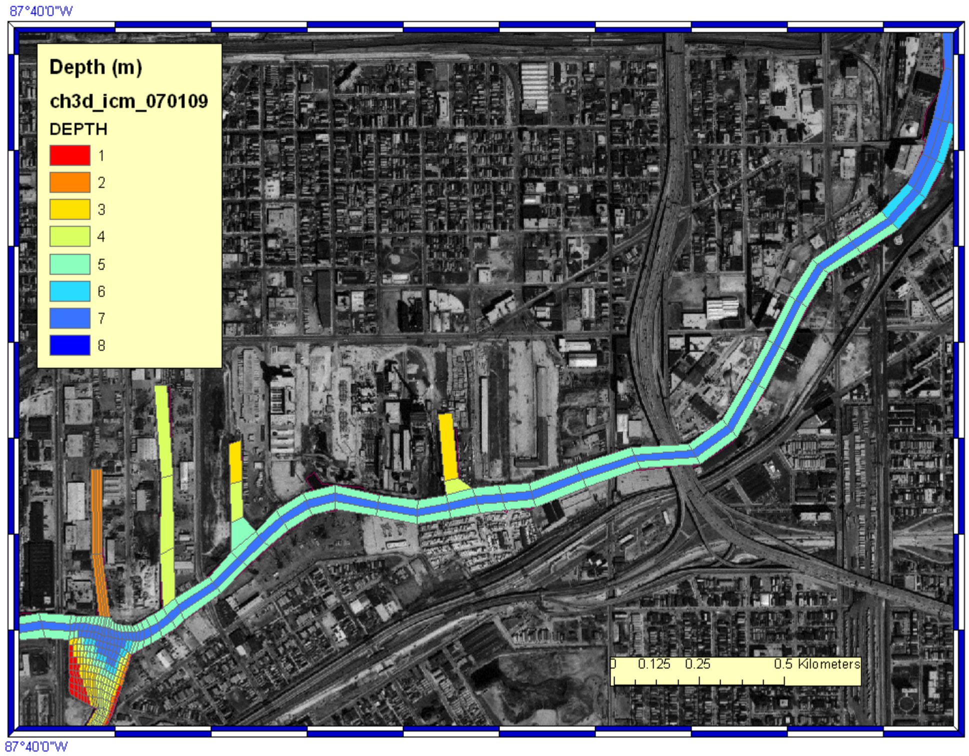
Appendix A. Figures to show the grid coverage.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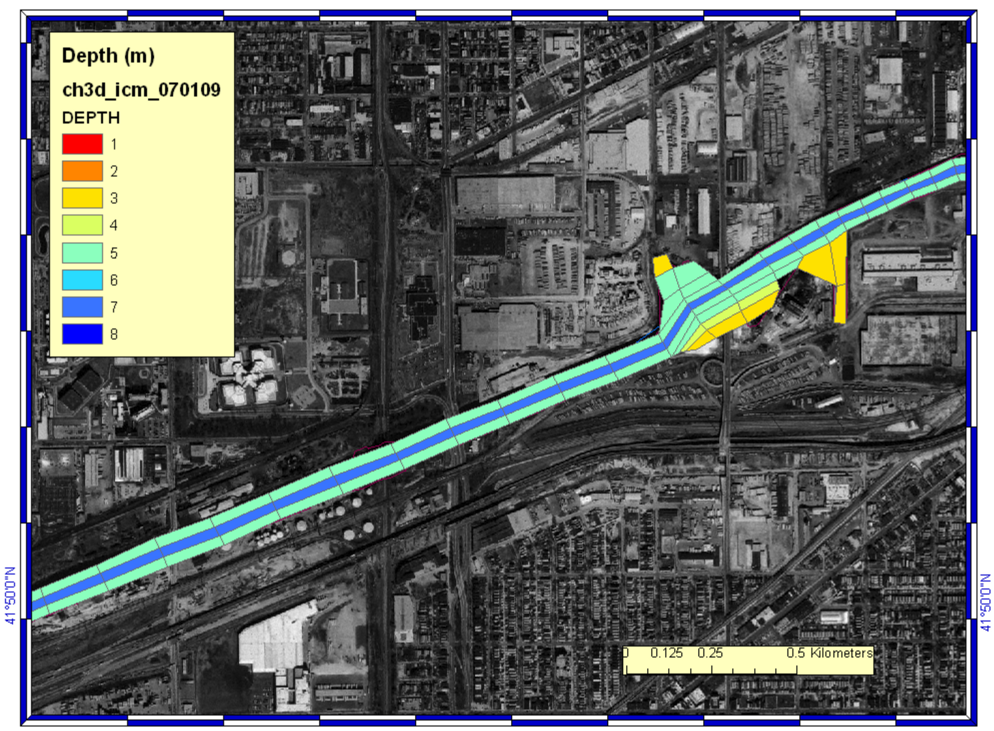
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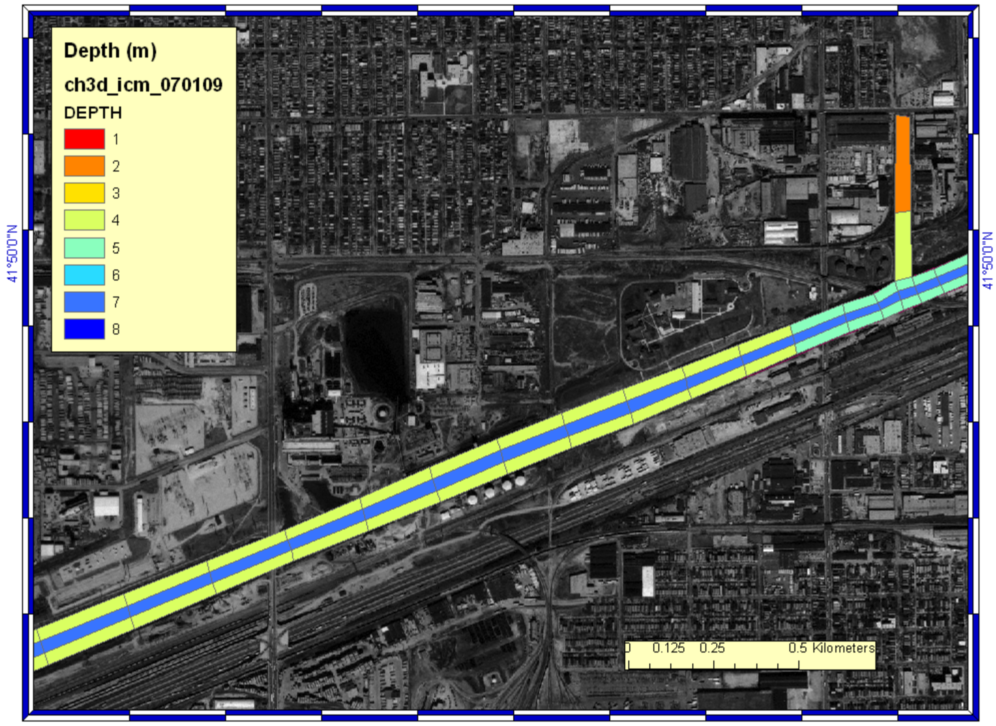
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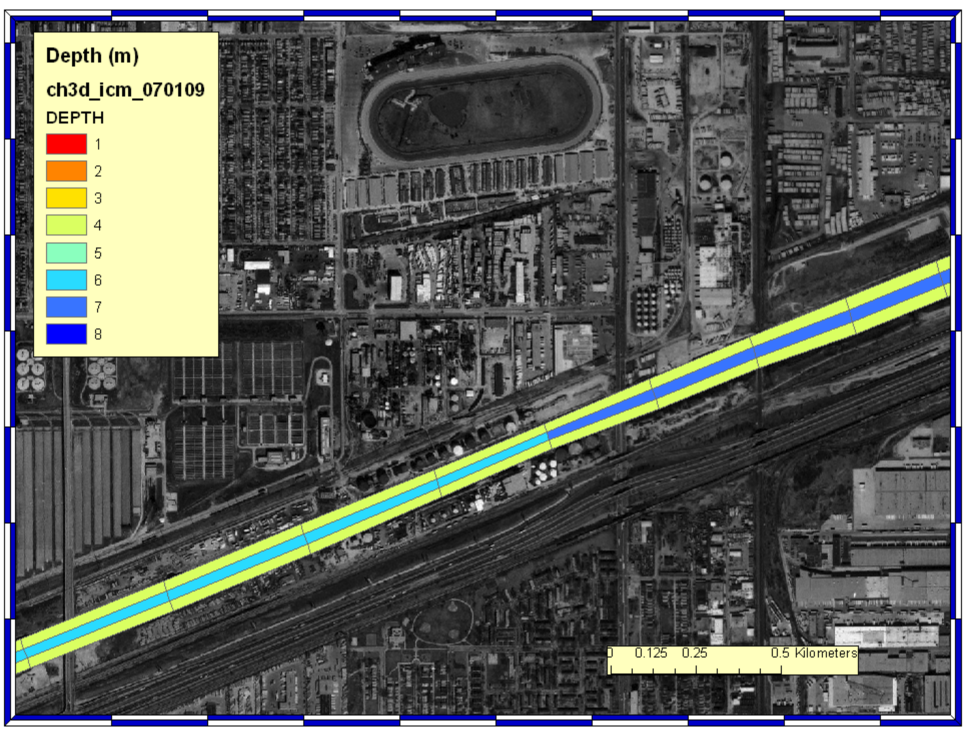
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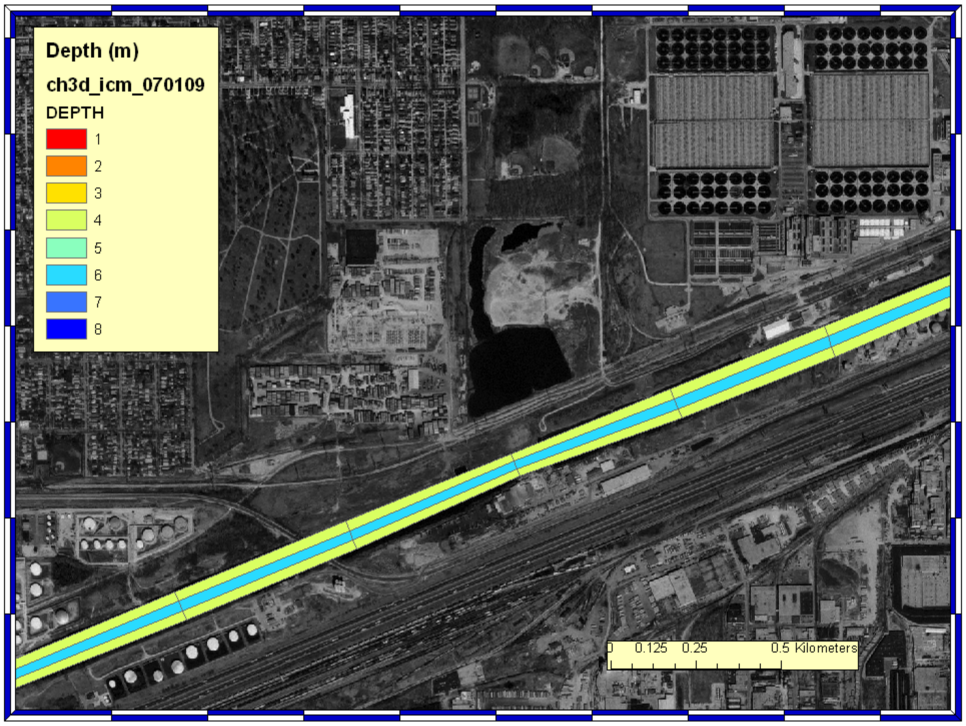
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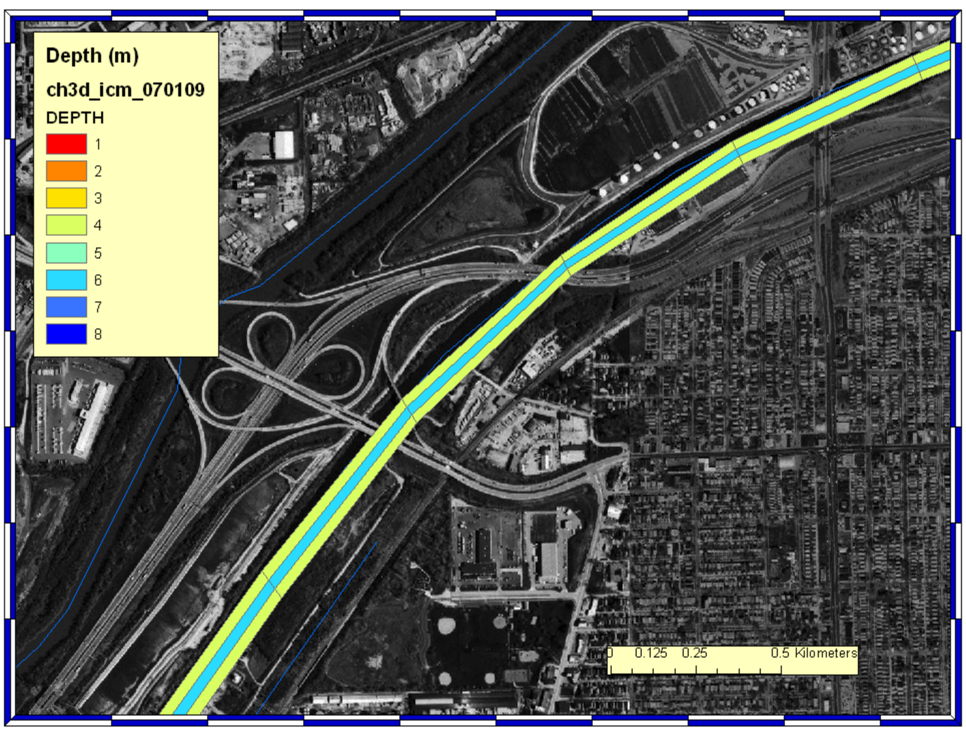
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87°50'0"W


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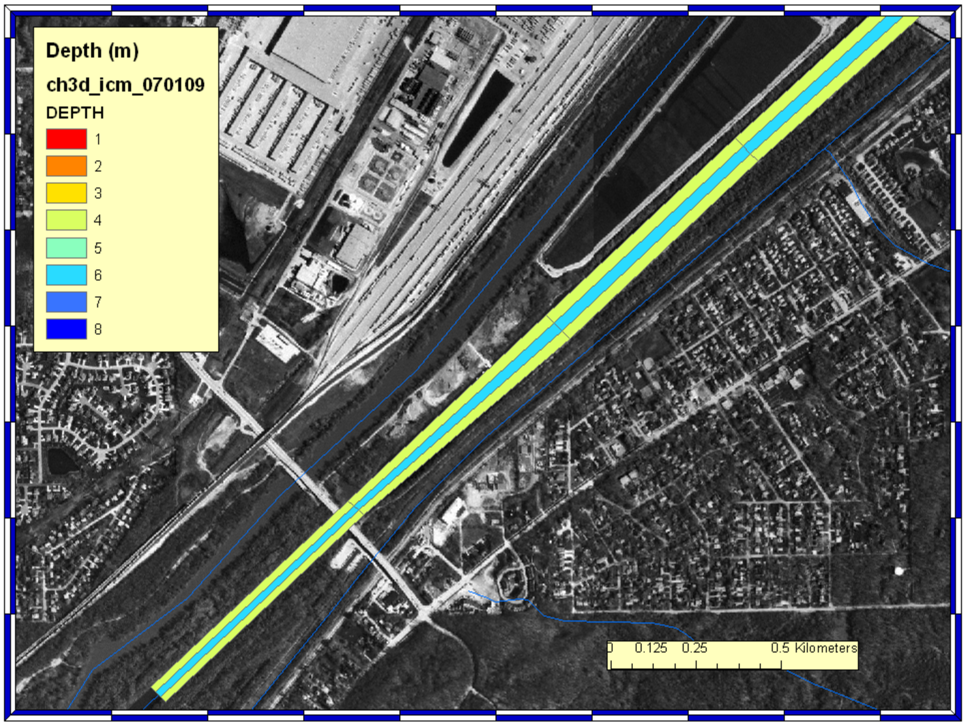
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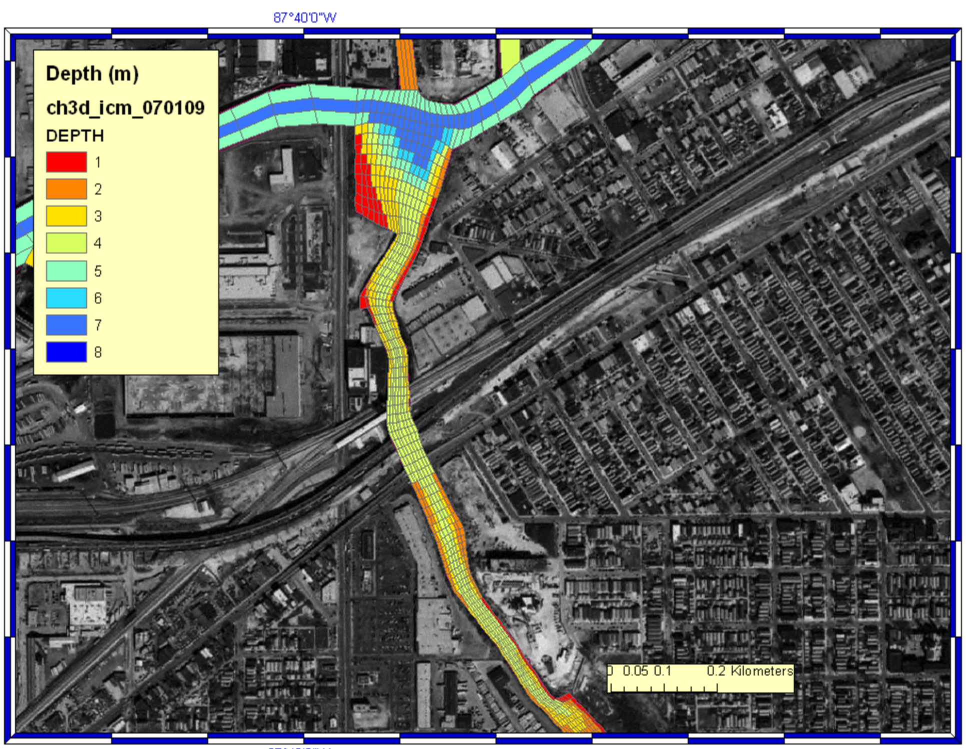
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Attachment 4:

Substrate Restoration Design for Bubbly Creek – ERDC, Environmental Laboratory

Substrate Restoration Design for Bubbly Creek

Paul R. Schroeder, PhD, PE
Environmental Laboratory
US Army Engineer Research and Development Center
15 November 2013

1. Background Sediment Quality. The South Branch of the Chicago River, aka Bubbly Creek, is well known for poor environmental conditions including sediment quality. Efforts are underway implement an environmental restoration project on Bubbly Creek which will restore sediment quality to support aquatic macrophytes and improve habitat. Bubbly Creek's nickname originates from sediment gas production originating in historical deposits of organic matter. This water body served as an outfall for meat packers and other industrial activities in the late 19^{th} and early 20^{th} century. Natural decomposition of these sediments exerts an oxygen demand (SOD) that limits the establishment of fish habitat. In addition, these sediments release gas bubbles (predominantly methane), which suspend fine sediments in the water column increasing turbidity and limit light penetration to support submerged aquatic vegetation. Therefore, the sediments require a coarse-grained substrate to reduce sediment oxygen demand in the rooting zone, to improve oxygen transfer at the sediment surface, and to reduce sediment resuspension and turbidity.

