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Master's Program in Systems Science

Thesis Title:

Water Management Modelling in the Simulation of Water Systems in Coastal Communities

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Abstract

It is no longer a question of scientific debate that research declares our climate is changing. One of the most important and visible impacts of this phenomenon is sea level rise which has impacts on coastal cities and island communities. Sea level rise also magnifies storm surges which can have severely damaging impacts on different human made infrastructure facilities near the shorelines in coastal zones. In this research we are concerned about the proximity of water systems as one of the most vulnerable infrastructures in the coastal zones because of the impact of stormwater combining with sewage water. In Canada, the government has plans to address these issues, but to date, there needs to be further attention to stormwater management in coastal zones across the country. This research discusses the impacts of severe environmental events, e.g., hurricanes and storm surge, on the water systems of selected coastal communities in Canada. The purpose of this research is to model coastal zone water systems using the open source StormWater Management Modelling (SWMM) software in order to manage stormwater and system response to storms and storm surge on water treatment plants in these areas. Arichat on Isle Madame, Cape Breton, one of the most sensitive coastal zones in Canada, is the focal point case study for this research as part of the C-Change International Community-University Research Alliance (ICURA) 2009-2015 project.

Keywords: Climate change, sea level rise, stormwater, water systems

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Table of Contents

List of Figures

List of Tables

Glossary

µg/L	Micrograms per Liter
BOD	Biochemical Oxygen Demand
CMS	Cubic Meter per Second
COD	Carbon Oxygen Demand
EMC	Event Mean Concentration
GPS	Global Positioning System
ha	Hectare
HGL	Hydraulic Grade Line
hr	Hour
HRT	Hydraulic Residence Time
ICURA	International Community-University Research Alliance
IPCC	Intergovernmental Panel on Climate Change
Kg	Kilograms
Kw-Hr	Kilowatt-Hour
long-lat	Longitude-Latitude
L	Liter see mg/l
m	Meter
Max	Maximum
mg/L	Milligram per Liter
Min	Minimum
mm	Millimeter
MSWM	Municipal Stormwater Management
P	Phosphorus
Pb	Lead
PIEVC	Public Infrastructure Engineering Vulnerability Committee
RSBC	Revised Statutes of British Columbia
RTK	Real Time Kinematic
Sec	Second
SQ	Status Quo
SWMM	StormWater Management Modelling
TKN	Total Kjeldahl Nitrogen
TPO	Treatment Plant Outfall
TSS	Total Suspended Solids
Zn	Zinc

1. Introduction

This document presents research in the Master's Program in Systems Science in the form of a thesis in partial fulfillment of the M.Sc degree in Systems Science at the University of Ottawa. The research is undertaken as part of the C-Change International Community-University Research Alliance (ICURA) program with particular focus on the C-Change community of Isle Madame, Cape Breton, Nova Scotia (C-Change, 2011).

1.1. Motivation / Problem Definition

It is no longer a question of scientific debate that research declares our climate is changing. Climate change has significant effects on the physical environment and the industrial aspects of human life (Ministry of Environment, Ontario, 2010). One of the most important and visible impacts of the changing climate is sea level rise which has impacts on coastal cities and island communities all over the world. This issue has notably received a great deal of attention since even a small rise in sea level might have significant impacts on coastal environments.

Sea level rise also magnifies storm surges which can have severely damaging impacts on different human made infrastructure facilities near the shorelines in coastal zones such as sewage and water systems, energy systems (including nuclear), and different industrial facilities (Ministry of Environment, Ontario, 2010). Therefore, it is important to pay more attention to securing these facilities from storm water, storm surge, and rising seas because their failure will have inevitable and potentially catastrophic impacts on human life and health. One example is the case of the nuclear facilities at Fukushima and the Japanese tsunami of March 2011 (311). The resulting tsunami from the powerful earthquake caused immense damages in different infrastructures such as water systems, nuclear and conventional power plants (Fukushima, 2012). Another example is Hurricane Katrina that hit New Orleans in August 2005. Katrina was one of the strongest storms in United States coastal zones during the last 100 years (National Climatic Data Center, 2005). Katrina caused devastating impacts on the central Gulf Coast of the US. Impacts included loss of life, severe flooding from the breaching of the levees that displaced thousands of people, and economic impacts on the oil industry and power outages. According to the Louisiana Department of Health, the official number of deaths was 1646 people (Louisiana Department of Health and Hospitals, 2006*)*, but it is said that the real number of deaths was more than this number and possibly more than 3000. More than 1.7 million people lost power after this strong storm in the Gulf States. In New Orleans, drinking water was not available because of broken water mains. This research is concerned about the proximity of water systems as one of the most vulnerable infrastructure services in the coastal zone because of the impact of stormwater combining with sewage water or source water systems and the resulting impacts on the population that depends on clean water for survival.

In Canada, communities in the coastal zone have tried to manage water systems problems through different stormwater management plans led by provincial government. Provincial/territorial jurisdictions are responsible for governing water systems in Canada. The provinces are "owners" of the water resources and responsibilities related to water systems are defined for them in the day-to-day management of these fundamental services. As well, the federal government has a significant role in aquatic research and an important role in water management in Canada (Environment Canada: Federal Policy & Legislation, 2012).

Many small coastal communities in Canada are still using natural water systems not protected through a stormwater management system. Stormwater is a kind of water collected from rain, snowmelt or any other kind of precipitation that the ground receives. The environment will naturally move water through the water cycle. Stormwater is not an exception and it follows the water cycle as well (Ministry of Environment, Ontario: Stormwater management, 2010). Stormwater can have effects on both the drinking water system and the sanitation system (Ministry of Environment, Ontario: Stormwater management, 2010). Pollution may enter a coastal community's fresh water supply, and the community's water systems should be managed so that the population's health is not endangered from water borne pollutants.

Water systems management policies exist all over Canada in small municipalities, in regional district service areas, in improvement districts which are self-government authorities responsible to provide local services for benefit of residents of the community (Ministry of community services, 2006), in private water utilities, in water user's communities such as the public corporate bodies incorporated under the Water Act and administered by the Ministry of Environment of British Columbia (Ministry of Environment British Columbia, RSBC, 1996), on First Nation reserves, in individual private wells and domestic licensees (Ministry of Health Services, BC, 2002). Similar legislation exists in all provinces.

Some of the problems associated with water systems in Canada are attributed to the difficulties experienced on First Nations' reserves (Simeone, 2010). This fact occurs despite the assertion that pure and healthy water should be accessible for everyone. Access to clean water is taken as a human right and the right to water places responsibilities on governments.

In some parts of Canada this problem has been a preoccupation of local communities and an issue for governments tasked to resolve it, especially native Canadians on reserves. In Canada, there are approximately 89,897 houses on First Nations reserves (Project Blue, 2008). It is estimated that around 2,000 of these houses do not have access to any water services and around 5,000 do not have access to sewage systems (Project Blue, 2008). Although the government promised to allocate additional funding for the recognized First Nations water problem, there are still too many boil water advisories and in some cases "do not consume" orders for reserves. In some cases, these communities live under this situation for extended periods of time, e.g., years (Project Blue, 2008).

According to Project Blue, the number of First Nations communities which were living in this situation decreased in 2008. According to Health Canada (2012), on July 2011, 126 First Nations' communities were under water advisories. Operators in these communities have not been able to certify local water systems to ensure proper testing and treatment. Water systems infrastructures in these communities are in high risk of pollution and affect human health. Problems with water systems in First Nations are attributed to different issues such as lack of filtration, low quality treatment and stormwater effects. The government has plans to address these issues, but to date, there is further attention needed to stormwater management in First Nation's reserves across the country The recent media attention around Attawapiskat highlighted water problems on First Nations communities. Attawapiskat, an isolated first nation located in the Kenora District in Northern Ontario, is facing water quality issues caused by human pollution and threats to safe water due to erosion, turbidity, agricultural runoff, fertilizer and manure. The case of Attawapiskat has garnered much public attention to the plight of First Nations' lack of capacity in securing safe water.

Canada experienced its worst *E.coli* contamination on May 2000, known as "The Walkerton Tragedy" (Lindgren, R.D., 2003). Walkerton is a community in southwestern Ontario, located within and governed by the municipality of Brockton. Walkerton is 200 km from Toronto, Canada's largest city. The Walkerton water supply became contaminated because of the runoff from farms that were spreading manure for fertilizer that then leached into nearby drinking water wells. This water was carrying *E.coli* bacteria, which are extremely dangerous to human health (Pollution Probe, 2004). Although the Ontario Medical Officer of Health issued a boil water advisory, seven people lost their lives because Walkerton Public Utilities Commission, who was in charge of this community's water supply, did not accept that the water supply was contaminated for several days during which time about 2,500 people became sick. This disaster forced the government to allocate funds to clean up the water system in Walkerton. Many governmental agencies were blamed because of the failure to apply water guidelines and policies.

A similar tragedy took place in 2001 in North Battleford, Saskatchewan (Laing, 2002). These events made people believe that water can easily become contaminated and it is not always correct to assume that our water supply is safe. As a consequence of these events, protection of water from contamination is recognized as an essential policy that is necessary for human health. To keep the water supply clean, contamination and pollutants must be prevented from entering our water sources (Pollution Probe, 2004).

Water management is an important issue. Access to clean water is of importance. Water quality for people, plants and animals should be improved. In this regard, governments should consider plans regarding surface water, drinking water, bathing water and ground water. Even waste water is important and should meet certain standards, because this water after treatment flows into the environment and will be back into the water cycle.

There are many sources of contamination of water. Natural water is not pure and it always contains minerals. Some of these natural minerals may cause health risk for humans. As well, water contamination may be the result of human activities. Agriculture and industrial activities may impact quality of water. Industrial discharges, municipal wastewater effluents, septic systems and landfill sites are possible sources of water contamination (Pollution Probe, 2004). Governments allocate huge amounts of budget on water treatment systems in order not to let polluted water combine with source water. However, some factors such as storm water may also cause contamination (Pollution Probe, 2004). According to the Federation of Canadian Municipalities (FCM) all federal, provincial and municipal governments have shared responsibility for water management (FCM, 2012)

While water quality concerns have increased in recent years in Canada, there is little attention paid to contamination from stormwater. This is especially true for the management of water systems in some parts of Canada especially First Nation's reserves. Sea level rise and storm surge is happening due to the changing climate, and its effects on coastal communities' water systems may be devastating without proper water systems management. Moreover, overflow conduits or flooded manholes may cause intrusion of salt water into the waste water system which can be considered as one source of the water pollution. In wastewater systems, salt will kill the treatment plant biology, therefore there will be discharge of partially or untreated effluent, with its consequences and risks. This consideration of water systems management is the focus of the proposed research.

This research discusses the impacts of the changing coastal environment on the waste water systems of a selected coastal community in Canada. The focus is on water infrastructure modelling and the analyses of the sensitivity of water systems to increasing severe storms that put pressure on the sewage water systems. The purpose of this research is to simulate a coastal water system using the StormWater Management Modelling (SWMM) (EPA, 2010) software in order to manage the sewage water and its response to storms on water treatment plants. One of the most sensitive coastal zones in Canada, Isle Madame, Cape Breton Island in Nova Scotia is the case study for this research.

1.2. Research questions and objectives

The fundamental research questions in this research are:

1) What are the characteristics of sewage and stormwater systems in coastal communities and how can they be described?
2) What are the impacts of severe coastal storms on the community's water system?
3) What is the performance of community sewage systems to address severe storms?
4) What are the communities' responses to stormwater management planning and strategies?

In response to the research questions the associated objectives of this research are as follows:

1) To model a selected community's sewage and stormwater systems components using SWMM.
2) To acquire data on storms and storm impacts of stormwater on selected coastal communities
3) To evaluate simulated storm impacts using the SWMM stormwater model.
4) To communicate the results to communities and find strategies for managing adaptation in selected communities.

1.3. Plan of the Thesis

The thesis document has seven main sections. The current section provides the introduction and motivation for the proposed research project. The second section is a literature review on the different aspects of the project. The third section contains the methodology of the research. Research process is explained in section four. Results and analysis are discussed in section five of the thesis. In section six, conclusions, suggestions and recommendations for future studies are provided. And, finally, section seven presents the bibliography of the research. Appendices are also provided in this document to identify data and complete analyses.

2. Literature review

The literature review of this chapter below is divided into four main sections namely: (2.1) Water and Climate Change; (2.2) Coastal Communities; (2.3) Water Infrastructure Modelling; and (2.4) Applications. Each main section is further subdivided into subsections of particular interest and literature related to this research.

2.1. Water and Climate Change

One effect on the environment associated with climate change is global warming which results in other inevitable impacts. Global warming due to greenhouse effects has impacts on the water life cycle, for instance, more evaporation and more precipitation is expected, glaciers and ice sheets will experience melting, the volume of sea water will expand, storms will become more frequent and intense, and the sea level will rise leading to more flooding (Mimura et al., 2007). Many aspects of daily life are dependent on water systems and water resources, and any change in this regard would have impacts on economic and social aspects of human life. Moreover, serious negative impacts of the kind of change noted on the quality of the environment and human well-being are inevitable (Arnell, 1999). However, climate change is only one of the pressures on water systems. Climate and water resources are interconnected in different ways (Kundzewicz et al., 2007). The connections between human activities, climate change and water resources are shown below in Figure 2.1.

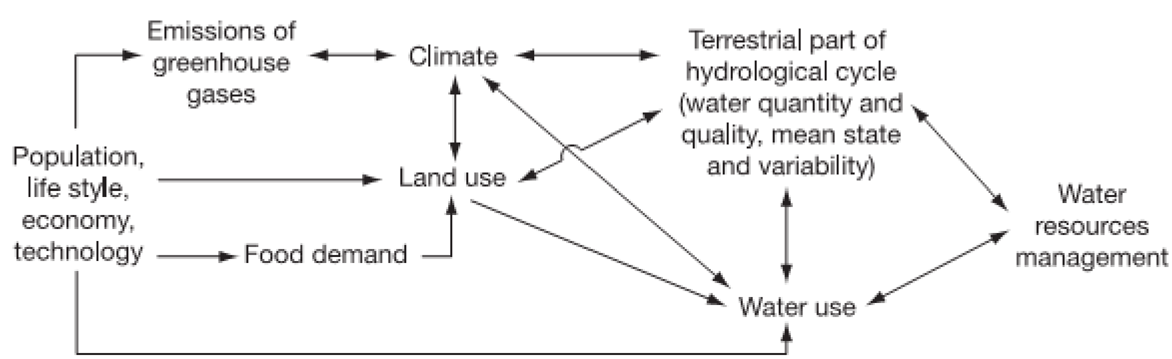

Figure 2.1. The connections between human activities, climate change and water resources (Source: Kundzewicz et al., 2007)

Climate change also impacts different components of global freshwater. Changes in precipitation intensity and snowmelt volume result in changes in river flows, and water levels in lakes and wetlands. Changes in temperature, solar radiation, and humidity have effects on evaporation. These factors impact climate change on surface water and runoff. Groundwater systems are affected by climate change more slowly. Groundwater levels mostly depend on precipitation and rarely are affected by the change in temperature. Moreover, one of the most important impacts of climate change is on weather systems and severe storms leading to environmental disasters. The increase in precipitation, changes in weather patterns, and increases in floods are further evidence of the impacts of climate change. More frequent storms cause pollution in stormwater, drinking water, and treated water which overall results in more contaminated water (Mimura et al., 2007).

Water quality is therefore exceptionally affected by climate change. Increases in water temperature reduce the amount of oxygen in water and cause problems for aquatic biological systems, e.g., fishes, reptiles, molluscs, crustaceans and other aquatic organisms. More intensity in rainfalls causes contaminants to wash into the water system. These contaminations may cause different water-related diseases either from drinking or from water consumption by plants irrigated by polluted water and consumed. It is evident that the changing climate is capable of causing changes in water systems that, in turn, causes economic, social and environmental disruption leading to disasters (Kundzewicz et al., 2007).

2.2.Coastal Communities

The impacts of climate change are particularly important in coastal zones as they are more vulnerable than other zones (Lane et al., 2010). Water related impacts of climate change are more obvious in coastal communities affected by rising sea level and coastal storm surge. Therefore coastal communities need to be adaptive to climate change. According to the fourth IPCC assessment report (Mimura et al., 2007), coastal hazards include: rising CO_2 and decreases in ocean surface pH, sea level rise, and increases in global sea surface temperature, erosion and ecosystem loses. These hazards are the reasons behind increased floods and increases in storm runoff which cause damages to infrastructure in the coastal zone. Water system infrastructure, fishery infrastructure, marinas, and harbours are all affected by these changes in water behavior (Moy et al., 2010). Canada has the longest coastline in the world (Mostofi, 2011); therefore, these zones attract more attention for climate change research. Thus, the coastal zone, characterized as it is by increasing human activities, e.g., encroaching communities and increasing demands for infrastructure, are more and more vulnerable than ever before (Mimura et al., 2007, Pilkney and Young, 2010). An example could be Hurricane Sandy. Sandy was a late-season hurricane that hit south-western Caribbean Sea coastal zones and the eastern US seaboard during 22-29 October 2013 (Blake et al., 2013). Based on preliminary estimates Hurricane Sandy caused almost $50 billion in damages and unfortunately 199 people were killed (Atlantic Flyway Shorebird Business Strategy Planning Team, 2013). This was one of the costliest hurricanes to occur on the east coast US since 1900 (Blake et al., 2013). It had damaging impacts on human life, infrastructures, wildlife and environment.

2.3. Water Infrastructure Modelling

In Canada, the Ontario Ministry of the Environment is responsible for stormwater management for protecting communities' water systems through provincial regulations (Ministry of Environment, Ontario, 2010). In Ontario, municipalities apply the Municipal Stormwater Management (MSWM) regulations. Municipal Stormwater Management programs include conventional stormwater management and source control equipment and activities for the systems as illustrated in Figure 2.2 below (Ministry of Environment, Ontario, 2010).

Figure 2.2, illustrates a typical municipal stormwater management system. Sanitation and stormwater received from precipitation and runoff are collected in the sewer system and stormwater system. Water in these 2 systems will flow into the receiving watercourses. The flow in the sewer system is treated in a treatment plant and the treated water flows out of the system. Source control facilities, as noted in Figure 2.2, may be on properties located as road rights-of-way that are controlled by municipalities, while other source control facilities (e.g., drains and prepared underground drainage ways, and culverts) are located on private properties. Conventional (natural, manmade or combined) stormwater management systems refer to conveyance facilities that may include vegetated filter strips, roadside ditches, storm sewers and perforated conduit conveyance systems and end-of-conduit facilities like ponds, oil grit separators, constructed wetlands, or most often for coastal communities, natural harbors (as "end-of-conduit" receiving watercourses illustrated in Figure 2.2). Managing stormwater at the lot level (private properties) is referred to as source control (Figure 2.2). Source control facilities may use infiltration, reuse and evapotranspiration methods as well as storage and treatment of stormwater. The importance of water conservation through reusing stormwater is recognized by source control (Figure 2.2). There are also connected systems whereby the stormwater collection system can run off into the wastewater system through manholes – as well as out through directed natural drains into conduits and out into receiving watercourses (as in the Figure 2.2). These connected systems are also discussed in section 2.3.1 where SWMM is introduced as the simulation tool in this research.

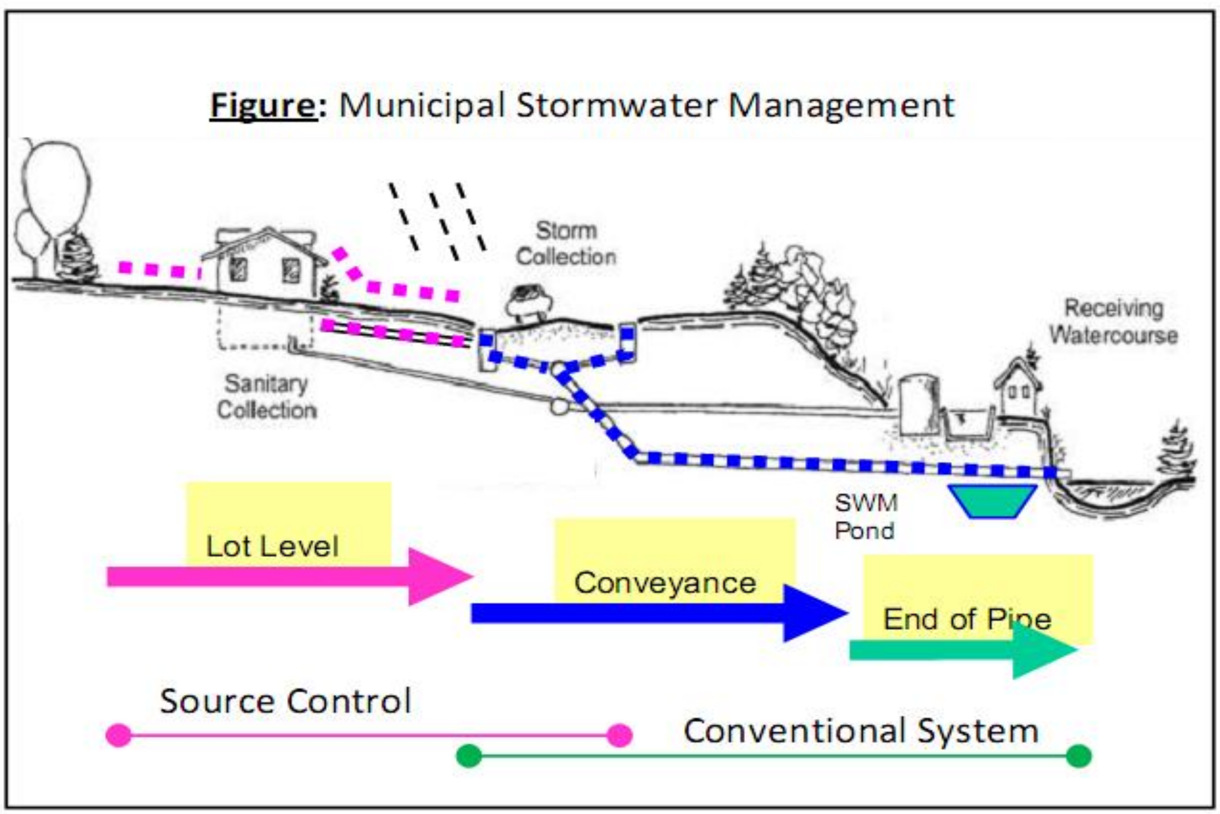

Figure 2.2. Municipal Stormwater Management System

(Source: Ministry of Environment,Ontario, 2010)

Most of the water systems work similarly. Sewage into pipes/conduits at residences/businesses, flows by gravity to meet other pipes at junctions, stormwater can enter via manholes and catchbasins/grates on side of road, flows to low points with minimum grade or slope to pipes, may get too deep for effective construction so pumps added to lift water to higher elevation pipe for further gravity flow towards treatment plant. Manholes/pumping stations are large structures whose volume allows storage of wastewater so pumps are not working 100%. Where feasible, manholes/pumping stations equipped with emergency overflows (a type of regulator) to prevent damage to system and flooding – these are an outfall or outlet from the system, as is Treatment Plant Outfall.

Stormwater management also recognizes the quality of water to protect health and the environment. Since stormwater connects with surfaces such as roads, landscapes and buildings through runoff from precipitation and snow melt, there are increases in the risk level of contamination from stormwater through the system that need to be taken into account (Ministry of Environment, Ontario, 2010). One of the most important examples related to water quality is Walkerton tragedy. Drinking water source in Town of Walkerton (located in south Ontario) was contaminated from manure spreading on farms near municipal wells during an unusual precipitation event in this small town. Seven people died in this tragic event, while almost 2300 people suffered a water quality related illness (Prudham, 2004).

In Ontario, the Clean Water Act is another program that affects community water systems. The Clean Water Act represents the provincial government's response to the Walkerton disaster. The Clean Water Act requires communities to evaluate water quality and declare potential water threats in their area, and to find solutions to clean the water through treatment. This program requires the public to participate in source protection. Moreover, through this program there are financial aids that can be allocated among farmers, landowners and small and medium businesses to take up activities that have the goal of reducing the need for water treatment and with the objective of maintaining clean water. The regulations referred to in these programs are available for other communities and municipalities who are partners in this Clean Water Act as well, so they can update current policies and develop tools to adapt stormwater management process (Ministry of Environment, Ontario, 2010).

British Columbia has one of the oldest water system infrastructures in Canada according to the average age of expected life span for such systems (Marshall, 2002). In recent years, water related improvement projects have been funded through the Canada-British Columbia Infrastructure program (Marshall, 2002). There are approximately 3,300 water systems infrastructures in BC today. Most of the BC population is served by 96 systems operating in large municipalities such as Vancouver and Victoria. Other systems include public and private systems such as those in small municipalities, regional district service areas, improvement districts, private water utilities, water users communities, First Nation reserves, individual private wells and domestic licensees plus other systems (BC Ministry of Health Services, 2002). The Ministry of the Environment of BC provided a guide book for stormwater management in BC which explains a special provincial-wide stormwater management plan for water systems in BC (Stephens et al., 2002). In recent years, BC boil-water advisories are in effect because of the lack of disinfecting abilities in BC's water systems. Despite this, there have been no reports of serious water-related disease outbreaks in BC since 1998 (Cash et al., 2008).

In the Yukon Territory, a boil-water advisory was issued in 2007, when some elementary school students suffered from an illness caused by *E.coli* bacteria in drinking water. Also, high levels of coliform were found in drinking water in the town of Tagish, Yukon (Cash et al., 2008). In Yukon, people get their water through a public drinking water system. Regulations applied on this system are according to the territory's Public Health and Safety Act Drinking Water Regulation (Yukon Water Resources, 2011).

The recent media attention around Attawapiskat highlighted water problems on First Nations communities. Attawapiskat, an isolated first nation community located in the Kenora District in Northern Ontario (Attawapiskat First Nation, 2012), is facing serious water-related issues caused by human pollution. This community has lacked safe drinking water because of erosion, turbidity, agricultural runoff, fertilizer and manure that leach into the water system through wells. While these are all legally licensed activities, the impact on the community's drinking water is severe. Until the facilities and capacities of the community are improved for water management, then the ability to maintain clean water for human consumption is in question and the health of the community members are at risk (The Council of Canadians, 2011).

These programs and cases are cited here as examples of the importance of community water management in Canada. Accordingly, despite Canada's best efforts at the federal, provincial, and municipal levels of government, water systems are still prone to problems which have the potential to affect human life and health. Consequently, there is considerable research underway with the objective of improving water quality for Canadians throughout the country.

2.3.1. SWWM5.0 as a Simulation Tool in Stormwater Management

There are advanced capabilities for modelling and simulating water systems schematically. In particular, software systems such as Storm Water Management Model (SWMM) are useful for planning and engineering development of waste water systems for communities. This software was developed by the United States Environmental Protection Agency and has been widely used around the world since 1969 (now in version 5) for exploring the capacity and efficiency of water systems. Water system modelling tools of this kind (while it is a dominant model, SWMM is not unique) are helpful in analyzing these systems in different situations including examining the impacts of sea level rise, stormwater capacity, and the impacts from increasing storms and incidences of high precipitation. SWMM is particularly useful for analyzing water systems in the case of combined water systems where sanitary sewage and stormwater runoff are collected in the same conduit. Therefore, it helps in managing stormwater capacity so that the system functions effectively, and potential water-borne pollutions can be understood in order to prevent contamination of the water system.

SWMM is a tool for simulating storm water systems. It permits modelling dynamic rainfall-runoff situations in a flow simulation model which can be applied to single event or continuous event simulation of specified quantity and quality of runoff such as may be experienced in a severe storm, hurricane, etc. This tool can be used for measuring runoff collected from land-based subcatchment areas which models precipitation and produces runoff and pollutant-laden water in the water system. In SWMM, water is transported through conduits, channels, storages, pumps and other water system components schematically. SWMM tracks runoff quantity and quality collected in each subcatchment. During a specified simulation period, factors such as flow rate, flow depth, and water quality in each component of the water system can be tracked through the water system until release to watercourses or after water treatment (Rossman, 2010).The applications of this tool are for planning, analysis and design in relation to stormwater runoff, combined sewers, sanitary sewers, pumps, and treatment plants. SWMM applications are developed through a network representation as a drawing of the particular physical components of the water system of interest. SWMM allows the possibility of selecting a set of analysis options after which the simulation will be run and the results illustrated (Rossman, 2010).

SWMM is useful in this research project that seeks to examine the impacts of severe storms on the water systems in coastal communities in Canada, as a tool for understanding stormwater management. Moreover, in selected coastal communities that are part of the C-Change ICURA research project, municipalities sometimes do not have the opportunity of analyzing and testing the stresses on their water systems, including planning for pending extreme environmental events arising from coastal climate change. Specifically, using available data and information for analyzing water systems in the municipalities through the SWMM modelling tool provides a valuable resource for understanding municipalities' water systems under different storm scenarios and to examine water system capacities and strategies for adapting to the impacts of more frequent coastal storms and sea level rise. Thus, the preparation of a locally prepared SWMM model will provide a useful resource for municipal planning not otherwise available to these coastal communities.

SWMM has a number of limitations. The SWMM hydraulic engine is a bit slower than other common hydraulic engines like INFOWORKS CS and MOUSE/MIKE URBAN so it has numerical instability. The other consequence of having a slow engine is that the simulation speed is less than common hydraulic engines. Another limitation on SWMM is that there is no formal support for SWMM by EPA. Moreover, SWMM does not have any direct GIS interface.

2.4. Applications

More frequent storms and rising sea levels continue to affect Canada's coastal zones (Pakdel, 2011). Of particular interest to this research, are those hazardous coastal areas in Canada that will be impacted by more frequent and severe storms in the short run, and sea level rise in the long run. The applications of storm impacts on the stormwater systems and modelling and simulation will be investigated for the Atlantic coastal community of Arichat on Isle Madame in Richmond County, Cape Breton, Nova Scotia. Isle Madame is an island community located off the south-eastern shore of Cape Breton Island (Figure 2.3).

Figure 2.3. Isle Madame map noting the village of Arichat along the southern shore (Source: C-Change, 2012)

A permanent settlement named Arichat began rising economically in Isle Madame in 1713. The population of this community began to increase and Arichat harbour became one of the most important international trading ports in Isle Madame. Currently, 3,455 residents live in 1,383 private dwellings on Isle Madame (Canada Census, 2006). According to the recent municipal strategic plan (Municipality of the County of Richmond, 2011), the central water supply in Arichat is located at Babin's Lake; most other Isle Madame communities are serviced by private wells and on-site (septic) sewage systems.

Recent studies completed on groundwater availability in Isle Madame report that the result is low potential risk for groundwater sources of drinking water (Municipality of the County of Richmond, 2011). Therefore, according to the report, water services are currently in a phase of further development in the Isle Madame area.

The stormwater system in this community is a natural one where water flows to the sewage system or to the ocean through culverts, drains and manholes. The wastewater collection was established in 1970s. This system contains 95 manholes, 5 wastewater-pumping stations and a treatment plant (Richmond County, 2013, Municipality of the County of Richmond, 2003). As this single wastewater treatment plant in Arichat is located on the shore in the Arichat Harbour area, there is some concern that this plant may be prone to impacts from rising sea level and coastal storms. Therefore, it is expected that some further development be applied to protect this treatment plant and other parts of the sewage and stormwater treatment system in Arichat in order to adapt to the pending climate changes.

3. Methodology

The Methodology chapter contains five sections namely: (3.1) Modelling with SWMM; (3.2) Physical Components in SWMM, (3.3) SWMM Settings and Inputs, (3.4) Outputs in SWMM and (3.5) Process of Research. These sections of the research methodology present general aspects of the SWMM modelling framework as used in the development of the Isle Madame wastewater system model, as well as the process for carrying out this research with emphasis on the implications of the model on community water systems policy and strategic decision making under the threats of changing coastal climates.

3.1. Modelling with SWMM

The SWMM Version 5 software has been used as a simulation modelling tool for exploring the water system in the community of Arichat on Isle Madame, Richmond County, Cape Breton, Nova Scotia, one of the C-Change project communities. This community is one of the hazardous coastal communities in Canada. According to the Municipality of County of Richmond (2011), currently water services are in a phase of further development in the Isle Madame area. Arichat has 95 manholes and one treatment plant (Richmond County, 2013, Municipality of the County of Richmond, 2003). The treatment plant is located near the coastline, which increases the risk of damage because of storm surges hitting the treatment plant during huge storms. On the other hand, stormwater flows into the sewage system naturally and it might cause overload in the system and may cause a decrease in water quality of this area. SWMM has the ability to simulate this system by simulating physical components of the system, precipitation time series, tidal levels and water quality.

SWMM considers assumptions which have impacts in this research using SWMM for modelling Arichat water system. SWMM can model sewage system and subcatchments as linkages of the natural stormwater system to sewage for modelling Arichat sewage water system. SWMM cannot model individual water sources system, .e.g., wells, and septic tanks. Moreover, SWMM is a deterministic modeler; therefore no random variables are defined for simulations in SWMM. SWMM is therefore most useful in exploring the sensitivity of alternative parameter values in the system, e.g., pump performance, elevation levels, etc.

In the following sections, SWMM ability of simulating water systems is discussed.

3.2. Physical Components in SWMM

In SWMM, the study area water system can be modeled with a set of physical components. Some of the physical components in SWMM includes: (i) subcatchments, (ii) junctions, (iii) storage units, (iv) conduits, (v) pumps, (vi) outfalls and (vii) regulators. For example, SWMM's junction-conduits representation of sewage and stormwater flow needs to be described as in Figure 3.1. In this figure $Q_1(N-1)$ is the net flow (Inflow-outflow) into the node J through conduit N-1 and $Q_1(N)$ is the same value for node J+1. Node J is a storage manhole. Astore is the manhole surface area and node J as a manhole with storage has Astore but node J+1 as a non-storage node does not have any surface area. In this figure $AS_{N-1}(J)$ and $AS_N(J)$ are the surface area contributed by the conduits connected to the node. Based on this figure, in SWMM flow depth at the end of a conduit can be calculated as the difference between the ZCROWN (J) (which is the head of the node) and the invert elevation of the conduit (Rossman, 2006). This figure is a proper representation of a water system modelling in SWMM which shows how manholes and conduits are connected and how water flows into the system. To apply SWMM, data sets are also needed to describe the water system under investigation. Physical components' representations in SWMM are discussed below.

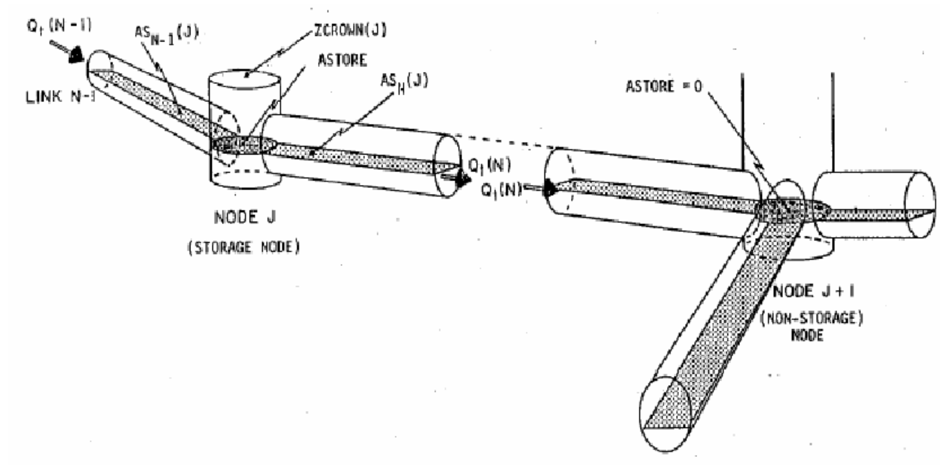

Figure 3.1. Junction-Conduits representation in SWMM

(Source: Rossman, 2006)

(i) Subcatchments

In order to obtain results in SWMM, the study area should be divided into smaller spatial sub-areas, each of them provided with specific properties. This icon is considered for subcatchments in the SWMM toolbar. Choosing this icon, it will be possible to shape the subcatchment area to a desired shape by defining its vertices (as determined by the physical map of the area under investigation). Figure 3.2 shows an example of a subcatchment, which is taken from the water system model in Arichat, developed in this research. Figure 3.2 shows the shape of the area and subcatchment's name, S3.

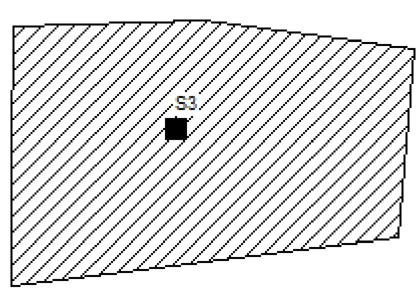

Figure 3.2. Representation of Isle Madame model subcatchment Area, S3 in SWMM

After drawing the physical components on the study area map in SWWM, the next step is to edit the properties for the component and provide related data for each component. Generally, subcatchments are divided into 2 subareas, the pervious area and the impervious area. Runoff can penetrate into the upper soil part of the pervious subarea, but this is not the case for the impervious area. Impervious areas mostly include parking lots and asphalt roads that lead water directly to the linked junction as runoff (Rossman, 2010).

For each subcatchment area, the percentage of impervious area, i.e., the subarea into which water cannot penetrate, and name of the labeled junction which receives the runoff from the subcatchment are included in the subcatchment properties window.

Finally, a rain gauge can also be assigned to the subcatchment properties window that directs precipitation to the catchment. Rain gauges are discussed below in section 3.3(ii) of this chapter.

(ii) Junctions

Junctions are physical components in SWMM which are drainage system nodes where conduits link together. Junctions assemble natural surface channels, conduit connections or manholes of the sewer system. Runoff from subcatchments or any other external inflows can enter the system through junctions. The most important input parameters for a junction are elevation of junction's "invert" (i.e., the level of the inside of the conduit), maximum depth of the manhole, initial depth of water present in the system, surcharge depth, and ponded area. "Maximum water depth" in the manhole is the distance from invert to the ground surface. If zero is used, then the distance from invert to top of highest connecting conduit will be used (Rossman, 2010). "Surcharge depth" is the excess depth in the manhole over the maximum depth before flooding happens. The "ponded area" is a storage area around the junction (manhole station) which can store excess water (above surcharge levels) which is more than the capacity of the system and overflows the system and is lost. Ponded area is an option for re-introducing the excess water to the system as the capacity of the system resumes (Rossman, 2010).

In SWWM, junctions are designated by this shape: . Figure 3.3 represents subcatchment S3 connected to junction j3 as its outlet junction. In Figure 3.3, junction j3 is connected to another junction (j10) through the conduit (c7).

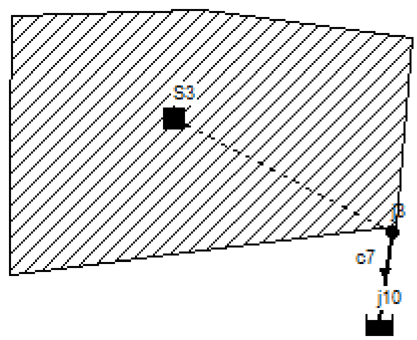

Figure 3.3. SWMM representation of Isle Madame model subcatchment S3 linked to junction j3 which is connected to storage unit j10 through conduit c7

(iii)Storage Units

Storage units are also drainage system nodes. Their difference with junctions is that they provide storage volume for the system. They can be the representation of a small catch basin or a big lake. Storage units can receive water as inflows and also release water as outflows. In addition to these, they also can lose water through surface evaporation or infiltration to the native soil. Similar to all other components, storage units have specific properties. Invert elevation and maximum depth are the two most important properties for storage units. Invert elevation for the storage unit is the elevation of the bottom of the unit. Storage units are shown with  in the SWMM toolbar. Also, in Figure 3.3 a representation of a storage unit in Isle Madame model is shown; j10 is the storage unit.

(iv) Conduits

Conduits also can be added to the area map using this icon: . Conduits are pipes or channels that transfer water from one junction to another. The most important properties of a conduit are inlet and outlet junctions, shape, length and maximum depth of the conduit. Conduit cross-sectional shapes are defined in SWMM from common shapes options.

Maximum depth of the cross-section can be considered as the diameter of the conduit in the system. The representation of a conduit, c7 which moves water from j3 to j10 is shown in Figure 3.3 in the Isle Madame model. The arrow on the conduit shows the direction of the water flow in the conduit.

(v) Pumps

A pump is represented in SWWM by the icon: . Typically, pumps are used to bring up water from a lower elevation to a higher elevation. In SWMM, 4 types of pumps are supported, differentiated by their curves. A pump curve explains the connection between a pump's flow rate and conditions at its inlet and outlet junctions.

Type 1 pump represents an off-line pump which is inside a wet well and its curve is shown in Figure 3.4 below. In this type of pump, flow increases incrementally with the wet well volume.

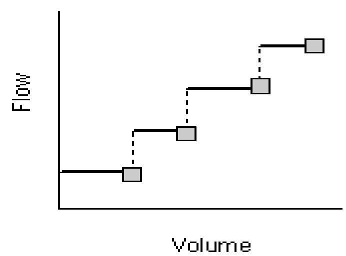

Figure 3.4. Type 1 pump curve

(Source: Rossman, 2010)

Type 2 is an in-line pump where flow increases incrementally with the depth of the inlet junction. Figure 3.5 represents type 2 pump curve.

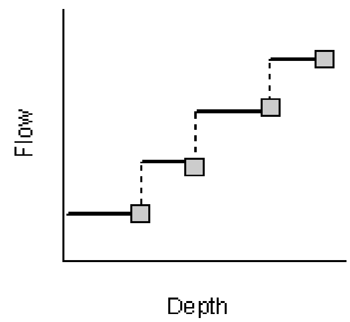

Figure 3.5. Type 2 pump curve

(Source: Rossman, 2010)

The third type of the pumps is an in-line pump where flow differs with head difference between the inlet and outlet nodes, continuously. Figure 3.6 depicts pump type 3 curve.

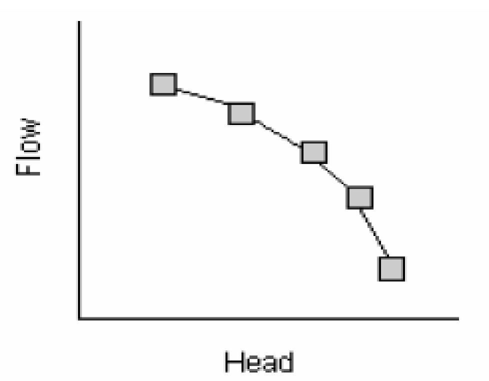

Figure 3.6. Type 3 pump curve

(Source: Rossman, 2010)

Another type of pump is type 4, this kind of pump is a variable speed in-line pump in which flow changes with the depth of the node continuously. In Figure 3.7 type 4 pump's curve can be found.

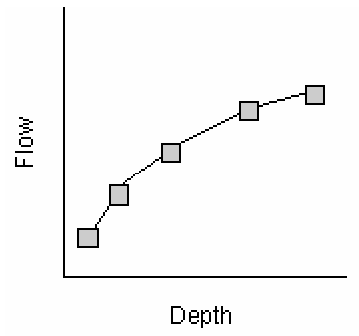

Figure 3.7. Type 4 pump curve

(Source: Rossman, 2010)

The last type of pump is an “ideal” transfer pump. In this type of pump the flow rate is the same as inflow rate at the inlet node. For these kinds of pumps there is no curve required. This type of pump should be considered as the only outflow conduit from the inlet node. The pump type which is of interest of this research is “Type 3” transfer pump.

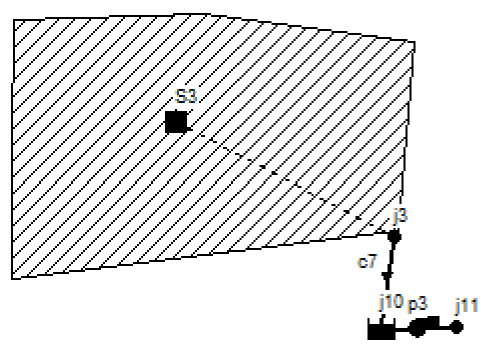

Figure 3.8. SWMM representation of Isle Madame model subcatchment S3 linked to junction j3 which is connected to junction j10 through conduit c7 and pump p3 linked to junction j11

Similar to other components in SWMM, pumps have specific properties. Firstly, the inlet and outlet junction for each pump must be determined. In Figure 3.8, j10 is the inlet junction and j11 is the outlet junction for pump p3. The inlet junction is considered as a storage unit so water is stored in j10 and then will be pumped to j11. It is possible to enter the specific pump curve type. If no curve is inserted for the pump, SWMM considers the pump as the ideal pump, i.e., the flow rate is the same as inflow rate at the inlet junction. Pump initial status can be shown as “on” or “off”. Most pumps have specific rules. These rules can be based on the pump startup depth, and the shutoff depth. It is also possible to consider control rules for each pump to turn off and on in “controls” section in the software.

(vi) Outfalls

An outfall is a type of junction that discharges wastewater into the environment and which is represented by the icon ▽ on the SWMM toolbar. Outfall is used to show the final junction in the system to assign final downstream boundaries under Dynamic wave flow routing. The dynamic wave flow routing method used in the Isle Madame model is discussed in section 3.3(*i*) below. Outfalls act as a junction for any other type of routing. Each outfall can only have one conduit to be connected to it as the outflow junction for the conduit. Outfalls have specific characteristics which can be shown by defining related parameters such as invert elevation, existence of a flap gate to prevent backflow, and tidal outfall which is a user defined time series of tidal level versus time. Figure 3.9 shows an outfall (outfall 3) as a terminal junction of the Isle Madame system.

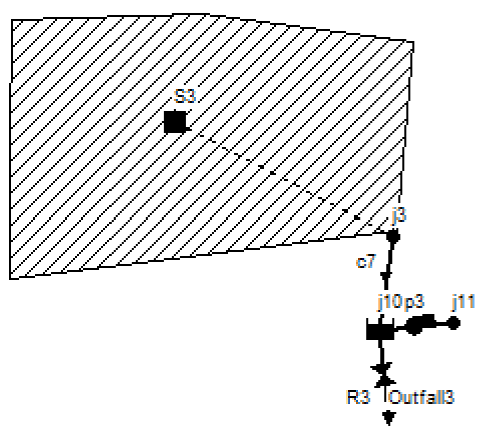

Figure 3.9. SWMM representation of Isle Madame model subcatchment S3 linked to junction j3 connected to junction j10 through conduit c7 and pump p3 linked to junction j11. J10 connected to outfall (Outfall3) and associated control regulator (R3) in SWMM

(vii) Regulators

The last physical component of SWMM introduced in this section and used in the development of the Isle Madame model is the regulators. Regulators are devices that control flows in the system. SWMM represents 3 kinds of regulators: orifices, weirs and outlets.

Orifices are represented by the  icon on the SWMM toolbar. Orifices are used as flow modelling diversion or outlet structures in water systems. Most of the orifices are openings in the wall of the manholes, storage facilities or control gates. For all regulators, it is possible to consider control rules to determine open and close time. Regulators may be fully open, fully closed or, fractionally open. The second type of regulators are represented as , addressed as "weirs". Weirs have the same responsibility as orifices; the difference is that weirs are located in a manhole on the side of a channel or inside a storage unit. Weirs have 4 types: transverse, V-notch (which is of interest in this research), side flow and trapezoidal. The principal input for weirs are the same as orifices in addition to its type and side slope. Weir properties can be the vertical height of the weir opening, horizontal length of the weir crest (or crown for V-notch) and the discharge coefficient for flow through the central portion of the weir. The third and the last type of regulators are outlets. Outlets are control structures that control outflows from the storage units which are shown as . All the regulators are represented with the same symbol, R, on model output diagram as a connection between two junctions in SWMM as shown in Figure 3.9 above.

For the case of Arichat, "regulators" do not exist, but there are emergency/overflow outlets (combined sewer overflows) for water release to the ocean in the case of near flooding at some manhole junctions. In this case, untreated water may be released. Accordingly, this is modelled in SWMM as a controllable parameter to simulate the case where these escape valves are sealed off and do not release untreated water. This assumption is helpful in defining scenarios as well.

Finally, SWMM has an option to load a backdrop image behind the model on the study area map. This map can be any relevant picture such as city map, topographic map and site development plan. Loading a geo-referenced map behind the model can help to locate the system components at their exact geographical (longitude and latitude) location on the map. Figure 3.10 shows the geographical map of Arichat which is taken from Google Earth (Google Earth, 2012). SWMM has the capability of determining x-coordinate and y-coordinate of each component on the map based on the map dimension. Specifying map dimensions, the software automatically recognizes the x-coordinate and y-coordinate of each component on the map. This feature is helpful to find the length of the conduits and area of the subcatchments (addressed as Auto-length in the software).

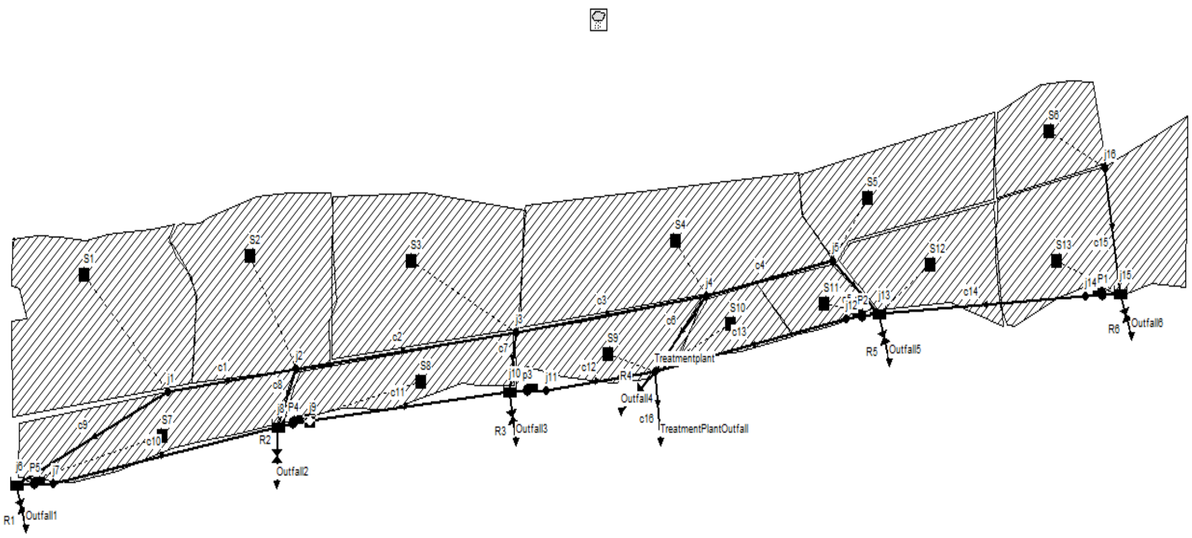

Figure 3.10. Isle Madame water system model developed in SWMM

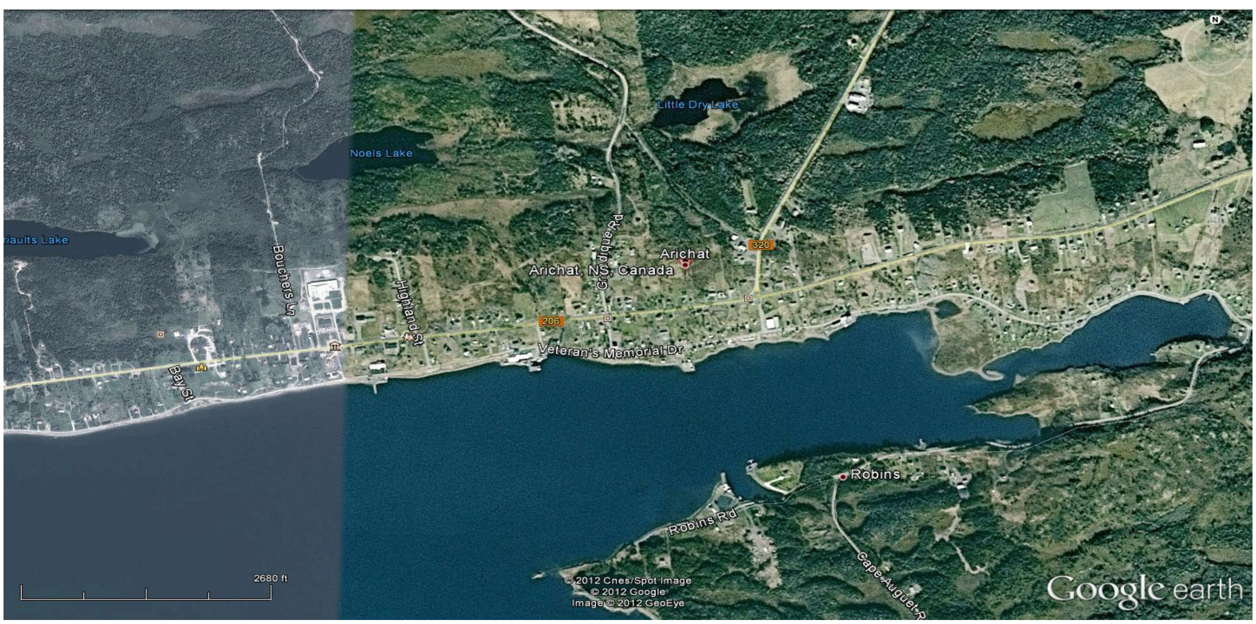

Figure 3.11. Geographical map of Arichat used as the backdrop image of the model Isle Madame water system model in SWMM

(Source: Google Earth, 2012)

3.3. SWMM Settings and Inputs

In the SWMM simulation option section, general settings, dates and time steps settings are required to be defined. In the general settings section, it is possible to choose process models among: (i) different flow routing options, (ii) precipitation, i.e., rainfall/runoff, (iii) tidal flow routing and (iv) water quality. Choosing any of these options results in a different final report on SWMM. In the miscellaneous settings section, there are options related to ponding allowance and report summary options. Infiltration models are other choices in this section. Infiltration models can be chosen from Horton equation, Green-Ampt method and Curve Number method. The Horton equation decreases infiltration exponentially from an initial maximum value to a minimum rate during a huge rainfall. Green-Ampt method assumes a wetting front in the soil in which an initial moisture soil is separated from saturated soil above. The third method estimates runoff based on its curve number and total infiltration capacity.

Date settings in SWMM include start analysis date, start and end reporting dates, start and end sweeping dates, and the number of anticipated dry days. For time steps in the simulation, reporting time steps, dry weather runoff, and wet weather runoff time steps are available options.

(i) Flow Routing Models

Flow routing model is the last option in the “general settings” in SWMM. Flow routing within a conduit in SWMM is governed by the conservation of mass and momentum equations for an unsteady flow. These equations are the Saint Venant flow equations. The flow routing option is used in this simulation and there are three ways to solve these equations. The first one is steady flow routing which is the simplest method to solve the equation. When this method is used, it means within each time step flow is uniform and steady. Therefore, the inflow hydrographs at the upstream end of the conduit flows to the downstream end without any delay or change in shape. This is the simplest routing method which cannot be considered for channel storage, backflow effects, entrance losses, exit losses, reverse flow and pressurized flow. This method is proper for preliminary analysis with long-term continuous simulations in SWMM (Rossman, 2010).

The second routing method is kinematic wave routing. This method allows flow and area to vary both spatially and temporally within a conduit, however this form of routing cannot account for backwater effects, entrance/exit losses and flow reversal. It can usually maintain numerical stability with moderately large time steps, therefore it can be used as an accurate and efficient routing method for long-term simulations (Rossman, 2010) .

The third method is dynamic wave routing. Unlike the other two methods, dynamic wave routing can be used for channel storage, backwater, entrance/exit losses, flow reversal and pressurized flow, which is the case in this research for Arichat.

(ii) Precipitation

The Climatology editor in SWMM is helpful to import time series from climate files. These time series can be related to precipitation, temperature, evaporation, wind speed, snow melt and areal depletion.

One of the most important hydrological inputs in SWMM is the precepitation time series. The Rain gauge is the representative input which is shown with the icon in the SWMM toolbar and can be added to the study area map in SWMM. Figures 3.10 and 3.11 show gauge1 which is a rain gauge for the Arichat developed model. Rain gauges, similar to other physical components, have their specific properties. Rain format is one of these properties with options for Intensity, Volume and Cumulative Volume. Time interval for precipitation data and precipitation time series are other principal properties for a rain gauge. It is possible to define different hourly precipitation time series in SWMM.

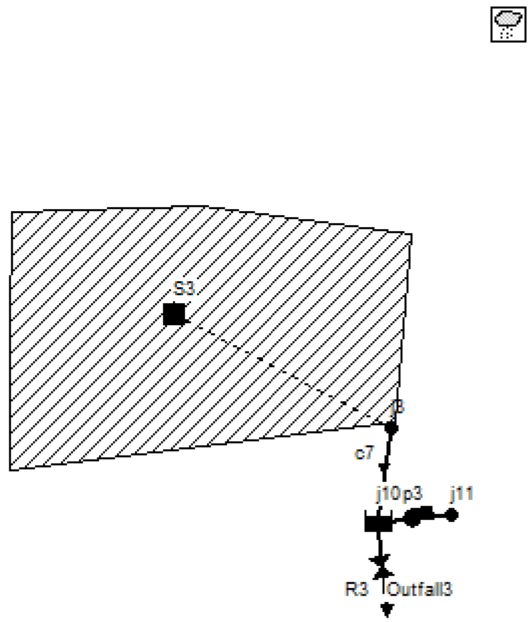

Figure 3.12. The raining cloud representation is the Rain gauge representation in SWMM

(iii) Tidal Flow

The Curve section in SWMM defines pump, storage, control, diversion, rating, shape and tidal curves. The most important curve for the Isle Madame coastal model is the tidal curve, which can be defined as hourly time series for tidal levels. Many factors cause tides at any coastal zone. These factors can be the consequence of the response of the ocean basin to the tide producing forces, to the variations of tides because of the shallow water effects of local rivers, or to the regional effects of weather change on water levels. Tides can be high or low. A high tide is produced by the horizontal flow of water toward the area with maximum solar and lunar attractions, which results in "heaping" action. On the other hand low tides are produced by a compensating withdrawal of water. The alternation of these two kinds of tides is the result of daily rotation of the Earth (International Hydrographic Bureau, 2005). Highest high tides and lowest low tide effects on the water system in Isle Madame are an interest of this research. One of the most important effects of tides can be the backflow in the water system, especially in the treatment plant, where seawater is forced up the outfalls by the high tides. The tidal curves for Arichat will be discussed in scenarios to check the capacity of the system under different tidal levels.

(iv) Water Quality

SWMM is capable of tracking water quality. Based on this capability, a variety of pollutants can be defined in SWMM. Defining these pollutants requires defining land uses of the subcatchments. In each land use definition, pollutant or effluent build-up and washoff functions are determinable. Effluent Build-up functions are described by a mass per unit of the subcatchment area. The Effluent Build-up, B is a function of the number of dry weather days and can be described in 3 kind of functions: 1. Power Function, 2. Exponential Function, and 3. Saturation Function. The SWMM functions for B are defined mathematically below.

1.Power Function

$$B = Min(C_1, C_2 t^{C_3})$$

where C_1= maximum build-up possible (mass per unit of area or curb length) and C_2= build-up rate constant, and C_3= time exponent.

2.Exponential Function

$$B = C_1(1 - e^{-C_2 t})$$

where C_1= maximum buildup possible (mass per unit of area or curb length) and C_2=buildup rate constant(1/days).

3.Saturation Function

$$B = \frac{C_1 t}{C_2 + t}$$

where C_1= maximum buildup possible(mass per unit area or curb length) and C_2= half-saturation constant (days to reach half of the maximum).

On the other hand, pollutant washoff may also be defined by one of these functions below:

1.Exponentail Washoff

$$W = C_1 q^{C_2} B$$

where C_1= washoff coefficient, C_2= washoff exponent, q= runoff rate per unit area(inches/hour or mm/hour), and B= pollutant buildup in mass units.

2.Rating Curve Washoff

$$W = C_1 Q^{C_2}$$

where C_1= washoff coefficient, C_2= washoff exponent, and Q= runoff rate in user-defined flow units.

3.Event Mean Concentration (EMC)

This function is a special case of rating curve washoff where C_2 is 1.0 and C_1 shows the washoff pollutant concentration in mass per liter. Typical EMC's for selected pollutants can be found in Table 3.1. The table shows the typical values for EMC in a water system. For each of them there is a maximum and a minimum limitation. These limitations show the safe range for the washoff for each pollutant. For example, in Arichat water quality modelling, TSS is used as one of the pollutants with EMC as the washoff function. The washoff coefficient is considered equal to 100. Therefore TSS (Total Suspended Solid) EMC, based on values in Table 3.1, should be between 180- 584 mg/L.

Finally, in each outfall it is possible to choose if the outfall is connected to a treatment plant or not. If yes, for each pollutant there should be a treatment expression which has the general form of one of the two forms below:

$$R = f(P, R\_P, V), \text{ or } C = f(P, R\_P, V)$$

where:

R = fractional removal, (units: % concentration)

C = outlet concentration, (units:mg/L)

P = one or more pollutant names,

R_P = one or more pollutant removals (prepend R_ to pollutant name),

V = one or more process variables (e.g., FLOW(in user-defined flow units), DEPTH(units: m), HRT(hydraulic residence time (units: hours), DT(routing time step (units: sec)), AREA(units: m^2).

Table 3.1. Typical EMC's for selected pollutants

(Source: Rossman, 2010)

Constituent	Event Mean Concentration
TSS(mg/L)	180-584
BOD(mg/L)	12-19
COD(mg/L)	82-178
Total P(mg/L)	0.42-0.88
Soluble P(mg/L)	0.15-0.28
TKN(mg/L)	1.9-4.18
NO2/NO3-N(mg/L)	0.86-2.2
Total Cu(µg/L)	43-118
Total Pb(µg/L)	182-443
Total Zn(µg/L)	202-633

Based on the inputs and the properties of the SWMM Physical components, a variety of scenarios can be defined to run the simulation of the water system. In order to define the simulation scenarios for analysis, the first step is to determine the controllable and uncontrollable variables of the system as those variables that are important for the purpose of the community water system policy analysis and research. For example, whether valves are open or closed can be considered as controllable variables for the scenarios in this research. Other variables such as precipitation time series, tidal level, initial depth of the junctions, and impervious percentage applied to the subcatchment areas are uncontrollable system variables. The precipitation time series can introduce heavy, mean and light storms based on the hourly values provided in the series. Tidal level time series can be helpful in defining highest high tide, lowest low tide and mean tide level in the whole data available for the tidal levels. Another option for tides can be starting the simulation at low tide or starting at high tide. All these alternatives make a difference in the scenarios and the simulation. Initial depth of the junctions can be considered equal to 0, which shows there is no water present in the junction at the time the simulation starts. Also it can be considered equal to the maximum depth of the junction at the time the simulation starts. Subcatchment impervious percentages can be low or high in different scenarios. Low percentage shows more infiltration in the subcatchments which results in less inflow into the water system and high percentage causes less infiltration and more runoff on the subcatchments. Considering these controllable and uncontrolable variables, scenarios can be defined with the combination of different alternatives for the SWMM model parameters defined above.

3.4. Outputs in SWMM

Once the model data sets and properties for all components have been defined, the simulation of the water system performance using SWMM can be carried out. Simulation results are presented in a variety of SWMM reports. The moment the simulation is completed, a small window appears on the screen which shows the mass continuty errors for surface runoff, flow routing and quality routing. These errors show the difference between initial storage plus total inflow and final storage plus total outflow for the whole water system. These errors are provided in percentage terms and should not exceed 10%, otherwise the simulation and data should be checked for possible problems. The most common reasons for a continuity error more than 10% can be long time steps, or too short conduits. In addition, the in-status report (explained in the following paragraph) lists the junctions of the system that have excessive continuity errors. If junctions have continuity errors greater than 10% then it signals that a review of the junctions should be conducted in order to reduce the continuity errors.

SWMM reports results in different formats such as system status reports (e.g., re flooding of junctions), graphic reports (e.g., on flow rates), table reports (e.g., of junction depth levels) and statistical reports (e.g., of time in state). A status report includes analysis options, input summary, rain fall summary, error messages, control actions, continuity errors, stability results and also summary results for each physical component in the system. Moreover, in the status report, flooding junctions and surcharged conduits and junctions are specified. Results can be analyzed based on different kinds of graphs in SWMM including time series plots (e.g., of preciptiation events over the simulated period), scatter plots (e.g., catchment runoff over nodes total inflow), and profile plots (e.g., tracking water flow in the lower road conduits).

Time series graphs depict the value of a particular variable (e.g., flow rates, water depth) versus the elapsed time. These values are chosen based on the object category (junctions, conduits, subcathments, total system). Values may include inflow, outflow, depth, and flooding for each object chosen from the study area map. Also if data, which are pre-defined data based on some limitations and constraints, can be defined for any of the objects, SWMM has the ability to compare the computed values with the pre-defined data in a graph. Additionally, the table report displays all data related to graphs.

Scatter plots represent the association between a pair of variables, such as precipitation in a subcatchment and flow rate in a conduit. Scatter plots are helpful in identifying unstable flow routing results which SWMM is not capable of doing automatically. Unstable flow routing occurs when significant fluctuations take place at certain periods of time in some junctions.

Profile plots display water flow in conduits and junctions. In this type of plot, it is possible to track water flows in the system in an animated view. This plot shows what exactly happens to the junctions and conduits during a simulation.

Lastly, statistical reports can be created from the simulation results time series. This report separates the simulation period into non-overlapping segments such as; day, month, or by volume above some minimum threshold values. Then it computes a statistical value for each event such as mean, maximum or total sum of a variable in the event’s time period. Finally, it shows a summary of these statistics values for the entire values in all the events. Mostly, statistical analyses in SWMM are proper for long-term continous simulation runs.

For the Arichat model, status report, time series plots, profile plots and table reports will be used for analyzing the results.

3.5. Process of Research

In this section, the process of the research, SWMM model development and analysis for the Arichat water system is explained. This section describes how the process is applied.

SWMM is used in this research to model and simulate the water system in Arichat, Isle Madame. The stormwater system in Arichat is characterized as a natural system which associates with the sewage system through runoff from catchments to the manholes in the system. The Arichat sewage system contains 5 pumping stations, one treatment plant, and 95 manholes (Municipality of the County of Richmond, 2003) from which only eleven manholes will be represented in the model developed in this research. In the Arichat SWMM model, some of these manholes are connected to subcatchment areas which connect the natural stormwater system and the sewage system in Arichat.

SWMM is used to develop rudimentary models of the Arichat water system with physical components (junctions, conduits, subcatchments, storage units, pumps, outfalls and gauges) and directional flow. Also, a geo-referenced map of Arichat harbour is provided for the backdrop map in the model and the area of the water system being modelled is defined on this map. After that, geo-referenced and elevation data on pumping stations and manhole sites are collected and linked to the mapping system in SWMM (the elevations are the result of comparing handheld GPS elevation values and the RTK report (Tienaah et al., 2011) elevation values). Technical data and properties on pumping stations is collected from the Municipal Public Works for each station in Arichat. On the other hand, for the water quality aspect, an example from the SWMM manual for water quality information for the system is used (in the absence of actual obsevations). For the inputs of SWMM, historical data on precipitation (Environment Canada) and on tidal patterns is gathered. Then, events of interest, storms, surge, hurricanes etc. are analysed. The next step is to define scenarios. For this purpose, the controllable and uncontrollable parameters of the water system are validated and verified with the Municipality. In addition to defining variables, a 24-hour simulation model of storm events based on tidal level information and precipitation time series is prepared. The 24-hour time period for the deterministic simulation is chosen based on an accepted practice (based on feedback from NS government workshop on

February 2013), the duration of severe storms, and the expected peak capacity impacts on the water system. This time period enables the analysis of the extreme event and its impacts on the water system.

Based on the defined controllable and uncontrollable variables, simulation model scenarios and output results for analysis are designed. After running the simulation model under each scenario, results and outputs are analyzed. Model results are explicitly examined for system policy considerations (i.e., observed water system “weak spots”, implications of severe storm event impacts, implications of strategic policy changes (e.g., closing outfall regulator valves, etc.)). The simulation model water system recommendations are reported and finally a review of future modelling updates, and monitoring and observations is provided.

In Chapter 4 the detailed research process and the results of the simulation are discussed. Chapter 5 presents the analysis of the results and outputs.

4. Research Process

Chapter 4 contains 2 main sections namely: (4.1) Modelling the Arichat Water System Using the SWMM Modelling System and (4.2) Arichat Hydrologic Settings and Inputs for Applying Different Scenarios in SWMM. These sections of the research process focus on the modelling of the Arichat water system in SWMM. Different modelling and simulation scenarios are defined in this chapter to explore environmental conditions of the coastal community, to investigate community water policy options, and to present the results and impacts of simulated severe weather conditions on the water system.

4.1. Modelling the Arichat Water System Using the SWMM Modelling System

The water system in Arichat is investigated in this research. The Arichat water system includes its explicit sewage system and the implicit link to the natural stormwater system. As mentioned in Chapter 3 above, the first step for modelling the system using SWMM is defining the system's physical components and their properties. In this section, Arichat specific physical components are explained. These physical components include the geographical location, subcatchments, rain gauges, junctions, outfalls, storage units, conduits, pumps and weirs. Each of these components specific to the Arichat water model is described in further detail below.

4.1.1. Arichat Map

Figure 4.1 is the geo-referenced map of the Arichat water system area. This map represents the background for the SWMM water system model. Figure 4.1 is the satellite image of the area map of Arichat as a snapshot of Google Earth maps (Google Earth, 2012). The map of Arichat shows the coastal community clustered left to right around a coastal road running up to the head of the Arichat harbour (known as the "Lower Road" or "Chemin d'en bas"), and a parallel road running above the coastal road (known as the "High Road" or "Chemin d'en haut"), a segment of Nova Scotia Highway 206. As implied by the name, the coast slopes up to a ridge that runs the length of the roads from left to right and defined by the Finger Lakes (Theriault's Lake, Noel's Lake, and Little Dry Lake in Figure 4.1) that lie behind the ridge and oriented in the same direction as the roadways. Community dwellings lie along the length of the roads on the shore side of the Lower (coastal) Road, and either side of the High Road evident from the map of Figure 4.1.

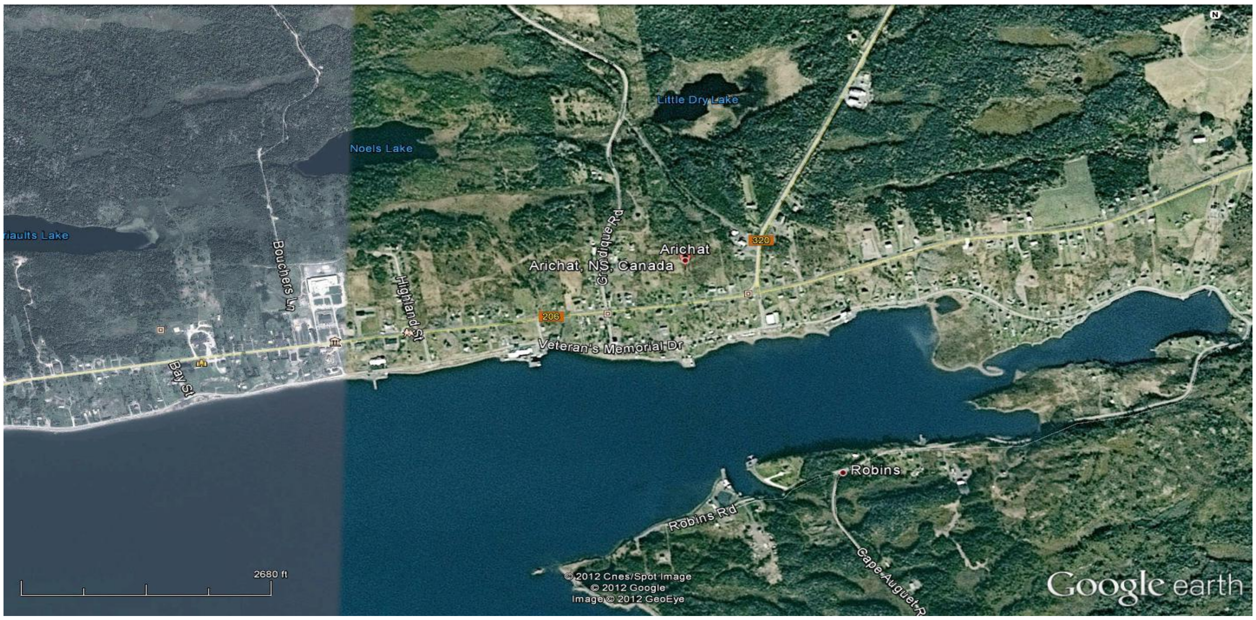

Figure 4.1. Arichat model's backdrop image (Arichat map)

Source: (Google Earth, 2012)

4.1.2. Model Subcatchments

The water system in Arichat, Isle Madame is developed in SWMM by applying the junction-links model to the pipe system there. The developed model is shown in Figure 4.2. The area is divided into 13 subcatchments from “S1” to “S13”. As noted in Figure 4.2, subcatchments S1 to S6 lie between the High Road and the high point of the ridge overlooking the Finger Lakes. These subcatchments above the High Road are located in a residential area of relative low density of houses and buildings. Subcatchments S7 to S13 are defined in the space between the High Road and the Lower Road and are more developed areas with residential and commercial usages.

Figure 4.2. Arichat water system model in SWMM with subcatchments S1 to S13 indicated

Subcatchment properties are shown in Table 4.1 below for the case of subcatchment S3 along the High Road. Each of the subcatchments receives precipitation from “Gauge1”. Since the area under investigation is relatively small, the precipitation is assumed to be the same in the different subcatchments of the whole model. Therefore, in this model only one gauge is applied to the whole area for injection of precipitation data to the water system. These precipitation data are defined as time series in SWMM.

Table 4.1. Sample subcatchment properties:S3

Property	Value
Name	S3
Rain Gauge	Gauge1
Outlet	j3
Area	23.89 ha
Impervious (%)	25 %
Land uses	2 (High Road area, “less developed”)

In the subcatchment property table of Table 4.1, junction "j3" is considered as the "Outlet" for subcatchment S3 which receives the catchment runoff. In the Arichat model, subcatchments S1 through S13 are connected to the junctions: j1, j2, j3, j4, j5, j16, j6, j8, j10, TreatmentPlant, j13, j13 and j15, respectively.

4.1.3. Arichat Map Geo-referencing

SWMM has an auto-length option, which determines the length of the pipes ("conduit") and area of subcatchments automatically when real geo-referenced coordinate values are provided in the model data. In this regard, the upper-right and lower-left X-coordinate and Y-coordinate are defined in the "dimensions" window in SWMM. In order to find these values, the map scale is used to find the length of each pixel of the background image used for this model. This scale can be found on the left bottom of the backdrop image in Figures 4.1 and 4.2 above. The length of the scale line on the map is 2680 feet (or 817 m) as noted on the image scale. Using the Photoshop software (Photoshop, 2013), the number of pixels of this line is determined as 291 pixels. Therefore based on the ratio calculation 2680 feet/291 pixels, each pixel in this image is 9.21feet/pixel (2.8 meters/pixel).

Now by finding the number of pixels on the length and width of the image it is possible to find the real values for these lengths. Photoshop was used to determine the pixel values for the background image. The length of the image is 1440 pixels and the width is 817 pixels. As noted above, each pixel is 2.8 meters. Therefore, the length of the map image (Figure 4.1 and 4.2) is determined as 1440 pixels in length or 1440 pixels times 2.8 meters/pixel or 4032 meters. Similarly, the width of the map image is determined as 817 pixels in length or 817 pixels times 2.8 meters/pixel or 2287.6 meters.

The lower-left X-coordinate and Y-coordinate are considered as the map origin or point (0, 0). Based on this assumption, the upper-right X-coordinate and Y-coordinate is point (4032, 2287.6). With these values, the auto-length option in the software is turned "on", and the real length of the pipes and real area for subcatchments are determined automatically from the map grid.

4.1.4. Subcatchment Impervious Characteristics

As noted in Chapter 3, subcatchments are characterized by their pervious and impervious natures. In general, the impervious percentage (impenetrable proportion of land with respect to water run-off) for the upper model subcatchments (i.e., subcatchments S1 through S6 above the High Road) that are comprised mainly of fields and undeveloped terrain, is set at 25% representing the estimate of precipitation that can be considered to flow off the soil and into the sewer system (through manholes) or the natural stormwater system (in culverts and drains). The higher subcatchments are located in more undeveloped area and this causes more ground infiltration and respectively a lower impervious percentage. The impervious value is higher (estimated at 50%) for the lower coastal subcatchments (S7 through S13) along the Lower Road because these subcatchments are more residential and contain paved or graveled driveways, parking lots and asphalt roads.

4.1.5. Water Quality and Common Pollutants

Before assigning land uses to the subcatchments, each subcatchment needs to be defined in terms of water quality and pollutants present. Unfortunately, there are insufficient data available regarding the status of water quality and the presence of pollutants in Arichat. To compensate for these data deficiencies, data applied to the Arichat model is taken from standard water system water quality examples provided in the SWMM manual (Rossman, 2010). In this example, two types of common water borne pollutants are introduced, TSS (Total Suspended Solids) and Lead, a co-pollutant of TSS. The co-fraction of TSS for Lead is considered as 0.25 in this model applied as an estimate (as stated in the SWMM manual example).

4.1.6. Land Use

Buildup and wash-off of pollutants from each subcatchment are assigned through the land uses option on the subcatchment properties table. Two types of land uses are defined in this model: residential and undeveloped. For residential land use, the exponential function is chosen as the build-up function for TSS with maximum build up equal to 75 kg per unit of normalizer with the normalizer being the area of the subcatchment, e.g., as noted in Table 4.1 for S3. Normalizer is the variable to which TSS build up is normalized on a per unit basis. The constant rate is 1 for this equation. The Wash-off function for TSS in residential land use is an Event Mean Concentration (EMC) function with 100 as its coefficient. For undeveloped subcatchments areas, there is no function defined for build-up and wash-off function as no such pollutants (e.g., TSS) are produced in these kinds of areas. As mentioned before, the subcatchments on the High Road are primarily undeveloped terrain and the subcatchments in between the upper and the lower road are mostly residential. Taking this into account, land use percentages can be defined for each subcatchment. Residential percentages will be stated here. Subcatchments S1 through S13 have their residential percentages equal to 10%, 40%, 25%, 25%, 30%, 30%, 30%,75%, 75%, 80%, 80%, 75% and 75%, respectively.

4.1.7. Model Junctions and Storage Units

The Arichat water system has 95 manholes (Municipality of the County of Richmond, 2003). Elevenof these manholes are used for the description of the Arichat water system. These 11 manholes are shown as junctions in this model (Figure 4.2). Five other junctions are represented as storage units that are used to represent inlet of pumps to the model. In this model, the pipeline along the High Road of Figure 4.2 is defined as being on a rise with specific elevations and slopes relative to the coastal area. As such, the modelled system, like the actual case for Arichat, uses gravity whereby water which comes from catchments at higher areas goes through connections ("j1" through "j16") between upper (higher) and lower (near coast) pipes. At the end of the water system, sewage water exits the system through the outlet ("TreatmentPlantOutfall (TPO)") corresponding to the town's treatment plant in Arichat harbor at the junction near LeNoir Forge (Figure 4.2).