2. Stream Characteristics. The stream characteristics of Bubbly Creek are not representative of a backwater slough because of its historical use for stockyard and meat packing waste disposal and urban drainage. Presently, the flow conditions are artificial without a defined watershed providing substrate for the sediment bed. The flow is created episodically by the Racine Avenue Pumping Station (RAPS), which discharges at a rate of up to 6000 cfs. RAPS was constructed in 1938 and, historically, the pumps discharged runoff about 50 times per year. TARP was put in service as portions were completed starting in 1985 and therefore the pumps presently discharge about 16 times per year. After McCook Reservoir is brought on line in 2017, the discharges from RAPS are likely to decrease to 3 or 4 times per year. The solids loadings from RAPS are predominantly organic solids and fine grit and soils from urban streets at anticipated TSS concentrations of 60 to 200 mg/L (with the low end potentially being more likely if the discharges occur on the waning side of the hydrograph).

3. Hydrodynamic Setting. Bubbly Creek is a stagnant water body off the Chicago River that drains a small urbanized watershed. Most of the runoff from the watershed is captured in the Chicago's combined sewer system that overflows into Bubbly Creek on an irregular and infrequent basis. An added feature of Bubbly Creek is that its upper end serves as the outfall of the Racine Avenue Pump Station (RAPS). RAPS pumps storm water to the Stickney Treatment plant from combined sewers and the Tunnel and Reservoir Plan (TARP). TARP collects waters from storm sewers and combined sewers during peak runoff events to prevent uncontrolled discharges with the intent that they can be treated later. TARP allows diversion, holding, and treatment of the combined sewers thereby decreasing the occurrence of uncontrolled discharges from combined sewers outfalls (CSO). However, under certain conditions RAPS discharges directly to the southern end of Bubbly Creek. The discharge flow rate, up to 7500 cfs, provides significant flushing of Bubbly Creek at stream velocities of up to 7 fps. However, about 95% of the existing sediment bed experiences velocities below 3.5 fps during RAPS discharge events. When pumping ceases, there is little flushing and water soon stagnates, as there is not another significant source of water to Bubbly Creek other than RAPS or CSOs. The high flow during RAPS discharges erodes sands as well as unconsolidated fines and organic matter; therefore, any coarse-grained media placed in Bubbly Creek would need to be armored with gravels or small stones.

4. Existing Sediment Physical Characteristics. The sediment in Bubbly Creek reflects the artificial character of the water body and its historic use. The sediment bed has adapted to current conditions and is stable except for the organic matter on the surface that is resuspended by gas ebullition and RAPS discharges. The surface sediments are sands and gravels in the shallow upstream end of the creek, sandy silts in the fairly shallow reaches immediately below the upstream end and then clays in the deep downstream reach. More specifically, based on cone penetrometer testing, the sediment characteristics vary along the length of the creek based on the creek dimensions. At the RAPS discharge, the sediment consists of sands and gravels, presumably due to the energy of the RAPS discharge and the shallow water depth. Over the next 2000 ft, the sediment bed consists primarily of sandy silts. At this point, there is a constriction in the creek for about 300 ft and the bed surface begins to contain more clay while remaining predominantly silts below the surface. At 3500 ft from the RAPS discharge, there is a further constriction of the creek and the water depth increases from 5 ft to 10 ft. At the constriction, the bed is predominantly clay for 500 ft followed by a 1000 ft of silts and clays. The sediment surface of the last 2000 ft of the creek is predominantly clay. In summary, the sediment bed shows that sands and gravels tend to fall out at the RAPS discharge but are not stable where the channel is constricted. In the constricted areas, the bed consists of erosion resistant, cohesive clays, apparently present before RAPS construction.

5. Substrate Restoration Designs. Two designs are being considered for Bubbly Creek. One substrate restoration design is simply a sand/gravel layered system and the other is a gas venting system. The sand/gravel design addresses all of the sediment quality issues discussed in sediment quality section but provides less effectiveness in controlling sediment oxygen demand and sediment resuspension by gas ebullition. The gas venting design provides nearly complete isolation of the existing sediment and its impacts. Conceptual designs of the two systems are shown in Figure 1.

6. Armor Requirements. Hydrodynamic and granular sediment transport models were run for the worst case flow conditions on Bubbly Creek with existing bathymetry and 22-inch shallower bathymetry (due to cap placement). The model results produced a range of stable particle size as a function of channel conditions (depth, cross-sectional area, position in bends, etc.) as shown in Figure 2. Without dredging, the D_{50} of the armor material would need to be about 50 mm or 2 inches; however, if dredged before substrate placement in order to maintain the same channel conveyance, the D_{50} of the armor material would need to be about 20 mm or 0.8 inches. These sizes are sufficient to prevent any movement; smaller sizes could be used if limited bedload transport were permitted or if the design velocity were decreased to the 95th percentile value evaluated spatially (3.5 fps with dredging or 4.5 fps without dredging), instead of the maximum velocity of up to 7 fps. The required thickness of an armor layer is 1.5 times the D_{100}, which is often about 1.5 times to two times the D_{50} for small stone sizes. Therefore, the minimum required armor thickness is about 6 inches for undredged conditions and 2.5 inches for dredged conditions. However, armor layers should not be placed thinner than about 5 inches due to bottom irregularities and construction limitations in precision.

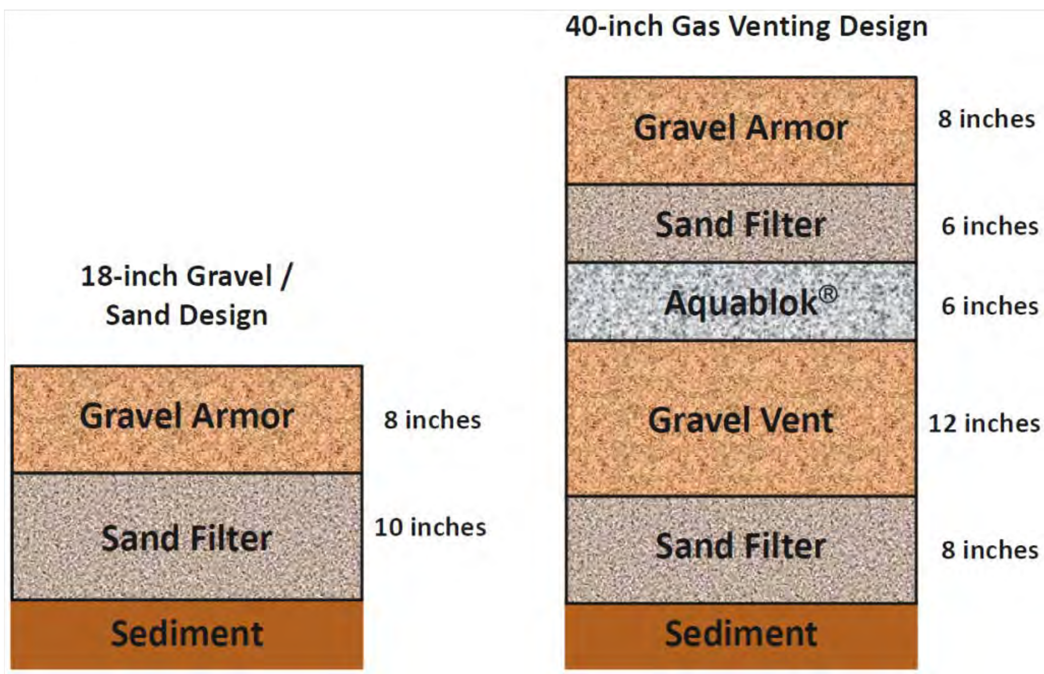
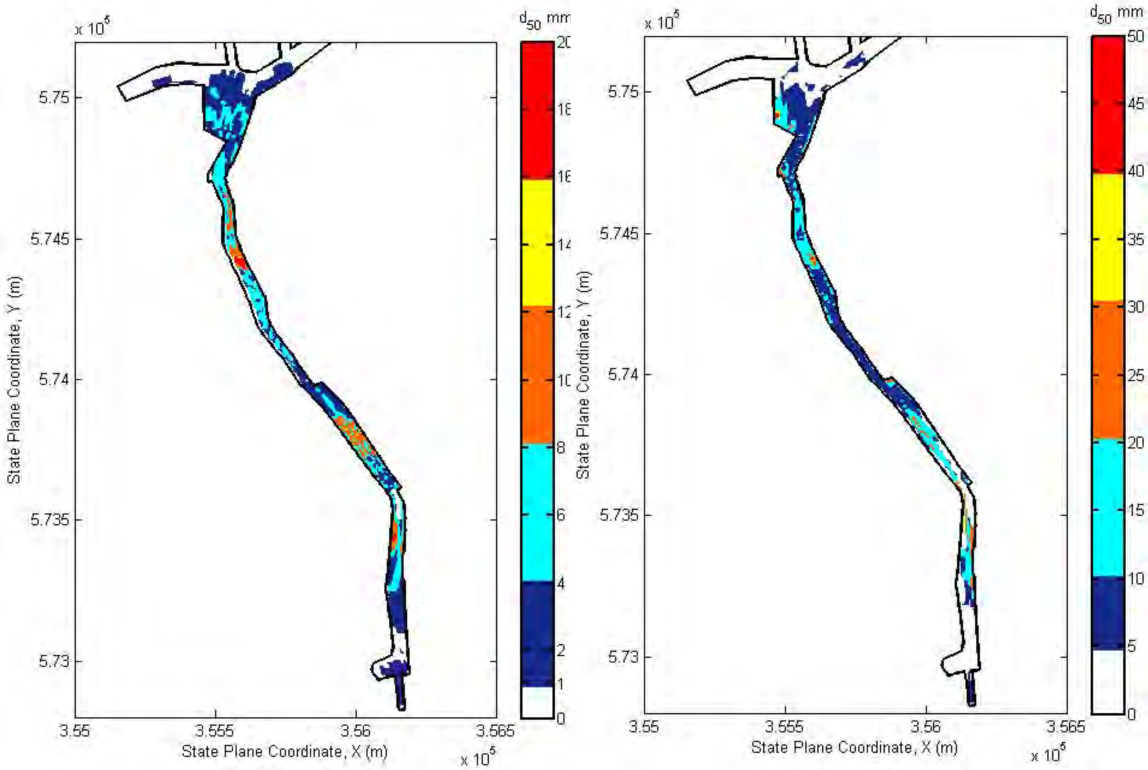


Figure 1. Schematic of two substrate restoration designs.

7. Isolation Requirements. Separation between the existing sediment and the bioactive and rooting zones provide a reduction in transport of sediment oxygen demand. Greater separation reduces the gradient to drive the transport; however, most of the reduction is achieved in the first four to eight inches of isolation. Initially, the contribution of sediment oxygen demand from the capped sediments will be nothing, but will increase over the first 30 years to peak at about 2% of the existing uncapped contribution. Isolation is less effective in highly permeable materials such as gravels and stones and is best achieved with sands, silts and clays.

8. Filter Requirements. Filter material should be sized to satisfy Vicksburg Terzaghi USACE retention criteria, permeability criteria and internal stability criterion. IDOT FA-2 class sand is appropriately sized to act as a filter between the fine-grained sediment and gravel armor material. A sand filter layer should be at least 4 inches thick.

9. Mixing Allowance. Soft organic sediments are soft and easily disturbed by construction activities, which can lead to mixing with the first two inches of placed substrate.



a) Existing grade **b) 22 inches above existing grade**

Figure 2. Stable D_{50} particle sizes under peak RAPS flow conditions.

10. Sand/Gravel Design. The design specifications are given in Table 1. The minimum D_{50} values would satisfy the conditions for limited transport in about 5% of the area under maximum discharge conditions as opposed to being completely stable. Actual RAPS discharges are seldom greater than 75% of the maximum discharges; therefore, the minimum D_{50} values should be acceptable. The target media thickness is 18 inches, 8 inches of coarse gravel underlain by 10 inches of sand as shown in Figure 1. The 10 inches of sand satisfies the 2 inches of mixing, the minimum of 4 inches of filter, and 6 inches of isolation and provides 2 inches for construction tolerance. Typical construction precision for placement of granular layers is 2 to 4 inches. The maximum construction shortage error of 4 inches would still provide the minimum filter and isolation requirements of 4 inches. The isolation component also serves as the filter component of the sand layer. This layer should not be less than 6 inches in thickness to satisfy all functions of the layer. The eight inches of gravel satisfies armor thickness, construction tolerance, bioactive zone and rooting zone. The armor layer should not be less than 6 inches thick for the 2-inch very coarse gravel and 5 inches thick for the 0.8-inch coarse gravel.

Table 1. Sand/Gravel Design Specifications

Material	Thickness (inches)			Size		Purpose
	Minimum	Target	Maximum	D_{100} (mm)	D_{50} (mm)	
Gravel	6	8	10	75 max 40 min	50 max 20 min	Protect against RAPS discharge erosion, provide clean media for rooting and habitat
Sand	8	10	12	IDOT FA-2	IDOT FA-2	Isolate water column from sediment, serve as filter, and provide bedding for cap

11. Constructability of Sand/Gravel Design. The sand/gravel design is readily constructible. A similar design was constructed 18 months ago on Reaches 1 & 2 of the West Branch of the Grand Calumet River. The substrate materials were placed in two layers using a barge mounted spreader. In four months, 1.8 miles of river averaging 150 ft in width was covered. The design consisted of a 6-inch sand base covered with a 12-inch layer of 0.5-inch gravel, constructed in 1000-ft sections. The dimensions of the Bubbly Creek substrate restoration are very similar, about 1.4 miles long and 140 ft wide. Assuming dredging and grading to maintain the existing conveyance for RAPS discharges, the sand/gravel design for Bubbly Creek would consist of a 10-inch sand base covered with an 8-inch layer of 0.8-inch gravel, constructed in short sections starting at the upstream end. The construction should minimize the potential erosion of the sand base from a RAPS event by allowing only a small reach of exposure at any time (perhaps 500-ft reaches).

12. Gas Venting Design. The design specifications are given in Table 2. The target thickness is 40 inches as shown in Figure 1. The top eight inches of gravel satisfies armor thickness, construction tolerance, bioactive zone and rooting zone. The armor layer should not be less than 6 inches thick for the 2-inch very coarse gravel and 5 inches thick for the 0.8-inch coarse gravel. The 6 inches of sand satisfies the minimum of 4 inches of filter and provides 2 inches for construction tolerance. Aquablok® serves as a liner to control gas and groundwater transport to the habitat zones. Six inches of Aquablok® provides enough thickness to create a seal, confining pressure and strength to preserve the integrity of the layer. The combined thickness of the top three layers should not be less than 16 inches to provide sufficient confining pressure or loading on the gas vent layer to counteract the buoyancy of the gas being accumulated and vented beneath the Aquablok® layer. The lower gravel layer is designed to be 12 inches of very coarse gravel to provide sufficient transmissivity for the gas to pass laterally under the mild irregular side slope of the channel. The permeability of the gravel is sufficient to displace the water and transmit gas to a collection and venting system at the maximum rate of gas generation. The bottom 8 inches of sand satisfies 2 inches of bedding and mixing with the sediment and the minimum of 4 inches of filter, and provides 2 inches for construction tolerance.

Table 2. Gas Venting Cap Specifications

Material	**Thickness (inches)**			**Size**		**Purpose**
	Minimum	**Target**	**Maximum**	**D_{100} (mm)**	**D_{50} (mm)**	
Gravel	6	8	10	75 max 40 min	50 max 20 min	Protect against RAPS discharge erosion, provide clean media for rooting and habitat and provide confining pressure
Sand	4	6	8	IDOT FA-2	IDOT FA-2	Serve as filter and provide confining pressure for gas
Aquablok®	4	6	8	NA	NA	Inhibit vertical gas and groundwater migration
Gravel	10	12	14	75 max 40 min	50 max 20 min	Vent gas laterally
Sand	6	8	10	IDOT FA-2	IDOT FA-2	Filter sediment from gas vent and provide bedding for cap

13. Constructability of Gas Venting Design. The gas venting design would present many challenges during construction. Similar designs are not readily available. The construction would require stricter grade controls and more access along the banks to install a gas collection header and flaring system. In addition, the design uses three materials instead of two and five layers instead of two, and the overall design thickness is more than twice as great. Therefore, construction of the gas venting design would likely take at least 12 months to complete. The long construction time presents concerns for potential erosion of materials from a RAPS event. Potential differential subsidence of the soft sediments would also pose a concern for the gas collection header.

14. Summary. Two substrate restoration designs were developed. Both designs provide greatly improved sediment quality for habitat development. The gas venting design couples the sand/gravel design with a liner and vent system that provides the added benefits of controlling gas ebullition and groundwater advection. However, these processes are expected to have only a small effect on post-restoration sediment quality. The vast majority of the improvement and maintenance of sediment quality is achieved by the surficial sand/gravel components of the two designs. The added cost, maintenance and construction difficulties associated with the gas venting design are not warranted considering the ongoing discharges from the RAPS and CSOs. In addition, considering the infrequent and short duration of most RAPS discharge events, the infrequent occurrence of peak discharge rates and the small fraction of the area requiring greater protection, a large pea gravel, coarse river run gravel or piedmont gravel corresponding to the minimum D50 specification for armoring should be able to control substrate transport, particularly when stabilized by roots. Therefore, implementation of the sand/gravel design is recommended.

Attachment 5:

Restored Bubbly Creek DO Levels – ERDC, Environmental Laboratory

Restored Bubbly Creek DO Levels

Purpose

This document presents predictions of dissolved oxygen levels in a restored Bubbly Creek, IL. Assumptions and analytical techniques are documented along with results.

Background

South Branch of the Chicago River, aka Bubbly Creek, is well known for poor environmental conditions including water quality. Efforts are underway implement an environmental restoration project on Bubbly Creek which would increase habitat, decrease sediment loads, and as a side benefit improve water quality. Bubbly Creek's nickname originates from sediment gas production originating in historical deposits of organic matter. This water body served as an outfall for meat packers and other industrial activities in the late 19^{th} and early 20^{th} century. Natural decomposition of these sediments release gas bubbles which suspend fine sediments in the water column. The sediment surface also exerts an oxygen demand (SOD) directly to the water column. The end result is that the combined effects of gas production, sediment resuspension, and SOD result in depressed dissolved oxygen (DO) levels in portions of the system.

An added feature of Bubbly Creek is that its upper end serves as the outfall of the Racine Avenue Pump Station (RAPS). RAPS pumps storm water to the Stickney Treatment plant from combined sewers and the Tunnel and Reservoir Plan (TARP). TARP collects waters from storm sewers and combined sewers during peak runoff events to prevent uncontrolled discharges with the intent that they can be treated later. TARP allows diversion, holding, and treatment of the combined sewers thereby decreasing the occurrence of uncontrolled discharges from combined sewers outfalls (CSO). Under certain conditions RAPS discharges directly to the southern end of Bubbly Creek. The flow rate discharged provides significant flushing to Bubbly Creek. When pumping ceases, there is little flushing and water soon stagnates as there is not another significant source of water to Bubbly Creek other than RAPS or CSO. Without a natural flushing inflow replenishing DO levels, conditions in the water column deteriorate in response to the sediment DO sinks and with water column BOD of the RAPS and CSO discharges.

Environmental restoration intends on attacking the issues of Bubbly Creek from many angles. Among the items proposed are capping the sediments which will decrease the SOD levels greatly and prevent the suspension of organic matter in the water column. Due to the expected high flows during Racine Pump Station operation, this cap will be armored to maintain cap stability. The expected result of this work is that SOD values will decrease from reported level of multiple grams O_2/m^2-day.

One issue to be addressed is the water column oxygen demand. Removal of sediment oxygen demand will not affect the overlying water column demand that originates in the discharges. Specifically, waters discharged from RAPS and CSOs are expected to have elevated BOD levels and upon cessation of pumping, it is conceivable that all waters in Bubbly Creek will have originated from RAPS and CSOs. The purpose of this work is to assess what the quality of this water will be under these conditions.

Dissolved Oxygen Balance Components

Components of the dissolved oxygen balance in Bubbly Creek include SOD, BOD, water column DO, reaeration, and flow. SOD represents a historical unsatisfied oxygen demand, while BOD represents the current water column oxygen demand. Water column DO represents the only source of oxygen available to satisfy the demand and reaeration, the process for adding oxygen to the water column. Flow is included as it is a means of bringing in water with additional dissolved oxygen. Each of these components can under specific conditions be the dominant process in the DO balance. They are discussed below.

SOD

Sediment Oxygen Demand is a major component in the current oxygen dynamics of Bubbly Creek. There are two parts to this SOD. The first is a result of a combination of low flushing and high loadings of organic material in the later 19^{th} and early 20^{th} centuries that resulted in thick deposits of organic material. This portion of the SOD is referred herein as a historic SOD. The second part is the SOD resulting from material depositions occurring after CSO and RAPs discharges.

The historic SOD consists of layers of putrefying material that impacts water column DO in three ways. First, a sediment surface oxygen demand is formed due to oxidation of materials at the surface or in the interstitial waters. Second, anaerobic conditions deeper in the sediments result in formation of methane gas that migrates upward through the sediments till it reaches the surface and escapes to the water column. A portion of this gas is stripped as the bubble passes through the water column resulting in an oxygen demand. Third, escape of gas bubbles to the water column results in suspension of clouds of fine particulate matter that exerts an oxygen demand.

SOD is a continuous oxygen demand especially in systems like Bubbly Creek. Due to the magnitude of the deposits generating the historic SOD, there is no likelihood of it being fully expended in the foreseeable future. Therefore, the only manner to effectively deal with it is to remove its influence from the system. This can be accomplished by removing the material via dredging or leaving in place and capping.

From a modeling stand point, SOD occurs at the bottom of the water column and is represented as a flux rate (mass per area per time). This rate is divided by the height of the water column to apply the effects of SOD over the whole water column.

BOD

Biochemical Oxygen Demand is a measure of the amount of oxygen required to oxidize unspecified material in the water column. BOD is a widely used descriptive term for the amount oxidizable material in water and wastewater. BOD can vary from near zero for pure clean streams to the thousands for readily biodegradable waste streams.

In the case of Bubbly Creek, BOD in the creek will most likely originate from the Racine Avenue Pump Station as it pumps waters from combined sewers and TARP. Waters in TARP are a combination of storm water and Combined Sewer Overflows (CSOs). As storm water tends to have a large volume with a lower BOD, it will dilute the higher BOD of the CSOs and regular sewers. TARP will also serve as a stilling basin where settable BOD may be removed. Primary treatment (settling) in wastewater treatment plants removes up to 35% of the BOD. The possibility of settling in TARP and storm water dilution leads to a reasonable expectation that the BOD of the water discharged to Bubbly Creek would be much lower than raw wastewater and have lower levels of settleable solids. The settleable solids are important, as they will contribute to future SOD if not flushed by a RAPS discharge event.

Following a significant discharge event to Bubbly Creek it is realistic for analysis purposes to assume that all water in Bubbly Creek originated from TARP and RAPS during the preceding event. On this basis it is conservative to assume that the BOD is the same throughout the water column.

Water Column DO

The single largest source of dissolved oxygen in Bubbly Creek after a discharge event is the dissolved oxygen that was in the water discharged from Racine Avenue Pump Station. Just as the water discharged from RAPS has some level of BOD, it will have some level of DO. The issue is the relative level of DO and BOD. Provided the discharge event was significant enough to completely flush Bubbly Creek, a reasonable assumption is that the DO throughout the water column after the event would be uniform.

Reaeration

Reaeration is the process of replenishing the DO levels in the water column. The driving force is the difference between water column DO level and DO saturation. The greater the difference is then the greater the gradient driving the process. The rate that reaeration occurs at is the reaeration rate. Reaeration occurs at the surface of the water column. The rate of reaeration is divided by the depth of the water column to distribute the effects of reaeration over the whole water column.

There is no universal model of reaeration and over time a multitude of models has been developed. They vary from molecular diffusion to those based upon empirical relationships of flow, wind, or water body type. As such, care must be exercised when selecting a reaeration rate, especially for a system such as Bubbly Creek. Too low a reaeration rate can result in artificially low DO levels and too high a reaeration rate results in DO levels near saturation. The problem with both of these cases is that they may not be representative of the actual system.

Flow

Flow is a critical component of the DO balance in that water movement affects reaeration, water column mixing, and exchange. Higher flow rates translate into higher turbulence which causes greater amounts of surface aeration. Turbulence also mixes the water column and diminishes the overall impact of SOD. Finally, flow is the major component of exchange in a riverine type system. Inflowing waters displace waters currently in the system. In the absence of inflow, there is no horizontal transport and little mixing which results in stagnant conditions where the only source of DO is reaeration.

Dissolved Oxygen Balance

The sources and sinks in the DO balance are summed to determine the resulting water column DO. In simplistic terms the relations ship is shown below:

$$DO_{in} + Reaeration + Q_{in}\, C_{in} – (BOD_e + SOD + Q_{out}\, C_{out}) = DO_{new}$$

Where:

DO_{in} = initial water column DO, (g/m^3)

DO_{new} = future water column DO, (g/m^3)

Reaeration = reaeration process, (K_a/h) (DO_{sat} - DO_{in}), (g/(m^3- d))

K_a = Reaeration rate (1/d)

h = water column depth, (m)

DO_{sat} = DO at saturation, (g/m^3)

Q_{in} C_{in} = mass loading of oxygen in water coming into system (g/(m^3- d))

Q_{out} C_{out} = mass loading of oxygen in water leaving system (g/(m^3- d))

BOD_e = fraction of water column BOD exerted during specified time, BOD_e = BOD * 10^{-kt} (g/m^3)

SOD = fraction of sediment oxygen demand exerted, (g/(m^2 –d))

In reality, the above equation must be corrected to account for difference in units, process descriptions, and time. A corrected relationship is shown below:

$$DO_{in} + (Reaeration + Q_{in}\, C_{in})\, dlt – (BOD_e + SOD/h + Q_{out}\, C_{out})\, dlt = DO_{new}$$

Where:

dlt = time interval, (d)

Let DO_{Del} represent the change in Dissolved Oxygen.

$$DO_{Del} = DO_{new} – DO_{in}$$

Also, assume that flow is steady-state and low so that Q_{in} = Q_{out} = 0. Therefore the revised equation is:

$$(Reaeration – BOD – SOD/h)\, dlt = DO_{Del}$$

or

$$(K_a\, (DO_{sat} - DO_{in})/h – BOD_e – SOD/h)\, dlt = DO_{Del}$$

A half-saturation term is added to the BOD_e calculation to incorporate the effects of decreasing DO levels on BOD uptake. The purpose of this term is to throttle the removal of BOD during low DO conditions and prevent its removal when no oxygen is present.

$$((K_a\ (DO_{sat} - DO_{in})/h) - BOD_e\ (DO_{in}/(K_h + DO_{in})) - SOD/h)\ dlt = DO_{Del}$$

Where:

K_h = Half saturation DO concentration for BOD uptake, (g/m^3)

The above relationship has been coded in a spreadsheet for analysis of the various components of the DO balance. Care must be used in this analysis as there is not direct feedback on the individual processes. Using small time steps generates smoother results and illustrates better whether DO conservation is being maintained. A user must exercise caution and realize that this is a screening tool.

Spreadsheet DO Balance Analysis

Users are only required to input limited information for this spreadsheet model consisting of initial water column DO, BOD, SOD, water column depth, reaeration rate, BOD decay rate, and time step. A limited set of the results generated are shown here. Other combinations can be generated if requested. In the following sections the various components of the spreadsheet model are discussed and their relative impact upon DO conditions in Bubbly Creek investigated.

BOD Decay

Figure 1 shows the first order decay of BOD with time based upon the decay rate of 0.08 day^{-1}. This is a reasonable BOD decay rate for settled storm water. The decay rate does not affect the magnitude of the ultimate BOD, only the rate at which it is obtained. Decay rates vary according to the source of the water and the ease with which microorganisms can utilize water the organic material in the water. Typical values are from 0.05 to 0.20 (Linsley et al., 1992; Metcalf and Eddy, 1979). Higher decay rates would remove BOD quicker resulting in a steeper BOD remaining curve. Lower BOD decay rates would result in a slower decrease in BOD and a milder BOD remaining curve. The importance of this is that BOD decrease corresponds directly to DO uptake so higher BOD decay rates correspond to higher levels of DO uptake. BOD decay is also a process over which there is little control. It is true that BOD decay rates are temperature dependent but there is little in terms of process control that can be done to affect these rates. The example in Figure 1 is for an initial BOD of 10 mg/l. By day 6, over 2/3 of the BOD has been removed. This period corresponds to the highest oxygen demand on the water column and the generation of the lowest DO levels. After day 6 remaining BOD is still being removed but at a slower rate than earlier which allows the DO to be replenished by reaeration. This curve would have the same shape if the initial BOD were larger or smaller provided ample dissolved oxygen were available, the only difference would be the values on the vertical axis. This is critical because the higher the initial BOD, the greater the amount of DO required to satisfy the oxygen demand. For example, if the initial BOD were 15 mg/l, then approximately 10 mg/l of BOD would be exerted in the first 6 days requiring 10 mg/l of DO. Such a high DO demand would likely exceed the initial water column DO unless reaeration or flushing were able to add sufficient oxygen to prevent hypoxic conditions.

Reaeration Effect

Figure 2 indicate the results for a series of simulations. All simulations have the same initial DO (5.5 mg/l), no SOD, and the only oxygen demand was an initial BOD of 20 mg/l. Computational depth of 2 m was used for the water column. These values were thought to be representative of what might be expected in a remediated Bubbly Creek after a discharge event. The assumptions are that the creek has been fully filled with water from the RAPS with the characteristics listed earlier. Saturation DO was assumed to be 7.5 mg/l which would be representative of warm weather conditions, 28^{o}C.

The reaeration rates used in Figure 2 represent a range of values that are felt to reasonably reflect conditions of Bubbly Creek. There are many empirical equations for computing reaeration rates which incorporate flow, velocity, or some characteristic of the water body in their assumption or computations. As there is no appreciable velocity in Bubbly Creek under the prescribed conditions, equations incorporating flow were not used. The Liss equation was used to compute a reaeration rate based upon measured wind speed. The details of this equation can be found in Appendix B of the CEQUAL-W2 Manual (Cole and Wells, 2010). The Liss equation is shown below:

$$K_L = 0.156\ W^{0.63}$$

$$K_a = K_L/h$$

Where

K_L = Reaeration coefficient (m/d)

W = mind speed measured at 10 m elevation (m/s)

This form of the Liss equation is valid for wind velocities less than 4.1 m/s. Another form of the equation is used for higher velocities.

To obtain an appropriate wind velocity, the meteorological records for Midway airport were analyzed for summer average conditions for the years 2002-2007. No attempt was made to distinguish between individual events. The result was an average wind speed of 3.84 m/s which resulted in a K_L of 0.364. Instead of using this exact value in computations, values bounding (0.33 and 0.4) it were used due to the overall uncertainty in reaeration rate computations. A third reaeration rate of 0.5 is felt to be unrealistic for Bubbly Creek, but was included in analysis to demonstrate the sensitivity or lack of sensitivity at times in the computations.

In Figure 2, three reaeration rates were used to demonstrate DO sensitivity to this parameter. In the long term there is no difference in this example, as all rates will return the water column to near saturation levels when the BOD is fully utilized. Higher reaeration rates will reach equilibrium more quickly. The differences between the reaeration rate effects are at the beginning of the simulation. The high rate of DO uptake required to satisfy BOD utilization results in precipitous drops in DO. Only the highest reaeration rate is able to supply adequate oxygen to the water column to prevent computations of DO values of 0. When the DO is near 0 mg/l, reaeration is still supplying oxygen to the water column but it is at a rate that cannot match the demand for BOD utilization. The oxygen in the water column is being used as quickly as it can be replaced by reaeration.

SOD Influence

Figure 3 contains the results for simulation performed with both BOD and SOD. Simulations presented in Figure 3 used a lower initial BOD, 10 mg/l, than was used in Figure 2 so as to not obscure the relative processes occurring. In all three cases shown in Figure 3, there were initial drops in DO due to the BOD demand. The greatest drop occurred for the simulation using the lowest reaeration rate (0.33 d^{-1}) while the smallest decrease occurred for the case with the largest reaeration rate (0.5 d^{-1}).

When Figures 2 and 3 are compared, two things are obvious. First, the simulations in Figure 3 reached different DO levels at the end of the simulations. This resulted from the SOD that was used in the simulations shown in Figure 3. The difference represents the imbalance between reaeration and SOD. The second item is that the ultimate DO at the end of the simulation is a function of SOD. Simulations in Figure 2 had higher ultimate DO levels than those shown in Figure 3 even though Figure 3 simulations used lower initial BOD values. The effects of BOD are temporary because once it is utilized, it is gone and does not impact water column DO levels again. However, SOD is continuous and it continues to exert an oxygen demand that depresses DO levels. More simply, BOD can depress DO levels, SOD can keep them depressed.

The value of SOD used in the simulations of Figure 3 was 0.5 g/m^2-d. This is an expected SOD for natural systems, (Cole and Wells, 2010). For a system like Bubbly Creek, SOD values can be much higher. Use of SOD values similar to that expected for BC would result in lower equilibrium DO levels. The simulation results of Figure 3 did not indicate that anoxic conditions were predicted. However, low values were predicted that in some cases were sustained for an extended period of time.

The sensitivity of the model to SOD magnitude is shown in Figure 4. As indicated, increased SOD has sustained effects on DO levels. Values of 2.0 g/m^2-d, which are not uncommon in a system like Bubbly Creek, result in very low sustained DO levels. Higher reaeration levels will increase water column DO as will flushing with waters containing a higher DO. However, the end result is that excessive SOD depresses water column DO.

Figure 4 illustrates the significance of decreasing SOD in improvement of Bubbly Creek water quality. Higher SOD values depress DO in a water column even after the effects of an event have passed. A value of 0.5 g/m^2-d is felt to be a conservative estimate of what is possible for a restored Bubbly Creek.

Figure 5 shows the results for a set of simulations in which the depth of water was 1 m and SOD was 0.5 g/m^2-d. This depth is thought to be representative of the conditions in shallower parts of restored Bubbly Creek and the SOD a conservative estimate of restored conditions. The initial water column conditions are the same as that shown in Figure 2, initial DO = 5.5 mg/l and BOD = 20 mg/l. When comparing Figures 2 and 5, two differences are evident. First, the same BOD will depress DO less in shallower waters because reaeration is distributed over less volume, i.e., shallower depth = lower volume. The second difference is that the simulations in the two figures have different final DO levels. This is due to the difference in the SOD between the simulations in the figures. DO levels predicted in Figure 5 were low for a portion of the simulation but not as low as for the deeper water cases shown in Figure 2. It must be remembered that this is a simplistic assessment and no horizontal transport is being included in these computations.

Figure 6 contains the results for simulations similar to those shown in Figure 5, only with lower BOD initial values of 10 mg/l. SOD is a uniform 0.5 g/m^2-d. As is evident the lower BOD decreases the DO less and results in a smaller sag and a faster return to equilibrium values.

Sensitivity to Initial DO

Earlier results indicated that BOD values of 20 mg/l could result in DO levels below desired levels, Figures 2 and 5. The degree and duration that DO is depressed below desirable levels is a function of initial water column DO. For these figures, initial DO was 5.5 mg/l. Figure 7 indicates DO levels for the same 20 mg/l BOD and differing initial DO levels. Higher initial DO values resulting from turbulent reaeration during pumping are possible. Likewise, lower initial DO values resulting from less turbulent reaeration or depletion of the DO while in the combined sewers or TARP are also possible. Review of 2002-2004 RAPS data indicated that DO samples from RAPS discharges varied from 0 to 9.3 mg/l as shown in Table 1. The mean DO of the measurements was 4.8 mg/l.

Figure 7 illustrates several items. First, when BOD utilization (and DO uptake) exceeds reaeration, water column DO decreases. Higher initial DO values can still be overwhelmed by the magnitude of the DO uptake during BOD utilization. Second, DO replenishment is a function of the difference between water column DO and DO saturation, 7.5 mg/l. In the cases shown in Figure 7 all conditions responded in similar manners. The time difference among the different simulations required for the DO to return to 4.0 mg/l was 2.5 days. Finally, it is unrealistic to expect initial DO along with reaeration to fully satisfy BOD requirements without encountering low water column DO values unless the BOD levels were low.

Sensitivity to BOD Magnitude

BOD magnitude is the ultimate determiner of the water quality that will exist in a rehabilitated Bubbly Creek. As indicated earlier, future SOD is expected to have less of an impact on water column DO. SOD values used in this assessment will depress DO slightly but not to the degree that occurred historically. BOD originating from RAPS and CSO discharges still can deplete water column DO depending upon the discharge BOD magnitude. RAPS and CSO 5-day BOD values vary greatly. A review of 2002-2004 BOD values (Table 1) indicates RAPS discharge events varied from a high of 190 mg/l to a low of 21 mg/l. The mean 5-day BOD was 97 mg/l. Therefore, on this basis, approximately 100 mg/l dissolved oxygen would be required to satisfy the first five days of oxygen demand for water with the mean BOD. The only manner of supplying this amount of oxygen is either dilution with higher DO receiving water or mechanical aeration. The lowest RAPS discharge BOD measure of 21 mg/l would result in DO responses similar to those shown in Figures 2 and 5 depending upon water column depth. Higher RAPS BOD values were observed as shown in Table 1. The important item is the BOD level in water at the end of the RAPS event because this is the water that remains in Bubbly Creek.