Junctions and storage units are the most important components in this model. The most significant property of a junction and a storage unit is its elevation. For the High Road junctions' elevation, two sets of data are investigated in this regard. The first set is taken from Tiennah et al. (2011) where a Real Time Kinematic (RTK) GPS survey was undertaken on Isle Madame, Nova Scotia. The second set is observations from a hand held GPS. Both datasets are used to establish elevations for different points in Arichat. In both of the sets, selected coordinates are specified for each point. The two sets of data are compared. Common points were found in order to compare and contrast the different elevation estimates. Four points were found in common based on the longitudes and latitudes, as shown in Table 4.2. The Treatment Plant is set as the benchmark of the system with the elevation of the Treatment Plant considered as 0, or the same as the sea level. Other point elevations are considered as relative elevations to the Treatment Plant benchmark elevation level. It is considered, despite their predicted higher accuracy that the relative elevation for the limited RTK point values recorded in Tiennah et al. (2011) (Table 4.3) appear to be different from expectations. Therefore, given the more complete set of the hand held GPS data, these values (Table 4.2) are chosen as the applicable data for higher road junctions' elevations. In the Arichat SWMM simulation model, the relative elevations of Table 4.2 were used as the main data in the model.

The Municipality of County of Richmond in which Arichat is located have provided documents regarding the Arichat pumping stations site location plan, as well as the Arichat pumping station electrical plans sections and details (CBCL Limited, 2010). For the Lower Road invert elevations, the technical data available in these documents are used to define the junctions' characteristics. Invert elevation, maximum depth and surcharge depth for junctions and storage units are all available in these documents. Table 4.4 presents the junction's properties for junction j11 (Figure 4.2). Table 4.5 presents the storage unit properties for junction j10 (Figure 4.2).

On the Lower Road, 5 pumping stations exist. Each pumping station has an inlet and an outlet. Outlets are represented by junctions and inlets are represented by storage units. In the documents provided by the Municipality, inlets are introduced as wet wells. In sewage systems which contain pumping stations, wet wells are used to store sewage and then when the level of sewage increases to a specific level in the wet well, the pump will start working and will pump up the sewage, stopping when a specific low level is achieved. In SWMM, wet wells can be shown as storage units. Therefore, it is obvious that the elevation of each junction should be higher than the elevation of each storage unit in a pumping station so the pump can lift the water from the lower elevation to the higher elevation.

The only junction which has an extra property is the "TreatmentPlant" junction. In order to simulate Arichat's treatment plant capacity, a "ponded area" is used for this junction. The ponded area for this junction is considered equal to $100m^2$ and for other junctions this area is 0. The initial depth of each junction is considered as a controllable variable for scenarios which can be either 0 or equal to the maximum depth.

Table 4.2. Hand held GPS data used for the simulation

Points	Description	GPS Long-Lat	GPS Elevations(m)	Relative GPS Elevations(m)
1	Near Dede and Thomas Boudreau's house	45 30.486 -61 02.308	9.144	2.74
2	Michel Samson's house	45 30.576 61 01.700	7.31	0.91
3	Funeral Home	45 30.656 61 01.013	6.4	0
4	Sewer system-Lenoir forge(benchmark)	45 30.649 61 00.752	6.4	0
5	Herman Samson's house	45 30.730 61 00.179	7.31	0.91
6	Before the two wharves	45 30.766 60.59.516	6.7	0.3
7	Corner of intersection with the church	45 30.615 61 01.976	28.96	22.56
8	Jeanties mini mart	45 30.649 61 01.723	24.08	17.68
9	Dan Mac Hardware store	45 30.648 61 01.616	20.73	14.33
10	Pharmacy	45 30.652 61 01.526	21.94	15.54
11	Old liquor store	45 30.658 61 01.436	24.69	18.29
12	Shirley's spud wagon	45 30.692 61 01. 288	22.86	16.46
13	Anglican church	45 30.704 61 01.116	23.47	17.07
14	Post office	45 30.722 61 00.971	22.55	16.15
15	Court house	45 30.725 61 00.886	21.94	15.54
16	Sporty's	45 30.740 61 0.616	26.51	20.11

Table 4.3. RTK survey and hand held GPS elevations and their relative elevations

Points	Description	RTK long-lat	GPS Long-Lat	RTK Elevations(m)	GPS Elevations (m)	RTK Relative Elevations (m)	Hand Held GPS Relative Elevations(m)
1	Near Dede and Thomas Boudreau's house	45 30 29.16 -61 02 18.58	45 30.486 -61 02.308	-9.0	9.144	3.52	2.74
2	Michel Samson's house	45 30 34.21 61 1 41.27	45 30.576 61 01.700	-10.763	7.31	1.76	0.91
3	Funeral Home	45 30 39.29 61 1 0.70 And 45 30 39.31 61 1 0.83	45 30.656 61 01.013	-10.9 (average of two close points)	6.4	1.62	0
4	Sewer system-Lenoir forge	45 30 36.70 61 0 42.67	45 30.649 61 00.752	-12.521	6.4	0	0

Table 4.4. Model Sample junction properties: j11

Properties	Value
Name	j11
Invert elevation (m)	1.64
Maximum depth (m)	4.274
Initial depth (m)	0
Surcharge depth (m)	1.683
Ponded area (m^2)	0

Table 4.5. Sample storage unit properties: j10

Properties	Value
Name	j10
Invert elevation (m)	-0.92
Maximum depth (m)	4.347

4.1.8. Model Conduits

Other important components of the Arichat model are conduits (pipes) which are the connections between the junctions. The three available properties for pipes are: (i) pipe diameter which is shown as the maximum depth of the pipe in the software; (ii) the length of pipe; and (iii) the shape of each pipe. For pipes along the High Road (“c1” to “c4”), the Lower Road (“c10” to “c14” and “c15”) and the pipes connecting lower and higher road pipes, the diameter of the pipes is 0.1524 meters (6 inches), 0.2560 meters (10 inches) and 0.7112 meters (28 inches), respectively (Boudreau, 2013). The connecting pipes which connect high to low road pipes are considered as culverts, that is why their diameter is considered larger than normal. The length of each pipe is defined according to the auto-length option in SWMM. Shape of the all the pipes is circular (Boudreau, 2013). Table 4.6 shows conduit c7 properties as an example.

Table 4.6. Sample conduit properties: c7

Property	Value
Name	c7
Inlet junction	J3
Outlet junction	J10
Shape	circular
Max. depth	0.7112 meters
length	156.41
Flap gate	No

4.1.9. Pumps

In the water system in Arichat there are 5 pumping stations. As explained in section 4.1.7, the pumping stations each have a junction as the outlet and a storage unit as the inlet point of the station. Similar to all other components, pumps have different properties. In the documents provided by the Municipality of Richmond County, the technical properties of the specific pumps are available. Based on the specific codes provided for each pump in these documents, it is possible to find pump performance curves. The manufacturing company of the pumps has provided these curves on its website (Xylect Professional, 2013).

Properties of a sample pump, p3, are shown in Table 4.7. Pump p3 performance curve is also shown in Figure 4.3.

Table 4.7. Sample pumps properties: p3

Property	Value
Name	P3
Inlet junction	j10
Outlet junction	j11
Pump curve	P3
Initial status	ON
Startup depth (m)	1.524
Shutoff depth (m)	1.067

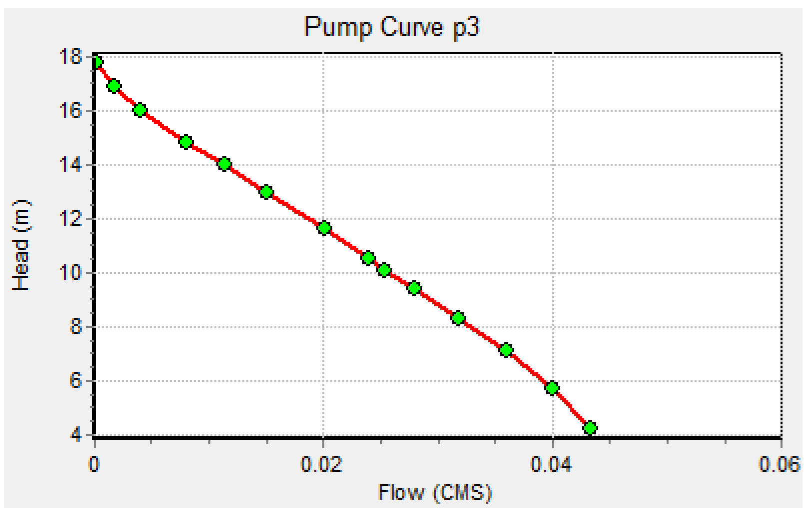

Figure 4.3. A sample pump, p3, performance curve

In Figures 4.4 and 4.5, images of pump stations are shown to give an actual view of the area. The location of each manhole and each pumping station is defined with the help of the longitude and latitude from the handheld GPS. These coordinate values are found in Table 4.2 above. These values are then matched with the background image in this model. The pictures shown in Figure 4.4 and 4.5 are of the 5 pumping stations in Arichat in addition to pictures of the treatment plant facilities in Arichat. As noted in the illustrations, these stations are recently installed new stations for the delivery of Arichat sewage water to the Treatment Plant near LeNoir forge.

4.1.10. Regulators

In coastal zones, storm surge affects the stormwater system. As the storm generates increased precipitation, there will be potential stormwater backups in the system if flow capacity is exceeded with the result being flooded junctions and possibly flooded basements. In order to prevent and control these backups, 6 regulators and 6 outfalls (to Arichat Harbour directly) are included in the modeled Arichat water system. Therefore, whenever junctions j6, j8, j10, j13, j15 and the treatment plant reach flood status from the incursion of stormwater, the regulators help prevent backups as they have gates that are triggered to open automatically to the outfall when the local pipes and junctions reach a pre-specified level of capacity. These regulators and controls apparently do not exist in the actual system currently in place in the Arichat water system. These controls are added to the model in order to model their existence as potentially helpful to prevent backups and flooding. Another reason to use these regulators in the model is that in SWMM, it is not possible to show the connection of several conduits to a junction. On the other hand, the documents provided by Municipality of Richmond County for pumping station plans (CBCL Limited, 2010) show an extra connection to the storage units, an overflow which directs water into the Arichat harbour at a specific height of water in the storage unit. In order to show this connection, regulators are helpful representations. The height of the regulator can be considered as the specific height provided in the pumping station plan and the length of the regulator can be considered as the diameter of the connection. Based on these plans, the height for each regulator is 2.591 meters and the remaining height above this height will be considered as the surcharge depth for the outlet junction of the pumping station (refer also to section 4.1.3). The length of each regulator is equal to 10 inches (0.254 m) based on the real observations. Based on the technical data provided (Municipality of Richmond County, 2007), all the regulators existing in the real system are considered in SWMM as “weirs” of type “V-NOTCH”, as consistent with the technical data provided. Therefore, all the added regulators are considered similar to other regulators in the system. As mentioned in Chapter 3, V-NOTCH regulators have a standard discharge coefficient equal to 1.35 CMS. Table 4.8 shows R3 properties as a sample regulator.

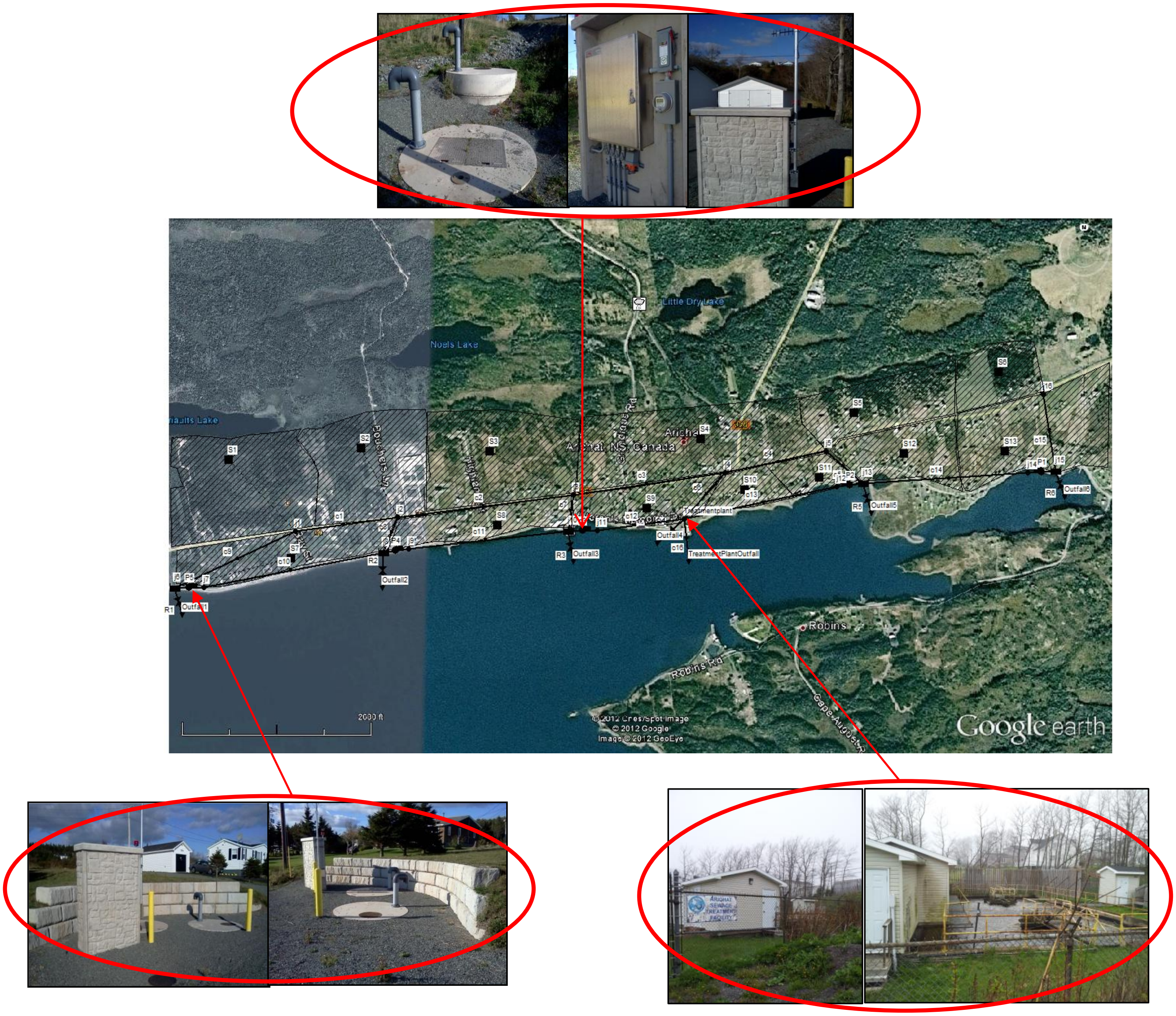

Figure 4.4. Arichat developed model with related pictures-part 1

Figure 4.5. Arichat developed model with related pictures-part 2

Table 4.8. Sample regulators properties: R3

Properties	Value
Name	R3
Inlet junction	j10
Outlet junction	Outfall3
Type	V-NOTCH
Height (m)	2.591
Length (m)	0.254
Discharge Coefficient(CMS)	1.35

It is also possible to define control rules for pumps and regulators in SWMM. Rules can be defined in a way to let the valves be open permanently, be closed temporarily or permanently and also they can be half-open in some specific situations or permanently be half open. These control rules are helpful to define the scenarios and are discussed in further detail below (section 4.2.1 below).

4.1.11. Outfalls

Outfalls are components in the Arichat water system that are the connection between the system and the open water of the Arichat harbour. The treated or untreated water flows into the harbour through these outfalls. The outfall connected to the Treatment Plant is the main outfall in this model. When there is no storm or any unusual precipitation in the system, the system would work normally which means the runoff from the subcatchments flows into the system, and the water will flow to the "TreatmentPlant" junction, the location of the Treatment Plant. The water is treated at the plant and then the treated water will flow to the "TreatmentPlantOutfall".

There are six other outfalls which are connected to the regulators. In case of overload in any of the storage units in the system, untreated water will flow into the harbour through these outfalls based on the control rules applied on the related regulators. Since these outfalls are the connections to the harbour, they directly sense the ocean tidal levels. Therefore, in the property table for outfalls there is a choice to choose tidal outfalls. The type of the outfalls other than "TreatmentPlant Outfall" is chosen as "tidal" and can be defined by a curve. The curve shows the relation between the sea level and the time of the day. Initial tidal data were applied hourly tide levels provided by Fisheries and Oceans Canada for North Sydney located in Nova Scotia and used as proxy tides for Arichat habour. Table 4.9 shows Outfall1 properties as an example. Since the treatment plant is usually the most downstream end of the collection system, it is represented as an outfall junction with a "fixed head" (Rossman, 2013). Therefore, "TreatmentPlantOutfall" type is a fixed head outfall with fixed stage equal to 0 instead of being an outfall with a curve. Moreover, as it is the outfall connected to the Treatment Plant, it should have the treatment option in its properties as mentioned previously in Chapter 3. In order to turn on the treatment option, the following treatment plant expression is used for TSS pollutant in this outfall:

$$C = 0.523 \times TSS^{0.5} \times FLOW^{1.2}$$

This equation is an example used in the SWMM model for water quality investigations (Rossman, 2010). This example is used because there are not enough data available for the treatment plant in Arichat. The invert elevation for each outfall is considered to be equal to the invert elevation of each storage unit, so it will be lower than the height of the regulator where water flows into the outfall. When the water level in the outfall gets to this height in the connected storage unit, it flows out into the harbor.

Table 4.9. Sample outfall properties: Outfall3

Properties	Value
Name	Outfall3
Inflows	NO
Treatment	NO
Invert elevation (m)	-0.92
Tide gate	NO
Type	Tidal
Curve name	Depending on the Scenario a Tidal Time Series is used as the tidal curve for an Outfall

4.2. Arichat Hydrologic Settings and Inputs for Applying Different Scenarios in SWMM

After modelling the Arichat stormwater system, different storm scenarios may be applied to the system. These scenarios can be defined by different precipitation intensity, different time series for rainfall gauges, different tide levels, and different properties for each component in the system. Uncontrollable and controllable variables can be defined in all these scenarios. In this section controllable and uncontrollable variables are introduced with their different alternatives to define the simulation scenarios.

In these simulations, control rules for regulators are considered as the controllable variables. Uncontrollable variables are introduced as precipitation intensity, tidal level, initial depth of the junctions, and impervious percentage for higher and lower subcatchments. For each of these variables different parameter values are provided to define the scenarios.

4.2.1. Controllable variables: Regulators

As mentioned above, regulators are considered as controllable variables. There are 6 regulators that can have 2 alternatives status settings: closed or open. In this research, all regulators are considered to have the same situation at all times, i.e., the 6 regulators are open or closed simultaneously. It means all the regulators obey the same control rule which makes them open or close. For each regulator, the depth of the storage unit is used as the factor that triggers opening closed regulators. This depth is equal to the height of the regulator. If water gets to this depth in the storage unit or more, then the regulator will open and water will flow directly to the harbour through the connected outfall. Resetting the weir of the regulator equal to 1 changes the regulator status from closed (=0) to open (=1). The control rules defined for opening regulators are the same for all regulators. j6, j8, j10, TreatmentPlant, j13, and j15 move from the initial status of closed to open when the depth of the node is greater than or equal to 2.591m. The SWMM code for these control variables are provided in Appendix B– SWWM Model Regulator Rules Definition.

The control rules for defining “closed” alternative for regulators is the same as above, but instead of 1 in front of the second line of each rule, one should consider 0, which is provided in Appendix B. It means that the regulators are closed at all times.

4.2.2. Uncontrollable variables

4.2.2.1. Precipitation

Uncontrollable variables play important roles in the simulation of this system. The precipitation data for running the Arichat water model are the precipitation time series provided by Environment Canada, Atlantic operations for Tracadie, Nova Scotia. Tracadie is used as a “proxy” for Arichat given that these data, presented in an Excel file, are the closest location to Arichat among the available data. The data are hourly data which start at 2:00pm on the 14th of April, 2004, and end at 4:00 AM on the 22nd of October 2011. The average monthly precipitation for these 7 years from these data are shown in Table 4.10 below.

Table 4.10. Monthly average precipitation for the seven-year data set, 2004 to 2011, considering 0 mm precipitations.

Source: (Environment Canada, 2012).

Number	Month	Average hourly Precipitation (mm)
1	January	0.13181338
2	February	0.099889
3	March	1.6
4	April	0.102839
5	May	0.09995
6	June	0.118096
7	July	0.116167
8	August	0.140054
9	September	0.103023
10	October	0.174237
11	November	0.159434
12	December	0.187238

Over the 7 year period, the maximum precipitation took place in March during these 7 years and the minimum precipitation took place in the month of February. The related histograms (Figure A1 and Figure A2) for these two months are found in Appendix A – Precipitation Data Summary.

In SWMM, a 24-hour time series is defined to introduce each precipitation alternative. Four alternatives are considered in the simulation model for applying the precipitation data. The first alternative is "Off" which shows a minimum precipitation in a 24-hour simulation in the system. The minimum hourly value for the precipitation in the seven years data is 0.3mm/hour. Therefore each hour in the "Off" scenario has precipitation equal to 0.3mm.

The second alternative is designated as "P1" which is the historical data for the maximum amount of precipitation in a single 24-hour dataset. To find the desired 24-hour with the maximum precipitation in the Excel file, the precipitation for all consecutive 24 hour periods was analyzed. From the Environment Canada dataset the maximum precipitation amount was found to be 86 mm overall. This 24-hour period started from 4:00 AM on 14^{th} of December 2010, to 3:00 AM on 15^{th} of December 2010. The exact data for this 24 hour period were used as the data for the "P1" case. In Table 4.11, the "P1" time series is shown. Moreover, related graphs can be found in Figure 4.6. In the third case, "P2" the same total precipitation data in "P1" are used but are spread out evenly over the 24 hour period. It means the overall precipitation is still 86mm but each hour the precipitation is 86/24=3.59 mm.

The fourth and the last precipitation alternative is "P3". For this case, the maximum amount of precipitation for an hour during the seven years of precipitation data was found. This maximum amount was 26.4 mm which took place at 9:00 AM on 26^{th} of June 2007. The next step was to front load a 24-hour period based on this maximum amount of precipitation by keeping the overall maximum precipitation for one 24-hour equal to the historical maximum of 86 mm. In this case, the first three hours are assigned 26.4 mm each, and the fourth hour receives 6.8 mm. For applying these data to the Arichat SWMM model, in order to match all the data, the timing used in the software is started from 00:00 and ends at 23:00 the same day.

Table 4.11. Second precipitation case “P1” :The historical data for the 24-hour period with the maximum precipitation, with 86mm for overall precipitation

Source: (Environment Canada, 2012)

Time	Value(mm)	Time	Value(mm)
4:00	0.6	16:00	1.7
5:00	4.7	17:00	0.3
6:00	0.6	18:00	0.8
7:00	7.2	19:00	3.8
8:00	2.8	20:00	3.4
9:00	1.7	21:00	3
10:00	8.9	22:00	4.8
11:00	9.6	23:00	0.4
12:00	2.5	00:00	1.5
13:00	5.4	01:00	4.1
14:00	6.8	02:00	6.6
15:00	4.2	03:00	0.6

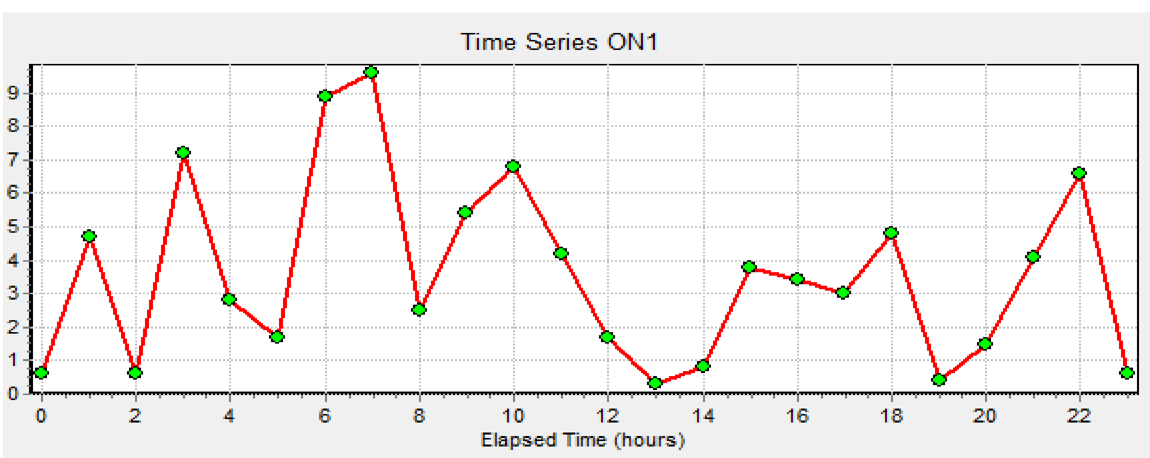

Figure 4.6. Second precipitation case “P1”: The historical data for the 24-hour period with the maximum precipitation. Total maximum rainfall for overall precipitation is 86mm

Source: (Environment Canada, 2012)

4.2.2.2. Tidal level

The second uncontrollable variable is the tidal level. The tidal data were obtained from hourly tide levels provided by Fisheries and Oceans Canada (2012) (Fisheries and Oceans Canada, 2012). The hourly tidal time series is actually for North Sydney, Cape Breton as a proxy for Arichat. These data are also available in an Excel file format. The data start at 14:00, 19th of April, 2004 and end at 4:00, 22th of November, 2011. For tidal levels, four cases are defined based on: (i) the highest high tide, (ii) the lowest low tide, (iii) the average tide levels starting from a low tide, and (iv) the average tide levels starting from a high tide.

For the first case, among the whole tidal data, the maximum tide level is found and considered as the highest high tide and then the 24-hour historical data, which includes this maximum high tide, are chosen for the 24 hour input data to define the tidal simulation scenario. The maximum tidal level was 2.158 meters that occurred at 22:00, 30th of October, 2011. The data table for the highest high tide values is presented in Table 4.12 and is illustrated in Figure 4.7.

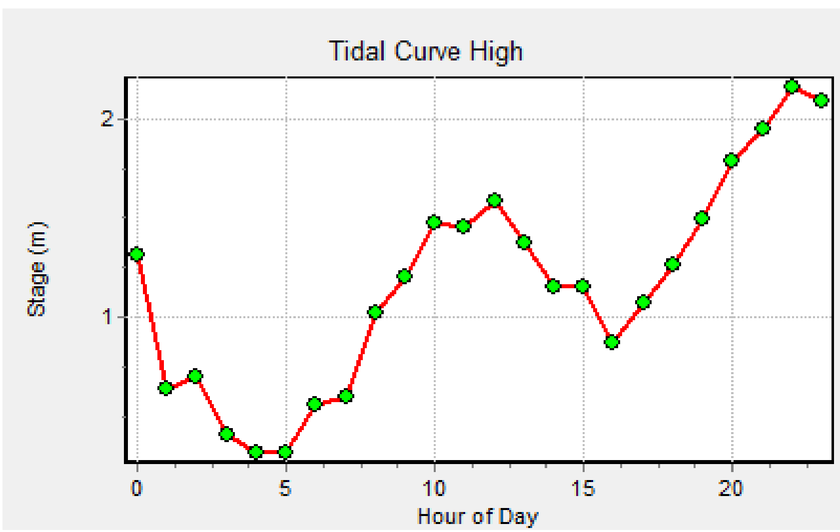

Figure 4.7. Tidal level case number 1: Historical highest high level tide

Source: (Fisheries and Oceans Canada, 2012)

Table 4.12. Tidal level case number (i): Highest high level tide, based on 2004-2011 data

Source: (Fisheries and Oceans Canada, 2012)

Time	Level(m)	Time	Level(m)
00:00	1.311	12:00	1.589
01:00	0.942	13:00	1.376
02:00	0.705	14:00	1.149
03:00	0.414	15:00	1.149
4:00	0.32	16:00	0.873
5:00	0.322	17:00	1.069
6:00	0.566	18:00	1.266
7:00.	0.597	19:00	1.491
8:00	1.028	20:00	1.79
9:00	1.201	21:00	1.95
10:00	1.472	22:00	2.158
11:00	1.455	23:00	2.091

For the second tidal case, among the whole dataset, the minimum tide level is found and considered as the lowest low tide and then the 24-hour historical data, which includes this minimum number, is chosen for the 24-hour input data for the simulation. The minimum tidal level for the dataset is -0.278 meters and it occurred at 16:00 21th of March, 2007. These data are shown in Table 4.13 and illustrated in the graph of Figure 4.8.

Table 4.13. Tidal level case number (ii): Lowest low tide level, based on 2004-2011 data. Source: Fisheries and Oceans Canada (2012)

Time	Level(m)	Time	Level(m)
00:00	0.967	12:00	0.734
01:00	0.797	13:00	0.405
02:00	0.505	14:00	0.008
03:00	0.221	15:00	-0.243
4:00	0.256	16:00	-0.278
5:00	0.337	17:00	-0.234
6:00	0.528	18:00	0.094
7:00.	0.721	19:00	0.257
8:00	1.022	20:00	0.732
9:00	1.098	21:00	0.917
10:00	1.166	22:00	1.112
11:00	0.961	23:00	1.107

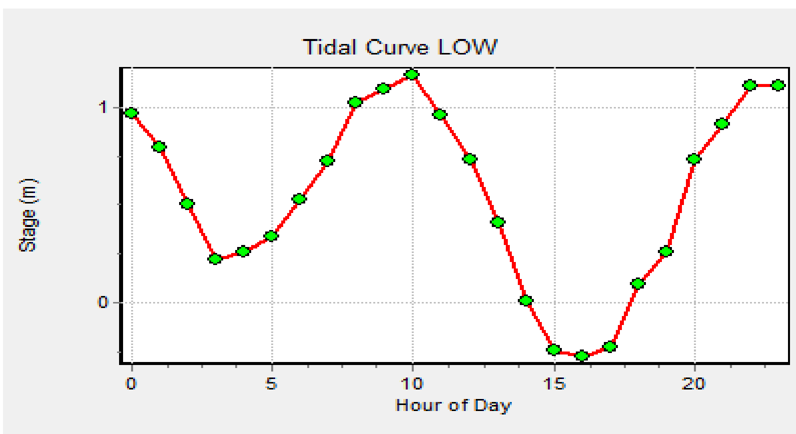

Figure 4.8. Tidal level case number (ii): Lowest low level tide, based on 2004-2011 data Source: (Fisheries and Oceans Canada, 2012)

For the third and fourth cases, the average maximum high tide and the average maximum low tide are found. The IF function was used in the Excel data file analysis to find the local maximum and local minimums in the data set by comparing a datum with its two neighbors. The average for both of them was then calculated. On the next step, the 48-hour period which includes both average low tides and average high tides are found which were 0.625m and 1.101 m respectively. In this 48-hour period, two 24-hour periods are selected. Each of which includes 0.625m and 1.101m values. One of these 24-hour periods starts with a low tide at the beginning, and the other starts with the high tide at the beginning. This first case is labeled as “Mean/Lowfirst” starts at 13:00, 28th of July, 2007 and ends at 12:00, 29th of July 2007. The second one is labeled as “Mean/Highfirst” starts at 5:00, 28th of July 2007 and ends at 4:00, 29th of July 2007. Tables 4.14 and 4.15 show the related data. Also Figure 4.9 and 4.10 show the related graphs. The time is shifted to the first hour of the day to match the precipitation time series.

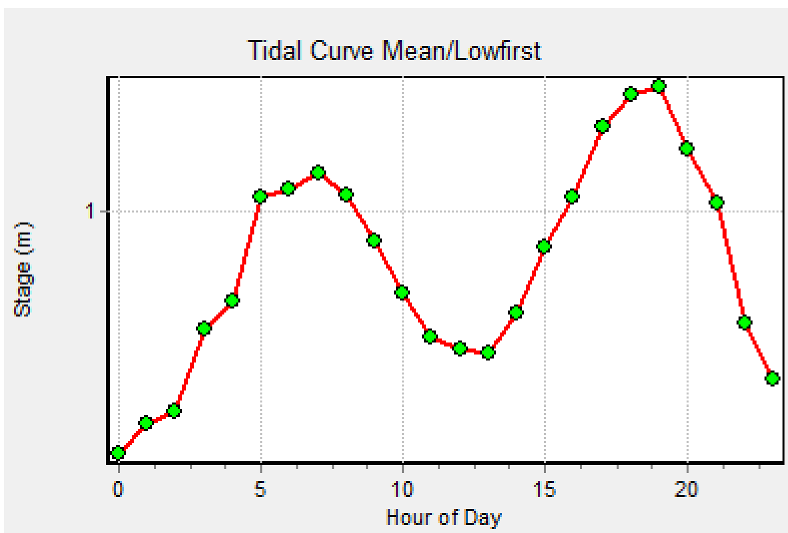

Figure 4.9. “Mean/Lowfirst” tidal alternative

Source: (Fisheries and Oceans Canada, 2012)

Table 4.14. "Mean/Lowfirst" tidal alternative

Source: (Fisheries and Oceans Canada, 2012)

Time	Level(m)	Time	Level(m)
00:00	0.363	12:00	0.635
01:00	0.441	13:00	0.625
02:00	0.471	14:00	0.733
03:00	0.689	15:00	0.904
04:00	0.764	16:00	1.035
05:00	1.035	17:00	1.223
06:00	1.06	18:00	1.308
07:00	1.101	19:00	1.327
08:00	1.045	20:00	1.163
09:00	0.92	21:00	1.021
10:00	0.783	22:00	0.703
11:00	0.667	23:00	0.557

Table 4.15. ”Mean/Highfirst” tidal alternative

Source: (Fisheries and Oceans Canada, 2012**)**

Time	Level(m)	Time	Level(m)
5:00	1.182	17:00	0.764
6:00	1.253	18:00	1.035
7:00.	1.28	19:00	1.06
8:00	1.239	20:00	1.101
9:00	1.029	21:00	1.045
10:00	0.893	22:00	0.92
11:00	0.6	23:00	0.783
12:00	0.494	00:00	0.667
13:00	0.363	01:00	0.732
14:00	0.441	02:00	0.635
15:00	0.471	03:00	0.625
16:00	0.689	04:00	0.733

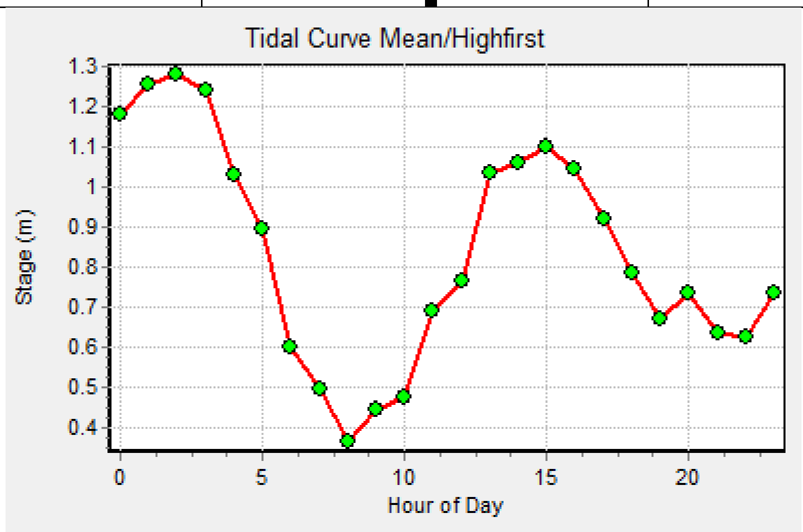

Figure 4.10. “Mean/Highfirst” tidal alternative

Source: (Fisheries and Oceans Canada, 2012)

4.2.2.3. Initial Depth

The third uncontrollable variable is the initial depth in the lower road junctions. As mentioned in section 4.1 above, the initial depth can be 0 meaning “dry”, i.e., at the time of starting the simulation there is no water in the junctions and storage units. The other case is to be equal to the height of the regulators at its worst case. As mentioned in section 4.1, the height of the regulator shows the water level that triggers the regulators to open. If the initial depth is equal to the height of the regulators, the length of time of pumping will increase.

4.2.2.4. Impervious Percentage

The fourth and the last uncontrollable variable is the impervious percentage for the lower and the higher subcatchments. By choosing this variable as one of the uncontrollable variables, we determine the amount of water which is assumed to flow into the sewage system through manholes, and the amount of water which infiltrates into the ground. The combination of the impervious percentage for the Higher Road and the Lower Road are defined as alternatives for this variable. For example, when the High Road impervious percentage is 25%, the Lower Road impervious percentage is set to 50%. Alternatively, when the High Road percentage is 50%, the Lower Road percentage is set at 75%. These different values for higher and lower impervious percentage are attributed to residential versus undeveloped land use of each subcatchment and are estimates of parameter values for which no empirical values are available. These estimates are assigned in order to reflect known SWMM outcomes for historical storm events.

The historical storm event of August 26, 2010 was used throughout the simulation analysis to help estimate these parameters by calibrating the model results under different parameter values to the well-known storm impacts, flooding events, road closures, etc. In this manner, validation of the simulation results was matched to the most appropriate and reasonable parameter values in the absence of actual data.

The controllable and uncontrollable variables combined, ten scenarios are defined for the simulation of the water system in Arichat. These scenarios are different combinations of controllable and uncontrollable variable alternatives. Applying each scenario to the developed model will result in status reports, graphs, tables and profile representations. The application and results of each of the scenarios are discussed in Chapter 5.

5. Results and Analysis

Chapter 5 contains three (3) main sections, namely: (5.1) Simulation Design; (5.2) Scenario Results; and (5.3) Analysis of the Results. In this chapter, ten simulation scenarios, their definitions and their inputs are discussed in 5.1. Results achieved from the simulation of these scenarios are discussed in section 5.2. Simulation run for each scenario results in different outputs. These outputs are analyzed in order to investigate the system's capacity, and to consider water quality (section 5.3).

5.1. Simulation Design

Based on the controllable and uncontrollable variables introduced in Chapter 4, different scenarios can be investigated with the combination of alternatives of controllable and uncontrollable variables for the purpose of this research. In this research 10 scenarios are introduced. Each scenario focuses on one or two of the variables. The objective is to use the SWMM simulation model to investigate the water quality and the capacity of the Arichat water system under alternative setting for the controllable and uncontrollable variables. With regard to water quality, the amount of untreated water entering the harbor is a factor that should be considered. Also, the number of junctions and storage units getting flooded is also a factor for analyzing and understanding the limits of the system's capacity. Table 5.1 presents the definitions of the 10 simulation scenarios of this analysis. Each scenario is discussed in more detail below.

5.1.1. Status Quo Scenario

The Status Quo (SQ) scenario is the first scenario of Table 5.1 and is considered as the benchmark scenario of the simulation model. Results of all other scenarios are compared to the results of this benchmark or "base case" scenario to investigate their differences. The Status Quo scenario definition is given in Table 5.1 by the variable values as follows:

- Regulators (designated in the model as R1 to R6, see also section 4.1.6 above) are denoted as the controllable variables of the simulation. For the Status Quo scenario, all Regulators are open, based on the control rules discussed in Chapter 4 as one of two alternatives for regulators' status.