Figure 8 shows the variation in DO for different initial BOD values in the shallow water areas where macrophyte plantings would occur along the banks and in the turning basin. A sediment oxygen demand representative of the long-term condition after accumulation of plant matter was used in the analysis as a worst-case condition. A BOD of 10 mg/l would be representative of fairly clean effluent discharged from a secondary wastewater treatment plant. A BOD of 30 mg/l would be representative of wastewater that has had some level of treatment or large stormwater dilution, but is still not “clean” as is indicated in Figure 8. For all BOD cases analyzed, decreases in water column DO ensue, but the magnitude and duration differ; a BOD greater than 40 mg/l could lead to short-term anoxic conditions. While water with 10 mg/l BOD is considered “treated“, it still can depress the DO significantly under the conditions evaluated. Normally when dealing with BOD values of effluent, the effluent is being discharged into receiving water that has significant volume, and through dilution receiving water DO is able to accommodate the BOD uptake requirement without significant DO depletion. In the case of Bubbly Creek, RAPS will fill the entire volume of the creek with water that exhibits a significant BOD without any receiving water dilution.

The unknown factor in this analysis is the issue of the magnitude of the water column BOD. All parameters in this analysis have some degree of variability. Reaeration varies in response to local wind and water surface conditions. Capping is expected to decrease the effect that SOD has on water column DO to the point that reaeration is able to maintain an oxygenated water column. In both reaeration and SOD, rates different than what were used in this analysis will result in a different equilibrium. However, as indicated the system is resilient to variations in these rates.

Summary

Analysis of the DO balance for Bubbly Creek indicates that there is potential for low DO events after storm water discharge from the Racine Avenue Pump Station. Assumptions made in this analysis were that there would be no flow after the event and the only sources of oxygen would be the initial water column DO and that supplied by reaeration. Oxygen demands would be the initial BOD in the water column along with SOD. Initial BOD values were assumed at 10 and 20 mg/l for computational purposes. The 20 mg/l BOD value was felt to be a realistic estimate of the lowest BOD discharged from RAPS at the end of a flood event. Simulations using a suite of BOD values, reaeration rates, initial DO levels, and depths were run to illustrate the sensitivity of the system DO response. In all cases, similar results are observed; just the magnitude and duration of the DO response differ as a result of conditions specified for the simulation. The TARP system may act as a settling basin by removing particulate BOD, thereby decreasing the BOD load.

In this work only initial BOD has the ability to completely deoxygenate the water column. Even the lowest observed BOD values for RAPS discharges would be adequate to result in low DO or anoxic conditions. Data indicates that there is an order of magnitude difference between the minimum and maximum observed RAPS discharge BOD values. Lowering BOD will have the greatest impact on future DO level. This can be accomplished by ceasing RAPS discharge to Bubbly Creek after BOD levels have decreased in the discharge water. This may occur for more significant storms where the water is comprised of more storm water and less combined sewer flow. An alternative is to draw the water into the RAPS, thereby replacing the water in Bubbly Creek with higher DO and lower BOD waters from the Chicago River. The waters drawn in from Bubbly Creek would have to be pumped to Stickney for treatment and discharge.

Based upon the results presented here it is apparent that temporary DO decreases in Bubbly Creek will occur as a result of a discharge event even after restoration. The duration and severity of these events is highly dependent upon issues such as initial water column DO and BOD that are not easily adjusted. Meteorological conditions, temperature and wind speed are constant in these analyses. The sensitivity of these results to changes in those conditions is unknown.

Reducing the SOD has the greatest long term impact on increasing DO levels and also allows the water column to respond better to periodic BOD loadings. Successful placement of a cap on Bubbly Creek sediments should greatly decrease existing SOD levels. The cap would isolate the water column from historic sediment deposits and the SOD they generate. Particulate sedimentation from RAPS events after the cap is installed will fall on the cap where they can be dislodged and flushed out by future RAPS events. Therefore, in a future Bubbly Creek with capped sediments, it is realistically anticipated that SOD will be low. Analysis of conditions for a restored Bubbly Creek in this study used a conservative estimate of SOD of 0.5 g/m^2-day. Lower SOD rates are possible.

Figure 4 summarizes potential differences between the existing and restored Bubbly Creek. Restoration will greatly lower SOD through capping. In Figure 4 it can be seen that, in both cases, BOD representing a RAPS event depresses DO temporarily. Once BOD is satisfied, the system returns to equilibrium. For the restored system, this equilibrium concentration for DO is much greater due to lower SOD. Repeating this analysis with different BOD loads will yield similar results. DO levels will be higher after RAP events when the SOD is lower.

Finally, BOD oxygen depletion is an acute phenomenon as opposed to SOD which is a chronic phenomenon. There is a limited amount of BOD in the water column so once it is utilized and the DO replenished, the system should return to its prior state. This is based upon the assumption that the flow conditions in the restored system prevent the accumulation of new sediment deposits by limiting RAPS discharges to only the occasional high flow events that will erode and flush new sediment deposits, limiting the generation of SOD. SOD will continue to utilize water column DO for extended time without abatement until capped or physically removed.

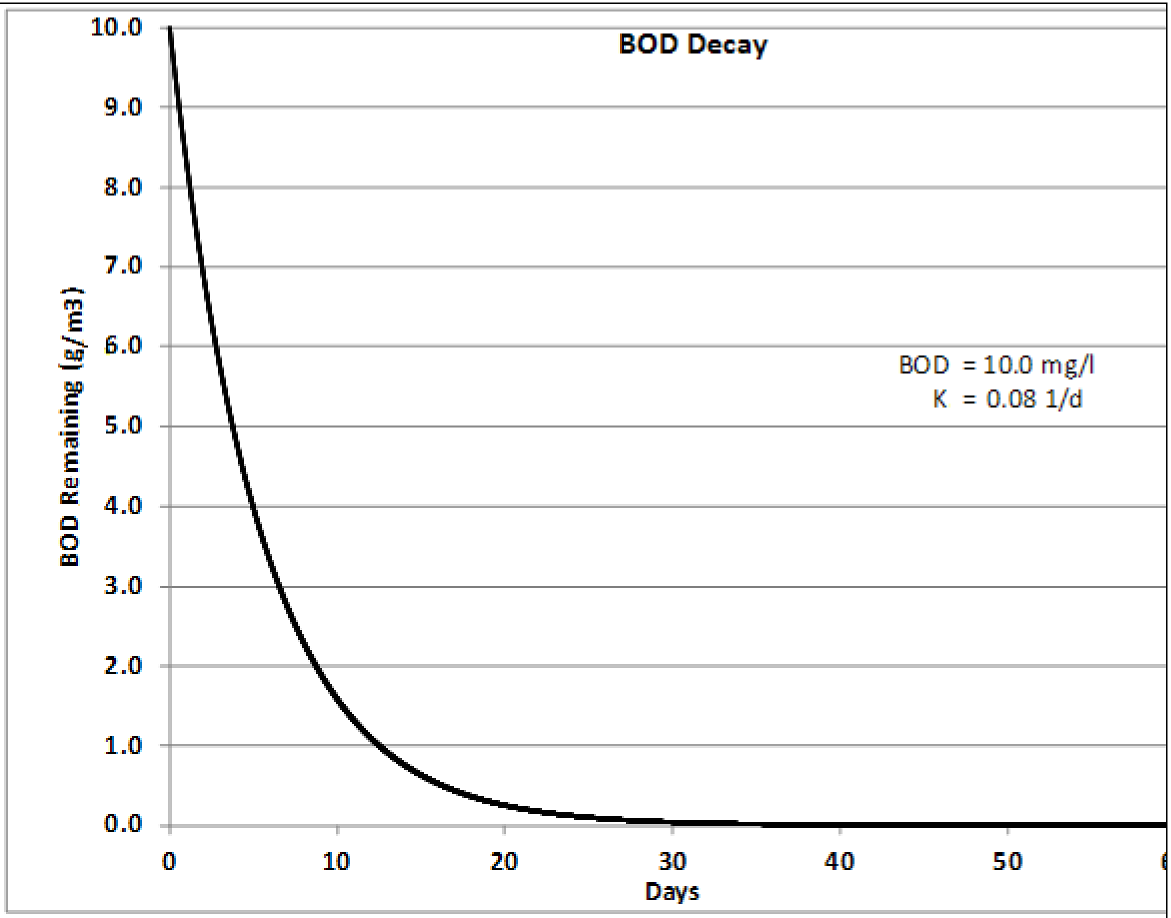


Figure 1. BOD decay over time.

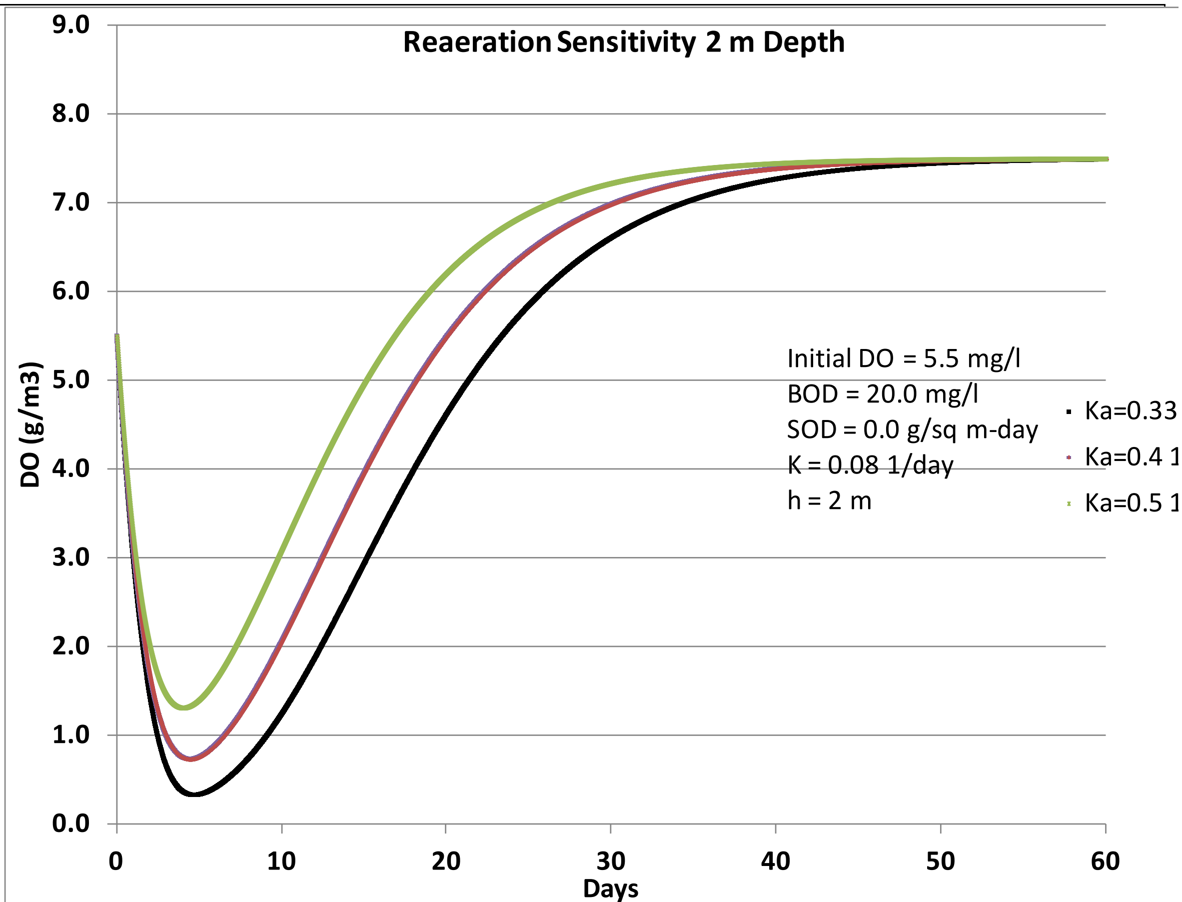


Figure 2. DO-BOD simulation over time for differing reaeration rates with 2 m depth and 20 mg/l BOD.

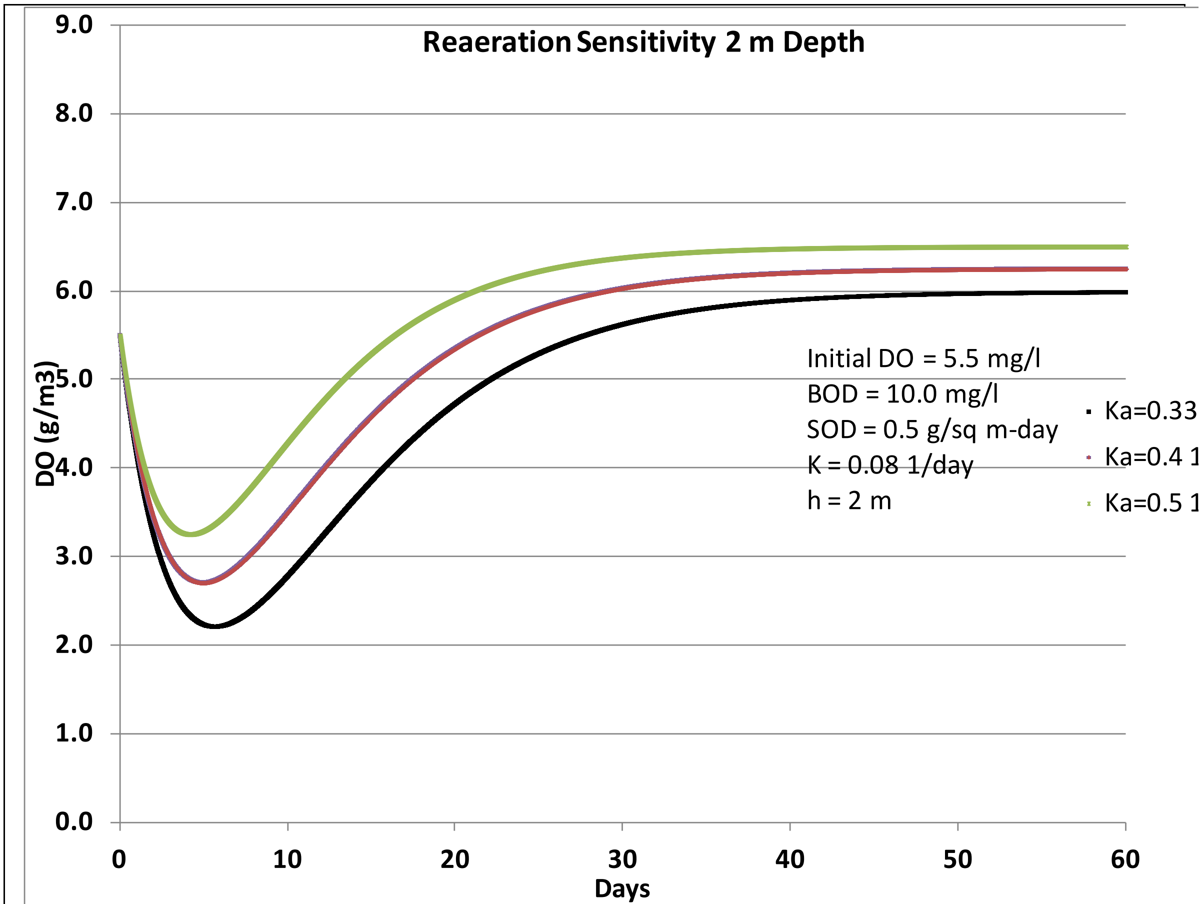


Figure 3. DO-BOD-SOD simulation over time for differing reaeration with 2 m depth and 10 mg/l BOD.

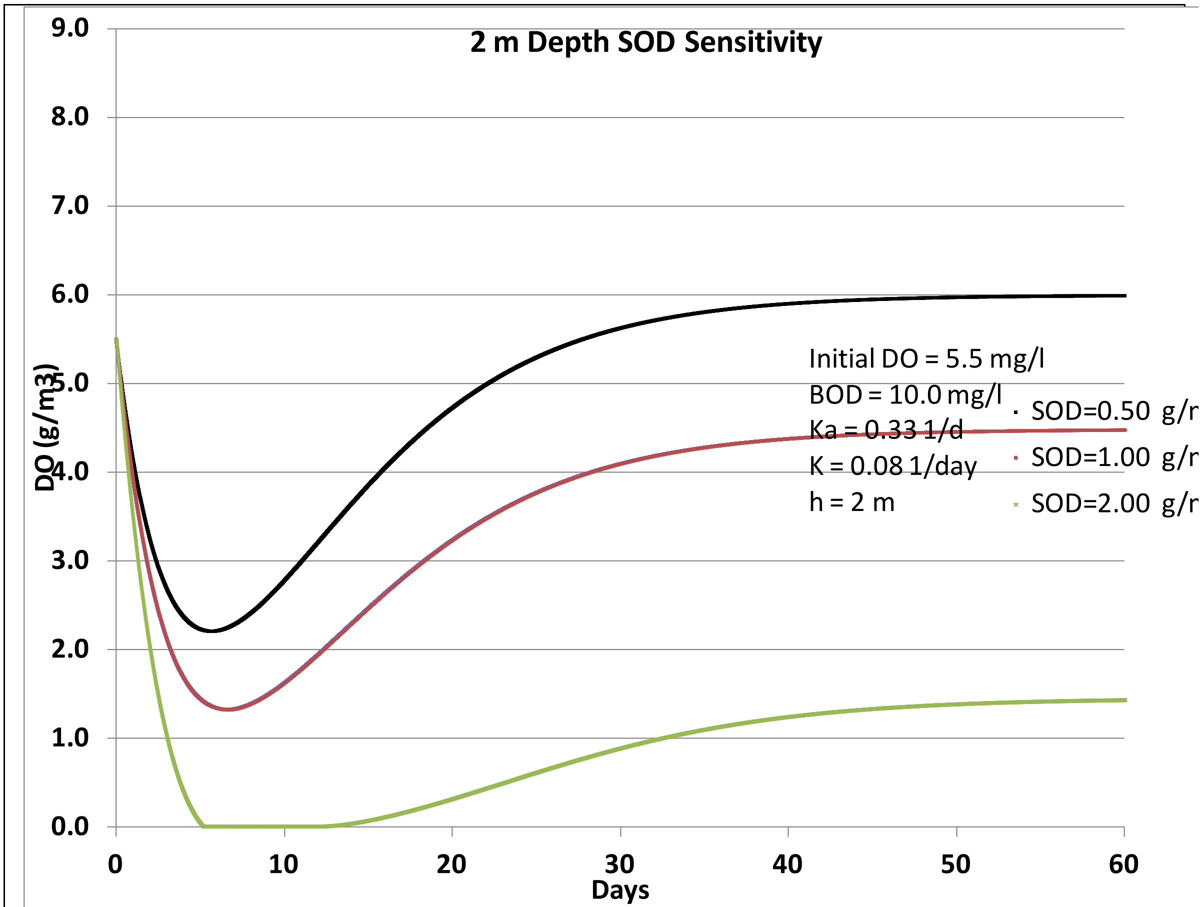


Figure 4. SOD Sensitivity

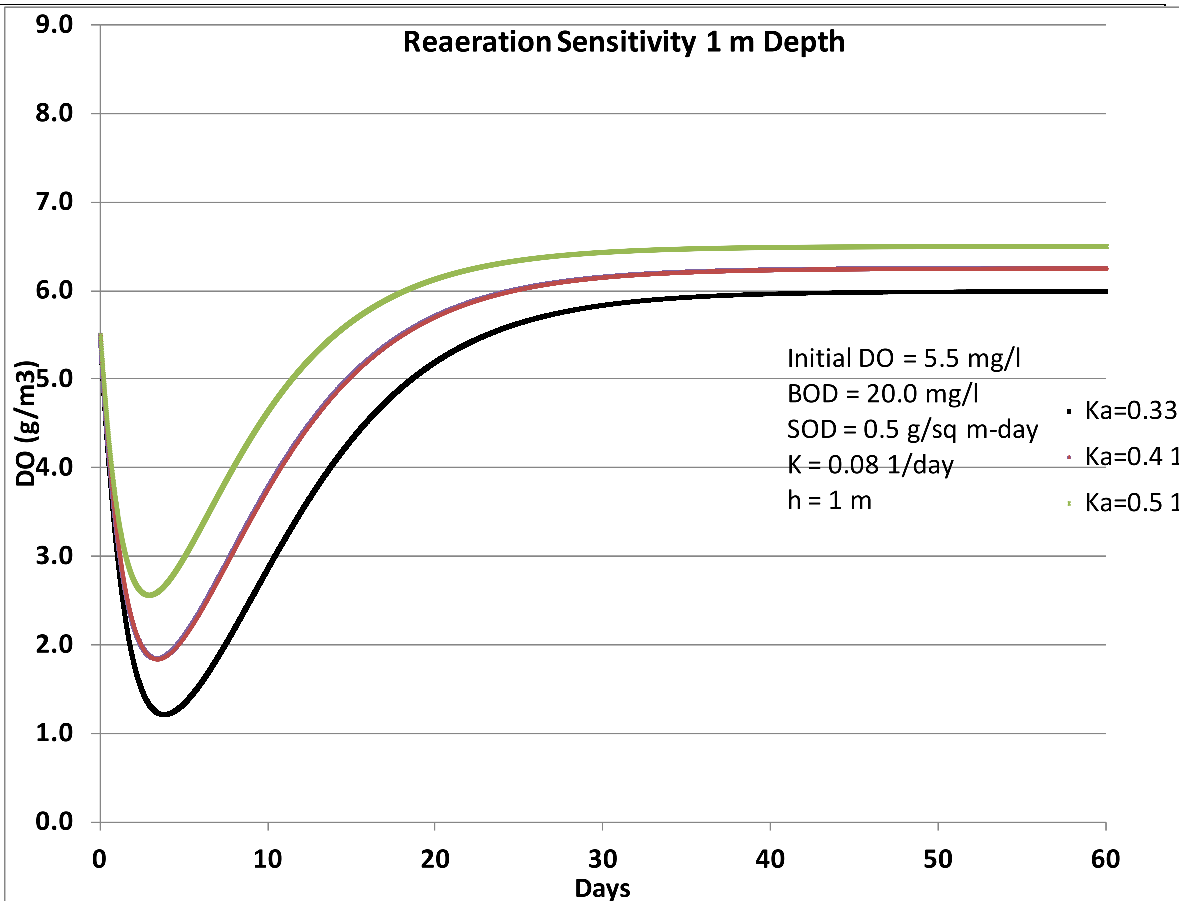


Figure 5. DO-BOD-SOD simulation over time for differing reaeration with 1 m depth and 20 mg/l BOD.

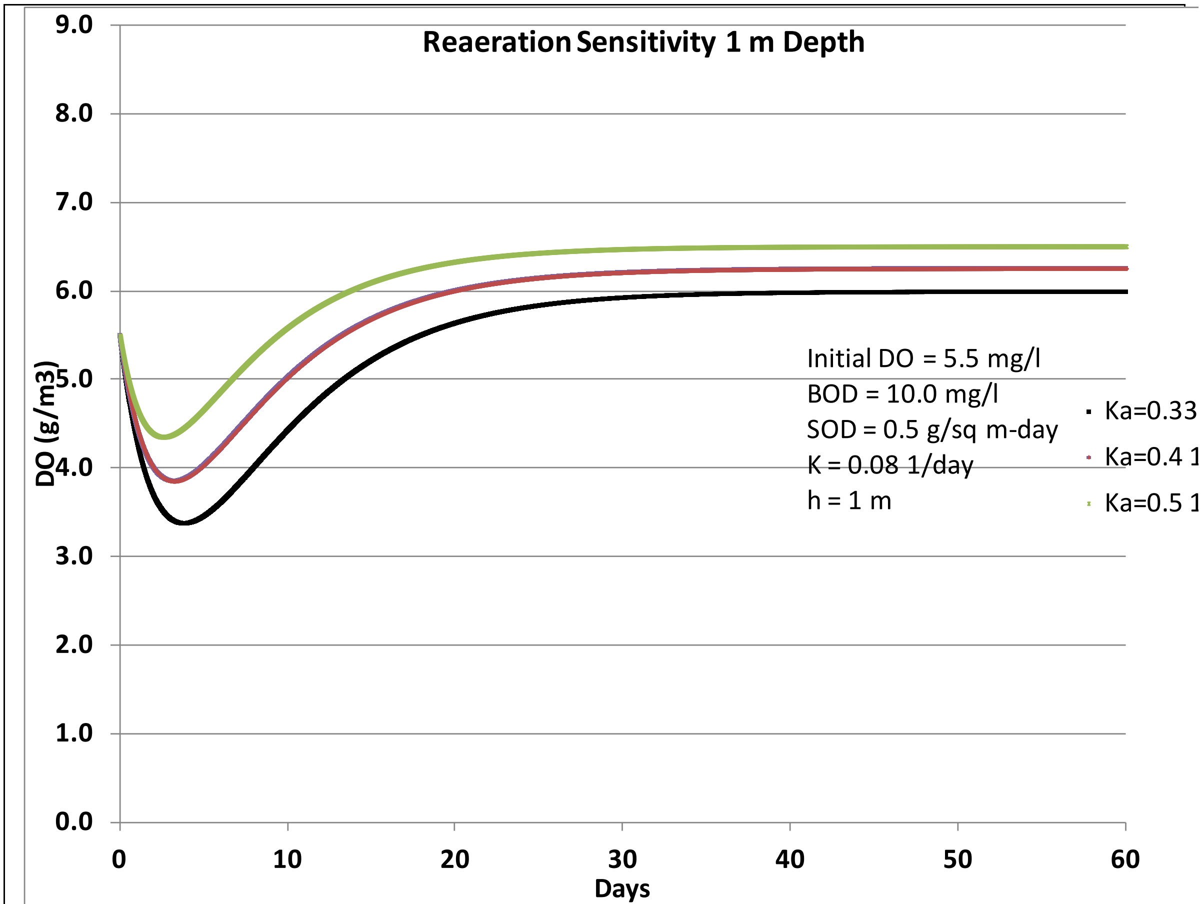


Figure 6. DO-BOD-SOD simulation over time for differing reaeration with 1 m depth and 10 mg/l BOD.

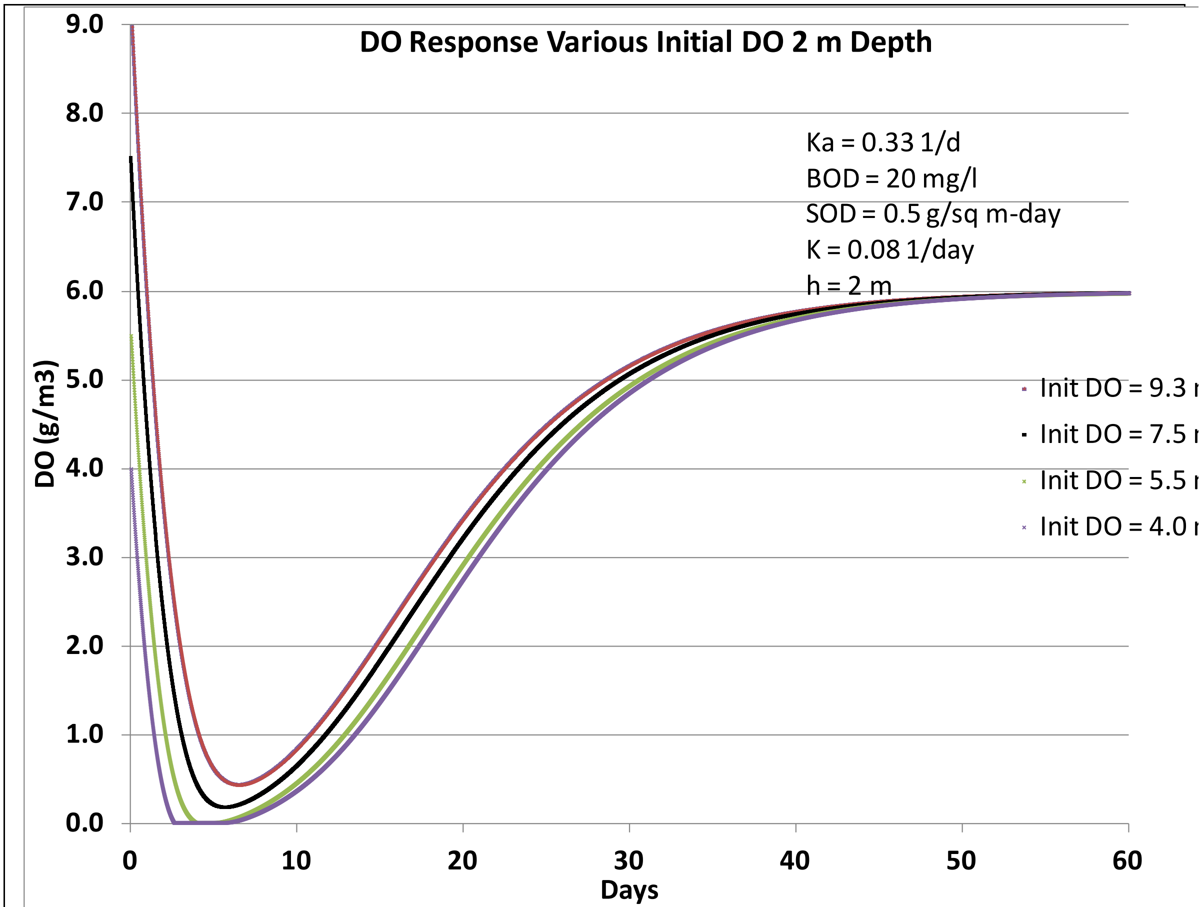


Figure 7. Impact if initial water column DO on sag magnitude and duration.

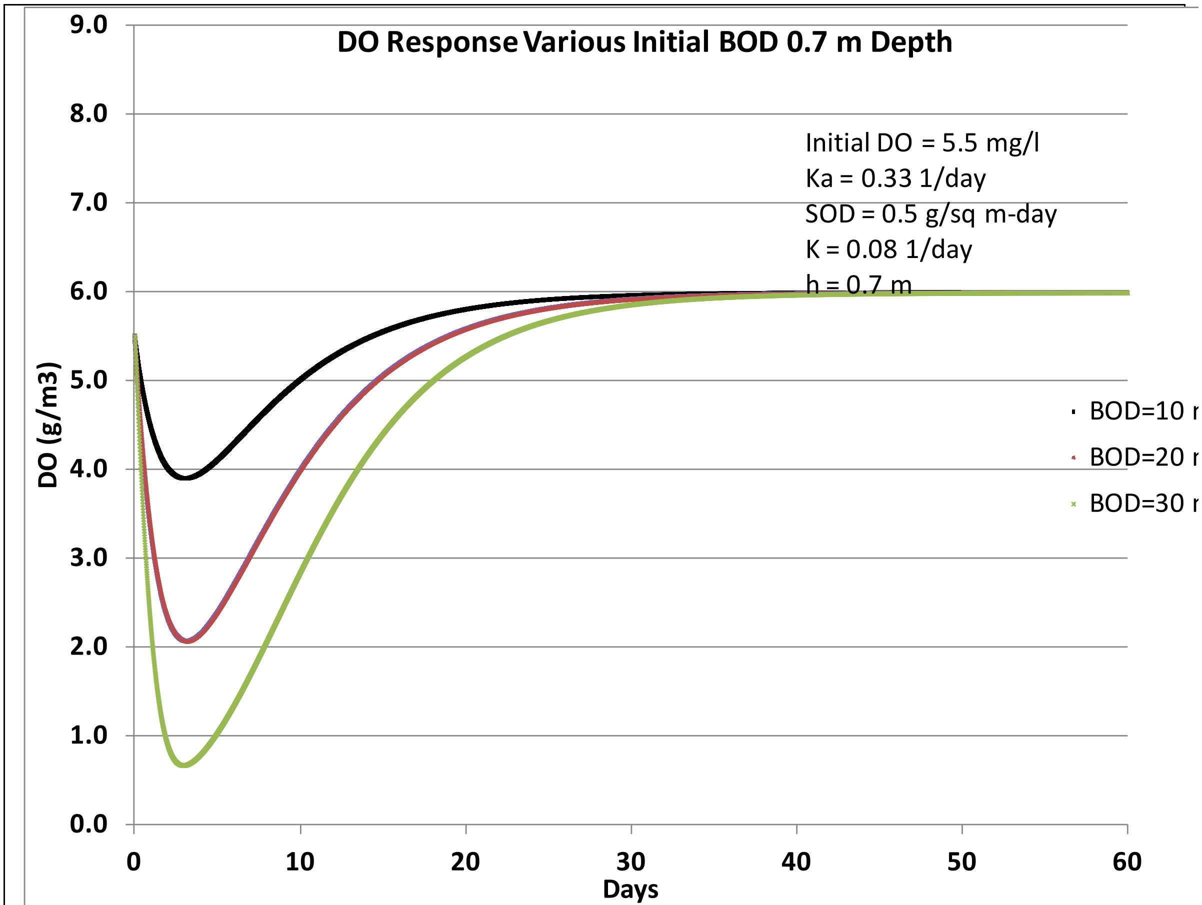


Figure 8. DO response for various water column BOD

	DO	BOD5
Date	mg/L	mg/L
09-Mar-02	2.1	122
09-Apr-02	2.5	83
11-May-02	6.5	62
12-May-02	9.3	49
16-May-02	2.2	105
4-Jun-02	0.0	190
10-Jun-02	0.0	124
9-Jul-02	4.5	95
22-Aug-02	5.8	39
23-Aug-02	8.0	21
04-Oct-02	8.1	N/A
04-Apr-03	3.7	190
1-May-03	8.3	75
4-May-03	0.0	131
9-May-03	5.3	103
11-May-03	2.4	67
15-Jul-03	3.5	126
17-Jul-03	5.0	27
3-Aug-03	8.8	61
14-Oct-03	4.0	157
18-Nov-03	3.8	NA
04-Mar-04	6.1	115
14-May-04	4.7	152
22-May-04	T/X	T/X
31-May-04	4.6	111
10-Jun-04	4.5	103
12-Jun-04	4.0	102
28-Aug-04	7.4	100
28-Aug-04	7.5	92
28-Aug-04	6.7	33
1-Nov-04	5.1	72
4-Nov-04	5.1	114
2002 avg	4.5	89
2003 avg	4.5	104
2004 avg	5.6	99
3yr avg	4.8	97
min	0	21
max	9.3	190

Table 1. Measured DO and BOD values for RAPS events 2002-2004.

References

Cole, T.M., and Wells, S. A. (2010). "CE-QUAL-W2: A two-dimensional, laterally averaged, Hydrodynamic and Water Quality Model, Version 3.7," Department of Civil and Environmental Engineering, Portland State University, Portland, OR.

Linsley, R.K, Franzini, J.B., Freyberg, D.L., and Tchobanoglous, G. (1992). Water Resources Engineering, 4th ed, McGraw Hill.

Metcalf &Eddy, Inc (1979). Wastewater Engineering Treatment/Disposal/Reuse 2nd ed, McGraw Hill.

Attachment 6:

Basement Flooding Impacts from Increased Bubbly Creek Stages

Final

Basement Flooding Impacts from Increased Bubbly Creek Stages

Prepared for

United States Army Corps of Engineers

June 2014

CH2MHILL®

115 South LaSalle

Suite 2400

Chicago, Il 60603

Contents

Tables **Page**

Figures

Acronyms and Abbreviations

CAWs	Chicago Area Waterways
CCD	Chicago City Datum
CDWM	Chicago Department of Water Management
CSO	Combined Sewer Overflow
CTSM	Chicago Trunk Sewer Model
GIS	Geographic Information System
GLMRIS	Great Lakes Mississippi River Interbasin Study
HEC-DSS	Hydrologic Engineering Center Data Storage System
HEC-RAS	Hydrologic Engineering Center River Analysis System
MWRDGC	Metropolitan Water Reclamation District of Greater Chicago
NGVD29	National Geodetic Vertical Datum of 1929
RAPS	Racine Avenue Pump Station
RTC	Real-time Controls
TARP	Tunnel and Reservoir Plan
TNET	TunnelNET
WRP	Water Reclamation Plant
WSEL	Water Surface Elevation
USACE	United States Army Corps of Engineers

SECTION 1

Introduction

1.1 Bubbly Creek Study Background

The United States Army Corps of Engineering (USACE) is performing a feasibility study for the ecosystem restoration of the South Fork of the South Branch of the Chicago River, also known as Bubbly Creek. Bubbly Creek is a 1.25-mile channel beginning near the Racine Avenue Pump Station (RAPS) and flows north into the South Branch. Bubbly Creek is a severely impaired ecosystem due to a history of hydrologic alterations, combined sewer overflows (CSOs), and contaminated sediments.

The USACE's proposed Bubbly Creek ecosystem restoration includes placing new substrate over existing sediments, and the addition of vegetation with a goal of improving the habitat structure of the aquatic ecosystem. The USACE developed a Hydrologic Engineering Center River Analysis System (HEC-RAS) model to determine stage impacts on Bubbly Creek under three scenarios: without project (without new substrate), with a 22-inch substrate restoration, and with the post-settling after substrate placement and following vegetation establishment.