Table 5.1. Simulation design:Scenarios description

Scenarios / Variables	1 SQ	2 Best Case	3 Worst Case	4 Precipitation Focus	5 Tide Focus	6 Tide & Initial Depth Focus	7 Regulators & Precipitation Focus	8 Tide & Impervious% Focus	9 Depth & Precipita-tion Focus	10 Depth & Impervious% Focus
Controllable Variable										
Regulators (R1-R6)	Open	Closed	Open	Open	Open	Open	Closed	Open	Open	Open
Uncontrollable Variable										
Precipitation (mm)	Off	Off	P3	P3	Off	P1	P3	P2	P3	P3
Tidal Level(m)	Mean/Low	Low	High	Mean/Low	High	High	Mean /Low	High	Mean /Low	Mean /High
Initial Depth(m)	0	0	2.591	0	0	2.591	0	0	2.591	2.591
Impervious percentage (Higher Road Catchments)	25%	25%	50%	25%	25%	25%	25%	50%	25%	50%
Impervious percentage (Catchments between higher and lower Road)	50%	50%	75%	50%	50%	50%	50%	75%	50%	75%

- With respect to the uncontrollable variables for the Status Quo, Precipitation is at its lowest value which in this case designated as “Off” for the precipitation variable, i.e., the minimum (0.3mm per hour) rain or snow is recorded for this case.
- The Status Quo Tidal level as noted in Table 5.1 is set at an intermediate level. In this case, tides are designated as “Mean/Low”, which is the representation for the tidal level time series containing the mean historical value starting with low tide.
- The initial depth of the junctions and storage units is set equal to 0 meters. Zero means that there is no water in the system at the beginning point of the 24-hour simulation.
- The last variables are the impervious percentages for the subcatchments above the High Road, and the impervious percentage for the subcatchments between the High and Lower Roads. For the Status Quo scenario, these values are set at 25% and 50% respectively. These choices help to guide less water into the system in comparison with alternative impervious percentages for the subcatchments. These percentages are not based on empirical valuation, but are simple estimates of water entering the sewage system through the catchments.

5.1.2. Best Case Scenario

The Best Case scenario is that case in which less water is present in the system (from uncontrollable variables) and untreated water does not flow into the harbour. Also, Best Case is considered as such since it has a minimum number of junctions getting flooded and the number of hours of flooded junctions is negligible. Therefore, to define this scenario, all Regulators’ control rules are set on “close” at all times, and water is prevented from flowing directly into the ocean and harbor, except through the treatment plant outfall and after aeration and settling of the effluent. Low precipitation is considered for the system as “Off” (Table 5.1). The level of tides for this case is set at its lowest historical level, i.e., the “Lowest Low” tidal level is used for the Best Case definition (Table 5.1). The Initial depths of the junctions and storage units are set at zero, and the impervious percentages for subcatchments are at their lowest assumed values (25% for High Road subcatchments, and 50% for Lower Road subcatchments). This situation applies less pressure to the system and reduces the probability of flooding junctions.

5.1.3. Worst Case Scenario

The Worst Case scenario is the opposite of the Best Case. In this case, it is assumed that untreated water may enter the harbour and the number of junctions flooded and duration of flooding is high. In order to define this scenario, the Regulators are "Open" when the depths of storage units get to a specific depth (2.591 meters), which is the height of the Regulators, and the height of the outflow of the storage units (Table 5.1).

For Precipitation, the time pattern "P3" is chosen. P3 precipitation has the highest amount of rainfall in the first 4 hour period of the 24 hour simulation (refer to section 4.2.2.1). This significant precipitation places high pressure on the system from the beginning of the simulation. The Worst Case Tidal level is set as the "Highest high tide" that places further pressure on the junctions and outfalls of the system. Consequently, the model shows that initially, due to the tidal pressure, there will be a flow from the harbour back into the system. Initial depth is also considered at its highest value, which is equal to 2.591 meters as explained in Chapter 4, section 4.2.2.3. The impervious percentages are considered at their highest values that effectively puts more water into the system which are 50% for the highest subcatchments, and 75% for the lower subcatchments. By choosing these percentages, less water is assumed to infiltrate into the ground and more water will flow into the sewage system junctions.

5.1.4. Precipitation Focus Scenario

This Precipitation Focus scenario focuses on the singular impacts of significant precipitation on the system. All the variables except the precipitation have the same values as the "Status Quo" scenario described above. The results of this scenario will be compared directly with the result of the Status Quo scenario and the differences attributed to the significant precipitation. To define this scenario, the precipitation variable (P1, P2, or P3) was considered. The scenario was examined 3 times and each time precipitation case results among "P1", "P2" and "P3" compared. Then, the simulation that showed the highest incidence of junctions flooded and the release of more untreated water flowing directly into the harbor is considered as the most significant precipitation case for defining the Precipitation Focus scenario. The most significant precipitation variable on the water system was found to be P3 which resulted in 5 flooded nodes compared with no flooded nodes for P1 and P2 precipitation variables in their results. The simulation is run under the situations for the precipitation focus scenario considering P3 as the precipitation alternative for the Precipitation Focus scenario.

5.1.5. Tide Focus Scenario

In the Tide Focus scenario, the focus is on the tidal level. For this case, all other variables are set to have the same values as the SQ scenario with the exception of the tidal level variable. This scenario aims to analyze the effects of alternative tidal levels on the system. For the tidal level in this scenario, the same process for choosing the most significant precipitation variable is applied to find the most significant tidal level among the 4 tidal cases, namely: “High”, “Low”, “Mean/Low” and “Mean/High” tides. Running the simulation model under the different tidal level time series shows that none of the tidal cases result in flooded junctions, but the simulations with “High” tide time series results in surcharging 6 junctions. Thus, the most significant tidal level is designated as the “High” tidal level time series, and is chosen for this scenario for the tidal level alternative.

5.1.6. Tidal Level and Initial Depth Focus Scenario

This scenario focuses on the tidal level and the initial depth of the junctions at the same time. The precipitation case for this scenario is set as “P1”, corresponding to the historical time series for the August 24, 2010 storm that happened in Arichat area (Environment Canada, 2012). The tidal level here is considered as the most significant tidal level which is defined in section 5.1.5 above (i.e., “High” tide) to represent the focus of this scenario for the tidal level. The initial depth is the highest one, namely 2.591 meters to show the focus on the initial depth. Finally, impervious percentages and regulators status remain the same as the Status Quo scenario as noted in Table 5.1.

5.1.7. Regulators and Precipitation Focus Scenario

The Regulator and Precipitation Focus scenario is a scenario with focus on the variables for regulators (controllable) and precipitation (uncontrollable). For this scenario, the regulators are kept closed throughput the simulation thereby preventing the release of untreated water into the harbour. For precipitation, as investigated in section 5.1.4 above, the most significant precipitation case is P3. The remaining scenario variables for tidal level, initial depth and impervious percentages are set the same as for the SQ scenario defined in 5.1.1 above (Table 5.1).

5.1.8. Tidal Level and Impervious Percentage Focus Scenario

The Tidal Level and Impervious Percentage Focus scenario defines the most significant tidal level and the highest variable values for the impervious percentages. Therefore, the tidal level is set as the "High" tidal level time series. For this scenario, the impervious percentages are 50% for the higher subcatchments and 75% for the lower subcatchments. "P2" is the precipitation case for this scenario. The Initial depth of junctions is set at 0, the same as the SQ scenario, and regulators are "Open" in the simulation of this scenario. This scenario is designed to investigate the significance of the combined tide and runoff to the water system.

5.1.9. Initial Depth and Precipitation Focus Scenario

This Initial Depth and Precipitation Focus scenario combines the effects of the initial depth of the junctions, and the precipitation uncontrollable variables on the system. The difference between this scenario and SQ scenario is in the Initial Depth values which are set at the highest level of 2.591m here. The Precipitation case that is judged to be the most significant precipitation is found in section 5.1.4 and denoted as P3. The rest of the variables are the same as the SQ scenario (Table 5.1): Regulators are open throughout the simulation time period (24 hours); Impervious percentages for the higher and lower subcatchments are 25% and 50%, respectively; and, the tidal level is the mean tidal level time series starting with a low tide at the beginning of the simulation time ("Mean/Low").

5.1.10. Initial Depth and Impervious Percentage Focus Scenario

The last scenario defined for the purpose of this research is the Initial Depth and Impervious Percentage Focus scenario. This scenario focuses on the initial depth of the junctions and the impervious percentage of the subcatchments. This scenario is comparable with "Precipitation Focus" scenario. For this scenario, regulators are open during the simulation. The precipitation case is "P3 "which is the most significant precipitation variable. This precipitation case is chosen the same as Precipitation Focus scenario of 5.1.4 above. The tidal level is set at the mean tidal level time series starting with a high tide. The Initial Depth of the junctions is set at the high level of 2.591 meters as the focus is on the initial depth. The Impervious Percentages are set at high levels of 50% and 75% for the higher subcatchments and the lower subcatchments, respectively. Comparing this scenario with the "Precipitation Focus" scenario results in finding effects of Initial Depth and Impervious Percentage focus on the system (Table 5.1).

5.2. Scenario Results

In this section the simulation model results for each scenario are presented and discussed. Related graphs and tables for all model results are found in Appendix C – "SWMMModel Simulation Scenario Results". As mentioned in the previous section, the results for all the scenarios are compared with the benchmark case, the SQ scenario. SQ results are discussed fully in the next section and the results for all other scenarios are presented in comparison to the scenarios' results with SQ results presented below in 5.2.1.

5.2.1. Status Quo Scenario Results

The model simulated as the Status Quo scenario is defined in section 5.1.1 above. Tables C.1 to C.21 in Appendix C present the status report for this scenario.

Continuity Errors. The first result of the SWMM simulation model that is shown after simulation run is the SWMM model continuity errors, i.e., the continuity error for (i) surface runoff, (ii) flow routing and (iii) quality routing for the 24 hour simulation period. The SQ scenario results are (i) -0.06%, (ii) 1.6% and (iii) -1.52%, respectively for the continuity error set. As discussed in Chapter 3, Section 3.4, when the continuity error is less than 10%, it is considered as "acceptable". In the status report for the SQ scenario results, which is provided in Appendix C, Section C.1 – "Status Quo Scenario Simulation Status Report", the continuity error for junctions with the highest error is recorded. Based on the SQ scenario results, the errors are under 10% for all the junctions. Thus, all continuity errors for the SQ are in an acceptable range.

Subcatchment Reports. In the subcatchment runoff summary of this report for the SQ benchmark scenario, the results show that the total precipitation for all the subcatchments is equal as they receive the precipitation from the same rain gauge (Gauge 1, Figure 3.11). Each subcatchment's total infiltration is calculated based on the impervious percentage for each subcatchment area. For example, for the S1 subcatchment (Figure 4.2), the total infiltration value is equal to 75% of the total precipitation value for S1.

The status report on the "Subcatchment Washoff Summary" provides the estimated amount of pollutants that are washed off each subcatchment into the system. Each of the subcatchment estimates is calculated based on the land use percentages provided in the subcatchment properties (Table 5.1). The smaller the residential percentage provided for a subcatchment, the fewer pollutants are washed off into that subcatchment. Also, the area of the subcatchment has an impact on the amount of wash-off in a subcatchment. More pollutants are washed off from the subcatchments to the system when the area of a subcatchment is larger. These wash off values include the "Total Suspended Solids" (TSS) and Lead as the standard pollutants. Model estimates for TSS and Lead are calculated based on the EMC (Event Mean Concentration) function which is defined in the SWMM model for these pollutants. The values for Lead are 0.25% of the value of TSS because Lead is defined as the co-pollutant for TSS with the 0.25 as the fraction value. This value is provided in the SWMM manual water quality example which is used for the purpose of this research.

Node Depth Summary. The "Node Depth Summary" of the SWMM results for the SQ scenario shows the average depth summary, the maximum depth, maximum HGL (Hydraulic Grade Line) and the time of maximum occurrence. Discussing these values is important whenever a junction gets surcharged or flooded. In the SQ scenario, none of the junctions and storage units is flooded or surcharged. The reason for this is because of the precipitation and the choice of tidal level in this scenario. The precipitation value is low and the tidal level has a medium level. Therefore, the average depth in the water system is not high for most of the junctions and represents a base validation of the model under these circumstances, as expected. This value for the Outfalls is higher than other junctions because of the tidal level which has effects on the level of water in the outfalls and water in the system flows into these outfalls whenever the control rules for the valves become true and the valves are opened. For all the outfalls and all the junctions connected to the outfalls, the maximum depth occurs at the same time that the tidal level is at its highest value. At 19:00 (7:00pm) the value of the tidal level time series starting with a low tide is 1.327 meters based on Table 4.14 and Figure 4.9. Figure 5.1 shows all outfall depths change during the time of the simulation. The graph of Figure 5.1 follows the same pattern as the tidal level graph which is presented in Figure 4.9. This is evidence, as expected, that the maximum depth for the outfalls and junctions connected to outfalls are at the same time as the highest tidal level.

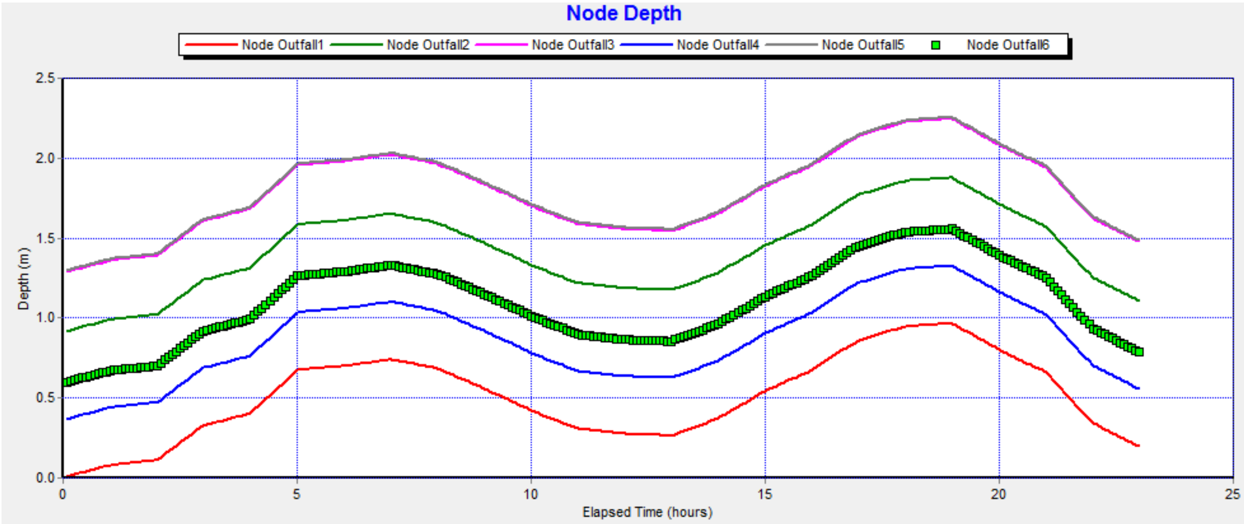

Figure 5.1. Outfalls Depth

Node Inflow Summary. The Node Inflow summary represents the amount of water that flows into each junction. Junctions can have inflows from different sources. These sources can be external or internal. In the node inflow table, maximum lateral inflow is the inflow from external sources shown for each junction. External sources are inflows from subcatchments which enter the junction. For junctions j7, j9, j11, j12, j14, TPO (TreatmentPlantOutfall) (Figure 4.2) and all the outfalls (Outfall1 through Outfall6), the lateral inflow volume is 0mm as these nodes are not connected to any subcathment. Other junctions have lateral inflows as they are connected to subcatchments. The volume of lateral inflow for these junctions is dependent on the area of the subcatchment and the impervious percentage of each subcatchment. This table also presents each junction's maximum total inflow and shows the time of the maximum occurrence. The total inflow volume values in this table are 0 mm for j7 and j14. The reason is that the pumping summary table at the end of the status report in Appendix C (Table C.21) shows that pump p1 and pump p2 have never started up during the SQ benchmark simulation. This means that there is no water pumped up to j7 and j14 during the simulation.

Surcharged and Flooded Nodes. The SQ benchmark simulation model report on surcharged and flooded nodes shows no junction or storage unit as surcharged or flooded (Tables C.14 and C.15) and this means that in the absence of significant precipitation and with a mean value for tidal level as inputs for the SQ benchmark scenario, the capacity of the system is sufficient to deal with this case.

Outfall Loading. The "Outfall Loading" table shows the results related to the flow through the six outfalls in the model for the SQ scenario. The results show the 'flow frequent percentage' or the percentage of time that there is a flow in these outfalls as water released to the harbour. For most outfalls, this percentage is close to 100%. The flow in most of the outfalls is attributed to the tidal level. The total volume of water in the outfall connected to the treatment plant (TPO, Figure 4.2) is higher than other outfalls as it is the very last outlet of the system and all the water in the system is guided to this outfall through the pipes in the system and treated before releasing to the harbour. This table also presents the amount of TSS and Lead which flow into these outfalls as estimated parts of untreated water. Treatment plant outfall has the lowest value for TSS which is expected since untreated water gets treated at this outfall, therefore it is assumed that treated water (aerated and settled) has less TSS. Other outfalls (with the exception of Outfall4) have higher values for TSS as the water flows to these outfalls is not treated, therefore it is assumed to contain a higher level of TSS and Lead pollutants. Outfall4 does not have any value for TSS as all the water containing TSS flowed into the treatment plant and was treated there (through aeration and settling); therefore, no water flows into Outfall4. Outfall4 is a support or bypass outfall for the treatment plant, in the case that the depth of the treatment plant node exceeds to 2.591 meters, regulator R4 is opened and untreated water flows into this outfall. Table 5.2 compares Outfall3, TPO and Outfall2 TSS values. As it is obvious in the graph Outfall2 and Outfall3 in some parts of the day have high values for TSS, this is because of an assumption in SWMM modelling system. This assumption considers the time people generally get up – around 7-10 AM, and when families go to bed – around 9-11 PM, i.e., when they tend to emit sewage flow (this timing is shown in the graph). In comparison to j10, j8 is connected to larger subcatchments and that causes more sewage to be released into Outfall2 in comparison to Outfall3.

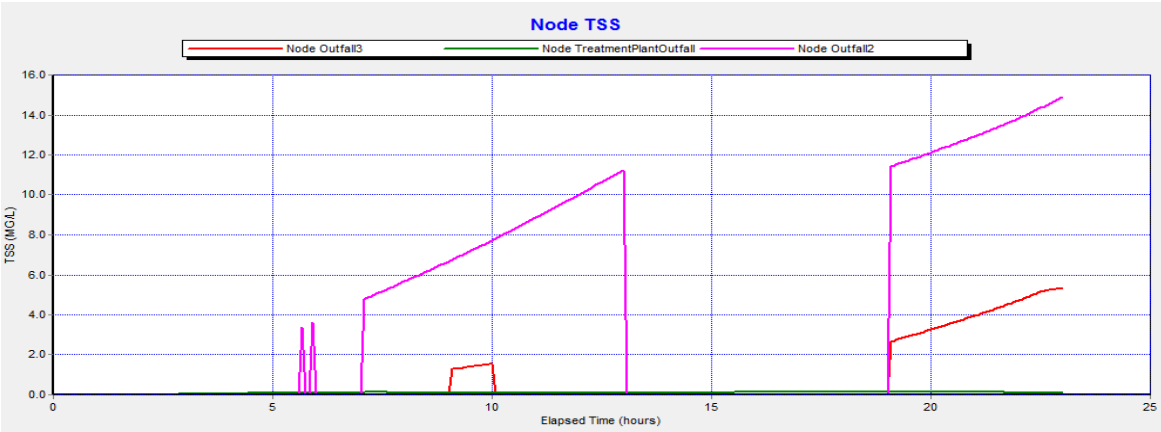

Figure 5.2. TSS values in Outfall2, Outfall3 and TPO

Pumping Summary. The last output table of interest is the pumping summary table. For the SQ scenario, p1 and p5 have not started up during the simulation. This is because in the node depth summary table, the value for the maximum depth of nodes j6 and j15 which are connected to these to pumps are 0.96 meters and 1.56 meters, respectively. On the other hand, based on Chapter 4 description the startup depth for both of the pumps p1 and p5 is 1.829 meters. Comparing nodes depth and the pump start up depth, it is understood why these pumps have not started up during the simulation. On the other hand, the other pumps p2, p3, and p4 have maximum depths higher than their pump start up depth; therefore, they have started up one time during the simulation. As well, in the "Node Inflow summary" table referred to above, the inflow for j6 and j15 is less than other junctions. Moreover, among the other 3 junctions which are connected to other pumps, j8 has the lowest inflow volume. This means that pump p2 was working for less time (20.91%) in comparison with the other 2 pumps, p3 (85.29%) and p4 (80.4%) which have higher inflow volume over the 24 hours of the simulation period.

In order to analyze the whole system, Figure 5.3 shows the total inflow and outflow of the system during the 24-hour simulation. From this graph, it is obvious that whenever inflow is high outflow is low. Inflow starts from a high value because based on the "OFF" case for the precipitation, which is the input for SQ scenario, from the first moment of the simulation there is a precipitation in the system with 0.3mm as the value. One of the sources of inflow to the system is tidal level time series, which is the most effective one in SQ scenario as precipitation is at its lowest value for this scenario.

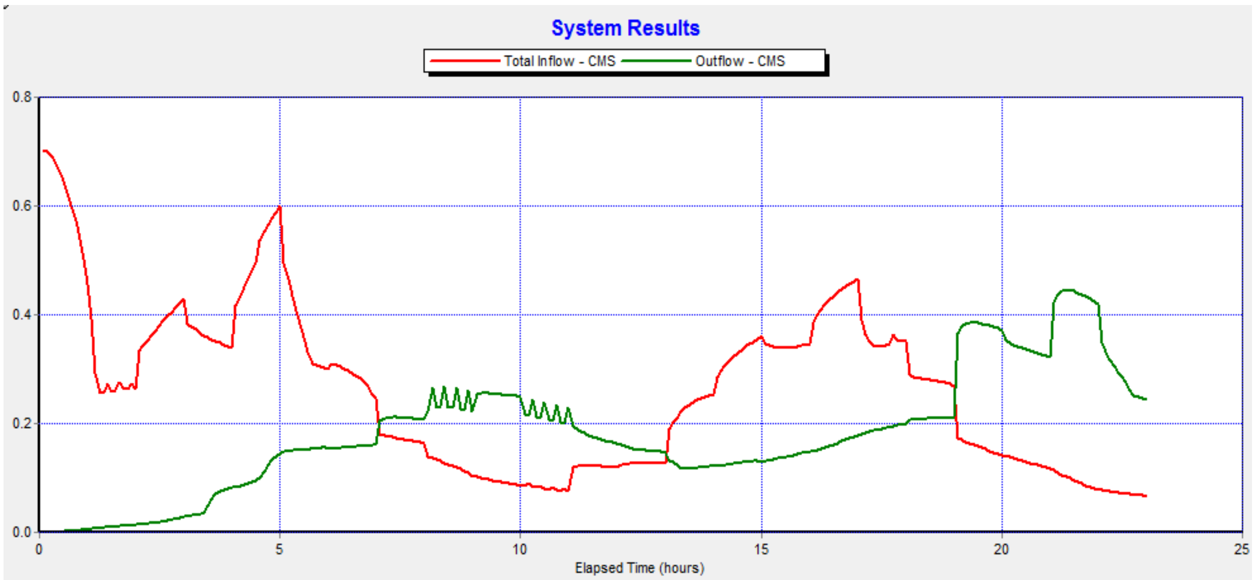

Figure 5.3. System Total Inflow and Outflow for SQ Scenario

In Figure 5.3, the red line shows the total inflow in this graph. Comparing this line with Figure 4.9 which is the graph for "Mean/Low" tidal level, the total inflows follow the same pattern as the tidal graph of Figure 4.9. The green line shows the outflow of the system, and although it is lower than the red line in some parts, it is increasing during the simulation time; this is because anyway water has to leave the system at any moment of the simulation. Therefore, the outflow graph is an ascending graph.

Table 5.2 represents a summary on the SQ scenario results.

Table 5.2 represents a summary of the SQ scenario results. It shows average number of hours junctions are flooded or surcharged. Also it shows average flow for outfalls in CMS (Cubic Meter per Second) and the comparable value is shown for TPO (TreatmentPlantOutfall). TSS and Lead values are also shown in kg in this table. Pumps information, the utilization percentage and the number of startups are also shown as important results in this table.

Table 5.2. Status Quo Scenario Simulation Results Summary

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls(CMS)	Total TSS in TPO(kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO (kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
SQ Scenario	---	---	0.12	0.388	TPO:1.201	56.217	TPO:0.015	0.029	P1: 0%,0 P2:80.4%, 1 P3:85.29%,1 P4:20.91%,1 P5:0%,0

5.2.2. Best Case Scenario Results

The model is simulated under best case scenario conditions discussed in section 5.1.2. Tables C.22 to C.42 in section C.2 of Appendix C present the status report for this scenario.

The best case scenario continuity errors for surface runoff, flow routing and quality routing are: 0.06%, 1.00% and -5.51%, respectively. The error values are all less than 10%, so they are in the acceptable range. In the status report for this scenario, junctions j6, j13 and j8 have continuity errors which exceed 10%. If these junctions are found important for the purpose of the research then their continuity error should be reduced. As there are no junctions flooded or surcharged in this simulation scenario, it is assumed that there is no need to be concerned about these junctions.

In comparison to the Status Quo scenario, the Best Case "Runoff Summary" is not different. This is because the precipitation case is the same for both of these scenarios and that is why all the subcatchments in this scenario receive the same amount of water as the SQ benchmark scenario.

Similarly, the "Subcatchment Washoff Summary" table is the same as that for the SQ scenario results. This is again because of the same inputs for the precipitation and the same example used for the pollutants.

In the "Node Depth Summary Table", the Best Case depth of junctions on the High Road is the same as the corresponding values in the SQ scenario results. The value for the "TreatmentPlant" junction depth is less than what resulted in SQ scenario node depth table. Also, for j7, j9, j11, j12and j14, the average depth is 0. The reason for having these results is that by closing the regulators in the system the effect of tides on the junctions connected to the outfalls will be eliminated. Tides only have effects on the outfalls and there is no way for them to connect to the system. Therefore the depth of the water in junctions j6, j8, j10, j13 and j15 are less than what is recorded in the SQ scenario, and also are less than the startup depth of the pumps. Therefore, the model pumps never start up and do not lift water up to the pumps' outlet junctions which are j7, j9, j11, j12and j14. That is why these junctions never have any water available in them. All these values result in lower depth in the "TreatmentPlant" junction. Also, it causes lower values as the depth for outfall junctions. Comparing maximum depth for the outfall with the maximum value in the "Low" tidal level time series in Figure 4.8 and Table 4.13 shows that outfall depths are due to low tidal level time series in Best Case Scenario. For all the outfalls, the maximum depth happened at 10:00 AM and the highest tide in the lowest low tidal level time series happened exactly at 10:00 AM with 1.166 meters as the value.

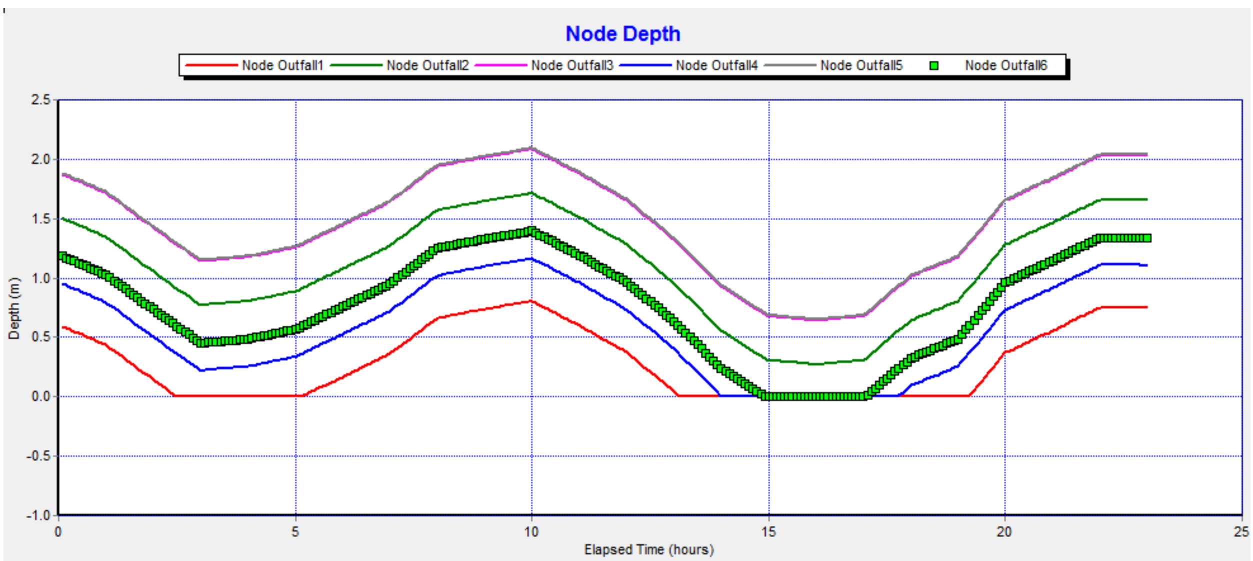

Figure 5.4. Outfall Depths for Best Case Scenario

Figure 5.4 shows all the outfalls' depth during the 24-hour simulation under Best Case scenario inputs. Some hours of the day depth of the nodes are 0, the reason is that in some hours of the day the tidal level is negative or 0 so it does not have any impact on the depth of the outfalls.

Based on the explanation above, the Best Case inflow for the junctions on the High Road remains the same as the SQ scenario result, as these junctions receive the same precipitation as in the SQ scenario. Treatment plant inflow is lower than the SQ scenario as no water is lifted up to outlets of the pumps, and therefore the inflow source from these outlets is not considered for the treatment plant. Outfalls do not have any inflow as the regulators connected to these outfalls are closed based on the inputs for the Best Case scenario.

No junctions are flooded or surcharged in the Best Case simulation, as expected, which is the same as for the SQ scenario results for flooding and surcharging.

Outfall loading summary for the Best Case scenario shows that only the “TreatmentPlantOutfall” has loadings during the simulation. This is because of the closed regulators status that does not let water flow into outfalls from storage units. Also, the amount of TSS in this outfall is very small, although the total amount of TSS in the system (based on the “washoff results” table) is 193.21 kg. This happens because the water remains in the storage unit until it is lifted up with the pumps. Therefore, washed off TSS also remains in the storage units and does not get to the treatment plant during the 24 hours of the simulation.

Figure 5.5 represents the whole system’s runoff; the runoff in the system starts from 0 and increases smoothly until it approaches a constant maximum value after approximately 6 hours into the 24 hour simulation. This is because of the precipitation in the system which adds up to the runoff little by little until it gets to a constant value (“OFF” case for the precipitation in these two scenarios has the minimum value: 0.3 mm). This value is less than the total precipitation due to the infiltration that happens in the system from subcatchment runoff. Comparing Figures 5.5 and 5.6, showing the total inflow and outflow of the system, the absence of tidal level impacts on the inflow of the system, the inflow line is exactly the same as the runoff line in Figure 5.5. It shows that runoff from the subcatchments is the only inflow into the system. The outflow line has lower values than the inflow because storage units store water and do not let it flow out of the system. Therefore, outflow speed is slower than the inflow.

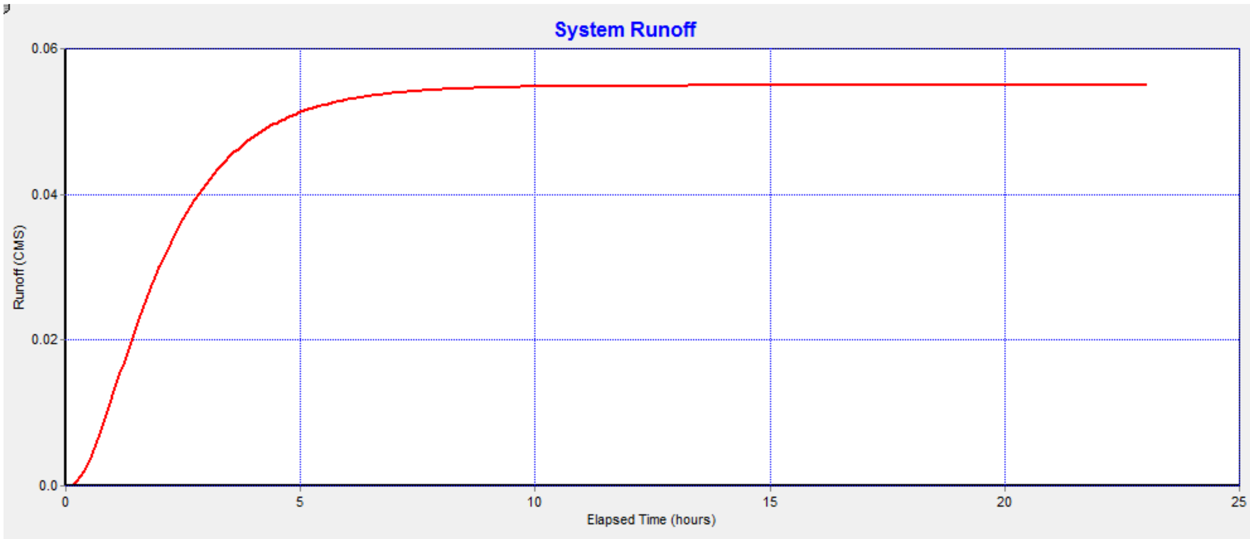

Figure 5.5. System Runoff for the Best Case Scenario

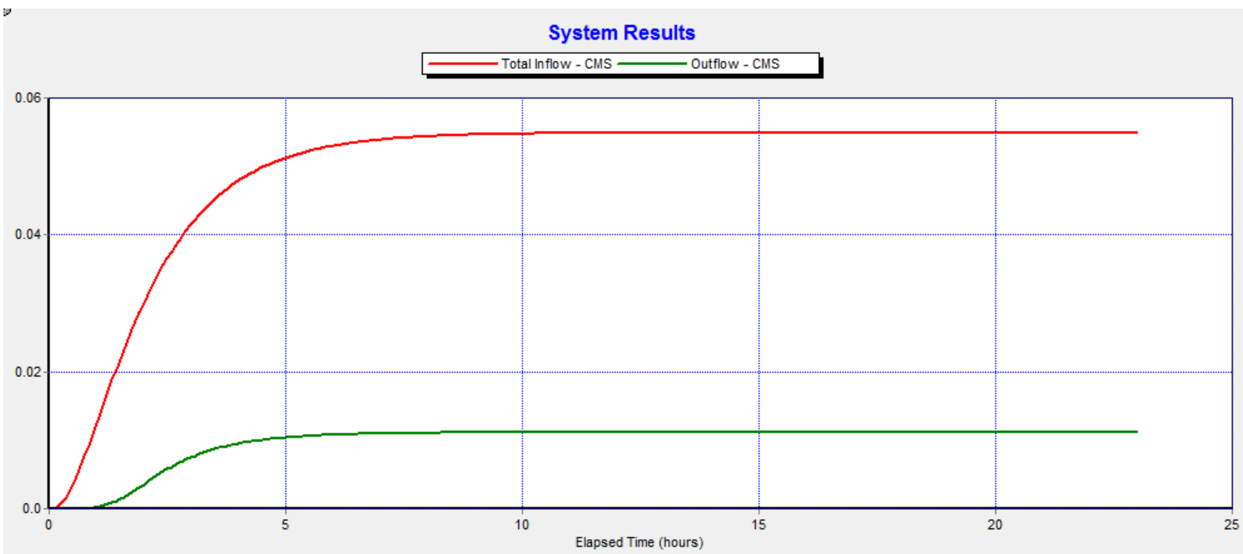

Figure 5.6. System Total Inflow and Outflow for the Best Case Scenario

Table 5.3 represents a summary of the results of the Best Case scenario. As explained above, the Best Case scenario is considered as "best" because it controls the release of untreated water through the outfalls so that less pollutant enter the harbour. This scenario in comparison with the SQ scenario reduces the amount of TSS and Lead entering the harbour. Comparing the summary results of the two scenarios shows this difference. In the SQ scenario, all the outfalls have high values for the TSS and Lead which enter the harbour directly. In the Best Case scenario, the summary results show that only the treatment plant outfall releases TSS into the ocean. The amount of the release of treated water (aerated and settled) is only 0.013kg, which is a small number in comparison with the total TSS loading into outfalls in the SQ scenario.

Table 5.3. Best Case Scenario Result Summary

Scenario	Average Number of Flooding Hours	Average Number of Flooding Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls(CMS)	Total TSS in TOP (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO (kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Best Case Scenario	---	---	0.01	0.01	0.013	0.013	TPO:0.0	0.01	P1:0%,0 P2:0%,0 P3:0%,0 P4:0%,0 P5:0%,0

5.2.3. Worst Case Scenario Results

The model simulated as the Worst Case scenario is defined in section 5.1.3 above. Tables C.43 to C.63 in part C.3 of Appendix C present the status report for this scenario.

The continuity error for surface runoff, flow routing and quality routing are acceptable: 0.04%, -0.1% and -0.1%, respectively. Junctions j7 and j14 have the highest continuity error but these values are also less than 10%.

As discussed in section 4.2.2.1, the total amount of the highest level of precipitation, P3 is 86mm. This value is the exact value shown in the runoff summary table in the status report of the simulation for the Worst Case scenario. The infiltration value for subcatchments in this scenario is less than the SQ scenario because of the higher impervious percentage for each subcatchment. This results in more inflow into the Worst Case scenario system results.

The "Washoff Results" table shows a larger number for the total amount of TSS and Lead in the system compared to the SQ benchmark results. This is because of higher values for the runoff which cause washing more TSS and Lead into the system. The higher the runoff is, the more pollutants are washed from the subcatchments into the system.

In the "Node Depth Summary Table", it is shown that junctions on the High Road have higher depths in this scenario in comparison with the SQ scenario because they receive more runoff from the subcatchments. The Treatment plant junction has a value for this simulation which is also higher than the value in the SQ scenario simulation. This is again because of the higher values for the precipitation time series and also because of the choice of highest high tide for all the outfalls. And, as all the water in the system aims to get to the treatment plant as the final destination, the value for the depth and volume of the “TreatmentPlant” junction is higher than other junctions in the system. However, the “TreatmentPlantOutfall” depth has a lower value than the “TreatmentPlant” junction. This means that some part of the water flowed into “Outfall4” because the maximum depth in the “TreatmentPlant” is 8.38 meters. This value shows that the regulator R4 was open at some time during the simulation because 8.38 meters is higher than the 2.591 meters, the control level rule that opens R4 so that untreated water can flow into the Outfall4. Also, because of the initial depth value for the Lower Road junctions, they mostly have a high maximum depth and the average depth for the junctions in this simulation is higher than the comparable SQ result. Most of the maximum depths for the junctions occured during the first hours of the simulation. This is due to the front loaded time series of heavy precipitation. The first 4 hours of this time series have the highest precipitation value as Worst Case loading. The rest of the time, precipitation is 0mm.

Figure 5.7 reflects the same concept for the storage units in the system. The first 4 hours of the simulation, because of the high precipitation value, the depth of junctions is increasing. Each junction depth starts from the initial depth (2.591 meters) and goes up to a maximum level. This maximum level is dependent on the area of the subcatchments conncted to these junctions. After this maximum level is achieved, the depth decreases since the precipitation decreases to zero for the rest of the simulation and since then the only inflow into the system comes from the high tidal level. The second reason is because of the control rules on the regulators. Regulators become open when the depth of the storage unit connected to them gets more than 2.591 meters. As in Figure 5.7, all the storage units’ depths are higher than 2.591 meters at the beginning of the simulation because of the initial depth consideration for this scenario. After this depth is attained, the regulators are opened and the pumps are all working causing the depth in the nodes to decrease. Since then, the graphs are following the same shape as the high tide time series which is shown in Figure 4.7. Regulators are closed after a reduction in the junctions’ depth and when depths get to 1.829 meters, the pumps start working. For some junctions, e.g., j13, j10 and j8, their depth is always higher than 1.829 meters (for j13 and j8) and for j10 higher than 1.524

meters (which are the startup depths of the pumps connected to these junctions). These pumps work 100% of the time.