1.2 Study Objective

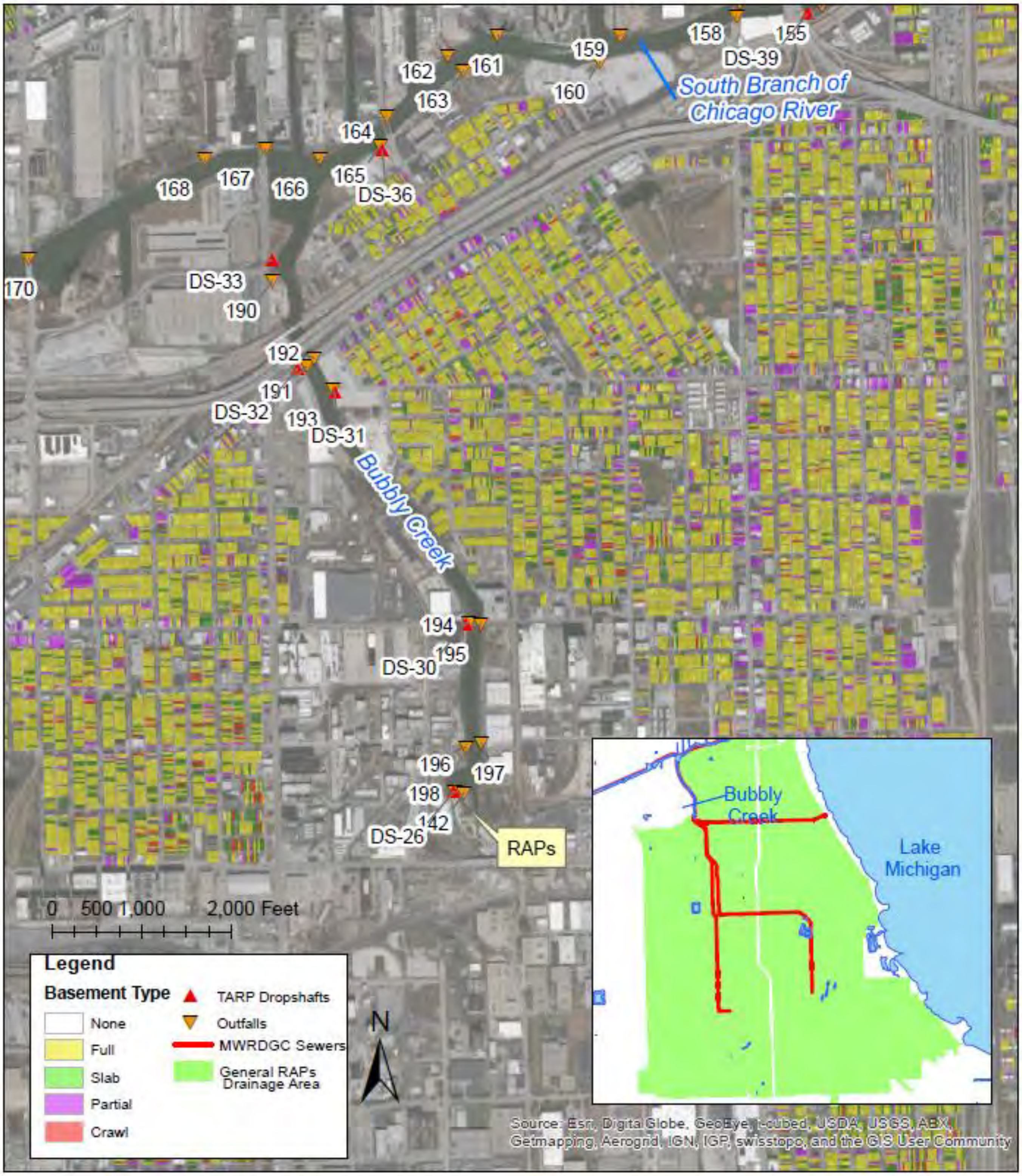
The objective of this study is to quantify the additional risk of basement and/or street flooding resulting from sewer system backups influenced by the increased river stage along Bubbly Creek associated with future project conditions. This study builds upon the existing conditions (2012) Chicago combined trunk sewer model (CDM , 2007) to identify increased basement flood risk in affected areas for a range of storm return periods. The quantitative outputs include both an estimate of the increase in water level and identification of when project conditions contribute to increased basement flood risk (as defined in Section 2.6 below). Areas with increased risk of basement flooding are identified by comparing with-project to without-project basement flooding results and identifying areas at increased risk.

1.3 Study Area Overview

The study area was defined to include the full extent of areas experiencing sewer water level increases due to the proposed project conditions. Because of with-project water level increases at the RAPS, the study area extends far south of the study area, encompassing a significant portion of the RAPS tributary area.

The study area includes parcels with a variety of basement types. Figure 1-1 illustrates basement type data by parcel, based on data from the Cook County Tax Assessor's Office (year 2005).

FIGURE 1-1
Bubbly Creek Location and Basement Type



Note: Parcels without basements are shown as clear.

SECTION 2

Modeling Approach

2.1 Baseline Model

The Chicago Trunk Sewer Model (CTSM), developed by the Chicago Department of Water Management (CDWM), was used as the basis of this study. The CTSM includes all combined sewer pipes 42 inches in diameter and greater, all pipes leading to CSOs, and significant hydraulic features including pump stations and sluice gates to the regional Tunnel and Reservoir Plan (TARP) facilities. The CTSM is constructed using InfoWorks modeling software, and uses EPA-SWMM runoff routing methodology to represent the surface runoff response to rainfall. The CTSM also includes head-discharge relationships for each subcatchment representing the effect of Rainblocker inlet restrictors installed throughout the Chicago service area.

The most up-to-date existing conditions model of Chicago was used for this study, representing sewers constructed through the end of 2012. The Citywide model, including the whole of Chicago (as opposed to regional networks representing specific subsystems) is the CTSM and is described further in the Chicago Trunk Sewer Model Protocol (CDM, 2007).

2.2 Waterway Representations

The CTSM includes level hydrographs at CSO locations to represent water levels in the Chicago Area Waterways (CAWs), which provide tailwater boundary conditions to the sewer system but are not explicitly modeled within the CTSM. Bubbly Creek water levels are the sole difference between the without-project and two project conditions, from the hydraulic modeling perspective, and thus the source of any differences in basement flooding risk for with-project conditions.

The USACE provided level hydrographs (personal correspondence with David Kiel, 7/9/2013) at four locations for each of the modeled simulations (described in Section 2.5). The level hydrographs were assigned to the nearest cross-section along Bubbly Creek. For areas outside of Bubbly Creek, level hydrographs were based upon data previously provided for the USACE Great Lakes Mississippi River Interbasin Study (GLMRIS).

Table 2-1 displays the peak level at two stations (the furthest upstream station and the third station downstream) for the 25-year (yr) 3-hour (hr) storm and the 100-yr 3-hr storm for "Tunnel Only" TARP conditions. Levels are provided at the RAPS, where Bubbly Creek originates, and at CSO-191, located roughly 0.3 mile from the confluence of Bubbly Creek and the South Branch. For all simulations, with-project level increases are greatest at the RAPS and decrease in the downstream direction.

TABLE 2-1

Peak Baseline Level and Incremental Increase for with-Project Conditions at Bubbly Creek Stations under Existing TARP Conditions

	25-yr-3-hr			100-yr-3-hr		
Station	Baseline Level	CAP Increase	STL Increase	Baseline Level	CAP Increase	STL Increase
CSO-191	2.97 ft CCD	0.03 ft	0.03 ft	3.98 ft CCD	0.10 ft	0.11 ft
RAPS	3.01 ft CCD	0.12 ft	0.12 ft	4.33 ft CCD	0.43 ft	0.46 ft

Note: ft = feet; CCD = Chicago City Datum

CAP: With-project condition after placement of 22" of new substrate atop channel bottom

STL: With-project condition after placement of new substrate and 12" of settlement and vegetation establishment.

2.3 TARP Representation

The Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) TARP system receives combined sewer overflows in areas of Cook County served by combined sewer systems. When TARP system relief is available, overflows within the Chicago system are discharged to TARP, not to the CAWs. Increased tailwater levels along Bubbly Creek due to project conditions only affect the Chicago combined sewer system when the TARP system is unavailable (i.e., gated dropshafts are closed due to lack of storage capacity or for other operational reasons). Inflows from the Chicago combined sewers into the TARP system are controlled by sluice gates that control flow to the dropshafts, although numerous dropshafts are not gated.

Two TARP conditions were evaluated for this study. The "existing conditions" TARP condition includes tunnel storage only, and no McCook Reservoir capacity. It is therefore also representative of a future condition when the McCook Reservoir is full, but tunnel storage is available. Secondly, a 2029 Future Condition was evaluated with full McCook Reservoir storage capacity available. The hydraulic modeling approach used to evaluate each condition is described below.

2.3.1 Existing Conditions TARP Modeling Approach

The CTSM model representation of TARP was used to simulate the existing TARP condition. The CTSM model utilizes a storage node as a simplified, volumetric representation of the current Mainstream system tunnel storage (a second node is used to represent the Calumet TARP system, but is not relevant to this study). Real-time Controls (RTC) are used in the model to close TARP gates when the Mainstream TARP storage node reaches a level of 385.38 ft (National Geodetic Vertical Datum of 1929 [NGVD29]). This is the level when 70 percent of tunnel storage is utilized, a threshold chosen based on current MWRDGC operating procedures.

The existing condition TARP representation is a simplified means of representing the existing TARP tunnel capacity that does not include routing effects within the tunnel system. Available TARP storage volume is utilized rapidly during the 25-yr and 100-yr return period events modeled for this study, at which point the TARP system is unavailable to relieve the Chicago sewer system and the CAWs become the effective downstream boundary condition.

2.3.2 Future Conditions TARP Modeling Approach

The USACE used the TunnelNET (TNET) model to represent the TARP system under 2029 conditions as part of the GLMRIS. The TNET model includes a representation of both (1) reservoir storage capacity available at the McCook reservoir and (2) hydraulic routing limitations within the TARP tunnels that convey flows to the McCook Reservoir. The TNET results were used to represent the TARP future condition.

CH2M HILL obtained TNET level outputs at nine locations throughout the Mainstream TARP system. These TNET level hydrographs were used to represent hydraulic levels in specific reaches of TARP, which were applied as level boundary conditions to specified outfall nodes in the modeled system. All modeled sluice gates representing control gates to TARP along a given reach were linked to the boundary node, and sluice gates were closed when the level at the boundary node reached 549.48 ft (NGVD29), indicating that TARP is full in that part of the system (CH2M HILL, 2013).

As expected, the TARP system provides significantly more relief under the future 2029 conditions when the McCook Reservoir is online.

2.4 Precipitation

Design rainfall events for two recurrence intervals and two storm durations were analyzed as part of this study. Rainfall depths from Bulletin 71 (Huff and Angel, 1992) were used for this study, as summarized in

Table 2-2. The beginning of the modeled rainfall event was set to the same time as the HEC-RAS model for Bubbly Creek.

The rainfall depth was distributed according to Huff quartile distributions based on storm duration, as described in Bulletin 71. Areal reduction factors, which represent the reduced probability of uniform rainfall across a large area, were retained from the CTSM for this analysis.

TABLE 2-2
Bulletin 71 Rainfall Depths

Annual Exceedance Probability	3-hr	12-hr
4% (25-yr)	3.53 in.	4.79 in.
1% (100-yr)	4.85 in.	6.59 in.

2.5 Modeling Simulations

The USACE defined a range of simulation conditions to test potential impacts of the Bubbly Creek project on potential basement flood risk under a range of conditions. The following conditions were defined:

- Project Condition: Baseline (BAS), with a 22-inches of new substrate (CAP), and with new substrate following 12 inches of settling and vegetation establishment (STL)
- Return Period: 25-yr and 100-yr
- Storm Duration: 3-hr and 12-hr duration storms
- TARP Condition: Existing (tunnel-only) and future TARP conditions

These simulation conditions resulted in 24 unique scenarios, listed in Table 2-3. Based on discussions with the USACE, the 25-yr return period is most likely to influence the determination of impacts on Chicago residents. Therefore, the 25-yr simulation with the greatest impact (E_3_CAP_25) was used to summarize results.

TABLE 2-3
Scenarios Evaluated for Basement Flooding Impacts

		TARP Existing		TARP Future	
Channel Geometry	**Storm**	**3-hr**	**12-hr**	**3-hr**	**12-hr**
No New Substrate (Baseline)	25-yr	E_3_BAS_25	E_12_BAS_25	F_3_BAS_25	F_12_BAS_25
	100-yr	E_3_BAS_100	E_12_BAS_100	F_3_BAS_100	F_12_BAS_100
With New Substrate	25-yr	E_3_CAP_25	E_12_ CAP _25	F_3_ CAP _25	F_12_ CAP _25
	100-yr	E_3_ CAP _100	E_12_ CAP _100	F_3_ CAP _100	F_12_ CAP _100
With New Substrate and Full Settlement	25-yr	E_3_STL_25	E_12_ STL _25	F_3_ STL _25	F_12_ STL _25
	100-yr	E_3_ STL _100	E_12_ STL _100	F_3_ STL _100	F_12_ STL _100

2.6 Performance Definition

Model results for the scenarios shown in Table 2-3 identify the increase in peak water levels throughout the Chicago system under alternative conditions. Basement flood risk was defined, consistent with CDWM modeling practices, when the modeled water level in the sewer exceeds the following thresholds:

- 6 ft below ground for trunk sewers 36 inches in diameter and greater
- 4 ft below ground for trunk sewers less than 36 inches in diameter
- Top of the pipe crown, when it is higher than the thresholds defined above

The basement flood risk threshold is an approximation defined at the subcatchment scale (e.g., 4-block area, roughly 20 acres in size). A variety of factors may contribute to basement flooding risk, including the intensity and duration of rainfall events, insufficient trunk sewer capacity, insufficient local sewer capacity, private lateral condition, downstream effects of TARP management, and the presence of backflow prevention devices on private property. Within a subcatchment, variation in the level of properties could lead to differences in flood risk not represented in this metric. Thus the basement flood risk metric should be interpreted as an indication that flood risk may exist for properties within a given subcatchment, rather than as a statement that all structures within a subcatchment would definitively experience basement flooding for a specific storm event.

Areas of increased basement flood risk were identified by comparing a subcatchment's flood risk under alternative conditions to baseline (without-project) conditions. Areas with flood risk under with-project conditions, but not under baseline conditions, were of particular interest, as damages incurred in going from no basement flooding to even a small amount of basement flooding are expected to be far greater than the incremental damages associated with small increases in flooding depth.

SECTION 3

Results

Tables summarizing peak flood levels and an indicator of flooding or street flooding for each of the 24 modeled scenarios is included in Appendix A. Table 3-1 summarizes the baseline area at flood risk, while Table 3-2 summarizes the incremental increase in flooded area due to the two Bubbly Creek bottom channel configurations.

TABLE 3-1
Baseline Total Area at Risk for Flooding (No New Substrate Alt)

Event	TARP Condition	
	Existing	Future
25-yr 3-hr	98,240 ac	90,849 ac
25-yr 12-hr	74,610 ac	58,049 ac
100-yr 3-hr	107,259 ac	102,581 ac
100-yr 12-hr	95,961 ac	90,307 ac

TABLE 3-2
Incremental Area at Risk for Flooding for Alternative Bubbly Creek Channel Bottom Configurations

Event	Alternative Channel Bottom Configuration	TARP Condition	
		Existing	Future
25-yr 3-hr	With New Substrate	127 ac	0 ac
	With New Substrate & Settling	121 ac	0 ac[a]
25-yr 12-hr	With New Substrate	92 ac	0 ac
	With New Substrate & Settling	92 ac	0 ac
100-yr 3-hr	With New Substrate	49 ac[a]	0 ac
	With New Substrate & Settling	31 ac[a]	0 ac
100-yr 12-hr	With New Substrate	161 ac	54 ac
	With New Substrate & Settling	203 ac	54 ac

[a] Results summary was modified to remove one or more subcatchments deemed to flood solely due to numerical anomaly instead of increased tailwater conditions along Bubbly Creek.

Table 3-3 and Figure 3-1 illustrate the subcatchments that are at increased flood risk with the proposed new substrate, but are not at flood risk under baseline conditions for the 25-yr, 3-hr event under existing TARP conditions. The table includes the number of parcels with basements in the subcatchments that are flooded due to increased levels resulting from the proposed new substrate. Refer to Appendix A for a complete list of subcatchments at an increased basement flooding risk for each model simulation.

In general, the duration of flooding at each of the subcatchments with an increased risk of flooding is short. For example, the subcatchment with the largest incremental rise listed in Table 3-3 (3C0173) has a water surface elevation (WSEL) above the flood level for a 5-minute time span.

TABLE 3-3
Incremental Flood Risk Summary for Scenario E_3_CAP_25

Subcatchment	Contributing Area (ac)	Max WSEL (ft CCD)	Flood Level (ft CCD)	Incremental Rise (ft CCD)	Impacted Parcel Basement Type
3A0483	29.4	4.09	4.05	0.05	6 Full, 1 Partial
3B0172	2.7	6.57	6.50	0.10	Parcels with No Basements
3B0233	5.0	6.12	6.07	0.09	24 Full, 25 Slab, 1 Partial, 1 Crawl
3C0173	10.9	9.17	9.00	0.31	Parcels with No Basements
3C0310	8.8	7.04	7.00	0.25	5 Full, 1 Slab
3C0433	21.8	8.15	8.04	0.18	44 Full, 9 Slab, 12 Partial, 6 Crawl
3C0443	5.7	8.15	8.05	0.18	Parcels with No Basements
3C0561	5.7	6.94	6.90	0..30	Parcels with No Basements
3C0679	9.7	12.41	12.30	0.28	Parcels with No Basements
3C0931	16.7	9.99	9.90	0.12	Parcels with No Basements
3C0986	10.5	10	9.96	0.12	Parcels with No Basements

SECTION 4

Discussion

4.1 General Summary of Findings

The 24 modeled scenarios result in a significant range of system responses, varying based upon the degree and extent of baseline condition flood risk, the modeled storm duration and intensity, the amount of Bubbly Creek level rise associated with each scenario, and the availability of the TARP system during the storm. While the variability of impacts is considerable due to the range of variables involved, several key points are useful for interpreting the results of this analysis. First, the CAP and STL conditions resulted in similar impacts, due to similar stage increases along Bubbly Creek (the increase in the channel's roughness due to vegetation after full settlement lessens the effect of the 12-inch settlement). Unless otherwise noted, the CAP scenario is generally referenced in this report to represent with-project conditions, channel with new substrate. Secondly, the existing TARP condition, representing tunnel storage volume only, results in the greatest increase in basement flood risk. The modeled behavior of TARP is described further below. Finally, the 25-yr 3-hr and 100-yr 12-hr storms result in the greatest amount of incremental basement flood risk (although the 100-yr 3-hr storm results in greater overall basement flood risk, the 100-yr 12-hr storm results in greater additional flood risk). Based on discussions with the USACE, the higher frequency 25-yr return period storm event is more important for assessing the need for potential mitigation and/or compensation of property owners experiencing basement flood risk due to the Bubbly Creek project.

Modeling results indicate that only under worst-case conditions, a small group of homes that previously was not subject to basement flooding experience minor impacts if the gravity discharge exists. The risk associated with this scenario is significantly less than 4% in any given year (25-year storm). These results were based on modeling of the 25-year storm and assuming the McCook reservoir is full and unavailable for additional storage of stormwater at the start of an event. Modeling of the TARP system shows that the reservoir is expected to be full or near full (50 feet from top of pool) about 0.5% of the time. As such, the minimal basement impacts under this worst-case scenario would have a coincident frequency of 0.02% or 1/5000 chance of occurring in any given year.

4.2 Area of Incremental Basement Flood Risk

While areas tributary to the other outfalls along Bubbly Creek also experience some impacts due to the proposed project, the RAPS is the outlet for a much larger drainage area. Thus the primary cause of increased basement flooding risk is the impact of increased levels at the RAPS (at the upstream extent of Bubbly Creek, where level rise is greatest). As shown in Table 3-3, many areas that technically exceed the basement flood risk threshold do not have basements (based on 2005 Cook County Tax Assessor's data), and thus would not experience adverse impacts due to incremental rise in sewer levels.

For all existing TARP conditions, the WSEL at the RAPS wet well rises from -19 ft CCD to almost 2 ft CCD very rapidly (less than 10 minutes). Table 4-2 displays the maximum increase in WSEL at the RAPS outfall to Bubbly Creek and at the wet well due to the proposed new substrate. Figures 4-1 and 4-2 display the level hydrograph at the RAPS wet well under baseline and with-project conditions for the 3-hr and 12-hr storm duration, respectively. In comparison to the level rise at the RAPS, the incremental increase due to the proposed new substrate is very small. However, this minor increase in level propagates upstream, leading to additional areas under flood risk.

TABLE 4-2
Incremental Max WSEL at the RAPS

Scenario	Incremental Max WSEL at RAPS Outfall (ft CCD)	Incremental Max WSEL at RAPS Wet Well (ft CCD)
E_3_CAP_25	0.12	0.25
E_3_CAP_100	0.43	0.37
E_12_CAP_25	0.19	0.06
E_12_CAP_100	0.50	0.42

The incremental level increase at the RAPS outfall is tabulated at the peak of the event. However, during the rising limb of the hydrograph, the with-project increase is greater than at the peak of the event. Due to timing effects, this greater level rise (roughly ¼ ft) is what contributes most to upstream level increases.

FIGURE 4-1
RAPS Wet Well Level Hydrograph for TARP Existing Condition, 3-hour Duration Scenarios

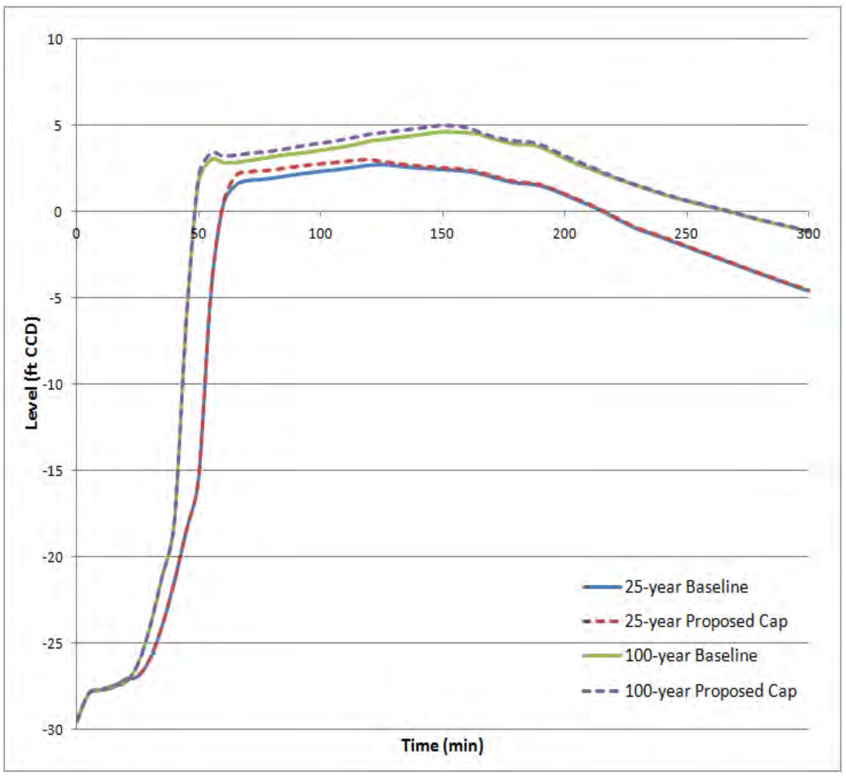
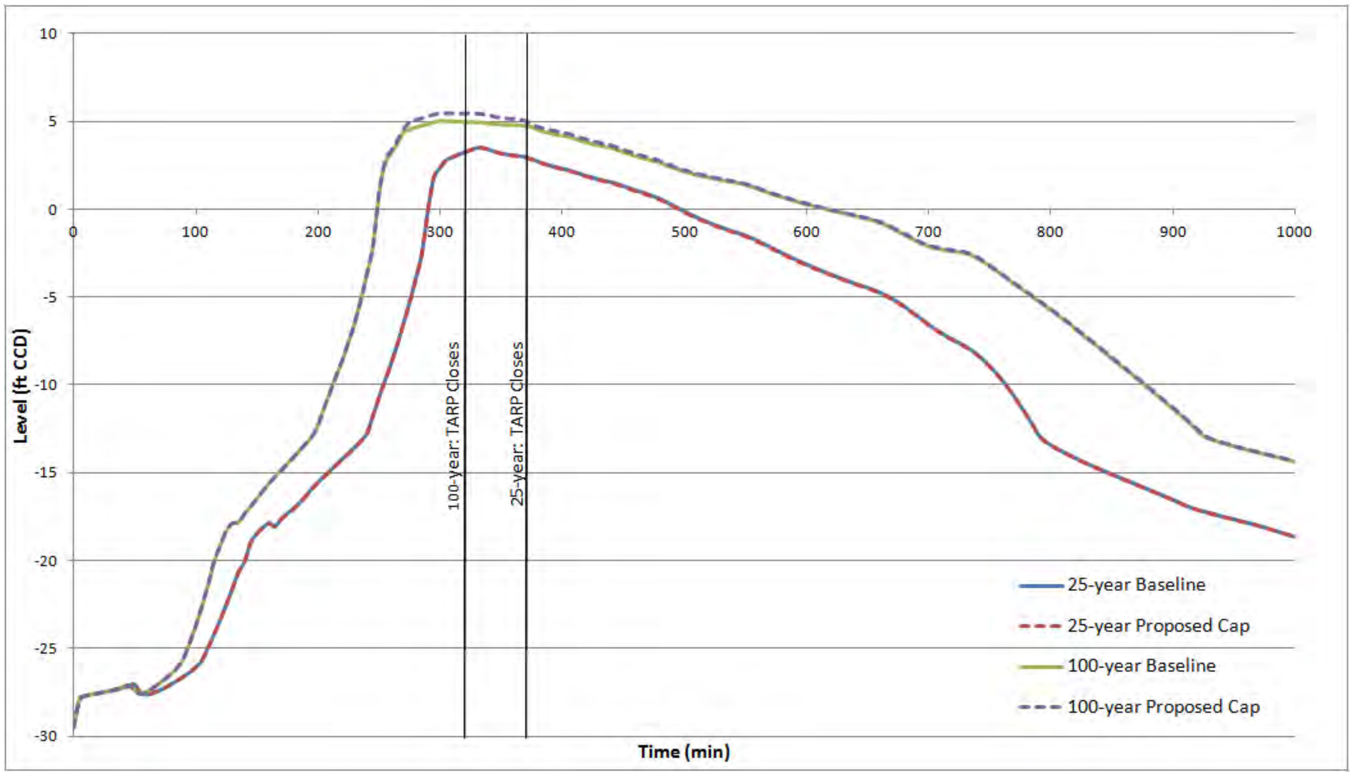


FIGURE 4-2
RAPS Wet Well Level Hydrograph for TARP Existing Condition, 12-hour Duration Scenarios



4.3 Behavior of TARP System

The TARP system performance is a critical component of the overall wet-weather response for the combined sewer portion of the modeled system. When TARP is available, areas served by combined sewers overflow to the TARP tunnel, which provides a downstream outlet, rather than the CAWs. The TARP system, therefore, isolates the sewer system from the impact of increased river levels (assuming that tide gates are present to prevent reverse flow into the sewers from the river). Table 4-1 displays the time during the simulation that TARP is no longer available and the sluice gates close. As described in Section 2.3.2, under future conditions TARP becomes unavailable at different times depending on the reach. Table 4-1 displays the closing time for the TARP Bubbly Creek boundary node.

Since TARP closes early into the simulation for both 25-yr and 100-yr storms under existing TARP conditions, the increased level at Bubbly Creek due to the proposed new substrate translates into increased additional risk of basement flooding in the system. For future conditions, the increased TARP capacity reduces or eliminates incremental basement flooding risk associated with the level increase due to the proposed new substrate. Even though the TARP system does become unavailable during the storm event, this generally occurs after the peak of the event within the sewer system and thus does not contribute to incremental basement flood risk except for the 100-yr 12-hr event. As shown in Table 3-2, under future TARP conditions for all storm events except the 100-yr 12-hr storm, no additional areas experience flooding with the proposed new substrate.

TABLE 4-1
TARP Closing Time

	TARP Condition	
Event	**Existing**	**Future[a]**
25yr-3hr	44 min	198 min
25yr-12hr	160 min	373 min

100yr-3hr	38 min	198 min
100yr-12hr	131 min	321 min

[a]At the Bubbly Creek boundary node

TARP is an operated system, and future operations may differ from those simulated in the TNET model used to represent future TARP performance. Depending on how the MWRDGC operates the TARP gates, which may be conservatively to prevent geysering or other adverse impacts, it is possible that the future TARP system will not provide as much basement flood risk reduction as represented in the TNET simulation.

4.4 Interpretation of Incremental Flood Risk

Many subcatchments in the area surrounding Bubbly Creek are at risk for basement flooding under baseline conditions (see Figures 3-1 and 3-2). The increase in Bubbly Creek's level due to the proposed project results in minor increases in the flood depth of already flooded areas, as well as the additional flooding of areas that were not at risk of basement flooding under baseline conditions. The latter category of incremental flood risk is much smaller, because level rises in the sewer system are small, and such rises must cause an area to cross the basement flood risk threshold; therefore, only subcatchments which were almost flooding under baseline conditions, and have sufficient incremental level rise to now cross the basement flooding threshold, contribute to incremental basement flooding risk as defined in this study. These conditions, where just a minor amount of flooding occurs when it would not have under without-project conditions, are generally of greater concern to property owners than a small incremental rise in flood depth. However, it is useful to consider that this basement flood risk threshold is a simplification of a variable real-world condition. The level of uncertainty associated with actual basement flood risk threshold levels of specific structures, due to differences in basement depth, ground elevation depth, and local sewer system behavior, is likely to be significantly greater than the roughly ¼-ft rise in sewer system level which may result from the proposed Bubbly Creek project (based on scenario E_3_CAP_25). Given the level of hydrologic resolution in the trunk sewer model, alternative approaches to reduce this level of uncertainty are not available. Incremental flood risk should be considered not so much as a prediction of where specific buildings will experience flooding for a specific storm, but rather as an indicator of the general area where a given scenario is likely to contribute to incremental basement flood risk.

4.5 April 2014 Addendum: New Information about Racine Avenue Pumping Station

Subsequent to the evaluation described above, the USACE obtained information from the MWRDGC indicating that the gravity bypass to the RAPs is not utilized, and would likely cause severe damage to the RAPs facility if it were utilized, because the HGL would be forced to rise to the level of Bubbly Creek and would put significant pressure on the floor of the pumping room (Personal communication with David Kiel, 3/14/14). Although this gravity bypass is included in the CDWM trunk sewer model, and is clearly shown on the original RAPs as-builts, according to the MWRDGC the gravity bypass is not utilized and has not been used in the past. This finding is significant, because in the absence of a gravity connection to Bubbly Creek at the RAPs, the RAPs tributary areas is not directly impacted by the increased HGL at the upstream extent of Bubbly Creek , which caused the vast majority of the impacts on basement flooding as described in the preceding sections.

Several modeling simulations were performed to confirm that without gravity overflow at RAPs, the basement flooding impacts described in the previous sections would not occur within the RAPs tributary area. The RAPs model representation was modified to remove the emergency gravity overflow connection that was previously included. The 25-yr 3hr and 25-yr 12 hr simulations were performed without project and with the new substrate in place under existing TARP conditions (E_3_BAS_25, E_3_CAP_25, E_12_BAS_25, E_12_ CAP _25).

The simulations results showed that, under these assumptions, the 'with project' condition caused no rise in the RAPs tributary area (any increases were less than 0.01 ft). Since areas with increased risk of basement flooding risk were predominantly in RAPs, and all such areas that included basements were in these areas, these simulation runs indicate that, based upon the best available data regarding RAPs operation, the proposed project is expected to have no impact in the RAPs tributary area.

It should be noted that for the 25-yr simulations, the modeled HGL in the wet-well still reaches the level where it would overflow by gravity if the gravity bypass were in place. This occurs because, even with the full pumping capacity from RAPs available, the RAPs is not able to keep up with the modeled inflows from its service area. Based upon anecdotal information from the MWRDGC, this condition has not been observed in the nearly 90 year history of the RAPs. It is possible that the CDWM trunk sewer model overpredicts peak inflows to the RAPs for events that exceed the local sewer capacity, like the 25-yr storm event. Further inquiry into this observation is beyond the scope of this study, and this aspect of model performance does not directly impact the conclusions stated above.

SECTION 5

Conclusions

The proposed project on Bubbly Creek will result in increased stages along Bubbly Creek, even after settlement. The increased levels are less than half a foot for all events analyzed, and lead to an increase in RAPs wet well level of roughly 0.25 ft for the E_3_CAP_25 condition. Initial simulations showed that the significant majority of potential basement flooding impacts for “with project” conditions were removed under the future TARP condition, and would be concentrated in the area draining directly to the RAPs. Based upon improved understanding of RAPs operations, simulations were performed under the assumption of no gravity bypass of RAPs to Bubbly Creek. Under this condition, there is no expected rise in peak levels within the RAPs tributary area. The areas draining to Bubbly Creek outside the RAPs tributary area would still experience small increases in peak level, however this did not result in additional areas with basements becoming subject to basement flooding risk that did not have such risk under “without project” conditions.

While the proposed Bubbly Creek restoration project results in small increases in peak level in affected areas, it would be difficult to measure this amount of increase. In addition, other forms of uncertainty, including uncertainty in the actual basement flood risk level for homes in the area affected, and local sewer system performance, are likely much higher than any potential increase in flood risk. Furthermore, these risks are further reduced when McCook Reservoir is online beginning in 2029 (assuming TARP is operated in a manner consistent with the TNET results used for this analysis). In summary, under “with project” conditions level increases in the adjacent sewers are minor, and do not cause new areas with basements to have basement flooding risk (at the scale of the trunk sewer modeling), and these minor increases in level are further reduced under the future TARP condition.

SECTION 6
References

CDM. 2007. *H&H Studies- Planning and Establishing Protocols. Work Plan and Protocols for Trunk Sewer Model Development.*

CH2M HILL. 2013. *Great Lakes and Mississippi River Interbasin Study: Hydrologic and Hydraulic Impact on Sewer Systems.*

Huff, F. A., and J. R. Angel. 1992. *Rainfall Frequency Atlas of the Midwest. Illinois State Water Survey, Champaign, Bulletin 71.* Illinois State Water Survey.

Appendix A
Subcatchments at Increased Basement Flooding Risk

Note: These results are from the original complete set of runs, performed assuming the emergency gravity bypass to RAPs is operational.

Simulation	Subcatchment	Contributing Area (ac)	Max WSEL (ft CCD)	Flood Level (ft CCD)	Incremental Rise (ft)
E_3_CAP_25	3A0483	29.37	4.09	4.045	0.052
	3B0172	2.66	6.57	6.5	0.099
	3B0233	5.03	6.12	6.07	0.091
	3C0173	10.87	9.17	9	0.314
	3C0310	8.79	7.04	7	0.254
	3C0433	21.82	8.15	8.039	0.184
	3C0443	5.66	8.15	8.05	0.176
	3C0561	5.71	6.94	6.9	0.298
	3C0679	9.67	12.41	12.3	0.279
	3C0931	16.68	9.99	9.9	0.122
	3C0986	10.54	10	9.955	0.116
E_3_STL_25	3A0483	29.37	4.09	4.045	0.045
	3B0172	2.66	6.56	6.5	0.087
	3B0233	5.03	6.11	6.07	0.077
	3C0173	10.87	9.12	9	0.265
	3C0310	8.79	7	7	0.22
	3C0433	21.82	8.12	8.039	0.155
	3C0443	5.66	8.12	8.05	0.147
	3C0679	9.67	12.39	12.3	0.262
	3C0931	16.68	9.98	9.9	0.103
	3C0986	10.54	9.98	9.955	0.098
E_3_CAP_100	3C0094	18.88	9.53	9.5	0.296
	3C0111	19.76	8.19	8.072	0.284
	3C0335	10.8	13.749	9.5	3.906
E_3_STL_100	3C0111	19.76	8.16	8.072	0.25
	3C0335	10.8	13.889	9.5	4.046
E_12_CAP_25	2D0421	45.2	6.25	6.25	0
	3A0050	5.04	6.76	6.75	0.022
	3C1034	34.87	11.02	11	0.027
	3C1686	7.01	9.01	9	0.023
E_12_STL_25	2D0421	45.2	6.25	6.25	0
	3A0050	5.04	6.76	6.75	0.024
	3C1034	34.87	11.02	11	0.027
	3C1686	7.01	9.02	9	0.024
E_12_CAP_100	3A0015	5.11	7.17	6.8	0.38
	3A0208	2.03	6.53	6.5	0.165
	3A1160	18.46	11.39	11.284	0.151
	3B0427	36.08	13.32	13.27	0.065
	3C0068	13.28	6.89	6.751	0.371
	3C0155	14.75	6.83	6.629	0.395
	3C0160	5.8	8.06	7.8	0.335
	3C0182	13.95	7	6.656	0.387

Simulation	Subcatchment	Contributing Area (ac)	Max WSEL (ft CCD)	Flood Level (ft CCD)	Incremental Rise (ft)
	3C0231	8.09	7.3	6.993	0.371
	3C0431	17.2	11.04	11	0.327
	3C0466	7.56	9.21	9	0.3
	3C0523	5.07	9.85	9.7	0.293
	3C1015	13.79	12.14	12.1	0.237
E_12_STL_100	3A0015	5.11	7.2	6.8	0.409
	3A0206	4.15	6.5	6.5	0.179
	3A0208	2.03	6.54	6.5	0.179
	3A1160	18.46	11.4	11.284	0.162
	3B0427	36.08	13.32	13.27	0.07
	3B1791	20.63	9.55	9.553	0.006
	3C0068	13.28	6.92	6.751	0.401
	3C0155	14.75	6.86	6.629	0.425
	3C0160	5.8	8.09	7.8	0.361
	3C0182	13.95	7.03	6.656	0.416
	3C0231	8.09	7.33	6.993	0.399
	3C0290	8.65	7.64	7.63	0.379
	3C0431	17.2	11.06	11	0.352
	3C0466	7.56	9.23	9	0.323
	3C0523	5.07	9.87	9.7	0.315
	3C1015	13.79	12.16	12.1	0.255
	3C1535	7.95	10.4	10.4	0.165
F_12_CAP_100	3C0421	7.19	7.88	7.7	0.187
	3C0693	10.21	8.56	8.508	0.133
	3C0924	8.45	9.09	8.7	0.407
	3C1085	17.77	11.1	11	0.124
	3C5359	10.02	8.58	8.5	0.123
F_12_STL_100	3C0421	7.19	7.86	7.7	0.164
	3C0693	10.21	8.54	8.508	0.116
	3C0924	8.45	9.05	8.7	0.372
	3C1085	17.77	11.08	11	0.109
	3C5359	10.02	8.56	8.5	0.108

Attachment 7:

Modeling Impacts from Adopted VE Study Measures

CELRC-TS-DH April 14, 2014

MEMORANDUM FOR CELRC-PM-PL

SUBJECT: Modeling Impacts from Adopted VE Study Measures

1. It was brought to my attention that there were some VE study recommendations that will be adopted. These include reducing the substrate layer overall thickness to 12 inches (6 inches sand and 6 inches rounded river rock), and in the deep areas the top 6 inches would utilize angular quarried rock instead of rounded river stone. These changes will have no adverse impacts on stage increases in Bubbly Creek and on basement flooding. The reason is that the previous HEC-RAS modeling included an analysis of a 22-inch cap with an assumed settlement of 12 inches. Despite the reduced cap thickness and reduced cap settlement, the assumed increase in channel roughness N values would remain the same for non-deep areas. The slight increase in N values due to the angular stone in deep areas will more than be offset by the reduction in N values resulting from the realization that vegetation is not expected to be sustainable in these areas (the reason given for using the less expensive stone). The impact from the reduced cap thickness and the angular stone would be that the increases in the channel stages due to the project will be further reduced. The recent InfoWorks modeling results showed that without a gravity overflow discharge from the Racine Avenue Pump Station that there would be no basement flooding impacts. The project changes due to the incorporation of the VE Study measures would not change these modeling conclusions.