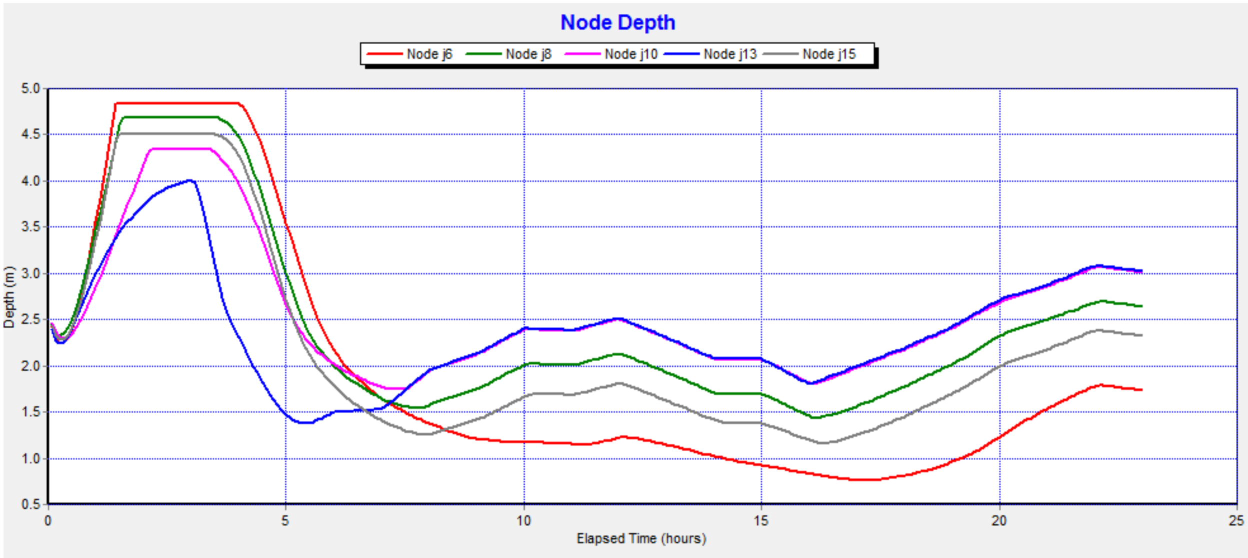

Figure 5.7. Storage Units Depth in Worst Case Scenario

Figure 5.8 shows similar trajectories for the outlet junctions of the pumps in the Worst Case scenario as in Figure 5.7. The same process is occurring for these junctions. Junction j14 depth increases and then decreases to zero. Pump p1 works only for 38% of the time and that is because of what is obvious in Figure 5.7, i.e., the depth of the j15 most of the time is less than the 1.829 meters therefore the pump needs to start up only once and then shut off soon after because the depth decreases to the shut off depth (1.219 meters).

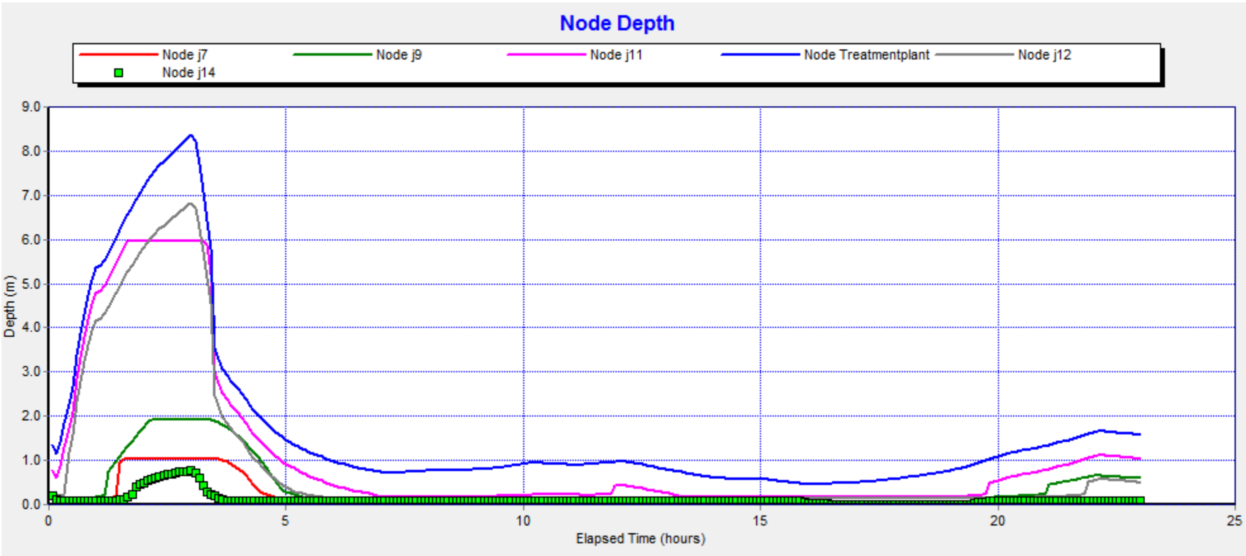

Figure 5.8. Pumps Outlet Junctions and Treatment Plant Depth in Worst Case Scenario

Figure 5.9 shows the outfall depths and as seen in the Figure 4.7, the shapes of these graphs follow the same relative trend as the high tide graph. Comparing Figure 5.7 and Figure 5.9, when the depth of a storage unit is less than others, the depth of the outfall connected to the storage unit is smaller in comparison with others. An example can be j6 and outfall1.

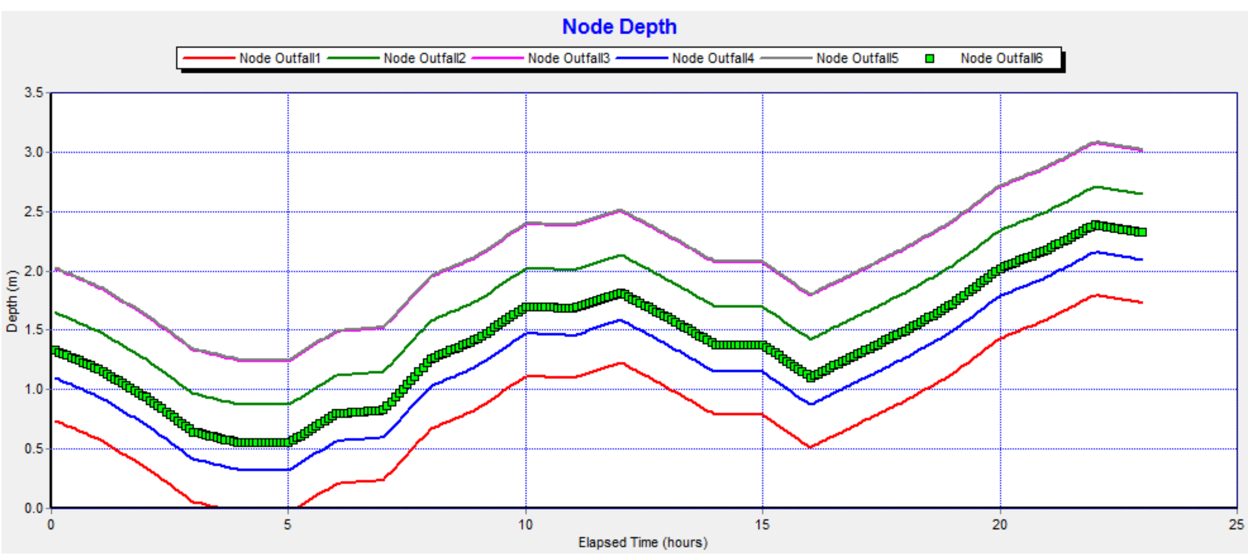

Figure 5.9. Outfall Depths in Worst Case Scenario

For the Worst Case scenario node inflow summary results, the "TreatmentPleant" has the highest inflow as all the water in the system aims to flow into the treatment plant. "Outfall4" and

“TreatmentPlantOutfall” have almost the same inflow, this is because of the high depth for “TreatmentPlant” which cause R4 to be open at some point in time and water flows into the harbour untreated. Other outfalls also have high values for inflow, which is one of the reasons that this scenario is labeled as the Worst Case scenario. This high value for inflows also shows the amount of untreated water that flows into the harbour without getting to the treatment plant. This causes more TSS and Lead to enter the harbour water.

The surcharge summary table for the Worst Case shows the junctions which are surcharged during the simulation. Because of the initial depth in the system, most of the junctions on the Lower Road are surcharged for at least one hour. j11 is surcharged for 11.11 hours because the surcharge depth for this junction is lower than other junctions. Figure 5.8 shows that its depth was almost higher than all of the other junctions. Other junctions have a reasonable number of hours surcharged. The “Node Flooding Summary” shows that j6, j7, j8, j9, j10, j11, j12, j14, j15 and the Treatmentplant are flooded in the Worst Case scenario. Junctions j9, j7, j12 and j14 are flooded for a negligible number of hours (0.01 hours), and also the flooding volume is 0 liter for all of them. The “TreatmentPlant” junction is flooded for 2.49 hours with a small amount of flooding volume which is 0.697 10^6 liters. This means that the capacity of the treatment plant in this system is not enough to manage the Worst Case scenario. With this high precipitation and high tide level the treatment plant gets flooded and untreated water flows out of the treatment plant outfall. This untreated water would flow into the ocean. Junction j10, the inlet of the largest pumping station (p3) is flooded for 1.21 hours with 1.65(10^6 liters) as the flooding volume. Both of these numbers are negligible in comparison to junctions j6, j8 and j15 which have flooding volumes equal to 8.859(10^6 liter), 5.257(10^6 liter) and 6.283(10^6 liter) respectively. The 3 pumping stations connected to these junctions are likely to be flooded in huge storms and high tide levels (Boudreau, 2013).

The last table for the Worst Case scenario is the “Outfall Loading Summary” table. Almost all the outfalls flow frequency percentage is 100%. Total volume for “TreatmentPlantOutfall” is higher than other outfalls, as expected. This means that there was lots of TSS and Lead entering the harbour from all other outfalls in the system. The proof of this is the values in this table related to total TSS and Lead in these outfalls. Outfall5 is connected to S12 and S11. Both of these subcatchments are highly residential which explains why although the Outfall5 loading is almost the same as other outfalls, the amount of pollutants is higher than others. The same reasoning appplies to Outfall6 which is connected to the highly residential subcatchment S13.

In comparison with the SQ scenario and the Best Case scenario, the Worst Case scenario experiences considerable flooded and surcharged junctions. Also, the Worst Case scenario releases more untreated water into the harbour due to the insufficient capacity of the treatment plant and the pumping stations to manage the high volume of water. Table 5.3 represents the summary results for the Worst Case Scenario.

Table 5.4. Worst Case Scenario Results Summary

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls(CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg))	Total Lead in TPO(kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Worst Case Scenario	(11 junctions) 1.94	(10 junctions) 6.45	0.405	2.805	36.229	4658.107	0.151	1.307	P1:84.35%,2 P2:100%,1 P3:100%,1 P4:100%,1 P5:38.82%,1

5.2.4. Precipitation Focus Scenario Results

The model simulated as the Precipitation Focus Scenario is presented in section 5.1.4, and Table 5.1. Tables C.64 to C.83 in part C.4 of Appendix C, present the status report for this scenario.

The continuity error for surface runoff, flow routing and quality routing for this scenario are: -0.03%, 0.24% and 0.64%, respectively. Continuity error values are all less than 10% and are considered acceptable. None of the junctions have a high continuity error in comparison with the SQ benchmark report.

The singular difference between this scenario and the SQ scenario is the precipitation time series. For the current scenario the precipitation series is P3. The precipitation series is the same as for the Worst Case scenario. However, the infiltration and total runoff are different because of the different impervious percentages considered for this scenario. The runoff is higher in this scenario than for the SQ scenario, and the resulting amount of TSS and Lead in the washoff is also higher than the SQ benchmark scenario. S13 has the highest value for TSS and Lead because its area is larger than other subcatchments and, because of its location, it also has a high percentage for the residential land use.

Node depths are higher in comparison to the SQ scenario and lower in comparison to the Worst Case scenario. The maximum depths for “TreatmentPlant”, j11, j12, j6, j8, j10, j13 and j15 are higher than the startup depth of the pumps, and also higher than the maximum depth for the regulators. Thus, for this simulation, the pumps are operating (i.e., all pumps are started up at least one time over the course of the simulation time), and regulator status is “Open”. According to Outfall Loading table, most of the outfalls were loading almost 100% of the time for this scenario. The total flow in the outfalls is higher than the total flow in the SQ scenario because of the higher precipitation. This scenario also results in more inflow into the system. Comparing Figure 5.3 and Figure 5.10 (below), total system inflow and outflow follow the same pattern as the P3 precipitation case. The higher value for the precipitation focus scenario leads to total inflow that is almost 12 CMS which is 20 times larger than the same value for the SQ scenario.

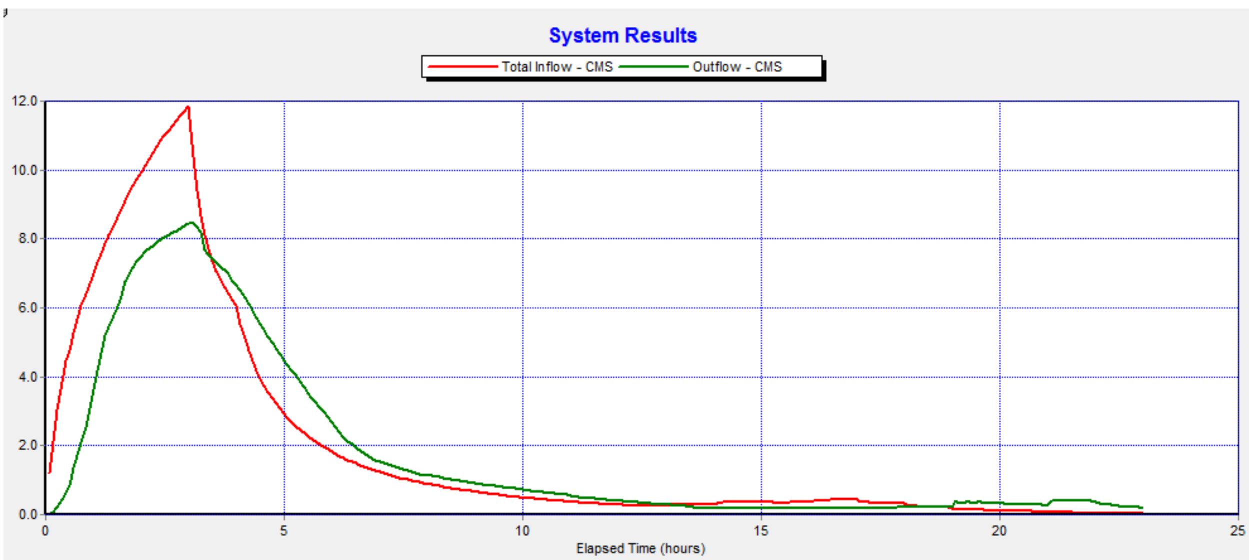

Figure 5.10. System Total Inflow and Outflow in the Precipitation Focus Scenario

Ten junctions are surcharged in the simulation of this scenario. These are: j6, j7, j8, j9, j10, j11, j12, j13, j15 and TreatmentPlant (Figure 4.2). In comparison to the Worst Case scenario, j14 is no longer surcharged. Other junctions, especially j11, are surcharged for fewer hours which is attributed to the elimination of the effect of the high tide time series, and reduction in the impervious percentage in the Precipitation Focus scenario in comparison to the Worst Case scenario.

The Node Flooding Summary table shows that 5 junctions are flooded in this scenario. In contrast, the SQ scenario did not have any junctions flooded or surcharged, and the Worst Case scenario had 10 junctions flooded. In comparison to the Worst Case Scenario, the junctions: j9, j7, j12, j14 and j10 are no longer flooded in this scenario. The first four junctions were not flooded for a significant number of hours in the Worst Case Scenario; therefore, it is not surprising if they are not flooded in the absence of the high level tides in this scenario. When the focus is on the precipitation, those connections which have more lateral inflows are more probable to get flooded; j10 only receives water from j3 which is connected to S3. The S3 subcatchment area is 24.63 ha which is not large in comparison to the total area of the subcatchments connected to the flooded junctions. Therefore, j10 is no longer flooded. On the other hand, j10 has a maximum depth as 4.04m and also the average depth of j10 is 2.21m, which means that pump 3 is working most of the time, and pumps the water into j11. As the surcharge depth of the j11 is lower than other junctions, it floods very quickly and that is the reason that j10 is not flooded and j11 continues to be flooded. The “TreatmentPlant” receives water from S9, S10 and S4 which is connected to j4 with j4 connected to the “TreatmentPlant”. As noted previously, all water in the sewage system aims to reach the treatment plant for treatment prior to being released into the harbour. Accordingly, with the high volume of water in the system, the treatment plant floods in this scenario. Junction “j6” receives water from S7 with 14.47 ha as its area, and also from “j1” which receives water from S1 with 26.01 ha as its area.

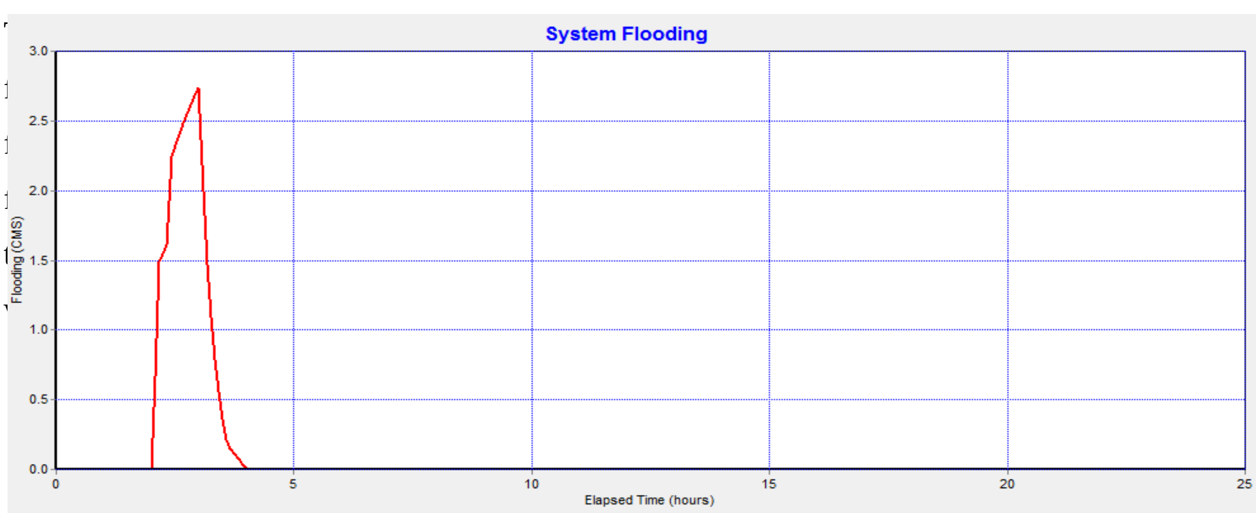

Figure 5.11. System Flooding for the Precipitation Focus Scenario

Figure 5.12 presents a profile plot for the Lower Road connections from j6 to j15. This figure is a screenshot of the SWMM simulation model which simulates how water flows into the system junctions. This screenshot is taken at 3:15 AM for this 24-hour simulation period, or 15 minutes after the last maximum flooding event (i.e., at 3:02 AM j11 flooded for 0.19 hours). The time step for this simulation is 5 minutes so no screenshot can be captured in any time less than 5 minutes. In this figure, junctions j11, j8, j6, j15 and the treatment plant are all flooded.

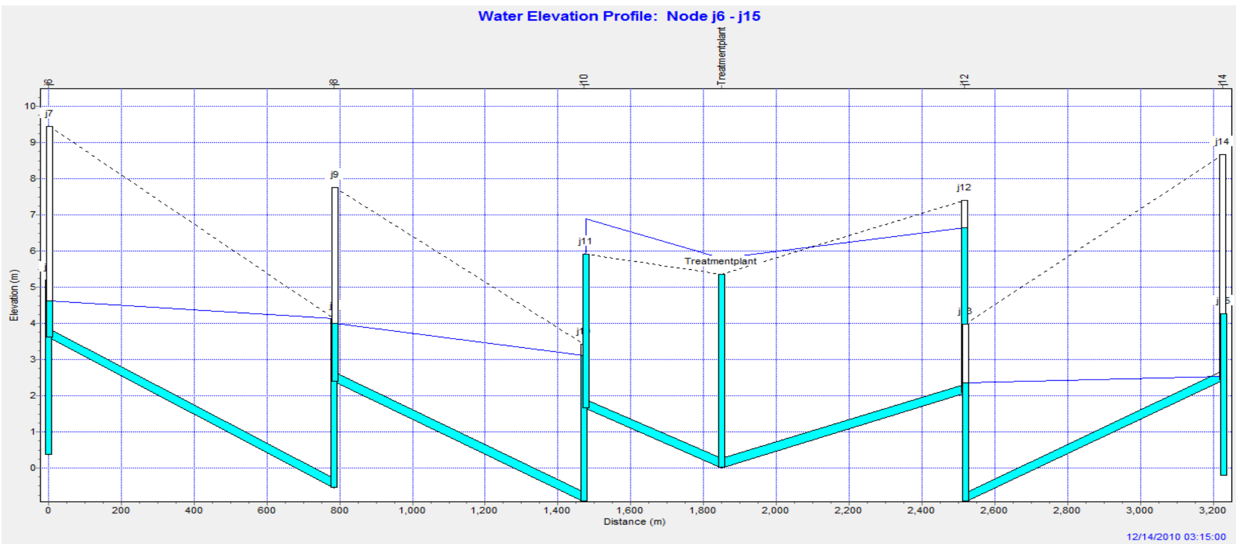

Figure 5.12. Profile plot for the Lower Road Connections from j6 to j15 at 3:15:00 for Precipitation Focus Scenario

As noted above, the amount of TSS and Lead flowing into the outfalls is more than the amount in the SQ scenario, but close to the amount of the Worst Case scenario. The amount of TSS and Lead released into the ocean from outfalls connected to the flooded junctions are more than the Worst Case Scenario. The reason for this can be found in the pumping summary table. All the pumps are started up at least for one time based on the pumping table for the precipitation focus scenario. P2 and P3 were working for most of the time because the flow in j13 and j10 is not so high, but the maximum depths for both pumps are high. The pumps start up because of the high maximum depth and because of the flow in the inlet junctions they do not stop working and continue working in order not to let water flow out from Outfall3 into the harbor without getting treated. Flow in j6, j8 and j15 is high and maximum depth is high as well, therefore the pump is not enough for lift the water up and regulators become open and most of the water flows into the harbour from the outfalls connected to these junctions. This is one of the reasons that the TSS value (which loads to these outfalls) is higher than other outfalls, and even higher than the Worst Case Scenario. Pumps are apparently not set properly in these stations to avoid overflows.

The Precipitation Focus scenario has higher Total TSS and Lead values loaded into the outfalls in comparison to the Worst Case scenario. The amount of TSS and Lead washed off from the subcatchments for these two scenarios is the same due to the same precipitation series, P3. On the other hand, the total runoff in the subcatchments in the Precipitation Focus scenario is less than the runoff in the Worst Case scenario because of lower impervious percentage in the Precipitation Focus scenario. This causes more concentration of TSS and Lead in the runoff in the Precipitation Focus scenario. Therefore, more TSS and Lead loads to the outfalls.

Table 5.5 represents the summary of the Precipitation Focus Scenario Results.

Table 5.5. Precipitation Focus Scenario Results Summary

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls(CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO (kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Precipitation Focus Scenario	(5 junctions) 1.23	(10 junctions) 4.293	0.328	2.288	29.909	4909.817	0.179	1.399	P1:39.58%,1 P2:96.45%,1 P3:95.77%,1 P4:70.12%,2 P5:41.89%,1

5.2.5. Tide Focus Scenario Results

The Tide Focus Scenario is presented in section 5.1.5. The continuity errors for surface runoff, flow routing and quality routing are -%0.06, 3.53% and 2.01%, respectively. Continuity error values are all less than 10%, which is acceptable. None of the junctions have a continuity error more than 10%.

All the values in the “Subcatchment Runoff Summary” and “Subcatchment Washoff Summary” tables are the same as those for the SQ scenario. This is attributed to the same precipitation case and the same impervious percentage for all the subcatchments in the SQ scenario and the Tide

Focus scenario. The total amounts of TSS and Lead is also the same in both the SQ scenario and the Tide Focus scenario.

The Node depths for the High Road junctions are the same as those in the SQ scenario again because they receive the same precipitation as in SQ scenario. For the junctions on the Lower Road, node depths are higher than the same values for the SQ scenario because of the "High" tidal level time series used for the Tide Focus scenario. Maximum depths are not high for most of the junctions and it means less water flows into the harbour from the outfalls as the pumps are helpful in lifting the water up to the pumping stations' outlet junctions. Maximum depth occurred at between 22:00 and 23:00 for all of the junctions; this is because of the "High" tide time series which has its highest value between 22:00 and 23:00 in the 24-hour simulation period. The High tide time series has an impact on the depth of the junctions. Figure 5.13 represents depth of outfalls in the Tide Focus scenario. Comparing this figure with Figure 4.7 and Table 4.12, the pattern of the graph follows the pattern of the high tide time series.

Based on the "Node Inflow Summary", outfalls have high values for the inflow because of the impact of the tidal level time series in this scenario. These tides are the source of inflow to the outfalls. This is verified by the maximum depth of the "TreatmeanPlant" (1.68 meters) which does not cause the regulator R4 to open and therefore, no water flows into the Outfall4 from the system itself, the Outfall4 inflow source is the tides which hit this outfall. Also, comparing the inflow of "TreatmentPlant" and "TreatmentPlantOutfall" it is obvious that almost all the inflow of the "Treatmentplant" flowed into the "TreatmentPlantOutfall" and nothing has flowed into the Outfall4.

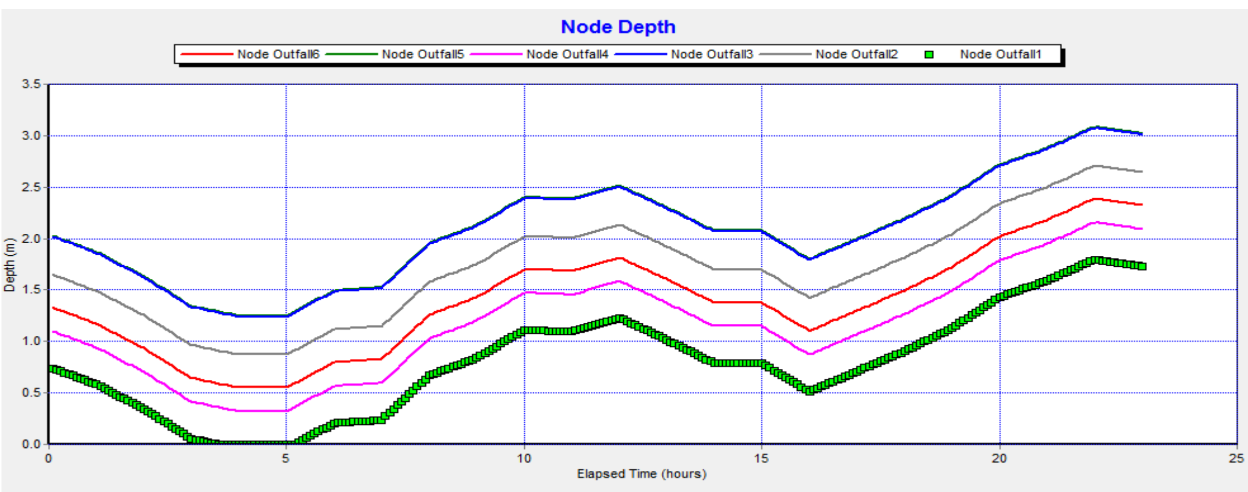

Figure 5.13. Outfall Depth for Tide Focus Scenario

Six junctions are surcharged in this simulation but none of them are flooded. This means that in these junctions water did not pass the surcharge depth limitation which defines the junction as being flooded. All of the junctions are surcharged for a negligible number of hours. j10 and j13 have high values for inflow and that is the reason that they are surcharged for almost 4 hours during the simulation.

The average flow in the outfalls is a little higher than the same value in SQ scenario. However, the amounts of TSS and Lead loads in the outfalls are lower than the SQ scenario. This is because there is more flow in the whole system in this scenario in compare to the SQ scenario, but the amount of TSS and Lead washed off to the outfalls is the same for both. This causes the concentration of the pollutants for this scenario to be lower than SQ scenario. Therefore, less TSS and Lead releases into the ocean during the same time, although the amount of water flows into the harbour is more than SQ scenario. Based on the "Pumping Summary Table", P5 does not work during the simulation. This is because of the maximum depth of the j6 which is 1.8 meters which is less than the start-up depth of the pump which is 1.829 meters. P1 works only 15.48% of time and p4 has the same situation (operates only 59.32% of time) and it seems these do not have good performance in comparison with other pumping stations. Performance for pumps means the amount of water they can lift up to a specific height in a specific time. Because of this weak performance, the inlet junctions of these pumping stations get surcharged and this causes R2 and R4 to open and empty untreated water into the harbour .

Table 5.6 represents the summary of results for Tide Focus Scenario.

Table 5.6. Summary of Results for Tide Focus Scenario

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls(CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO(kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Tide Focus Scenario	---	(10 junctions) 2.47	0.185	0.608	2.534	32.337	0.015	0.022	P1:15.48%,1 P2:96.67%,1 P3:93.15%,1 P4:59.32%,1 P5:0%,0

5.2.6. Tide and Initial Depth Focus Scenario Results

The Tide and Initial Depth Focus Scenario conditions are presented in section 5.1.6. The continuity errors for surface runoff, flow routing and quality routing are -0.2%, 0.04% and 0.75%, respectively and are well within the acceptable range. None of the junctions have a high continuity error more than 10%.

In this scenario P1 is used as the precipitation case, therefore the total precipitation is different from any other scenario discussed until now. The P1 scenario is based on the region's heavy rain historical series (Figure 4.6 above). The infiltration and runoff values are calculated based on the same impervious percentages as for the SQ benchmark scenario. As the total amount of precipitation over the 24-hour period is very close to the total amount of precipitation for the P3 (front loaded) precipitation case, the amount of washed off TSS and Lead is almost the same as those scenarios with P3 as the precipitation case, which represent significant flows.

In comparison to the SQ scenario, the average junction depth is high because of the high initial depth focus in this scenario representing full drains and culverts as the starting point. Outfalls have the same depth as the Tide Focus scenario because of the same tide time series considered for Tide and Initial Depth Focus scenario. As there is an initial depth in all of the Lower Road junctions and this initial depth is the same as the depth at which regulators are opened by the decision rule (section 4.2.1.1), the regulators are all open from the beginning of the simulation. Moreover, the initial depth of the junctions, 2.591 meters, is higher than the startup depth of all the pumps in the system, then the pumps are all started up at least one time during the course of the 24-hour simulation period. For the junctions on the High Road which are only connected to the subcatchments, the maximum depth happens at the same time that the P1 precipitation case has its highest precipitation value. This value is 9.6mm that occurs at 7:00 AM in the 24- hour simulation. Figure 5.14 shows the depth of the junctions on the High Road (j1 through j5 and j16). Comparing this figure with the P1 precipitation schedule of Figure 4.6, the highest depths happen at the same time as the highest precipitation values in the series.

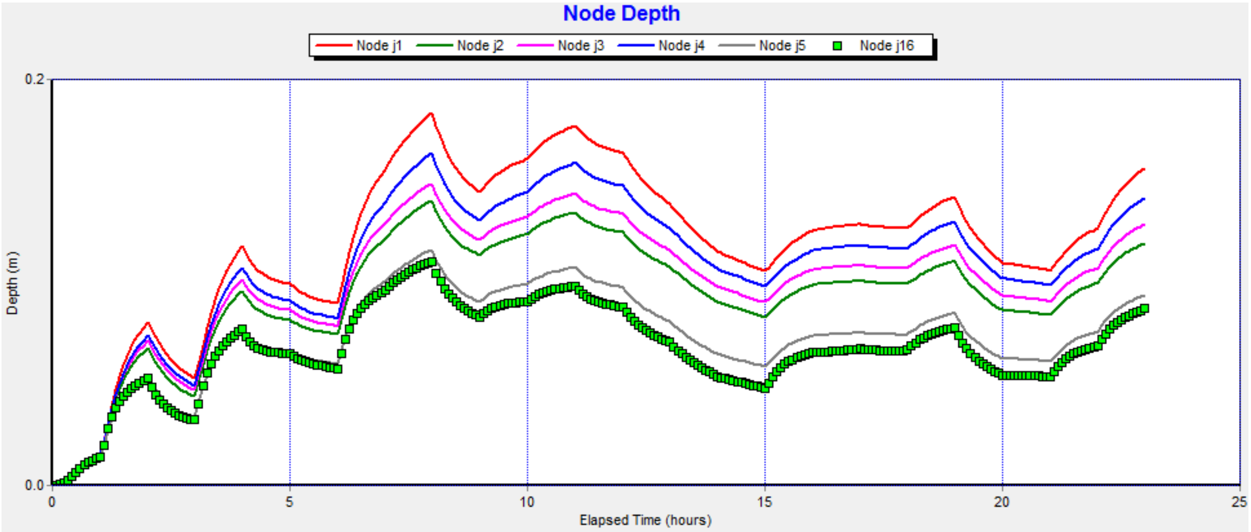

Table 5.14. High Road Junctions' Depth in the Tide and Initial Depth Focus Scenario

For storage units, the maximum depth happens at the same time as the maximum tidal level which occurs between 22:00 and 23:00.

In this water-laden scenario, 8 junctions are surcharged. These junctions are j7, j8, j9, j10, j11, j12, j13 and j14. Among these junctions, j11 and j12 have the highest number of hours surcharged. Because the initial depth (2.591 meters) is very close to the surcharge depth of these junctions (1.683 meters and 2.757 meters, respectively), then these two junctions are surcharged for a longer time than the other surcharged junctions. With respect to flooding, junctions J7, j9, j11, j12 and j14 are flooded in this scenario. The flooding period however is short at only 0.01 hours for the small amount of flooding value. Therefore, we consider these flooded junctions as negligible. Figure 5.14 does not show any value for the flooding in the system. This is due to the time steps assumed in the SWMM simulation model that are 5 minutes in length. Therefore, as these junctions are flooded only for 0.01 hour, or 0.6 minutes, and that this event happens at the start of the simulation at time 00:00, then the flooding graph (Figure 5.14) for this scenario does not show any flooding in the system. The reason that flooding happen at time 00:00 is because from the beginning of the simulation, the initial depth effectively surcharges the junctions in the system, the tides provide initial upward pressure on outflows, and the precipitation starts from the first moment of the simulation.

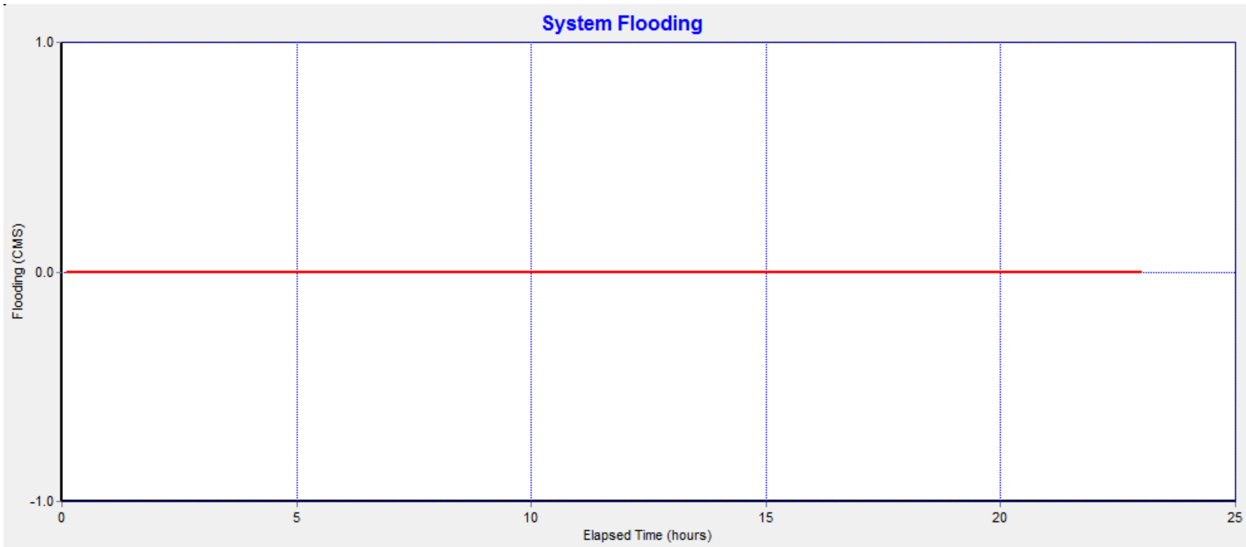

Figure 5.15. System Flooding in the Tide and Initial Depth Focus Scenario

The "Outfall Loading" table for this scenario shows that all the outfalls are loaded 100% of the time. This is because of the initial depth in the system, which causes the regulators to be open from the beginning of the simulation and the average junctions' depth are almost high and very close to the regulators opening height.

The average flows in the outfalls are higher than the SQ scenario and lower than the Worst Case scenario. These values are muchlower for the Outfall4 in comparison with the Worst Case scenario. Outfall4 shows that most of the water flows into the "TreatmentPlantOutfall" and that less untreated water flows into the harbour through Outfall4. This is provable from comparing the TSS values for the Outfall4 from the Worst Case scenario and Tide and Initial Depth Focus scenario. The Tide and Initial Depth Focus Outfall4 flow value is almost one third of that for the Worst Case scenario. In comparison with what is washed off from the subcatchments (5515.99 kg of TSS, and 1.379 kg of Lead), the amount of water which enters the harbour through outfalls indicates poor treatment in the system (3,773.241 kg of TSS, and 1.209 kg of Lead).

Table 5.7 represents the summary of results for the Tide and Initial Depth Focus scenario.

Table 5.7. Tide and Initial Depth Focus Scenario Results Summary

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls (CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO (kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Tide and Initial Depth Focus Scenario	(5 junctions) 0.01	(10 junctions) 5.0525	0.344	1.6	27.476	3773.241	0.272	1.209	P1:100%,1 P2:100%,1 P3:100%,1 P4:100%,1 P5:81.21%,2

5.2.7. Regulators and Precipitation Focus Scenario Results

The Regulators and Precipitation Focus Scenario is defined in section 5.1.7. The continuity error for surface runoff, flow routing and quality routing are -0.02%, 1.28% and 1.75%, respectively and indicate acceptable error values. As before, none of the junctions have a continuity error more than 10%.

The "Subcatchment Runoff Summary" and "Subcatchment Washoff Summary" tables are exactly the same as the Precipitation Focus scenario because the same precipitation and impervious percentages are used in both of these scenarios.

High Road junctions' depths are the same as the scenarios with P3 as the precipitation series input including the Worst Case and Precipitation Focus scenarios and are higher than the SQ scenario High Road junctions' depth. "TreatmentPlant" depth in this scenario is higher than any other scenario. In comparison with the Best Case scenario which has closed regulators throughput, this scenario has a higher depth for the "TreatmentPlant" junction. This is because of the difference in the precipitation time series which is the "OFF" case for the Best Case scenario and P3 is the case for the Regulators and Precipitation Focus Scenario representing the most significant precipitation case. All the junctions connected to the outfalls have high depth in this scenario as well.

Twelve junctions are surcharged in this scenario. Junctions j6, j7, j8, j9, j10, j13, j14 and j15 are surcharged for a significant number of hours - almost 22 hours – over the course of the 24-hour simulation period. Junction j6 is surcharged because regulator R1 is closed, and water cannot flow into the harbour. Pump P5 lifts up water to junction j7, and j7 gets surcharged as well. Water flows into j8 and the same surcharging situation happens for j8 and j9. The same process also occurs for paired junctions j12 and j13, j14 and j15. (Junction j10 is also surcharged but the water pumped from P3 does not force j11 to be surcharged for a long time.) Outfall4 is closed; therefore, all the water which flows into the treatment plant flows into the "TreatmentPlantOutfall", that is why "TreatmentPlant" junction is not surcharged for a long time in comparison with other junctions.

Nine junctions are flooded in this scenario. Junctions j6, j8, j10, j13 and j15 are flooded for a considerable number of hours due primarily to the fact that the regulators are closed. In comparison with the Precipitation Focus scenario, the number of junctions flooded and also the number of hours each is flooded is very high in this scenario. All the junctions connected to the outfalls are flooded again because of regulators being closed during the simulation. Closed regulators cause water being stored in the storage units to stay in the system and not flow out the outfall release into the harbor. As the pumps are started up only once in this scenario, it seems that they do not have enough power to force the water through the rest of the system. Junctions j11 and j12 have the lowest surcharge depths and accordingly, they get flooded sooner than other pumping station outlet junctions. The treatment plant is flooded for 5.07 hours in this scenario which in comparison with all other scenarios is high. Figure 5.15 shows the total system flooding. The highest flooding value happens during the first 5 hours of the simulation. This is because the front loaded precipitation series (P3) is used for this simulation. Then when it stops raining after the initial 5 hour period, water flows out of the system through the "TreatmentPlantOutfall" and the system also loses water because of being flooded.

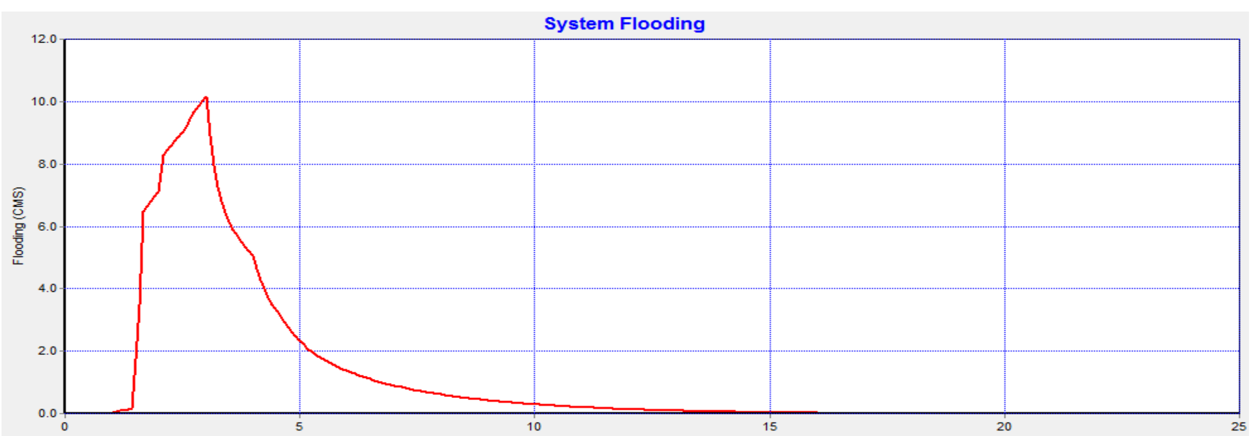

Figure 5.16. System Flooding in the Regulators and Precipitation Focus Scenario

All the outfalls except the "TreatmentPlantOutfall" do not have any flow because of the closed regulators preventing outflow. The average flow in the "TreatmentPlantOutfall" is 0.487 CMS and in comparison with SQ scenario (0.120 CMS) and even the Worst Case scenario (0.405 CMS), this flow is high. The total TSS loaded to this outfall is 118.214 kg and the total Lead loaded to this outfall is 0.327 kg which is more than the SQ scenario and only slightly below the Worst Case scenario total TSS and Lead outflows. As mentioned before, the Worst Case scenario is labeled accordingly because it releases the highest amount of TSS and Lead pollutants into the harbour in comparison with the other scenarios. Therefore, although the number of hours junctions are flooded in the Regulators and Precipitation Focus scenario is higher than the number in the Worst Case scenario, the Worst Case scenario still has the most significant water quality issue.

Figure 5.16 shows the total inflow and outflow for the Regulators and Precipitation Focus scenario. It is obvious from the graph that the inflow is much higher than the outflow, and this is because the regulators are closed. This forces the system to flow water to the "TreatmentPlantOutfall" and this outfall is the only outlet of the system. Moreover, being flooded for a long period of time causes the system to "lose" water, and this water will not be directed to its final destination at the "TreatmentPlantOutfall" and instead, it flows out of the storage units and back into the ground since the storage units or the junctions do not have enough capacity to hold this water. Accordingly, the excess capacity flows out of the system from the top of the flooded junctions and storage units back into the ground, on roadways, and yards.

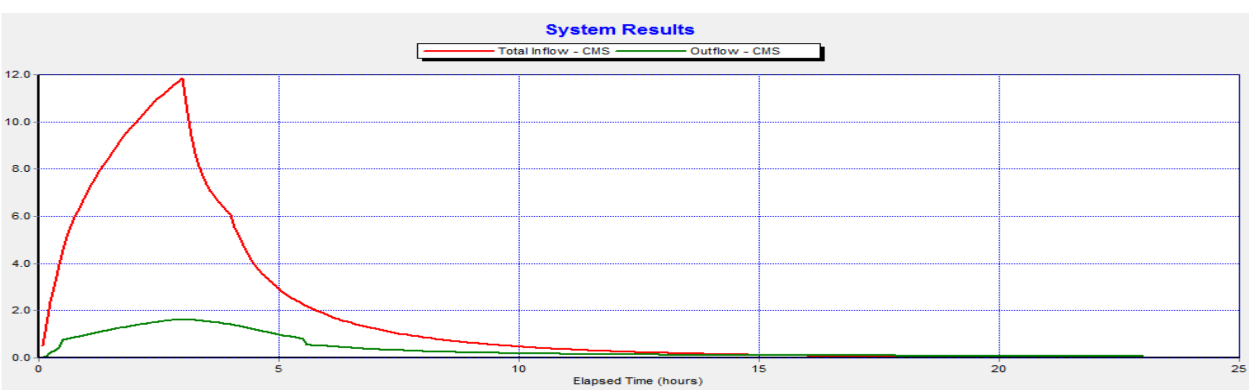

Figure 5.17. System Total Inflow and Outflow for the Regulators and Precipitation Focus Scenario

Table 5.8 represents the summary of the results of the Regulators and Precipitation Focus scenario.

Table 5.8. Regulators and Precipitation Focus Scenario Results Summary

Scenario	Average Number of Flooding Hours	Average Number of Flooding Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls (CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO (kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Regulators and Precipitation Focus Scenario	(9 junctions) 9.226	(12 junctions) 16.19	0.487	0.487	118.214	118.214	0.327	0.327	P1: 96.16%,1 P2:96.07%,1 P3:95.18%,1 P4:96.07%,1 P5:96.22%,1

5.2.8. Tide and Impervious Focus Scenario Results

The Tide and Impervious Focus Scenario is presented in section 5.1.8. The continuity error for surface runoff, flow routing and quality routing are -0.17%, 0.97% and 0.65%, respectively and are considered acceptable. None of the junctions have a continuity error of more than 10%.

In this scenario, the "Subcatchment Runoff Summary" and "Subcatchment Washoff Summary" results tables show values related to the P2 as the precipitation case and the higher impervious percentages are 50% and 75% which differ from the SQ benchmark scenario. Therefore the total precipitation, total infiltration and total runoff are different from other scenarios. The total washed off TSS is 5873.368 kg and the total washed off Lead is 1.468 kg. In comparison with the SQ scenario, Tide Focus scenario, and the Precipitation Focus scenario, this amount is high. But in comparison with the Worst Case scenario, the Tide and Initial Depth Focus scenario, and the Regulators and Precipitation Focus scenario, this value is very close to the same value in the aforementioned scenarios. These results are similar since they share the same high impervious rates that are in direct relation to the TSS and Lean washoff function.

Figure 5.17 represents total inflow and outflow for the Tide and Impervious Focus scenario. The beginning of the inflow line is between 1.5 CMS and 2 CMS due to a high tide at the beginning of the "High" tide time series (Figure 4.7). Then inflow falls down because of the decrease in the tidal level. Afterwards, because of the smooth precipitation of the P2 series, the inflow is smooth and increasing with a small slope because of the increase in the tidal level. Tidal level however does not have a significant effect on the inflow graph. The outflow is increasing because of the increase in the runoff during the simulation period. From Figure 5.17, the Outflow approaches its maximum level that corresponds to the maximum level in the inflow and tidal level.

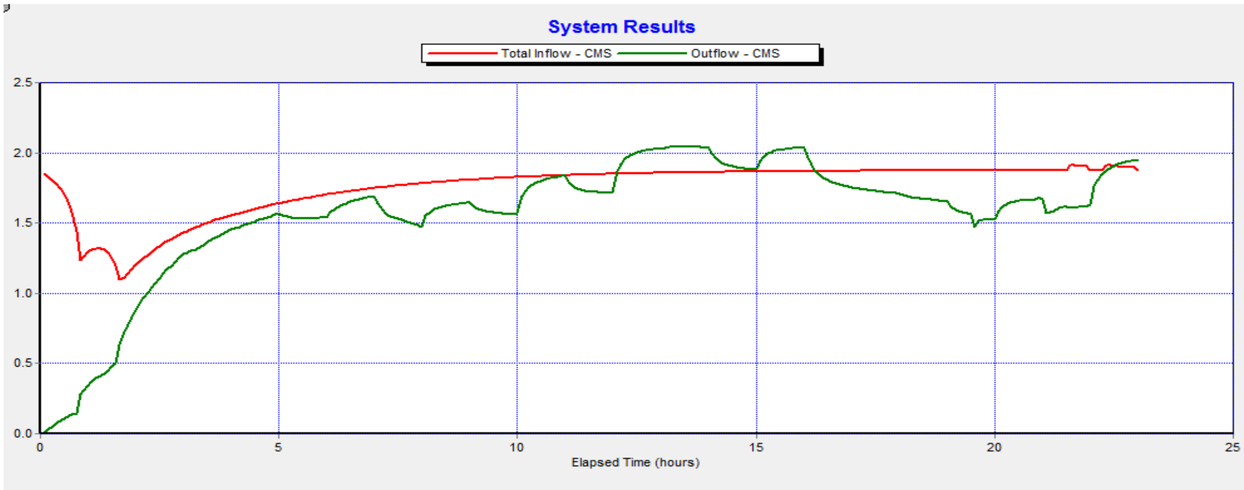

Figure 5.18. System Total Inflow and Outflow for the Regulators and Precipitation Focus Scenario

No junction is flooded in this simulation but 6 junctions are surcharged. Junction j11 is surcharged for 20.62 hours which is high number in the 24-hour simulation. This is due to the volume of water pump P3 has lifted up from j10 to j11 which is higher in comparison with the other pumps. As well, j11's surcharge depth is lower than other junctions in the system and causes surcharging to happen sooner than other junctions, all other things the same. Similar reasoning applies to junctions j12 and j13 where pump P2 lifts up a high volume of water into j13 causing it to become surcharged. Other surcharged junctions are surcharged only for a small number of hours.

In comparison with the washoff of TSS and Lead, it seems that this scenario loads a significant amount of TSS and Lead to the outfalls. In comparison with the Worst Case scenario, which has the same impervious percentage and the same tidal level time series, this scenario loads less pollutants to the outfalls. This is because of the smooth precipitation and absence of the higher initial depth in this scenario.

In the pumping summary, again P1 is working for less time in comparison with other pumps. All the pumps are started up once during the simulation. Table 5.9 shows the summary of the results of the Tide and Impervious Focus scenario.

Table 5.9. Summary of Results in the Tide and Impervious Focus Scenario

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls (CMS)	Total TSS in TPO(kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO (kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Tide and Impervious Focus Scenario	----	(6 junctions) 6.84	0.351	1.634	28.983	4080.723	0.298	1.311	P1: 69.38%,1 P2:96.88%,1 P3:94.28%,1 P4:74.24%,1 P5:74.42%,1

5.2.9. Initial Depth and Precipitation Focus Scenario Results

The Initial Depth and Precipitation Focus Scenario is presented in section 5.1.9. The continuity error for surface runoff, flow routing and quality routing are -0.03%, -0.60% and -0.25%, respectively are acceptable errors. None of the junctions have a continuity error more than 10%.

The “Subcatchment Runoff Summary” and “Subcatchment Washoff Summary” tables’ values are the same as other scenarios with P3 as the precipitation case. The total TSS washed is 6111.506 kg and the total Lead washed from subcatchments is 1.528 kg. In comparison with the SQ scenario, the junctions have higher average depth. For the High Road junctions, this difference is because of the higher precipitation values. For the Lower Road junctions, this difference is because of the initial depth considered for this scenario. The average depths for outfalls are almost the same as the SQ scenario values and this is due to the same tidal level time series used in both scenarios. As the entire maximum junction depth values are higher than 2.591meters for the storage units, the regulators connected to these storage units are opened by the rule (see section 4.2.1.1 above) from the beginning of the simulation and untreated water then flows into the harbour. At the same time, all the pumps are working because of the high maximum depth and high average depth for the storage units. The maximum depth for the High Road junctions and Lower Road junctions happened in the first 5 hours of the simulation and are due to the high precipitation during this time from the front loaded P3 precipitation series. On the other hand, outfall depths are dependent on the tidal level. Therefore, the maximum depth for the outfalls occurred at 19:00 which is the time of the highest tide for this scenario.

Figure 5.18 shows the total inflow and outflow of the system in the Initial Depth and Precipitation Focus scenario. Comparing this graph with Figure 5.10 shows that the total inflow is exactly the same because of the same P3 precipitation case used in both scenarios. The total outflow is almost the same. The only difference in total outflow is in the start point of the graph which is attributed to the different initial depths considered for the junctions. This means that outflow was occurring even at the first moment of the simulation due to the initial depth of the junctions and the amount of initial water in the system.

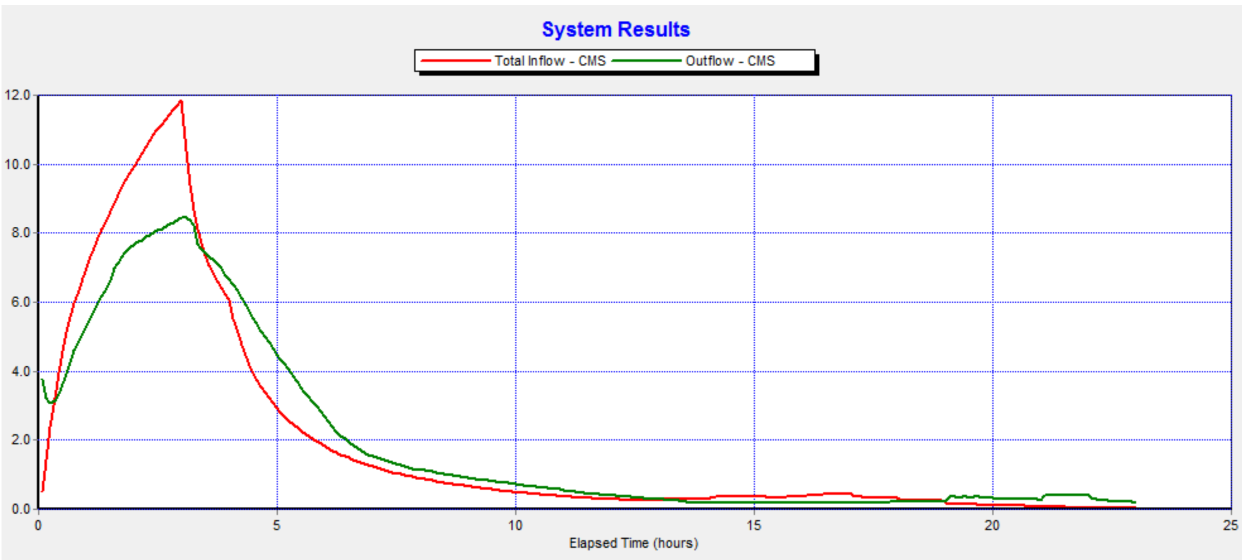

Figure 5.19. System Total inflow and Outflow for the Initial Depth and Precipitation Focus Scenario

Eleven junctions are surcharged and 9 junctions are flooded in this scenario. The number of hours the eleven junctions are surcharged is not significant except for j11 which always is surcharged for a high number of hours due to its low surcharge depth, and due to the volume of water pump P3 lifts up during the simulation. The results for the flooding junctions are almost the same as for the results in the Worst Case scenario. The difference is that junction j10 is not flooded in this scenario and the number of hours that junctions j6, j8, j11 and j15 are flooded are lower than for the Worst Case Scenario. Other flooded junctions (j7, j9, j12, 14) are flooded for an insignificant number of hours. The treatment plant is flooded because of the initial depth and also the high precipitation in the system. The treatment plant is connected to 2 subcatchments directly and to all other subcatchments indirectly through the pipes. Comparing the flooding results of this scenario with the Precipitation Focus scenario, all the values in the flooding junction tables for the "TreatmentPlant" junction are the same for both of the scenarios. The time of maximum occurrence, total flooded volume, total number of hours flooded, and the maximum ponded depth are the same for both of these scenarios. It seems that the initial depth does not have that much effect on the treatment plant getting flooded, but rather, it is all about the precipitation in the system. Also, junctions j6, j8 and j15 are flooded at 2:59 AM for both of the scenarios, but the number of hours being flooded is different and it is due to the initial depth for these junctions.

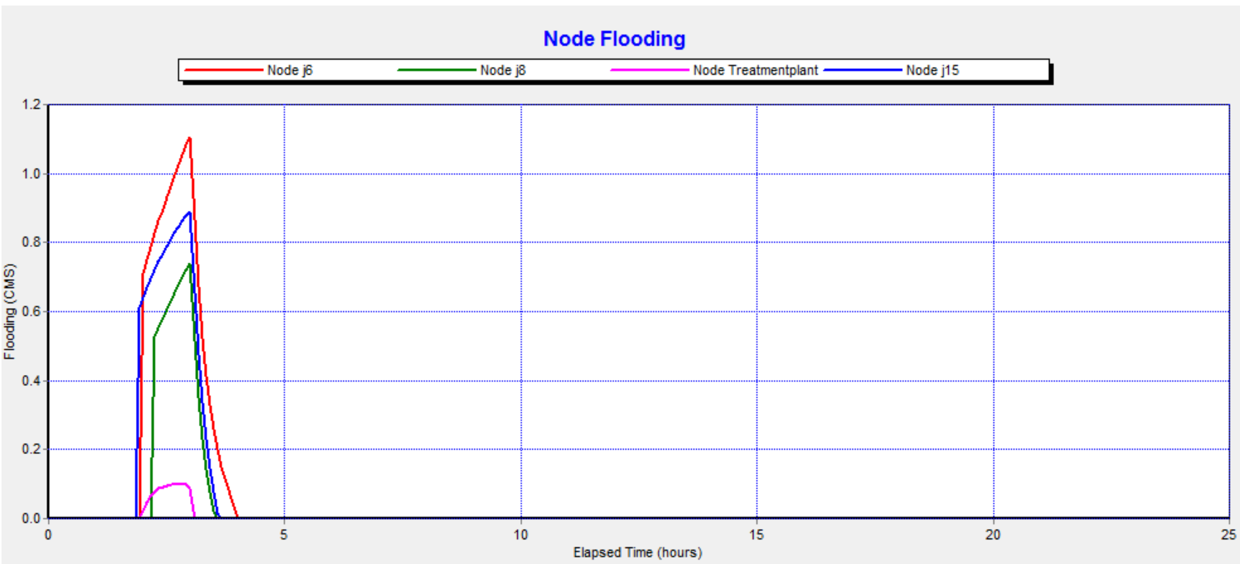

Figure 5.20. Junctions Flooding Summary for the Initial Depth and Precipitation Focus Scenario

Figure 5.19 represents flooding in the junctions flooded for a significant number of hours in this scenario. In this graph junction j11 is not shown because of the small number of hours getting flooded at the very beginning of the simulation. The time step for this simulation is 5 minutes and it cannot show occurrences within the first 5 minutes of the simulation. Junction j6 has the highest value for flooding as it is connected to the larger subcatchments in comparison with junction j8 and j15. The “TreatmentPlant” has lower flooding value although it is connected to many subcatchments. The reason for this low value is because “TreatmentPlant” does not need to wait for any outfall to open. Instead, it is assumed that it release water to avoid flooding into the “TreatmentPlantOutfall”, which is not the case for the other junctions. The “TreatmentPlant” has the initial depth which also causes R4 to be open and it helps the “TreatmentPlant” to have two outlets at the same time resulting in less flooding volume.

Figure 5.20 presents a profile plot for the Lower Road connections from j6 to j15. This figure presents a screenshot of the SWMM simulation which shows how water flows into the system. This screenshot is taken at 3:00 AMfor the 24-hour simulation beginning at 0:00 AM. This is the same time of the last maximum flooding (i.e., 2:59 for j6, j8 and j1). The time step for this simulation is 5 minutes so it is not possible to capture any screenshot in any time under 5 minutes. In this figure it is noted that junctions j11, j8, j6, j15, j12 and the Treatment Plant are "full" indicating that they are currently flooded.

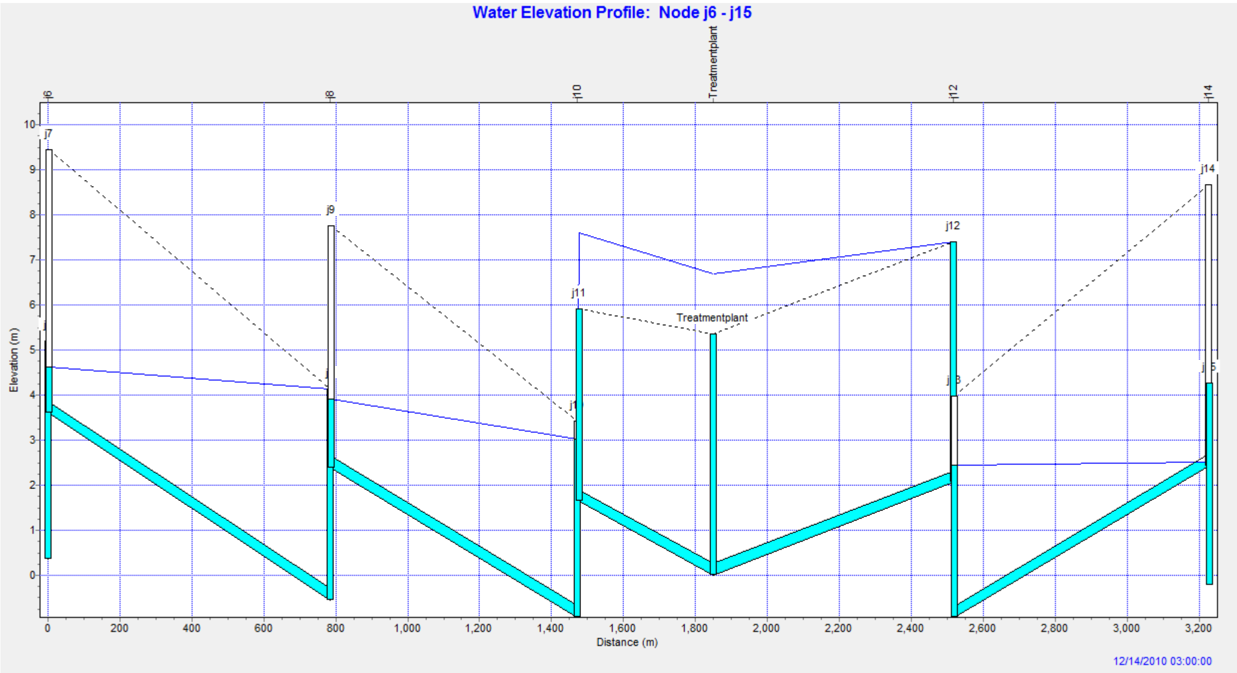

Figure 5.21. Profile plot for the Lower Road Connections from j6 to j15 at 3:00:00 for Initial Depth and Precipitation Focus Scenario

The total amount of TSS and Lead loaded into the outfalls is even higher than the worst case scenario. However, the difference is not big and it does not affect the definition of the "Worst Case" scenario.

Pumps P2 and P3 are working 100% of the time and that explain why junctions j10 and j13 are not flooded in this scenario. Other pumps are not working for the whole time. P4 is started up 2 times. Figure 5.21 provides validation for why this pump is started up twice during the simulation. A detailed look into Figure 5.21 showing the depth of the junction j8 makes it clear that, at first, the pump P4 starts working because the initial depth is 2.591 meters which is higher than the startup depth of the pump (1.829 meters). Then the pump turns on, and the depth of the junction decreases to 1.219 meters in which cause the pump turns off. Afterwards, at 19:00, the depth in junction j8 surpasses 1.829 meters which again causes the pump to restart as depicted in Figure 5.21 below. .

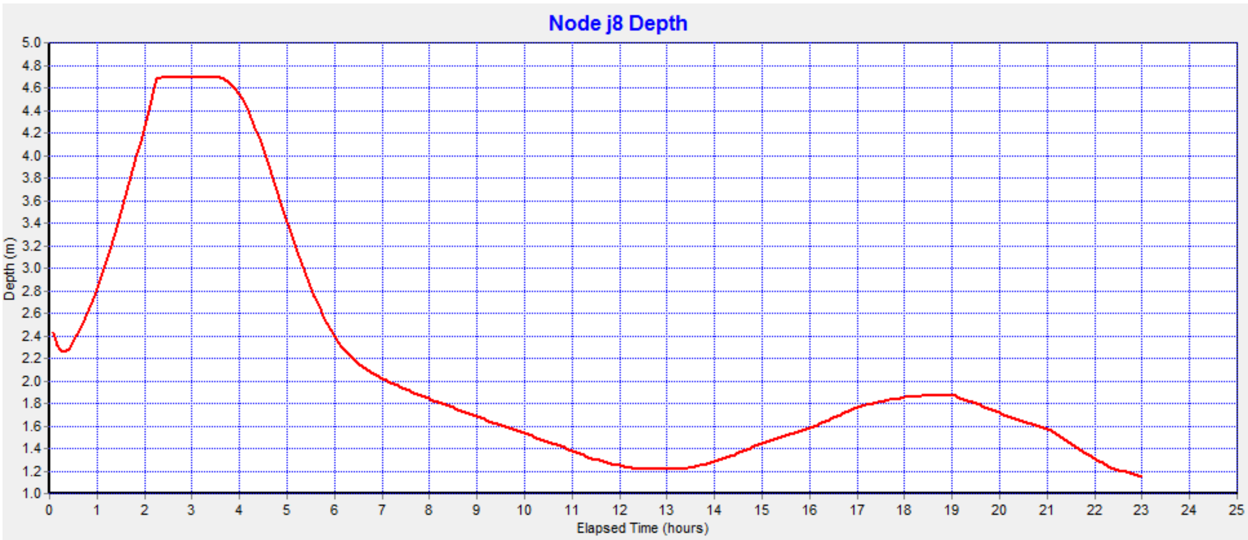

Figure 5.22. Junction j8 Depth in the Simulation of the Initial Depth and Precipitation Focus Scenario

Table 5.10 shows a summary of results for the Initial Depth and Precipitation Focus scenario.

Table 5.10. Summary of Results in the Initial Depth and Precipitation Focus Scenario

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO (CMS)	Average Flow for System's Outfalls (CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO(kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Initial Depth and Precipitation Focus Scenario	(9 junctions) 0.75	(11 junctions) 4.22	0.332	2.439	30.286	4926.923	0.181	1.405	P1: 43.59%,1 P2:100%,1 P3:100%,1 P4:74.1%,2 P5:45.87%,1

5.2.10. Initial Depth and Impervious Percentage Focus Scenario Results

The Initial Depth and Impervious Percentage Focus Scenario is presented in section 5.1.10. As before, the continuity errors for surface runoff, flow routing and quality routing are all acceptable at rates of -0.04%, -0.58% and -0.29%, respectively. None of the junctions have a continuity error more than 10%.

The runoff summary for this scenario is the same as for the Worst Case scenario because of the same impervious percentage for subcatchments, and the same front loaded precipitation case (P3) used for both scenarios. The Washoff summary is the same as the Worst Case and Precipitation Focus scenario because of the same amount of precipitation in the system for these scenarios.

Junctions' average depths on the High Road are the same as the depths in the Worst Case scenario and the Precipitation Focus scenario. The treatment plant average depth is lower than the Worst Case Scenario. For all the storage units, the average depths are very close to the values in the Worst Case scenario but higher than values in the Precipitation Focus scenario because of the initial depth considered for the junctions in the current scenario. The maximum depths for most junctions are very close to the Worst Case scenario. For the outfalls, because of the difference in the tidal level the average and maximum depth, flow is higher in the Worst Case scenario. Outfall flows are very close to the Precipitation Focus values.

Eleven junctions are surcharged and 10 junctions are flooded in this scenario. In comparison with the Worst Case scenario, the "TreatmentPlant" is surcharged for the same number of hours but other junctions are surcharged for fewer numbers of hours during the simulation. It seems that the change in the tidal level time series does not have any impact on the treatment plant surcharged time due to the same results for the two scenarios. Junction j11 is surcharged for less than 5 hours and this is attributed to j10 being surcharged for less than 3 hours so that less volume of water is pumped up to j11. The timing for the surcharged junctions is very close to the values in the Precipitation Focus Scenario, however, junction j14 is not flooded in the Precipitation Focus scenario as it is here.

For the flooded junctions, the same junctions are flooded as in the Worst Case scenario. All the junctions are flooded almost the same number of hours as for the Worst Case scenario except for j10 which is flooded for 0.14 fewer hours. According to the flooding results and the graph in Figure 5.22, all the flooding occurs in the first 5 hours of the simulated scenario because of the high precipitation during this time. In the Precipitation Focus scenario, only 5 junctions are flooded and the number of hours they are flooded is lower than the same value for the same junctions in the current scenario. This shows that the higher initial depth and impervious percentage in the junctions causes more flooding in the system. Comparing Figure 5.11 and Figure 5.22, the highest values of flooding are proof of the impact of the initial depth and the impervious percentage on the flooding volume.

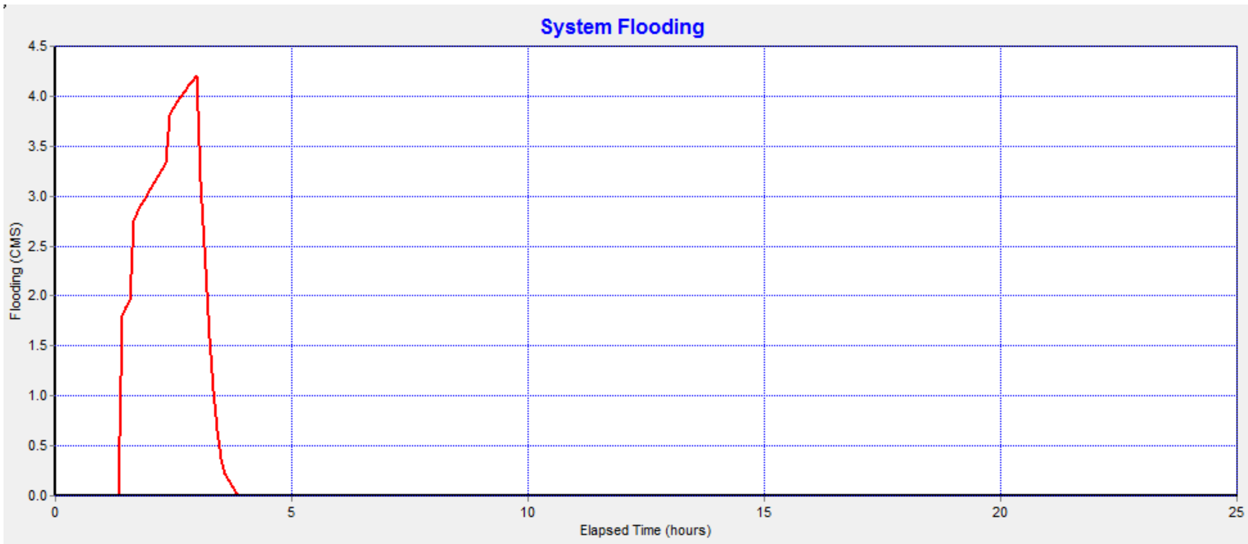

Figure 5.23. System Flooding for the Initial Depth and Impervious Focus Scenario

The amount of TSS and Lead which are loaded into the outfalls is close to the Worst Case scenario and lower than the Precipitation Focus scenario. The pumps are working very differently in this case compared to the Precipitation Focus scenario and the Worst Case scenario. The summary results for this scenario are shown in Table 5.11.

Table 5.11. Summary of Results for the Initial Depth and Impervious Percentage Focus Scenario

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO(CMS)	Average Flow for System's Outfalls (CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO(kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
Initial Depth and Impervious Percentage Focus Scenario	(10 junctions) 1.18	(11 junctions) 4.29	0.336	2.618	34.931	4667.476	0.152	1.310	P1: 33.81%,1 P2:100%,1 P3:100%,1 P4:69.46%,2 P5:37.98%,1

5.3 Analysis of the Results

Results of the 10 simulation model scenarios are presented above in section 5.2 and provide rationale for understanding the Arichat water system and the important occurrences discovered for each of the scenarios. Table 5.12 presents the collected summary of the scenario analysis results for all 10 scenarios as presented individually in Tables 5.2 through 5.11, respectively. This consolidated table is presented for comparative purposes and is referred to in the analysis presented below.

Table 5.12. Results summary for scenario 1 to 10

Scenario	Average Number of Flooding Hours	Average Number of Surcharging Hours	Average Flow for TPO(CMS)	Average Flow for System's Outfalls (CMS)	Total TSS in TPO (kg)	Total TSS Loaded to the Outfalls(kg)	Total Lead in TPO(kg)	Total Lead Loaded to the Outfalls(kg)	Pumps Working: The Percentage of Each Working, Number of Start ups
(1) SQ	---	---	0.12	0.388	1.201	56.217	0.015	0.029	P1:0%,0 P2:80.4%, 1 P3:85.29%,1 P4:20.91%,1 P5:0%,0
(2) Best Case	---	---	0.01	0.01	0.013	0.013	0.01	0.01	P1:0%,0 P2:0%,0 P3:0%,0 P4:0%,0 P5:0%,0
(3) Worst Case	(11 junctions) 1.94	(10 junctions) 6.54	0.405	2.805	36.229	4658.107	TPO:0.151	1.307	P1:84.35%,2 P2:100%,1 P3:100%,1 P4:100%,1 P5:38.82%,1
(4) Precipitation Focus	(5 junctions) 1.23	(10 junctions) 4.293	0.328	2.288	29.909	4909.817	0.179	1.399	P1:39.58%,1 P2:96.45%,1 P3:95.77%,1 P4:70.12%,2 P5:41.89%,1
(5) Tide Focus	---	(10 junctions) 2.47	0.185	0.608	2.534	32.337	0.015	0.022	P1:15.48%,1 P2:96.67%,1 P3:93.15%,1 P4:59.32%,1 P5:0%,0
(6) Tide and Initial Depth Focus	(5 junctions) 0.01	(10 junctions) 5.0525	0.344	1.6	27.476	3773.241	0.272	1.209	P1:100%,1 P2:100%,1 P3:100%,1 P4:100%,1 P5:81.21%,2
(7) Regulators and Precipitation Focus	(9 junctions) 9.226	(12 junctions) 16.19	0.487	0.487	118.214	118.214	0.327	0.327	P1:96.16%,1 P2:96.07%,1 P3:95.18%,1 P4:96.07%,1 P5:96.22%,1
(8) Tide and Impervious Focus	----	(6 junctions) 6.84	0.351	1.634	28.983	4080.723	0.298	1.311	P1:69.38%,1 P2:96.88%,1 P3:94.28%,1 P4:74.24%,1 P5:74.42%,1
(9) Initial Depth and Precipitation Focus	(9 junctions) 0.75	(11 junctions) 4.22	0.332	2.439	0.286	4926.923	0.181	1.405	P1:43.59%,1 P2:100%,1 P3:100%,1 P4:74.1%,2 P5:45.87%,1
(10) Initial Depth & Impervious Percentage Focus	(10 junctions) 1.18	(11 junctions) 4.29	0.336	2.618	34.931	4667.476	0.152	1.310	P1:33.81%,1 P2:100%,1 P3:100%,1 P4:69.46%,2 P5:37.98%,1

Table 5.13 shows the consolidated flooding summary for junctions flooded in the different scenarios, in hours. Also, it shows the frequency of being flooded for each junction.

Table 5.13. Flooding Summary for Junctions Flooded in Scenario 1 to 10

Scenarios / Junctions	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	Frequency
j4	-	-	-	-	-	-	0.67	-	-	-	1
j6	-	-	2.47	1.88	-	-	14.34	-	2.05	2.48	5
j7	-	-	0.01	-	-	0.01	-	-	0.01	0.01	4
j8	-	-	1.98	1.14	-	-	16.53	-	1.29	1.93	5
j9	-	-	0.01	-	-	0.01	-	-	0.01	0.01	4
j10	-	-	1.21	-	-	-	15.75	-	-	1.07	3
j11	-	-	1.66	0.19	-	0.01	4.41	-	0.19	1.65	6
j12	-	-	0.01	-	-	0.01	3.55	-	0.01	0.01	5
j13	-	-	-	-	-	-	10.38	-	-	-	1
j14	-	-	0.01	-	-	0.01	-	-	0.01	0.01	4
j15	-	-	2.09	1.56	-	-	12.33	-	1.73	2.14	5
TP	-	-	2.49	1.38	-	-	5.07	-	1.38	2.49	5

It is possible to analyze these flooding junctions based on the possibility of getting flooded which is concluded based on the frequency of flooding. Table 5.13 shows that j11 has the highest frequency of flooding in the 10 scenarios. Over the 10 scenarios, j11 is flooded in 6 of the 10 scenarios. Although the average number of hours this junction is flooded is low, because of the high number of occurrence for this junction, j11 is identified as the junction who has priority in terms of causing water quantity problems for the system. On other hand, j13 has the highest average number of hours being flooded (Table 5.13). The frequency of flooding occurrence for this junction is only one time in scenario 7. Therefore it is not a good decision if j13 may be introduced as a problematic junction since in 9 of 10 scenarios it is not at issue.

The next junctions with a high average number of hours flooded are j6, j8, j12, j15 and the TreatmentPlant. The frequency of occurrence for these junctions is also high. They are flooded in 5 of the 10 scenarios. Junction j12 was not flooded for a high number of hours; therefore, it is not designated as a problem junction for the system. The other 4 junctions can be introduced as junctions that may cause water quantity problems for the system. Based on personal communications with the Municipality of Richmond County Public Works engineer in Arichat, junctions j6 and j8 are more probable to get flooded in heavy storms in comparison with other junctions. This empirical report validates in part these model findings.