David L. Kiel

Attachment 8:

Evaluation of Value Engineering Substrate Restoration Design for Bubbly Creek

Evaluation of Value Engineering Substrate Restoration Design for Bubbly Creek

Paul R. Schroeder, PhD, PE
Barry W. Bunch, DE, PE
Environmental Laboratory

Ernest R. Smith, PhD
Coastal and Hydraulics Laboratory

U.S. Army Engineer Research and Development Center
Vicksburg, MS
29 April 2014

1. Impacts on Hydrodynamics

Past hydrodynamic modeling of Bubbly Creek predicted velocities and stage for the peak design flow of 6000 cfs discharge from the Racine Avenue Pumping Station plus up to 1000 cfs of additional discharges from the CSOs along the channel for two conditions: the existing bed elevation and the existing bed elevation raised 22 inches by adding new substrate. The Value Engineering design provides for raising the existing bed elevation by a minimum of 12 inches and possibly by 16 inches considering construction tolerances. Since the stage at the peak discharge rate increased only a few inches when raising the bed elevation by 22 inches, the change in cross-sectional area and channel velocity can be assumed to be linear functions of the change in bed elevation. Therefore, linear interpolation can be used to estimate the channel velocity for bed elevations between the two modeled conditions. The actual change in velocity at a location depends on cross-sectional area of the channel at the location, which varies throughout the length of the channel. Considering the relative change in average water depth in areas with high bed shear stress between the two modeled conditions, the interpolated relative increase in velocity over the existing baseline condition is about 7 percent when the bed elevation increases by 12 inches over the existing baseline condition. Therefore, the relative bottom velocity distribution should change less at high velocity than at other velocities, particularly in the upper reach where the channel has wider widths and shallower depths. The ratio of median velocity to peak velocity should decrease somewhat based on the thinner substrate layer, but the ratio should not change more than about 1 percent between the 95th percentile velocity and the peak velocity. The 95th percentile velocity corresponds to bottom velocity that is exceeded in only 5 percent of the area of the channel bed. Past hydrodynamic modeling for the existing conditions showed the ratio of 95th percentile velocity to peak velocity would be about 0.69, while the ratio of median to peak flow would be about 0.48. For the VE design, the ratio of 95th percentile velocity to peak velocity would be about 0.68, while the ratio of median to peak flow would be about 0.45.

All of the past actual discharge rates have been smaller than the design discharge rates; however, the relative decrease in velocity is less than proportional to the relative decrease in discharge rates from the peak design discharge rate to the maximum historic discharge rate due to corresponding decreases in cross-sectional area of flow at the lower discharge rate. Velocities at the highest actual discharge rate (5200 cfs) are likely to be up to about 10 percent smaller than for the peak design flow. The differences in discharge rates are unlikely to change the relative bottom velocity distribution; that is, the ratio of median velocity to peak velocity or the ratio of 95th percentile velocity to peak velocity should be unchanged.

Applying both the change in cross-sectional area (substrate thickness) and the change in design criteria to use the 95th percentile velocity instead of the peak velocity would decrease the design velocity from 2 m/sec to 1.27 m/sec, based on the interpolated change in modeled peak velocity and the computed ratio of the 95th percentile velocity to peak velocity estimated from the original hydrodynamic modeling. If the design RAPS discharge rate were also decreased to the peak historic discharge rate, the design velocity would decrease to about 1.15 m/sec based on the same analysis.

2. Impacts on Bottom Shear Stress and Design Armor Particle Size

The original design for a stable bed without potential for movement under a peak design velocity of 2 m/sec called for a D_{50} of 50 mm for rounded river stone. Bed shear stress is a function of the velocity squared; and stable particle size is a linear function of bed shear stress. Based on the velocities calculated above (a 7 percent reduction in peak bottom velocity), the thinner substrate layer would reduce the peak bottom shear stress by about 14 percent $\{1 – [(1 – 0.07)^2]\}$; therefore, a D_{50} of 43 mm [50 mm * (1 – 0.14)] would be stable for rounded river stone in all areas for the peak RAPS discharge rate of 6000 cfs.

If the design criteria were reduced to require the armor stone to be stable in only 95 percent of the area of the channel bed under the peak design flow and limited movement and scour in the other 5 percent of the channel areas, the design bottom shear stress would be only 40 percent $\{[(1.27 \text{ m/sec}) /(2 \text{ m/sec})]^2\}$ of the original design bottom shear stress and the required D_{50} of the rounded river stone would be 20 mm (50 mm * 0.4). (The area where scour would be allowed to occur is the same area where the bottom velocity exceeds the of 95th percentile velocity, which is why the 95th percentile velocity is used to compute the change in design bottom shear stress.) If the peak design RAPS discharge rate were also reduced to the historical maximum RAPS discharge rate, then the design bottom shear stress would be only 33 percent $\{[(1.15 \text{ m/sec}) / (2 \text{ m/sec})]^2\}$ of the original design bottom shear stress and the required D_{50} of the rounded river stone would be 17 mm (50 mm * 0.33).

Another change in the VE design calls for the use of angular quarry rock in some areas instead of rounded river stone. Angular quarry rock is more resistant to movement than rounded rock for the same weight or effective diameter, having a stability coefficient of 3 instead of 2.2 for rounded river stone. The effect of this difference in stability is a 10 percent reduction in the required effective D_{50} for the armor material. Hence, the required D_{50} for 95 percent of the area to be stable using angular quarry rock ranges from 15 to 18 mm depending on the design RAPS discharge rate.

The impact of allowing armor stone movement in 5 percent of the area, which are the areas with the highest bed shear stresses and bottom velocities, is unlikely to have significant impacts on maintenance or performance because these high bed shear stress areas currently would also have the coarsest existing bed substrate with the lowest organic loading and SOD throughout the channel. As such, the scour in these areas would be expected to be limited to a fraction of the new substrate in these areas and any exposure of the bed sediments would have minimal impacts on the overall system. Additionally, the reduced number and durations of the discharges in the future would also limit the quantity of stone that would move. The stones would not be suspended, but would transport along the bed and could replace other stone that has moved. The stones will rest in the channel at locations with lower bed shear stress.

3. Impacts on Water Quality and Dissolved Oxygen

Bubbly Creek substrate restoration consists of distinct layers of sand to isolate and filter the sediment and overlying rock to armor the sand and provide bed stability. Sand is required to reduce the high sediment oxygen demand of Bubbly Creek's organic detritus sediments and to distribute the weight of the overlying armoring stone on the weaker organic detritus sediments. Sand isolates the water column from organic detritus sediments by two methods. First, it is a filter that physically prevents entrainment of sediment particles in the water column during gas ebullition. Second, sand is a porous barrier between sediments and water column that reduces the concentration gradient of substances between the water column and sediments. Larger thicknesses, over which the differences in the water column and sediment concentrations are applied, result in smaller gradients driving the flux of oxygen demanding substances. The sand layer does not completely prevent chemical releases from the sediments to the water column. Instead, it can be viewed as throttling the existing release rate by providing the equivalent of back pressure that decreases the chemical amount ultimately released to the water column. The flux is linearly proportional to the concentration gradient and the gradient is inversely proportional to the thickness of the isolating media. In the existing sediment bed, the gradient exists in the thickness of the bioactive zone, estimated to be 2 cm in thickness for such a degraded substrate. The VE design would have a gradient thickness of 15 cm, which is predicted to reduce the flux by 87 percent [1 – (2 cm / 15 cm)] in the long run after breakthrough without taking any credit for the effectiveness of the armor stone to provide isolation. If the 15 cm of armor stone were equally effective at isolating the existing sediment bed, the gradient thickness would increase to 30 cm and the flux would be reduced by 93 percent [1 – (2 cm / 30 cm)]. The original design would reduce the flux by 92 percent [1 – (2 cm / 25 cm)] in the long run based on the 25-cm sand layer and 95 percent [1 - (2 cm / 45 cm)] when including the potential effectiveness of the 20-cm armor stone layer. Armor stone would not be expected to be as effective as sand due to greater exchange pore water with the water column due to its large pores and much greater permeability. Incorporation of habitat supporting media and sandy deposition into the armor layer will increase its effectiveness in providing isolation. Therefore, the expected effectiveness of the new substrate should be expected to fall between the estimated performance for the thickness of sand alone and the combined thickness of sand and armor.

The short-term flux reduction would be much greater since it is likely to take years for breakthrough to occur and to establish the gradient throughout the depth of the substrate. Alteration of the sand thickness will also impact the time required to re-establish dissolved chemical flux of the SOD from the sediments to the water column. Initial sand placement will result in immediate decrease in SOD from the current rates. Over time SOD will increase as the oxygen demanding substances infiltrate the sand layer and are released to the water column. This will occur 40 percent [1 – (15 cm / 25 cm)] sooner with a thinner sand layer than with a thicker sand layer or potentially only 33 percent [1 – (30 cm / 45 cm)] sooner if considering the armor material as well. Nevertheless, the end result is that SOD levels are decreased from the current level as given above for the long term regardless of the decrease in breakthrough time. The armoring stone layer and entrained sediment augment the isolation provided by the sand layer. SOD associated with recently deposited materials after substrate construction will still exist. This SOD should not be of the extent or magnitude of the historic sediment SOD due to the reduced discharges and the establishment of healthy benthic community. Recently deposited materials are more likely to be dislodged during subsequent flow event which removes their associated SOD.

Decreasing sand and armoring layer thickness result in the water column being slightly deeper (15 cm) than that generated if originally envisioned layers were used. However, the resulting system will be shallower than the current system by approximately 0.3 m. Water depth impacts two aspects of water quality dissolved oxygen levels; 1) reaeration, and 2) water column oxygen demand present. Reaeration at the surface has to be distributed over the water column, which is more readily accomplished with shallower systems. The VE design increases depth by about 6 percent, thereby on average increasing the reaeration required by 6 percent and decreasing the gradient driving the oxygen to the bottom by 6 percent. However, the system with the revised substrate design is still shallower than existing Bubbly Creek. Depth differences in plans with the original substrate thicknesses and the revised substrate thicknesses are small and within the natural variation of Bubbly Creek water levels. Considering this and the uncertainty of other processes affecting water column DO, slight increases in depth due to decreases in substrate thickness result in predicted recovery time for DO being only minimally impacted, perhaps about 10 percent.

Evaluation of Value Engineering Substrate Restoration Design for Bubbly Creek

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2. Impacts on Bottom Shear Stress and Design Armor Particle Size

The original design for a stable bed without potential for movement under a peak design velocity of 2 m/sec called for a D_{50} of 50 mm for rounded river stone. Bed shear stress is a function of the velocity squared; and stable particle size is a linear function of bed shear stress. Based on the velocities calculated above (a 7 percent reduction in peak bottom velocity), the thinner substrate layer would reduce the peak bottom shear stress by about 14 percent $\{1 – [(1 – 0.07)^2]\}$; therefore, a D_{50} of 43 mm [50 mm * (1 – 0.14)] would be stable for rounded river stone in all areas for the peak RAPS discharge rate of 6000 cfs.

If the design criteria were reduced to require the armor stone to be stable in only 95 percent of the area of the channel bed under the peak design flow and limited movement and scour in the other 5 percent of the channel areas, the design bottom shear stress would be only 40 percent $\{[(1.27 \text{ m/sec}) /(2 \text{ m/sec})]^2\}$ of the original design bottom shear stress and the required D_{50} of the rounded river stone would be 20 mm (50 mm * 0.4). (The area where scour would be allowed to occur is the same area where the bottom velocity exceeds the of 95th percentile velocity, which is why the 95th percentile velocity is used to compute the change in design bottom shear stress.) If the peak design RAPS discharge rate were also reduced to the historical maximum RAPS discharge rate, then the design bottom shear stress would be only 33 percent $\{[(1.15 \text{ m/sec}) / (2 \text{ m/sec})]^2\}$ of the original design bottom shear stress and the required D_{50} of the rounded river stone would be 17 mm (50 mm * 0.33).

Another change in the VE design calls for the use of angular quarry rock in some areas instead of rounded river stone. Angular quarry rock is more resistant to movement than rounded rock for the same weight or effective diameter, having a stability coefficient of 3 instead of 2.2 for rounded river stone. The effect of this difference in stability is a 10 percent reduction in the required effective D_{50} for the armor material. Hence, the required D_{50} for 95 percent of the area to be stable using angular quarry rock ranges from 15 to 18 mm depending on the design RAPS discharge rate.

The impact of allowing armor stone movement in 5 percent of the area, which are the areas with the highest bed shear stresses and bottom velocities, is unlikely to have significant impacts on maintenance or performance because these high bed shear stress areas currently would also have the coarsest existing bed substrate with the lowest organic loading and SOD throughout the channel. As such, the scour in these areas would be expected to be limited to a fraction of the new substrate in these areas and any exposure of the bed sediments would have minimal impacts on the overall system. Additionally, the reduced number and durations of the discharges in the future would also limit the quantity of stone that would move. The stones would not be suspended, but would transport along the bed and could replace other stone that has moved. The stones will rest in the channel at locations with lower bed shear stress.

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Bubbly Creek substrate restoration consists of distinct layers of sand to isolate and filter the sediment and overlying rock to armor the sand and provide bed stability. Sand is required to reduce the high sediment oxygen demand of Bubbly Creek's organic detritus sediments and to distribute the weight of the overlying armoring stone on the weaker organic detritus sediments. Sand isolates the water column from organic detritus sediments by two methods. First, it is a filter that physically prevents entrainment of sediment particles in the water column during gas ebullition. Second, sand is a porous barrier between sediments and water column that reduces the concentration gradient of substances between the water column and sediments. Larger thicknesses, over which the differences in the water column and sediment concentrations are applied, result in smaller gradients driving the flux of oxygen demanding substances. The sand layer does not completely prevent chemical releases from the sediments to the water column. Instead, it can be viewed as throttling the existing release rate by providing the equivalent of back pressure that decreases the chemical amount ultimately released to the water column. The flux is linearly proportional to the concentration gradient and the gradient is inversely proportional to the thickness of the isolating media. In the existing sediment bed, the gradient exists in the thickness of the bioactive zone, estimated to be 2 cm in thickness for such a degraded substrate. The VE design would have a gradient thickness of 15 cm, which is predicted to reduce the flux by 87 percent [1 – (2 cm / 15 cm)] in the long run after breakthrough without taking any credit for the effectiveness of the armor stone to provide isolation. If the 15 cm of armor stone were equally effective at isolating the existing sediment bed, the gradient thickness would increase to 30 cm and the flux would be reduced by 93 percent [1 – (2 cm / 30 cm)]. The original design would reduce the flux by 92 percent [1 – (2 cm / 25 cm)] in the long run based on the 25-cm sand layer and 95 percent [1 - (2 cm / 45 cm)] when including the potential effectiveness of the 20-cm armor stone layer. Armor stone would not be expected to be as effective as sand due to greater exchange pore water with the water column due to its large pores and much greater permeability. Incorporation of habitat supporting media and sandy deposition into the armor layer will increase its effectiveness in providing isolation. Therefore, the expected effectiveness of the new substrate should be expected to fall between the estimated performance for the thickness of sand alone and the combined thickness of sand and armor.

The short-term flux reduction would be much greater since it is likely to take years for breakthrough to occur and to establish the gradient throughout the depth of the substrate. Alteration of the sand thickness will also impact the time required to re-establish dissolved chemical flux of the SOD from the sediments to the water column. Initial sand placement will result in immediate decrease in SOD from the current rates. Over time SOD will increase as the oxygen demanding substances infiltrate the sand layer and are released to the water column. This will occur 40 percent [1 – (15 cm / 25 cm)] sooner with a thinner sand layer than with a thicker sand layer or potentially only 33 percent [1 – (30 cm / 45 cm)] sooner if considering the armor material as well. Nevertheless, the end result is that SOD levels are decreased from the current level as given above for the long term regardless of the decrease in breakthrough time. The armoring stone layer and entrained sediment augment the isolation provided by the sand layer. SOD associated with recently deposited materials after substrate construction will still exist. This SOD should not be of the extent or magnitude of the historic sediment SOD due to the reduced discharges and the establishment of healthy benthic community. Recently deposited materials are more likely to be dislodged during subsequent flow event which removes their associated SOD.

Decreasing sand and armoring layer thickness result in the water column being slightly deeper (15 cm) than that generated if originally envisioned layers were used. However, the resulting system will be shallower than the current system by approximately 0.3 m. Water depth impacts two aspects of water quality dissolved oxygen levels; 1) reaeration, and 2) water column oxygen demand present. Reaeration at the surface has to be distributed over the water column, which is more readily accomplished with shallower systems. The VE design increases depth by about 6 percent, thereby on average increasing the reaeration required by 6 percent and decreasing the gradient driving the oxygen to the bottom by 6 percent. However, the system with the revised substrate design is still shallower than existing Bubbly Creek. Depth differences in plans with the original substrate thicknesses and the revised substrate thicknesses are small and within the natural variation of Bubbly Creek water levels. Considering this and the uncertainty of other processes affecting water column DO, slight increases in depth due to decreases in substrate thickness result in predicted recovery time for DO being only minimally impacted, perhaps about 10 percent.