Table 5.14 shows the same values as shown in Table 5.13 for flooded junctions, in hours, but this time is prepared for surcharged junctions. Junctions getting surcharged are not necessarily flooded. Junction j8 to j13 have the highest frequency and among them, junction j11 has the highest mean number of hours flooded. Junction j6 has lower frequency with a high mean number of hours flooded. It seems j11 and j6 are the most problematic junctions in the system. Junction j15 is also surcharged 5 times with a mean duration of 7.914 hours which seems to be high in comparison with other junctions.

Table 5.14. Surcharging Summary for Junctions surcharged in Scenario 1 to 10

Scenarios / Junctions	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	Frequency
j4	-	-	-	-	-	-	0.96	-	-	-	1
j6	-	-	5	4.79	-	-	21.96	-	5.24	5.01	5
j7	-	-	3.09	2.39	-	0.05	21.4	-	2.62	3.05	6
j8	-	-	6.24	4.48	1.53	1.63	21.9	1.93	4.96	4.77	8
j9	-	-	5.8	3.37	1.97	2.16	21.21	2.42	3.59	4.04	8
j10	-	-	7.64	4	3.37	4.28	21.49	3.46	4.33	4.53	8
j11	-	-	11.11	8.81	3.28	18.6	8.32	20.62	9.4	6.72	8
j12	-	-	16.15	5.38	1.24	10.21	6.57	9.41	5.88	5.11	8
j13	-	-	6.25	2.31	3.43	3.44	21.89	3.46	2.36	3.61	8
j14	-	-	1.63	-	-	0.05	21.51	-	0.05	2.29	5
j15	-	-	4.46	4.14	-	-	21.94	-	4.55	4.48	5
TP	-	-	3.51	3.26	-	-	5.13	-	3.36	3.52	5

Table 5.15 shows a summary of the percentage of pumps utilized and the number of times they are started up in each scenario. Pump P1 seems to have poor performance. Based on Table 5.13,

j15, the inlet of the pump P1, is flooded in scenarios 3, 4, 7, 9 and 10, and mostly for a significant number of hours. But, as it is shown in Table 5.15, this pump is not working properly and the utilization percentage is very low for most of the scenarios. Pump P2 is working properly as j13, ,the inlet of this pump, is flooded only in scenario 7 and pump P2 utilization percentage for this scenario is close to 100%. Pump P3 also has a good performance. Junction j10 connected to this pump, is not flooded for a significant number of hours. P3 lifts up water to j11 and, j11 because of insufficient capacity, is flooded. Pumps P4 and P5 are connected to j8 and j6, respectively. These two junctions are the two junctions with high frequency of flooding and high average number of hours flooded. P4 and P5 performance seem moderate because the utilization percentage is high for most of the scenarios and for those scenarios that the utilization percentage is low, the number of hours the connected junctions are flooded is very low.

Table 5.15. Utilization Percentage for Pumps 1 to 5 in the Scenarios 1 to 10

scenarios / Pumps	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
P1	0%,0	0%,0	84.35%,1	39.58%,1	15.48%,1	100%,1	96.16%,1	69.38%,1	43.59%,1	33.81%,1
P2	80.49%,1	0%,0	100%,1	96.45%,1	96.67%,1	100%,1	96.07%,1	97.88%,1	100%,1	100%,1
P3	85.29%,1	0%,0	100%,1	95.77%,1	93.15%,1	100%,1	95.18%,1	94.28%,1	100%,1	100%,1
P4	20.91%,1	0%,0	100%,1	70%,2	59.32%,1	100%,1	96.07%,1	74.24%,1	74.1%,2	69.46%,2
P5	0%,0	0%,0	38.82%,1	41.89%,1	0%,0	81.21%,2	96.22%,1	74.42%,2	45.87%,1	37.98%,1

6. Conclusion, Suggestions and Recommendations for Future Study

This chapter has 3 sections: (6.1) Summary of Results, (6.2) Conclusions and Suggestions for Improvement of the Water System in Arichat, (6.3) Recommendations for Future Study. The first section of this chapter summarizes the results of this research in terms of the original research objectives (section 1.2). The second section presents the model conclusions and suggestions for improvement in the water system infrastructure in Arichat based on the results achieved in this research. The final section discusses recommendations for future study.

6.1. Summary of Results

The thesis research objectives were stated in Chapter 1, Section 1.2 and are presented again here. They are discussed in terms of the thesis results:

1) *Model a selected community's sewage and stormwater systems components using SWMM.* In achieving this objective, the community of Arichat, a coastal community located on the island of Isle Madame in Cape Breton, N.S is selected as the case study and SWMM, version 5 is used as the modelling tool in this research. In Chapter 3, SWMM capabilities are discussed. Chapter 4 explains how these capabilities are applied in modelling the Arichat water system. Water system components modeled in SWMM and the properties for each model component are presented in Chapter 4. This work accomplishes the stated objective.
2) *Acquire data on storms and storm impacts of stormwater on selected coastal communities.* In order to achieve this objective, precipitation and tidal level time series data were prepared in order to appropriately simulate the water system model in Arichat. These data are discussed in Chapter 4. Chapter 5 presents the simulation design for the structured investigation of simulated impacts on the water system. A suite of ten (10) simulation scenarios were developed and defined in Chapter 4 including defining the status quo scenario. The scenarios were constructed and used to test and validate the functioning and capacity of the actual system.
3) *Evaluate storm impacts using the SWMM stormwater model.* This objective is achieved through the structured analysis and results of the simulation of the water system model based on the 10 scenarios. The SWMM model was applied to all 10 scenarios using the data, and the impacts of all these scenarios analyzed and results presented in summary in

Chapter 5. The detailed results for all 10 scenarios are presented in Appendix B – Simulation Scenario Model Results. The model results assist in identifying problematic components in the system and uncovering solutions for problems that a future severe storm may cause. Chapter 5 discusses the SWMM model scenario by scenario analysis and compares each to the SQ Benchmark scenario in order to discover the sensitivity and the robustness of the Arichat water system. The final chapter, Chapter 6 suggests possible solutions for the problems.

4) *Communicate the results to communities and find strategies for managing adaptation in selected communities.* During the research process, this research was presented to community leaders in personal meetings, and at national and international conferences as part of the C-Change "community of practice" (Lane et al., 2013). Moreover, this thesis document has been prepared as a report that will be delivered directly to the Public Works Engineer at the Municipality of the County of Richmond, as well as to other C-Change project community partners. As noted elsewhere, the Municipality has been an excellent and most welcome provider of data for this research without whom the detail of the SWMM model would not have been as precise. Therefore, it is understood that the results and discussions of this research are of interest to and will be communicated directly to the Arichat community through this thesis report and pending presentations, etc.

6.2. Conclusions and Suggestions for Improvement of the Water System in Arichat

Section 5.3 discusses water system components which cause problems in the Arichat water system based on the observations of impacts from the simulations of the model in different scenarios. The following items present the focused summary of issues in the water system and conclude options for improved flow and performance of the Arichat water system.

6.2.1. Pump P1 Performance

Based on the discussion in Chapter 5, there is evidence that pump P1 does not perform appropriately and it seems its capacity for lifting up water from j15 is not sufficient. The performance of a pump is evaluated based on its performance curve. The suggestion here is to improve P1 performance by upgrading its performance curve. This can happen by either changing the pumps physical components, or by changing the pump itself with a new pump with a better characteristic performance curve.

6.2.2. Junction j11 Capacity

In addition to the pump with a problem, j11 is the most problematic junction in the system based on the analysis presented in section 5.3. The problem with this junction is its low surcharge depth and low maximum depth in comparison with other junctions in the system. Actually, junction j11 has the lowest values for these properties among all other junctions. The suggestion for improving the capacity of this junction is to change its surcharge depth and maximum depth to a higher value. It can happen by changing its physical appearance and/or lowering the level of the storage and manhole station.

6.2.3. Junctions j6 and j8 Capacities

Based on the results of section 5.3, junctions j6 and j8 are also junctions which cause problems. These junctions have also been pointed out as problematic in personal communication with the Arichat Public Works Engineer. These junctions receive water from large areas which cause lots of inflow into these junctions. As well, pumps P4 and P5 are only moderate performers based on the discussion in section 5.3. Consequently, under storm pressure, these two pumping stations do not have enough capacity to handle the large amount of water inflow into the system during a severe precipitation storm. It is suggested that adding another station to this area would reduce the burden on the two other pumping stations. Alternatively, another suggestion is to improve the performance of the pumps while increasing the capacity of these two junctions.

6.2.4. Treatment Plant Capacity

The ability of the water system to manage current and future more severe storms is limited by the capacity of the Treatment Plant. Although the treatment plant is able to aerate and settle sewage water in the system, some untreated water flows out of the system through the Treatment Plant outfall which is connected to the treatment plant. As noted above, the treatment plant is flooded in 5 of 10 scenarios simulated, which indicates that the capacity of the treatment plant is not enough during the large precipitation and storm events to treat all the water entering the system, and results in overflows and the release of untreated water into the harbour. The conclusion and suggestion here is that increasing the treatment plant capacity and improving the treatment plant facility, e.g., by including more efficient and rigorous aeration, more settling time, chlorine treatment, and a larger ponded area, would result in better overall water quality.

6.3. Recommendations for Future Study

A review of the research leads to recommendations for the future study of this work on coastal community water systems, especially in light of rising seas, and more frequent and severe storm surge. These recommendations are discussed below.

6.3.1 Data

The first and the most important item that should be considered in the future extension of this research is data. Each component in the SWMM can have different properties; lack of data for each component properties was one issue during this research. A recommendation can be finding more exact data for these components. A number of estimations were used in this research due to lack of site specific data. For example, precipitation time series and tidal level time series were proxy of Arichat for this research. Exact data related to the precipitation and tidal values can be searched and found for Arichat.

6.3.2 Water Quality

Water quality is an important issue in the water systems. The data used for estimating water-borne pollutants and treatment plant effectiveness in this model were taken from examples used for different contexts in SWMM. In order to achieve real results related to Arichat community, it is recommended that the actual values related to the treatment plant and pollutants existing in this community be used for this water system modelling in SWMM.

6.3.3. Application to Other Coastal Communities

This research and modelling work is provided for the ultimate use and application to other coastal communities' water system, e.g., Gibsons, British Columbia. The model is made available to other communities, e.g., in the C-Change project, and can be readily used as a basic guide for a better simulation of their water systems with SWMM.

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Appendix A
Precipitation Data Summary

The following histograms contained in this appendix are the maximum and minimum monthly average precipitation in the precipitation data set (Environment Canada, 2012). For a better view of the histogram data, please note that the precipitation values between 0 mm and 1 mm are eliminated from the data.

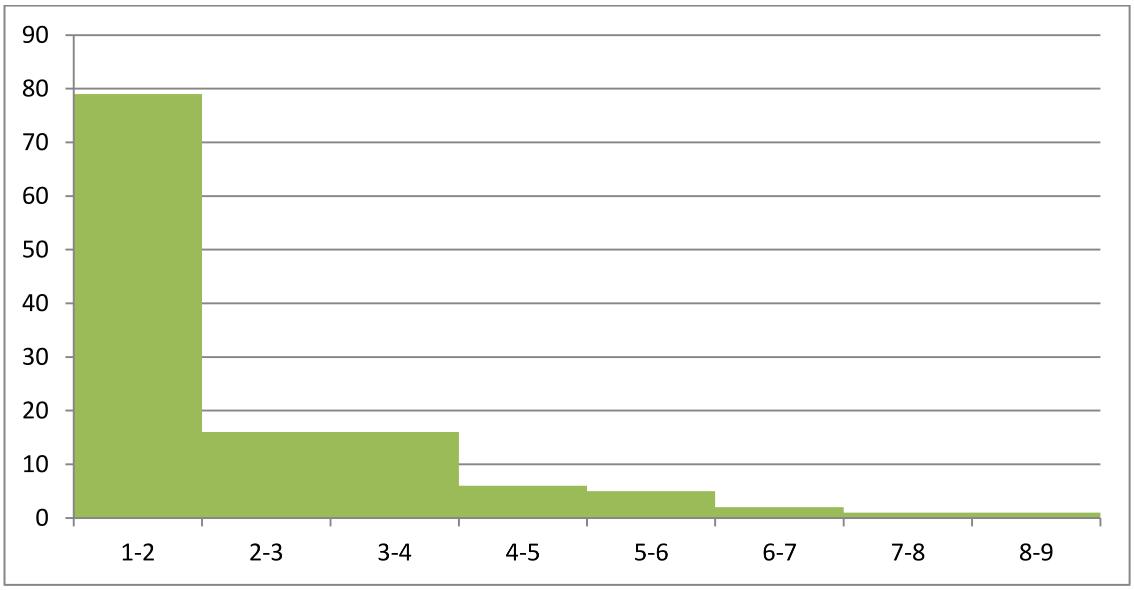

Figure A.1. Maximum monthly average precipitation histogram excluding 0mm to 1mm precipitation

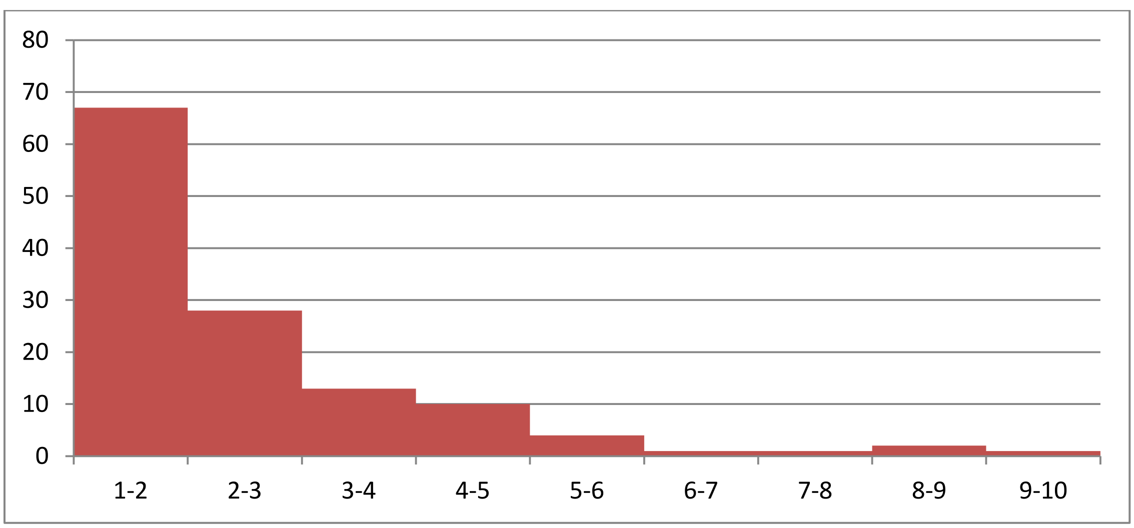

Figure A.2. Minimum Monthly average precipitation precipitation histogram excluding 0mm to 1mm precipitation

Appendix B

SWMM Model Regulator Rules Definitions

The following lines of code are used in the SWMM model to show the “Open” alternative for regulators in the Arichat water system. The code applies to all six (6) of the regulators in the system.

```
RULE R1
IF NODE j6 DEPTH >= 2.591
THEN WEIR R1 SETTING = 1
PRIORITY 1

RULE R2
IF NODE j8 DEPTH >= 2.591
THEN WEIR R2 SETTING = 1
PRIORITY 1

RULE R3
IF NODE j10 DEPTH >= 2.591
THEN WEIR R3 SETTING = 1
PRIORITY 1

RULE R4
IF NODE Treatmentplant DEPTH >= 2.591
THEN WEIR R4 SETTING = 1
PRIORITY 1

RULE R5
IF NODE j13 DEPTH >= 2.591
THEN WEIR R5 SETTING = 1
PRIORITY 1

RULE R6
IF NODE j15 DEPTH >= 2.591
THEN WEIR R6 SETTING = 1
PRIORITY 1
```

The following lines show the “Close” alternative for regulators in the Arichat water system modelling in SWMM. The code applies to all six (6) of the regulators in the system.

```
RULE R1
IF NODE j6 DEPTH >= 0
THEN WEIR R1 SETTING = 0
PRIORITY 1

RULE R2
IF NODE j8 DEPTH >= 0
THEN WEIR R2 SETTING = 0
PRIORITY 1

RULE R3
IF NODE j10 DEPTH >= 0
THEN WEIR R3 SETTING = 0
PRIORITY 1

RULE R4
IF NODE Treatmentplant DEPTH >= 0
THEN WEIR R4 SETTING = 0
PRIORITY 1

RULE R5
IF NODE j13 DEPTH >= 0
THEN WEIR R5 SETTING = 0
PRIORITY 1

RULE R6
IF NODE j15 DEPTH >= 0
THEN WEIR R6 SETTING = 0
PRIORITY 1
```

Appendix C
SWMM Model Simulation Scenario Results

The Arichat water system model output status reports for each of the 10 simulation scenarios is provided in this appendix.

NOTE: The summary statistics displayed in these reports are based on results found at every computational time step, not just on results from each reporting time step.

C.1. Status Quo Scenario Simulation Status Report

****

Table C.1. Analysis Options: SQ Scenario

****

Flow Units............... CMS
Process Models:
Rainfall/Runoff........ YES
Snowmelt............... NO
Groundwater............ NO
Flow Routing........... YES
Ponding Allowed........ YES
Water Quality.......... YES
Infiltration Method...... HORTON
Flow Routing Method...... DYNWAVE
Starting Date............ DEC-14-2010 00:00:00
Ending Date.............. DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step........ 30.00 sec

****

Table C.2. Runoff Quantity Continuity: SQ Scenario

****

	Volume Hectare-m	Depth mm
Total Precipitation	1.381	6.925
Evaporation Loss	0.000	0.000
Infiltration Loss........	0.924	4.636
Surface Runoff...........	0.413	2.071
Final Surface Storage....	0.044	0.222
Continuity Error (%).....	-0.060	

****
Table C.3. Runoff Quality Continuity: SQ Scenario

****

	TSS kg	Lead kg
Initial Buildup	6111.392	0.000
Surface Buildup	1.777	0.048
Wet Deposition...........	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss........	0.000	0.000
BMP Removal.............	0.000	0.000
Surface Runoff...........	193.201	0.048
Remaining Buildup........	5919.581	0.000
Continuity Error (%).....	0.006	0.000

****
Table C.4. Flow Routing Continuity: SQ Scenario

****

	Volume Hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.410	4.104
Groundwater Inflow	0.000	0.000
RDII Inflow..............	0.000	0.000
External Inflow	1.727	17.265
External Outflow	1.476	14.756
Internal Outflow	0.000	0.000
Storage Losses...........	0.000	0.000
Initial Stored Volume....	0.000	0.000
Final Stored Volume	0.627	6.271
Continuity Error (%).....	1.600	

****

Table C.5. Quality Routing Continuity: SQ Scenario

****

	TSS kg	Lead kg
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	192.420	0.048
Groundwater Inflow	0.000	0.000
RDII Inflow..............	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding........	0.000	0.000
External Outflow	56.181	0.029
Mass Reacted	58.510	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass........	80.662	0.020
Continuity Error (%).....	-1.524	-1.524

****

Table C.6. Highest Continuity Errors: SQ Scenario

****

Node j6 (7.63%)
Node j8 (4.32%)
Node j13 (4.08%)
Node j15 (3.45%)
Node j10 (2.97%)

****

Table C.7. Time-Step Critical Elements: SQ Scenario

****

None

****

Table C.8. Highest Flow Instability Indexes: SQ Scenario

****

Link R5 (47)
Link R3 (10)
Link R2 (3)

****

Table C.9. Routing Time Step Summary: SQ Scenario

****

Minimum Time Step : 30.00 sec
Average Time Step : 29.99 sec
Maximum Time Step : 30.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00

****

Table C.10. Subcatchment Runoff Summary: SQ Scenario

****

Subcatchmen	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	6.93	0.00	0.00	5.19	1.56	0.41	0.01	0.225
S2	6.93	0.00	0.00	5.19	1.58	0.35	0.00	0.227
S13	6.93	0.00	0.00	3.46	2.99	0.67	0.01	0.432
S11	6.93	0.00	0.00	3.46	3.28	0.14	0.00	0.474
S10	6.93	0.00	0.00	3.46	3.28	0.15	0.00	0.473
S9	6.93	0.00	0.00	3.46	3.19	0.29	0.00	0.460
S8	6.93	0.00	0.00	3.46	3.17	0.31	0.00	0.458
S7	6.93	0.00	0.00	3.46	3.10	0.45	0.01	0.447
S6	6.93	0.00	0.00	5.19	1.64	0.14	0.00	0.237
S3	6.93	0.00	0.00	5.19	1.57	0.39	0.01	0.226
S4	6.93	0.00	0.00	5.19	1.55	0.44	0.01	0.224
S5	6.93	0.00	0.00	5.19	1.61	0.23	0.00	0.233
S12	6.93	0.00	0.00	5.19	1.63	0.18	0.00	0.235

****

Table C.11. Subcatchment Washoff Summary: SQ Scenario

****

Subcatchment	TSS kg	Lead kg
S1	4.012	0.001
S2	13.769	0.003
S13	49.727	0.012
S11	11.069	0.003
S10	11.728	0.003
S9	21.341	0.005
S8	23.169	0.006
S7	13.386	0.003
S6	4.177	0.001
S3	9.528	0.002
S4	10.932	0.003
S5	6.778	0.002
S12	13.584	0.003
System	193.201	0.048

****

Table C.12. Node Depth Summary: SQ Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min
j2	JUNCTION	0.02	0.02	17.70	0 23:00
j4	JUNCTION	0.02	0.02	20.13	0 23:00
j1	JUNCTION	0.02	0.02	22.58	0 23:00
j5	JUNCTION	0.01	0.01	21.96	0 23:00
j16	JUNCTION	0.01	0.01	23.01	0 22:29
j3	JUNCTION	0.02	0.02	16.17	0 23:00
Treatmentplant	JUNCTION	0.44	0.69	0.70	0 19:01
j11	JUNCTION	0.15	0.17	1.81	0 04:26
j9	JUNCTION	0.03	0.13	2.52	0 19:33
j7	JUNCTION	0.00	0.00	3.59	0 00:00
j12	JUNCTION	0.12	0.15	2.21	0 06:46
j14	JUNCTION	0.00	0.00	2.43	0 00:00
TreatmentPlantOutfall	OUTFALL	0.22	0.31	0.31	0 19:01
Outfall4	OUTFALL	0.87	1.33	1.33	0 19:00
Outfall6	OUTFALL	1.10	1.56	1.33	0 19:00
Outfall1	OUTFALL	0.51	0.97	1.33	0 19:00
Outfall2	OUTFALL	1.42	1.88	1.33	0 19:00
Outfall3	OUTFALL	1.79	2.25	1.33	0 19:00
Outfall5	OUTFALL	1.80	2.26	1.33	0 19:00
j6	STORAGE	0.49	0.96	1.32	0 19:04
j8	STORAGE	1.31	1.88	1.33	0 19:00
j10	STORAGE	1.69	2.25	1.33	0 19:00
j13	STORAGE	1.78	2.26	1.33	0 19:00
j15	STORAGE	1.00	1.56	1.33	0 19:00

****
Table C.13. Node Inflow Summary: SQ Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	0.005	0.006	0	23:00	0.346	0.418
j4	JUNCTION	0.006	0.006	0	23:00	0.440	0.458
j1	JUNCTION	0.005	0.005	0	23:00	0.403	0.403
j5	JUNCTION	0.003	0.003	0	23:00	0.226	0.226
j16	JUNCTION	0.002	0.002	0	22:59	0.139	0.139
j3	JUNCTION	0.005	0.006	0	23:00	0.383	0.455
Treatmentplant	JUNCTION	0.006	0.206	0	19:00	0.430	10.013
j11	JUNCTION	0.000	0.043	0	03:23	0.000	3.064
j9	JUNCTION	0.000	0.029	0	17:40	0.000	0.504
j7	JUNCTION	0.000	0.000	0	00:00	0.000	0.000
j12	JUNCTION	0.000	0.031	0	04:31	0.000	2.050
j14	JUNCTION	0.000	0.000	0	00:00	0.000	0.000
TreatmentPlantOutfall	OUTFALL	0.000	0.206	0	19:01	0.000	9.930
Outfall4	OUTFALL	0.000	0.121	0	19:00	0.000	4.080
Outfall6	OUTFALL	0.000	0.102	0	05:00	0.000	3.320
Outfall1	OUTFALL	0.000	0.034	0	18:00	0.000	1.423
Outfall2	OUTFALL	0.000	0.118	0	05:00	0.000	3.681
Outfall3	OUTFALL	0.000	0.145	0	02:51	0.000	5.132
Outfall5	OUTFALL	0.000	0.504	0	00:00	0.000	4.455
j6	STORAGE	0.006	0.045	0	18:00	0.446	1.512
j8	STORAGE	0.004	0.126	0	05:00	0.308	3.092
j10	STORAGE	0.000	0.150	0	02:52	0.000	5.395
j13	STORAGE	0.004	0.504	0	00:00	0.320	4.348
j15	STORAGE	0.009	0.111	0	05:00	0.663	2.600

****
Table C.14. Node Surcharge Summary: SQ Scenario

****
No nodes were surcharged.

****
Table C.15. Node Flooding Summary: SQ Scenario

****
No nodes were flooded.

****

Table C.16. Storage Volume Summary: SQ Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	0.490	10	0	0.961	20	0 19:04	0.034
j8	1.309	28	0	1.876	40	0 19:00	0.091
j10	1.686	39	0	2.247	52	0 19:00	0.124
j13	1.777	36	0	2.257	46	0 19:00	0.105
j15	0.996	22	0	1.557	35	0 19:00	0.063

****

Table C.17. Outfall Loading Summary: SQ Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	99.71	0.120	0.206	9.930	1.201	0.015
Outfall4	100.00	0.049	0.121	4.080	0.000	0.000
Outfall6	100.00	0.040	0.102	3.320	27.706	0.007
Outfall1	97.65	0.018	0.034	1.423	7.202	0.002
Outfall2	100.00	0.044	0.118	3.681	12.870	0.003
Outfall3	100.00	0.062	0.145	5.132	2.748	0.001
Outfall5	99.96	0.054	0.504	4.455	4.489	0.001
System	99.62	0.388	0.704	32.021	56.217	0.029

****

Table C.18. Link Flow Summary: SQ Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.001	0 23:00	0.65	0.04	0.14
c6	CONDUIT	0.006	0 23:00	0.09	0.00	0.50
c9	CONDUIT	0.004	0 23:00	1.11	0.00	0.52
c16	CONDUIT	0.206	0 19:01	1.21	5.71	0.81
c5	CONDUIT	0.003	0 23:00	0.01	0.00	0.51
c11	CONDUIT	0.029	0 19:33	0.71	0.52	0.76
c13	CONDUIT	0.031	0 06:46	0.70	0.69	0.80
c14	CONDUIT	0.000	0 00:00	0.00	0.00	0.50
c2	CONDUIT	0.000	0 23:00	0.22	0.03	0.13
c4	CONDUIT	0.000	0 23:00	0.21	0.02	0.12
c8	CONDUIT	0.005	0 23:00	0.03	0.00	0.51
c10	CONDUIT	0.000	0 00:00	0.00	0.00	0.50
c15	CONDUIT	0.002	0 20:36	0.08	0.00	0.51
c12	CONDUIT	0.043	0 04:26	0.95	0.82	0.84
c3	CONDUIT	0.001	0 23:00	0.44	0.04	0.14
c7	CONDUIT	0.006	0 23:00	0.03	0.00	0.51
P2	PUMP	0.031	0 04:31		1.00	
p3	PUMP	0.043	0 03:23		1.00	
P5	PUMP	0.000	0 00:00		0.00	
P1	PUMP	0.000	0 00:00		0.00	
P4	PUMP	0.029	0 17:40		1.00	
R4	WEIR	0.121	0 19:00			0.51
R6	WEIR	0.102	0 05:00			0.60
R1	WEIR	0.034	0 18:00			0.37
R2	WEIR	0.118	0 05:00			0.72
R3	WEIR	0.145	0 02:51			0.87
R5	WEIR	0.504	0 00:00			0.87

****

Table C.19. Flow Classification Summary: SQ Scenario

****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
		--------Fraction of Time in Flow Class --------								
c1	1.00	0.00	0.00	0.00	0.01	0.99	0.00	0.00	1.69	0.0000
c6	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.04	0.0000
c9	1.00	0.00	0.00	0.00	0.87	0.13	0.00	0.00	0.44	0.0000
c1	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.48	0.0043
c5	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.01	0.0000
c1	1.00	0.00	0.77	0.00	0.23	0.00	0.00	0.00	0.11	0.0004
c13	1.00	0.00	0.20	0.00	0.80	0.00	0.00	0.00	0.39	0.0002
c14	1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.60	0.0000
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.59	0.0000
c8	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.02	0.0000
c10	1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c15	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.02	0.0000
c12	1.00	0.00	0.15	0.00	0.85	0.00	0.00	0.00	0.52	0.0003
c3	1.00	0.00	0.00	0.00	0.04	0.96	0.00	0.00	1.14	0.0000
c7	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.02	0.0000

****

Table C.20. Conduit Surcharge Summary: SQ Scenario

****

Conduit	--------- Hours Full -------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c16	0.01	0.01	0.01	19.57	0.01

****
Table C.21. Pumping Summary: SQ Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Curve Low	% Time Off Pump Curve High
P2	80.40	1	0.00	0.03	0.03	2.050	7.07	0.0	100.0
p3	85.29	1	0.00	0.04	0.04	3.065	7.40	0.0	100.0
P5	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P4	20.91	1	0.00	0.03	0.03	0.504	1.93	0.0	100.0

Analysis begun on: Tue Apr 23 17:35:51 2013
Analysis ended on: Tue Apr 23 17:35:51 2013
Total elapsed time: < 1 sec

C.2. Best Case Scenario Simulation Status Report

****
Table C.22. Analysis Options: Best Case Scenario

****
Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed .... YES
Water Quality .... YES
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****

Table C.23. Runoff Quantity Continuity: Best Case Scenario

****

	Volume hectare-m	Depth mm
	------------	----------
Total Precipitation	1.381	6.925
Evaporation Loss	0.000	0.000
Infiltration Loss	0.924	4.636
Surface Runoff	0.413	2.071
Final Surface Storage	0.044	0.222
Continuity Error (%)	-0.060	

****

Table C.24. Runoff Quality Continuity: Best Case Scenario

****

	TSS kg	Lead kg
	----------	----------
Initial Buildup	6111.392	0.000
Surface Buildup	1.777	0.048
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	193.201	0.048
Remaining Buildup	5919.581	0.000
Continuity Error (%)	0.006	0.000

****
Table C.25. Flow Routing Continuity: Best Case Scenario

****

	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.410	4.104
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	0.081	0.811
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.325	3.252
Continuity Error (%)	0.998	

****
Table C.26. Quality Routing Continuity: Best Case Scenario

****

	TSS kg	Lead kg
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	192.420	0.048
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	0.000	0.000
External Outflow	0.013	0.010
Mass Reacted	41.678	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	140.133	0.035
Continuity Error (%)	5.506	5.506

****
Table C.27. Highest Continuity Errors: Best Case Scenario

****
Node j6 (14.08%)
Node j13 (11.06%)
Node j8 (10.70%)
Node j10 (9.69%)
Node j15 (8.59%)

****
Table C.28. Time-Step Critical Elements: Best Case Scenario

****
None

****
Table C.29. Highest Flow Instability Indexes: Best Case Scenario

****
All links are stable.

****
Table C.30. Routing Time Step Summary: Best Case Scenario

****
Minimum Time Step : 30.00 sec
Average Time Step : 29.99 sec
Maximum Time Step : 30.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00

****
Table C.31. Subcatchment Runoff Summary: Best Case Scenario

****

Subcatchmt	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10^6 ltr	Peak Runoff CMS	Runoff Coeff
S1	6.93	0.00	0.00	5.19	1.56	0.41	0.01	0.225
S2	6.93	0.00	0.00	5.19	1.58	0.35	0.00	0.227
S13	6.93	0.00	0.00	3.46	2.99	0.67	0.01	0.432
S11	6.93	0.00	0.00	3.46	3.28	0.14	0.00	0.474
S10	6.93	0.00	0.00	3.46	3.28	0.15	0.00	0.473
S9	6.93	0.00	0.00	3.46	3.19	0.29	0.00	0.460
S8	6.93	0.00	0.00	3.46	3.17	0.31	0.00	0.458
S7	6.93	0.00	0.00	3.46	3.10	0.45	0.01	0.447
S6	6.93	0.00	0.00	5.19	1.64	0.14	0.00	0.237
S3	6.93	0.00	0.00	5.19	1.57	0.39	0.01	0.226
S4	6.93	0.00	0.00	5.19	1.55	0.44	0.01	0.224
S5	6.93	0.00	0.00	5.19	1.61	0.23	0.00	0.233
S12	6.93	0.00	0.00	5.19	1.63	0.18	0.00	0.235

****

Table C.32. Subcatchment Washoff Summary: Best Case Scenario

****

Subcatchment	TSS kg	Lead kg
S1	4.012	0.001
S2	13.769	0.003
S13	49.727	0.012
S11	11.069	0.003
S10	11.728	0.003
S9	21.341	0.005
S8	23.169	0.006
S7	13.386	0.003
S6	4.177	0.001
S3	9.528	0.002
S4	10.932	0.003
S5	6.778	0.002
S12	13.584	0.003
System	193.201	0.048

****
Table C.33. Node Depth Summary: Best Case Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min
j2	JUNCTION	0.02	0.02	17.70	0 23:00
j4	JUNCTION	0.02	0.02	20.13	0 23:00
j1	JUNCTION	0.02	0.02	22.58	0 23:00
j5	JUNCTION	0.01	0.01	21.96	0 23:00
j16	JUNCTION	0.01	0.01	23.01	0 22:29
j3	JUNCTION	0.02	0.02	16.17	0 23:00
Treatmentplant	JUNCTION	0.14	0.15	0.16	0 23:00
j11	JUNCTION	0.00	0.00	1.64	0 00:00
j9	JUNCTION	0.00	0.00	2.39	0 00:00
j7	JUNCTION	0.00	0.00	3.59	0 00:00
j12	JUNCTION	0.00	0.00	2.06	0 00:00
j14	JUNCTION	0.00	0.00	2.43	0 00:00
TreatmentPlantOutfall	OUTFALL	0.06	0.07	0.07	0 23:00
Outfall4	OUTFALL	0.56	1.17	1.17	0 10:00
Outfall6	OUTFALL	0.76	1.40	1.17	0 10:00
Outfall1	OUTFALL	0.29	0.81	1.17	0 10:00
Outfall2	OUTFALL	1.08	1.72	1.17	0 10:00
Outfall3	OUTFALL	1.45	2.09	1.17	0 10:00
Outfall5	OUTFALL	1.46	2.10	1.17	0 10:00
j6	STORAGE	0.30	0.67	1.03	0 23:00
j8	STORAGE	0.29	0.63	0.08	0 23:00
j10	STORAGE	0.19	0.41	-0.51	0 23:00
j13	STORAGE	0.22	0.47	-0.46	0 23:00
j15	STORAGE	0.33	0.73	0.50	0 23:00

****
Table C.34. Node Inflow Summary: Best Case Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	0.005	0.006	0 23:00	0.346	0.418
j4	JUNCTION	0.006	0.006	0 23:00	0.440	0.458
j1	JUNCTION	0.005	0.005	0 23:00	0.403	0.403
j5	JUNCTION	0.003	0.003	0 23:00	0.226	0.226
j16	JUNCTION	0.002	0.002	0 22:59	0.139	0.139
j3	JUNCTION	0.005	0.006	0 23:00	0.383	0.455
Treatmentplant	JUNCTION	0.006	0.011	0 23:00	0.430	0.838
j11	JUNCTION	0.000	0.000	0 00:00	0.000	0.000
j9	JUNCTION	0.000	0.000	0 00:00	0.000	0.000
j7	JUNCTION	0.000	0.000	0 00:00	0.000	0.000
j12	JUNCTION	0.000	0.000	0 00:00	0.000	0.000
j14	JUNCTION	0.000	0.000	0 00:00	0.000	0.000
TreatmentPlantOutfall	OUTFALL	0.000	0.011	0 23:00	0.000	0.811
Outfall4	OUTFALL	0.000	0.000	0 00:00	0.000	0.000
Outfall6	OUTFALL	0.000	0.000	0 00:00	0.000	0.000
Outfall1	OUTFALL	0.000	0.000	0 00:00	0.000	0.000
Outfall2	OUTFALL	0.000	0.000	0 00:00	0.000	0.000
Outfall3	OUTFALL	0.000	0.000	0 00:00	0.000	0.000
Outfall5	OUTFALL	0.000	0.000	0 00:00	0.000	0.000
j6	STORAGE	0.006	0.010	0 23:00	0.446	0.776
j8	STORAGE	0.004	0.009	0 23:00	0.308	0.702
j10	STORAGE	0.000	0.006	0 23:00	0.000	0.454
j13	STORAGE	0.004	0.007	0 23:00	0.320	0.527
j15	STORAGE	0.009	0.011	0 23:00	0.663	0.802

****
Table C.35. Node Surcharge Summary: Best Case Scenario

****
No nodes were surcharged.

****
Table C.36. Node Flooding Summary: Best Case Scenario

****
No nodes were flooded.

****

Table C.37. Storage Volume Summary: Best Case Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	0.304	6	0	0.667	14	0 23:00	0.000
j8	0.287	6	0	0.627	13	0 23:00	0.000
j10	0.186	4	0	0.410	9	0 23:00	0.000
j13	0.221	5	0	0.469	10	0 23:00	0.000
j15	0.326	7	0	0.733	16	0 23:00	0.000

****

Table C.38. Outfall Loading Summary: Best Case Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	97.07	0.010	0.011	0.811	0.013	0.010
Outfall4	0.00	0.000	0.000	0.000	0.000	0.000
Outfall6	0.00	0.000	0.000	0.000	0.000	0.000
Outfall1	0.00	0.000	0.000	0.000	0.000	0.000
Outfall2	0.00	0.000	0.000	0.000	0.000	0.000
Outfall3	0.00	0.000	0.000	0.000	0.000	0.000
Outfall5	0.00	0.000	0.000	0.000	0.000	0.000
System	13.87	0.010	0.011	0.811	0.013	0.010

****

Table C.39. Link Flow Summary: Best Case Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.001	0 23:00	0.65	0.04	0.14
c6	CONDUIT	0.006	0 23:00	0.28	0.00	0.12
c9	CONDUIT	0.004	0 23:00	1.13	0.00	0.48
c16	CONDUIT	0.011	0 23:00	0.34	0.31	0.22
c5	CONDUIT	0.003	0 23:00	1.39	0.00	0.34
c11	CONDUIT	0.000	0 00:00	0.00	0.00	0.50
c13	CONDUIT	0.000	0 00:00	0.00	0.00	0.30
c14	CONDUIT	0.000	0 00:00	0.00	0.00	0.50
c2	CONDUIT	0.000	0 23:00	0.22	0.03	0.13
c4	CONDUIT	0.000	0 23:00	0.21	0.02	0.12
c8	CONDUIT	0.005	0 23:00	1.71	0.00	0.45
c10	CONDUIT	0.000	0 00:00	0.00	0.00	0.50
c15	CONDUIT	0.002	0 20:35	1.29	0.00	0.51
c12	CONDUIT	0.000	0 00:00	0.00	0.00	0.30
c3	CONDUIT	0.001	0 23:00	0.44	0.04	0.14
c7	CONDUIT	0.006	0 23:00	2.07	0.00	0.30
P2	PUMP	0.000	0 00:00	0.00		
p3	PUMP	0.000	0 00:00	0.00		
P5	PUMP	0.000	0 00:00	0.00		
P1	PUMP	0.000	0 00:00	0.00		
P4	PUMP	0.000	0 00:00	0.00		
R4	WEIR	0.000	0 00:00			0.00
R6	WEIR	0.000	0 00:00			0.00
R1	WEIR	0.000	0 00:00			0.00
R2	WEIR	0.000	0 00:00			0.00
R3	WEIR	0.000	0 00:00			0.00
R5	WEIR	0.000	0 00:00			0.00

****

Table C.40. Flow Classification Summary: Best Case Scenario

****

Conduit	Adjusted /Actual Length	Fraction of Time in Flow Class Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
c1	1.00	0.00	0.00	0.00	0.01	0.99	0.00	0.00	1.68	0.0000
c6	1.00	0.00	0.00	0.00	0.98	0.02	0.00	0.00	0.29	0.0000
c9	1.00	0.00	0.00	0.00	0.85	0.15	0.00	0.00	0.52	0.0000
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.37	0.0001
c5	1.00	0.00	0.00	0.00	0.86	0.14	0.00	0.00	0.60	0.0000
c11	1.00	0.01	0.99	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c13	1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c14	1.00	0.01	0.99	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.60	0.0000
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.60	0.0000
c8	1.00	0.00	0.00	0.00	0.83	0.17	0.00	0.00	0.75	0.0000
c10	1.00	0.01	0.99	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c15	1.00	0.00	0.00	0.00	0.87	0.12	0.00	0.00	0.50	0.0000
c12	1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c3	1.00	0.00	0.00	0.00	0.09	0.91	0.00	0.00	1.13	0.0000
c7	1.00	0.00	0.00	0.00	0.76	0.24	0.00	0.00	1.16	0.0000

****

Table C.41. Conduit Surcharge Summary: Best Case Scenario

****

No conduits were surcharged.

****

Table C.42. Pumping Summary: Best Case Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Curve Low	% Time Off Pump Curve High
P2	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P3	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P5	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P1	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P4	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0

Analysis begun on: Thu May 02 14:03:27 2013
Analysis ended on: Thu May 02 14:03:27 2013
Total elapsed time: < 1 sec

C.3. Worst Case Scenario Simulation Status Report

****

Table C.43. Analysis Options: Worst Case Scenario

****

Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ......YES
Water Quality .... YES
Infiltration Method ...... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step ....30.00 sec

****

Table C.44. Runoff Quantity Continuity: Worst Case Scenario

****

	Volume hectare-m	Depth mm
Total Precipitation	17.148	86.000
Evaporation Loss	0.000	0.000
Infiltration Loss	0.776	3.891
Surface Runoff	16.362	82.061
Final Surface Storage	0.016	0.082
Continuity Error (%)	-0.039	

****

Table C.45. Runoff Quality Continuity: Worst Case Scenario

****

	TSS kg	Lead kg
Initial Buildup	6111.392	0.000
Surface Buildup	1768.215	1.528
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	6111.496	1.528
Remaining Buildup	1768.215	0.000
Continuity Error (%)	-0.001	0.000

****

Table C.46. Flow Routing Continuity: Worst Case Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-------------	-------------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	16.362	163.625
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	1.900	18.998
External Outflow	16.075	160.749
Internal Outflow	2.232	22.323
Storage Losses	0.000	0.000
Initial Stored Volume	1.347	13.472
Final Stored Volume	1.323	13.227
Continuity Error (%)	-0.104	

****

Table C.47. Quality Routing Continuity: Worst Case Scenario

****

	TSS kg	Lead kg
	----------	----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	6109.376	1.527
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	840.228	0.210
External Outflow	4658.834	1.307
Mass Reacted	568.539	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	53.855	0.013
Continuity Error (%)	-0.198	-0.198

****

Table C.48. Highest Continuity Errors: Worst Case Scenario

****

Node j7 (-3.94%)
Node j14 (-2.56%)

****

Table C.49. Time-Step Critical Elements: Worst Case Scenario

****

Link c7 (23.72%)
Link c16 (12.02%)

****

Table C.50. Highest Flow Instability Indexes: Worst Case Scenario

****

Link R5 (12)
Link R6 (6)
Link R3 (3)
Link R2 (2)
Link c12 (1)

****

Table C.51. Routing Time Step Summary: Worst Case Scenario

****

Minimum Time Step	:	14.57 sec
Average Time Step	:	26.69 sec
Maximum Time Step	:	30.00 sec
Percent in Steady State	:	0.00
Average Iterations per Step:		2.97

****

Table C.52. Subcatchment Runoff Summary: Worst Case Scenario

****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	86.00	0.00	0.00	5.63	80.32	20.89	1.65	0.934
S2	86.00	0.00	0.00	5.23	80.73	17.83	1.44	0.939
S13	86.00	0.00	0.00	1.95	83.92	18.69	1.60	0.976
S11	86.00	0.00	0.00	1.12	84.92	3.59	0.31	0.987
S10	86.00	0.00	0.00	1.13	84.90	3.81	0.33	0.987
S9	86.00	0.00	0.00	1.39	84.60	7.59	0.65	0.984
S8	86.00	0.00	0.00	1.43	84.55	8.27	0.71	0.983
S7	86.00	0.00	0.00	1.65	84.29	12.20	1.05	0.980
S6	86.00	0.00	0.00	3.51	82.51	7.05	0.61	0.959
S3	86.00	0.00	0.00	5.49	80.46	19.82	1.58	0.936
S4	86.00	0.00	0.00	5.87	80.07	22.85	1.78	0.931
S5	86.00	0.00	0.00	4.31	81.68	11.55	0.97	0.950
S12	86.00	0.00	0.00	1.50	84.47	9.49	0.82	0.982

****

Table C.53. Subcatchment Washoff Summary: Worst Case Scenario

****

Subcatchment	TSS kg	Lead kg
S1	193.765	0.048
S2	658.247	0.165
S13	1244.271	0.311
S11	252.091	0.063
S10	267.586	0.067
S9	501.170	0.125
S8	546.426	0.137
S7	323.388	0.081
S6	190.858	0.048
S3	458.710	0.115
S4	531.530	0.133
S5	316.011	0.079
S12	627.442	0.157
System	6111.496	1.528

****
Table C.54. Node Depth Summary: Worst Case Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.09	0.29	17.97	0	02:59
j4	JUNCTION	0.12	0.37	20.48	0	02:59
j1	JUNCTION	0.13	0.42	22.98	0	02:59
j5	JUNCTION	0.07	0.23	22.18	0	02:59
j16	JUNCTION	0.06	0.21	23.21	0	02:59
j3	JUNCTION	0.10	0.31	16.46	0	02:59
Treatmentplant	JUNCTION	2.22	8.38	8.39	0	03:00
j11	JUNCTION	1.54	5.96	7.60	0	00:00
j9	JUNCTION	0.52	8.14	10.53	0	00:00
j7	JUNCTION	0.23	9.12	12.71	0	00:00
j12	JUNCTION	1.35	8.11	10.17	0	00:00
j14	JUNCTION	0.14	9.89	12.32	0	00:00
TreatmentPlantOutfall	OUTFALL	0.38	0.50	0.50	0	00:33
Outfall4	OUTFALL	1.10	2.16	2.16	0	22:00
Outfall6	OUTFALL	1.33	2.39	2.16	0	22:00
Outfall1	OUTFALL	0.74	1.80	2.16	0	22:00
Outfall2	OUTFALL	1.65	2.71	2.16	0	22:00
Outfall3	OUTFALL	2.02	3.08	2.16	0	22:00
Outfall5	OUTFALL	2.03	3.09	2.16	0	22:00
j6	STORAGE	2.22	4.84	5.20	0	01:25
j8	STORAGE	2.63	4.69	4.14	0	01:32
j10	STORAGE	2.72	4.35	3.43	0	02:08
j13	STORAGE	2.50	4.01	3.08	0	03:00
j15	STORAGE	2.37	4.50	4.27	0	01:26

****

Table C.55. Node Inflow Summary: Worst Case Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	1.439	1.461	0	02:59	17.834	18.574
j4	JUNCTION	1.781	1.794	0	02:59	22.851	23.136
j1	JUNCTION	1.650	1.650	0	02:59	20.891	20.891
j5	JUNCTION	0.975	0.975	0	02:59	11.551	11.550
j16	JUNCTION	0.610	0.610	0	02:59	7.047	7.046
j3	JUNCTION	1.577	1.603	0	02:59	19.817	20.636
Treatmentplant	JUNCTION	0.982	2.781	0	02:59	11.404	48.205
j11	JUNCTION	0.000	0.080	0	03:01	0.000	3.675
j9	JUNCTION	0.000	0.029	0	00:00	0.000	2.409
j7	JUNCTION	0.000	0.024	0	01:26	0.000	0.643
j12	JUNCTION	0.000	0.031	0	00:00	0.000	2.504
j14	JUNCTION	0.000	0.015	0	01:44	0.000	0.726
TreatmentPlantOutfall	OUTFALL	0.000	1.013	0	03:01	0.000	29.066
Outfall4	OUTFALL	0.000	1.672	0	03:01	0.000	27.496
Outfall6	OUTFALL	0.000	1.125	0	01:27	0.000	22.903
Outfall1	OUTFALL	0.000	1.183	0	01:25	0.000	26.210
Outfall2	OUTFALL	0.000	1.158	0	01:50	0.000	24.472
Outfall3	OUTFALL	0.000	1.097	0	03:22	0.000	21.194
Outfall5	OUTFALL	0.000	2.051	0	03:00	0.000	28.404
j6	STORAGE	1.050	2.677	0	02:59	12.199	36.117
j8	STORAGE	0.713	2.185	0	02:59	8.271	32.319
j10	STORAGE	0.000	1.631	0	02:59	0.000	27.562
j13	STORAGE	1.126	2.099	0	02:59	13.081	30.827
j15	STORAGE	1.598	2.208	0	02:59	18.690	30.267

****

Table C.56. Node Surcharge Summary: Worst Case Scenario

****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
Treatmentplant	JUNCTION	3.51	5.788	0.000
j11	JUNCTION	11.11	5.703	0.000
j9	JUNCTION	5.80	7.889	0.000
j7	JUNCTION	3.09	8.871	0.000
j12	JUNCTION	6.15	7.851	0.000
j14	JUNCTION	1.63	9.631	0.000
j6	STORAGE	5.00	2.249	0.000
j8	STORAGE	6.24	2.099	0.000
j10	STORAGE	7.64	1.756	0.000
j13	STORAGE	6.52	1.416	0.893
j15	STORAGE	4.46	1.909	0.000

****

Table C.57. Node Flooding Summary: Worst Case Scenario

****

Flooding refers to all water that overflows a node, whether it ponds or not.

Node	Hours Flooded	Maximum Rate CMS	Time of Max Occurrence days hr:min	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
Treatmentplant	2.49	0.140	0 01:30	0.697	8.38
j11	1.66	0.080	0 03:01	0.272	5.96
j9	0.01	0.011	0 00:00	0.000	8.14
j7	0.01	0.006	0 00:00	0.000	9.12
j12	0.01	0.005	0 00:00	0.000	8.11
j14	0.01	0.001	0 00:00	0.000	9.89
j6	2.47	1.474	0 02:59	8.859	4.84
j8	1.98	0.998	0 02:59	5.257	4.69
j10	1.21	0.497	0 02:59	1.650	4.35
j15	2.09	1.073	0 02:59	6.283	4.50

****

Table C.58. Storage Volume Summary: Worst Case Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	2.222	46	0	4.840	100	0 01:25	1.203
j8	2.631	56	0	4.690	100	0 01:32	1.187
j10	2.717	62	0	4.347	100	0 02:08	1.140
j13	2.504	51	0	4.007	82	0 03:00	2.073
j15	2.370	53	0	4.500	100	0 01:26	1.135

****

Table C.59. Outfall Loading Summary: Worst Case Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	100.00	0.405	1.013	29.066	36.229	0.151
Outfall4	100.00	0.442	1.672	27.496	773.408	0.193
Outfall6	100.00	0.361	1.125	22.903	1033.754	0.258
Outfall1	100.00	0.403	1.183	26.210	345.273	0.086
Outfall2	100.00	0.380	1.158	24.472	894.690	0.224
Outfall3	99.97	0.330	1.097	21.194	417.940	0.104
Outfall5	100.00	0.484	2.051	28.404	1156.811	0.289
System	100.00	2.805	9.292	179.746	4658.107	1.307

****
Table C.60. Link Flow Summary: Worst Case Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days	hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.023	0	04:36	1.46	1.05	1.00
c6	CONDUIT	1.777	0	03:00	5.50	0.51	0.76
c9	CONDUIT	1.627	0	03:00	4.80	0.66	0.80
c16	CONDUIT	1.013	0	03:01	5.16	28.09	1.00
c5	CONDUIT	0.962	0	02:59	3.44	0.23	0.66
c11	CONDUIT	0.035	0	00:10	0.76	0.63	1.00
c13	CONDUIT	0.036	0	00:10	0.75	0.80	1.00
c14	CONDUIT	0.053	0	00:00	1.04	0.95	1.00
c2	CONDUIT	0.010	0	00:20	0.57	1.07	1.00
c4	CONDUIT	0.014	0	00:23	0.79	1.06	1.00
c8	CONDUIT	1.452	0	02:59	4.87	0.35	0.70
c10	CONDUIT	0.051	0	00:00	1.01	0.87	1.00
c15	CONDUIT	0.610	0	02:59	2.25	0.18	0.65
c12	CONDUIT	0.047	0	06:50	0.96	0.88	1.00
c3	CONDUIT	0.017	0	00:19	1.04	1.08	1.00
c7	CONDUIT	1.602	0	02:59	5.27	0.38	0.72
P2	PUMP	0.031	0	00:00		1.00	
p3	PUMP	0.043	0	00:00		1.00	
P5	PUMP	0.020	0	00:00		1.00	
P1	PUMP	0.010	0	00:00		1.00	
P4	PUMP	0.029	0	00:00		1.00	
R4	WEIR	1.672	0	03:01			1.00
R6	WEIR	1.125	0	01:27			1.00
R1	WEIR	1.183	0	01:25			1.00
R2	WEIR	1.158	0	01:50			1.00
R3	WEIR	1.097	0	03:22			1.00
R5	WEIR	2.051	0	03:00			1.00

****

Table C.61. Flow Classification Summary: Worst Case Scenario

****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
		--- Fraction of Time in Flow Class ----								
c1	1.00	0.00	0.00	0.00	0.31	0.69	0.00	0.00	1.30	0.0008
c6	1.00	0.00	0.00	0.00	0.73	0.27	0.00	0.00	0.63	0.0003
c9	1.00	0.00	0.00	0.00	0.75	0.25	0.00	0.00	0.52	0.0004
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.52	0.0239
c5	1.00	0.00	0.00	0.00	0.79	0.21	0.00	0.00	0.39	0.0001
c11	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.34	0.0005
c13	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.31	0.0007
c14	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.19	0.0011
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.38	0.0007
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.19	0.0007
c8	1.00	0.00	0.00	0.00	0.75	0.25	0.00	0.00	0.57	0.0002
c10	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.10	0.0011
c15	1.00	0.00	0.00	0.00	0.87	0.13	0.00	0.00	0.25	0.0001
c12	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.26	0.0015
c3	1.00	0.00	0.00	0.00	0.34	0.66	0.00	0.00	0.88	0.0007
c7	1.00	0.00	0.00	0.00	0.74	0.26	0.00	0.00	0.62	0.0002

****

Table C.62. Conduit Surcharge Summary: Worst Case Scenario

****

Conduit	Hours Full: Both Ends	Hours Full: Upstream	Hours Full: Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c1	4.11	4.11	4.12	5.26	4.11
c16	3.03	3.03	3.03	23.00	3.03
c11	5.80	5.80	5.81	0.01	0.01
c13	6.15	6.15	6.17	0.01	0.01
c14	1.58	1.58	1.60	0.01	0.01
c2	4.11	4.11	4.12	0.69	0.01
c4	3.38	3.38	3.38	0.45	0.01
c10	3.03	3.03	3.05	0.01	0.01
c12	11.11	11.11	11.13	0.01	0.01
c3	4.38	4.38	4.39	5.56	4.38

****
Table C.63. Pumping Summary: Worst Case Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Curve Low	% Time Off Pump Curve High
P2	100.00	1	0.00	0.03	0.03	2.504	9.89	0.0	88.4
P3	100.00	1	0.00	0.04	0.04	3.590	10.77	0.0	94.6
P5	38.82	1	0.00	0.02	0.02	0.643	1.80	0.0	99.9
P1	84.35	2	0.00	0.01	0.01	0.724	2.01	0.0	100.0
P4	100.00	1	0.00	0.03	0.03	2.409	6.31	0.0	100.0

Analysis begun on: Thu May 02 19:41:59 2013
Analysis ended on: Thu May 02 19:41:59 2013
Total elapsed time: < 1 sec

C.4. Precipitation Focus Scenario Simulation Status Report

****
Table C.64. Analysis Options: Precipitation Focus Scenario

****
Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method ...... ….HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step .... …00:05:01
Routing Time Step .... 30.00 sec

****

Table C.65. Runoff Quantity Continuity: Precipitation Focus Scenario

****

	Volume hectare-m	Depth mm
	---------	-------
Total Precipitation	17.148	86.000
Evaporation Loss	0.000	0.000
Infiltration Loss	1.436	7.200
Surface Runoff	15.615	78.312
Final Surface Storage	0.103	0.517
Continuity Error (%)	-0.033	

****

Table C.66. Runoff Quality Continuity: Precipitation Focus Scenario

****

	TSS kg	Lead kg
	----------	----------
Initial Buildup	6111.392	0.000
Surface Buildup	1329.555	1.528
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	6111.506	1.528
Remaining Buildup	1329.555	0.000
Continuity Error (%)	-0.002	0.000

****

Table C.67. Flow Routing Continuity: Precipitation Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-----------	-----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	15.615	156.148
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.757	7.574
External Outflow	14.758	147.579
Internal Outflow	0.959	9.588
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.616	6.160
Continuity Error (%)	0.241	

****

Table C.68. Quality Routing Continuity: Precipitation Focus Scenario

****

	TSS kg	Lead kg
	----------	----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	6109.957	1.527
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	397.202	0.099
External Outflow	4910.381	1.400
Mass Reacted	687.935	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	75.605	0.019
Continuity Error (%)	0.636	0.636

****

Table C.69. Time-Step Critical Elements: Precipitation Focus Scenario

****

Link c7 (20.76%)
Link c16 (9.62%)

****
Table C.70. Highest Flow Instability Indexes: Precipitation Focus Scenario

****
Link R5 (23)
Link R3 (5)
Link c12 (2)

****
Table C.71. Routing Time Step Summary: Precipitation Focus Scenario

****
Minimum Time Step : 16.34 sec
Average Time Step : 27.46 sec
Maximum Time Step : 30.00 sec
Percent in Steady State : 0.00
Average Iterations per Step: 2.88

****
Table C.72. Subcatchment Runoff Summary: Precipitation Focus Scenario

****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	86.00	0.00	0.00	9.12	75.82	19.72	1.34	0.882
S2	86.00	0.00	0.00	9.12	76.58	16.92	1.20	0.891
S13	86.00	0.00	0.00	5.25	80.71	17.97	1.45	0.938
S11	86.00	0.00	0.00	2.73	83.32	3.52	0.31	0.969
S10	86.00	0.00	0.00	2.78	83.26	3.74	0.33	0.968
S9	86.00	0.00	0.00	3.58	82.44	7.39	0.64	0.959
S8	86.00	0.00	0.00	3.70	82.31	8.05	0.69	0.957
S7	86.00	0.00	0.00	4.35	81.64	11.81	1.00	0.949
S6	86.00	0.00	0.00	6.20	79.83	6.82	0.58	0.928
S3	86.00	0.00	0.00	9.12	76.09	18.74	1.29	0.885
S4	86.00	0.00	0.00	9.12	75.34	21.50	1.42	0.876
S5	86.00	0.00	0.00	7.70	78.31	11.07	0.87	0.911
S12	86.00	0.00	0.00	6.96	79.06	8.88	0.73	0.919

****

Table C.73. Subcatchment Washoff Summary: Precipitation Focus Scenario

****

Subcatchment	TSS kg	Lead kg
S1	193.765	0.048
S2	658.248	0.165
S13	1244.271	0.311
S11	252.094	0.063
S10	267.589	0.067
S9	501.170	0.125
S8	546.426	0.137
S7	323.387	0.081
S6	190.859	0.048
S3	458.710	0.115
S4	531.530	0.133
S5	316.013	0.079
S12	627.445	0.157
System	6111.506	1.528

****

Table C.74. Node Depth Summary: Precipitation Focus Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.09	0.26	17.94	0	02:59
j4	JUNCTION	0.12	0.32	20.43	0	02:59
j1	JUNCTION	0.13	0.37	22.93	0	02:59
j5	JUNCTION	0.07	0.22	22.17	0	02:59
j16	JUNCTION	0.06	0.20	23.20	0	02:59
j3	JUNCTION	0.10	0.28	16.43	0	02:59
Treatmentplant	JUNCTION	1.63	6.70	6.71	0	03:01
j11	JUNCTION	1.13	5.96	7.60	0	02:55
j9	JUNCTION	0.29	1.63	4.02	0	03:23
j7	JUNCTION	0.17	2.88	6.47	0	02:16
j12	JUNCTION	0.90	5.35	7.41	0	03:01
j14	JUNCTION	0.04	0.11	2.54	0	03:13
TreatmentPlantOutfall	OUTFALL	0.35	0.50	0.50	0	00:57
Outfall4	OUTFALL	0.85	1.33	1.33	0	18:59
Outfall6	OUTFALL	1.08	1.56	1.33	0	18:59
Outfall1	OUTFALL	0.49	0.97	1.33	0	18:59
Outfall2	OUTFALL	1.40	1.88	1.33	0	18:59
Outfall3	OUTFALL	1.77	2.25	1.33	0	18:59
Outfall5	OUTFALL	1.78	2.26	1.33	0	18:59
j6	STORAGE	1.97	4.84	5.20	0	02:08
j8	STORAGE	2.20	4.69	4.14	0	02:24
j10	STORAGE	2.21	4.04	3.12	0	03:20
j13	STORAGE	2.05	3.40	2.47	0	03:03
j15	STORAGE	1.97	4.50	4.27	0	02:03

****

Table C.75. Node Inflow Summary: Precipitation Focus Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	1.202	1.224	0 02:59	16.917	17.851
j4	JUNCTION	1.424	1.437	0 02:59	21.501	21.846
j1	JUNCTION	1.341	1.341	0 02:59	19.721	19.722
j5	JUNCTION	0.872	0.872	0 02:59	11.074	11.073
j16	JUNCTION	0.578	0.578	0 02:59	6.818	6.817
j3	JUNCTION	1.293	1.319	0 02:59	18.740	19.768
Treatmentplant	JUNCTION	0.965	2.450	0 02:59	11.136	40.868
j11	JUNCTION	0.000	0.043	0 00:59	0.000	3.447
j9	JUNCTION	0.000	0.029	0 00:55	0.000	1.690
j7	JUNCTION	0.000	0.020	0 00:55	0.000	0.694
j12	JUNCTION	0.000	0.057	0 00:50	0.000	2.454
j14	JUNCTION	0.000	0.010	0 00:55	0.000	0.340
TreatmentPlantOutfall	OUTFALL	0.000	0.899	0 03:01	0.000	24.217
Outfall4	OUTFALL	0.000	1.461	0 03:01	0.000	19.180
Outfall6	OUTFALL	0.000	1.125	0 02:03	0.000	21.287
Outfall1	OUTFALL	0.000	1.183	0 02:08	0.000	25.078
Outfall2	OUTFALL	0.000	1.158	0 02:24	0.000	22.573
Outfall3	OUTFALL	0.000	0.975	0 03:18	0.000	19.185
Outfall5	OUTFALL	0.000	1.673	0 03:03	0.000	23.633
j6	STORAGE	0.995	2.312	0 02:59	11.815	30.685
j8	STORAGE	0.694	1.928	0 02:59	8.051	26.882
j10	STORAGE	0.000	1.347	0 02:59	0.000	22.859
j13	STORAGE	1.034	1.901	0 02:59	12.404	25.614
j15	STORAGE	1.450	2.027	0 02:59	17.976	25.433

****

Table C.76. Node Surcharge Summary: Precipitation Focus Scenario

****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
Treatmentplant	JUNCTION	3.26	4.111	0.000
j11	JUNCTION	8.81	5.703	0.000
j9	JUNCTION	3.37	1.375	3.738
j7	JUNCTION	2.39	2.630	2.974
j12	JUNCTION	5.38	5.099	0.000
j6	STORAGE	4.79	2.249	0.000
j8	STORAGE	4.48	2.099	0.000
j10	STORAGE	4.00	1.449	0.307
j13	STORAGE	2.31	0.804	1.505
j15	STORAGE	4.14	1.909	0.000

****

Table C.77. Node Flooding Summary: Precipitation Focus Scenario

****

Flooding refers to all water that overflows a node, whether it ponds or not.

Node	Hours Flooded	Maximum Rate CMS	Time of Max Occurrence days hr:min	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
Treatmentplant	1.38	0.099	0 02:45	0.312	6.70
j11	0.19	0.003	0 03:02	0.001	5.96
j6	1.88	1.108	0 02:59	4.229	4.84
j8	1.14	0.741	0 02:59	1.946	4.69
j15	1.56	0.892	0 02:59	3.412	4.50

****
Table C.78. Storage Volume Summary: Precipitation Focus Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min		Maximum Outflow CMS
j6	1.965	41	0	4.840	100	0	02:08	1.203
j8	2.201	47	0	4.690	100	0	02:24	1.187
j10	2.207	51	0	4.040	93	0	03:20	1.018
j13	2.053	42	0	3.395	69	0	03:03	1.699
j15	1.971	44	0	4.500	100	0	02:03	1.135

****
Table C.79. Outfall Loading Summary: Precipitation Focus Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	99.90	0.328	0.899	24.217	29.909	0.179
Outfall4	99.97	0.303	1.461	19.180	709.162	0.177
Outfall6	100.00	0.317	1.125	21.287	1189.206	0.297
Outfall1	99.27	0.366	1.183	25.078	407.423	0.102
Outfall2	100.00	0.332	1.158	22.573	1021.144	0.255
Outfall3	100.00	0.274	0.975	19.185	447.253	0.112
Outfall5	100.00	0.368	1.673	23.633	1105.720	0.276
System	99.88	2.288	8.450	155.153	4909.817	1.399

****

Table C.80. Link Flow Summary: Precipitation Focus Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days	hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c6	CONDUIT	1.419	0	02:59	4.62	0.41	0.72
c9	CONDUIT	1.317	0	03:00	4.07	0.53	0.76
c16	CONDUIT	0.899	0	03:01	4.58	24.92	1.00
c5	CONDUIT	0.859	0	02:59	3.12	0.21	0.65
c11	CONDUIT	0.033	0	05:55	0.73	0.59	1.00
c13	CONDUIT	0.034	0	06:24	0.73	0.76	1.00
c14	CONDUIT	0.015	0	03:26	0.41	0.27	0.72
c2	CONDUIT	0.010	0	00:26	1.26	1.07	1.00
c4	CONDUIT	0.014	0	00:38	0.79	1.06	1.00
c8	CONDUIT	1.215	0	02:59	4.81	0.29	0.68
c10	CONDUIT	0.027	0	04:58	0.66	0.46	1.00
c15	CONDUIT	0.578	0	02:59	4.40	0.17	0.64
c12	CONDUIT	0.047	0	09:30	0.96	0.88	1.00
c3	CONDUIT	0.017	0	00:22	1.03	1.07	1.00
c7	CONDUIT	1.318	0	02:59	4.79	0.32	0.69
P2	PUMP	0.031	0	00:49		1.00	
p3	PUMP	0.043	0	00:59		1.00	
P5	PUMP	0.020	0	00:55		1.00	
P1	PUMP	0.010	0	00:55		1.00	
P4	PUMP	0.029	0	00:55		1.00	
R4	WEIR	1.461	0	03:01			1.00
R6	WEIR	1.125	0	02:03			1.00
R1	WEIR	1.183	0	02:08			1.00
R2	WEIR	1.158	0	02:24			1.00
R3	WEIR	0.975	0	03:18			1.00
R5	WEIR	1.673	0	03:03			1.00

****
Table C.81. Flow Classification Summary: Precipitation Focus Scenario

****

Conduit	Adjusted /Actual Length	--- Fraction of Time in Flow Class ---- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
c1	1.00	0.00	0.00	0.00	0.33	0.67	0.00	0.00	1.30	0.0008
c6	1.00	0.00	0.00	0.00	0.74	0.26	0.00	0.00	0.61	0.0003
c9	1.00	0.00	0.00	0.00	0.75	0.24	0.00	0.00	0.56	0.0004
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.50	0.0171
c5	1.00	0.00	0.00	0.00	0.83	0.17	0.00	0.00	0.36	0.0001
c11	1.00	0.00	0.04	0.00	0.96	0.00	0.00	0.00	0.27	0.0008
c13	1.00	0.00	0.02	0.00	0.98	0.00	0.00	0.00	0.32	0.0008
c14	1.00	0.00	0.04	0.00	0.96	0.00	0.00	0.00	0.11	0.0002
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.41	0.0008
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.24	0.0008
c8	1.00	0.00	0.00	0.00	0.75	0.25	0.00	0.00	0.60	0.0002
c10	1.00	0.00	0.04	0.00	0.96	0.00	0.00	0.00	0.13	0.0005
c15	1.00	0.00	0.00	0.00	0.92	0.08	0.00	0.00	0.32	0.0001
c12	1.00	0.00	0.02	0.00	0.98	0.00	0.00	0.00	0.33	0.0008
c3	1.00	0.00	0.00	0.00	0.37	0.63	0.00	0.00	0.87	0.0008
c7	1.00	0.00	0.00	0.00	0.72	0.28	0.00	0.00	0.64	0.0002

****
Table C.82. Conduit Surcharge Summary: Precipitation Focus Scenario

****

Conduit	------------ Hours Full ------------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c1	4.35	4.35	4.36	6.24	4.35
c16	2.61	2.61	2.62	22.86	2.61
c11	3.37	3.37	3.38	0.01	0.01
c13	5.38	5.38	5.38	0.01	0.01
c2	4.35	4.35	4.36	1.15	0.01
c4	3.15	3.15	3.15	0.86	0.01
c10	2.39	2.39	2.40	0.01	0.01
c12	8.81	8.81	8.81	0.01	0.01
c3	4.78	4.78	4.79	6.63	4.78

****
Table C.83. Pumping Summary: Precipitation Focus Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Curve Low	% Time Off Pump Curve High
P2	96.45	1	0.00	0.03	0.03	2.440	10.90	0.0	90.9
P3	95.77	1	0.00	0.04	0.04	3.437	11.77	0.0	92.3
P5	41.89	1	0.00	0.02	0.02	0.694	2.10	0.0	100.0
P1	39.58	1	0.00	0.01	0.01	0.340	1.05	0.0	100.0
P4	70.12	2	0.00	0.03	0.03	1.690	4.99	0.0	100.0

Analysis begun on: Fri May 03 14:21:04 2013
Analysis ended on: Fri May 03 14:21:04 2013
Total elapsed time: < 1 sec

C.5. Tide Focus Scenario Simulation Status Report

****
Table C.84. Analysis Options: Tide Focus Scenario

****
Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method .... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****

Table C.85. Runoff Quantity Continuity: Tide Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-------------	-------------
Total Precipitation	1.381	6.925
Evaporation Loss	0.000	0.000
Infiltration Loss	0.924	4.636
Surface Runoff	0.413	2.071
Final Surface Storage	0.044	0.222
Continuity Error (%)	-0.060	

****

Table C.86. Runoff Quality Continuity: Precipitation Focus Scenario

****

	TSS kg	Lead kg
	----------	----------
Initial Buildup	6111.392	0.000
Surface Buildup	1.777	0.048
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	193.201	0.048
Remaining Buildup	5919.581	0.000
Continuity Error (%)	0.006	0.000

****

Table C.87. Flow Routing Continuity: Precipitation Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	0.410	4.105
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	3.028	30.281
External Outflow	1.994	19.939
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	1.323	13.233
Continuity Error (%)	3.528	

****

Table C.88. Quality Routing Continuity: Precipitation Focus Scenario

****

	TSS kg	Lead kg
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	192.455	0.048
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	0.000	0.000
External Outflow	32.331	0.022
Mass Reacted	55.510	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	100.753	0.025
Continuity Error (%)	2.006	2.006

****
Table C.89. Highest Continuity Errors: Tide Focus Scenario

****
Node j6 (9.41%)
Node j8 (5.61%)
Node j15 (5.26%)
Node j13 (4.91%)
Node j14 (4.13%)

****
Table C.90. Time-Step Critical Elements: Tide Focus Scenario

****
Link c16 (7.58%)

****
Table C.91. Highest Flow Instability Indexes: Tide Focus Scenario

****
Link R5 (18)
Link R3 (6)
Link R2 (5)

****
Table C.92. Routing Time Step Summary: Tide Focus Scenario

****

Minimum Time Step	:	28.10 sec
Average Time Step	:	29.89 sec
Maximum Time Step	:	30.00 sec
Percent in Steady State	:	0.00
Average Iterations per Step:		2.06

****

Table C.93. Subcatchment Runoff Summary: Tide Focus Scenario

****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	6.93	0.00	0.00	5.19	1.56	0.41	0.01	0.225
S2	6.93	0.00	0.00	5.19	1.58	0.35	0.00	0.227
S13	6.93	0.00	0.00	3.46	2.99	0.67	0.01	0.432
S11	6.93	0.00	0.00	3.46	3.28	0.14	0.00	0.474
S10	6.93	0.00	0.00	3.46	3.28	0.15	0.00	0.473
S9	6.93	0.00	0.00	3.46	3.19	0.29	0.00	0.460
S8	6.93	0.00	0.00	3.46	3.17	0.31	0.00	0.458
S7	6.93	0.00	0.00	3.46	3.10	0.45	0.01	0.447
S6	6.93	0.00	0.00	5.19	1.64	0.14	0.00	0.237
S3	6.93	0.00	0.00	5.19	1.57	0.39	0.01	0.226
S4	6.93	0.00	0.00	5.19	1.55	0.44	0.01	0.224
S5	6.93	0.00	0.00	5.19	1.61	0.23	0.00	0.233
S12	6.93	0.00	0.00	5.19	1.63	0.18	0.00	0.235

****

Table C.94. Subcatchment Washoff Summary: Tide Focus Scenario

****

Subcatchment	TSS kg	Lead kg
S1	4.012	0.001
S2	13.769	0.003
S13	49.727	0.012
S11	11.069	0.003
S10	11.728	0.003
S9	21.341	0.005
S8	23.169	0.006
S7	13.386	0.003
S6	4.177	0.001
S3	9.528	0.002
S4	10.932	0.003
S5	6.778	0.002
S12	13.584	0.003
System	193.201	0.048

****

Table C.95. Node Depth Summary: Tide Focus Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.02	0.02	17.70	0	23:00
j4	JUNCTION	0.02	0.02	20.13	0	23:00
j1	JUNCTION	0.02	0.02	22.58	0	23:00
j5	JUNCTION	0.01	0.01	21.96	0	23:00
j16	JUNCTION	0.01	0.01	23.01	0	22:28
j3	JUNCTION	0.02	0.02	16.17	0	23:00
Treatmentplant	JUNCTION	0.68	1.68	1.69	0	22:08
j11	JUNCTION	0.27	1.14	2.78	0	22:08
j9	JUNCTION	0.12	0.66	3.05	0	22:04
j7	JUNCTION	0.00	0.00	3.59	0	00:00
j12	JUNCTION	0.17	0.67	2.73	0	21:46
j14	JUNCTION	0.01	0.07	2.50	0	22:12
TreatmentPlantOutfall	OUTFALL	0.28	0.43	0.43	0	22:09
Outfall4	OUTFALL	1.16	2.16	2.16	0	22:00
Outfall6	OUTFALL	1.39	2.39	2.16	0	22:00
Outfall1	OUTFALL	0.80	1.80	2.16	0	22:00
Outfall2	OUTFALL	1.71	2.71	2.16	0	22:00
Outfall3	OUTFALL	2.08	3.08	2.16	0	22:00
Outfall5	OUTFALL	2.09	3.09	2.16	0	22:00
j6	STORAGE	0.72	1.80	2.16	0	22:03
j8	STORAGE	1.62	2.70	2.15	0	22:05
j10	STORAGE	1.98	3.07	2.15	0	22:04
j13	STORAGE	2.05	3.09	2.16	0	22:01
j15	STORAGE	1.30	2.39	2.16	0	22:01

****

Table C.96. Node Inflow Summary: Tide Focus Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	0.005	0.006	0	23:00	0.346	0.417
j4	JUNCTION	0.006	0.006	0	23:00	0.440	0.458
j1	JUNCTION	0.005	0.005	0	23:00	0.403	0.403
j5	JUNCTION	0.003	0.003	0	23:00	0.226	0.226
j16	JUNCTION	0.002	0.002	0	22:58	0.139	0.139
j3	JUNCTION	0.005	0.006	0	23:00	0.383	0.455
Treatmentplant	JUNCTION	0.006	0.421	0	21:59	0.430	15.554
j11	JUNCTION	0.000	0.043	0	01:35	0.000	3.347
j9	JUNCTION	0.000	0.029	0	09:22	0.000	1.429
j7	JUNCTION	0.000	0.000	0	00:00	0.000	0.000
j12	JUNCTION	0.000	0.031	0	00:46	0.000	2.465
j14	JUNCTION	0.000	0.010	0	19:27	0.000	0.133
TreatmentPlantOutfall	OUTFALL	0.000	0.412	0	22:09	0.000	15.250
Outfall4	OUTFALL	0.000	0.335	0	21:59	0.000	8.952
Outfall6	OUTFALL	0.000	0.138	0	00:00	0.000	4.110
Outfall1	OUTFALL	0.000	0.096	0	20:00	0.000	2.743
Outfall2	OUTFALL	0.000	0.236	0	00:00	0.000	5.452
Outfall3	OUTFALL	0.000	0.390	0	00:00	0.000	6.558
Outfall5	OUTFALL	0.000	1.001	0	00:00	0.000	7.154
j6	STORAGE	0.006	0.107	0	20:00	0.446	2.756
j8	STORAGE	0.004	0.236	0	00:00	0.308	5.259
j10	STORAGE	0.000	0.390	0	00:00	0.000	7.535
j13	STORAGE	0.004	1.001	0	00:00	0.320	6.814
j15	STORAGE	0.009	0.138	0	00:00	0.663	3.782

****
Table C.97. Node Surcharge Summary: Tide Focus Scenario

****
Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
j11	JUNCTION	3.28	0.883	3.137
j9	JUNCTION	1.97	0.402	4.711
j12	JUNCTION	1.24	0.418	4.676
j8	STORAGE	1.53	0.110	1.989
j10	STORAGE	3.37	0.483	1.273
j13	STORAGE	3.43	0.496	1.813

****
Table C.98. Node Flooding Summary: Tide Focus Scenario

****
No nodes were flooded.

****
Table C.99. Storage Volume Summary: Tide Focus Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	0.722	15	0	1.795	37	0 22:03	0.048
j8	1.621	35	0	2.701	58	0 22:05	0.087
j10	1.984	46	0	3.074	71	0 22:04	0.116
j13	2.052	42	0	3.087	63	0 22:01	0.128
j15	1.303	29	0	2.387	53	0 22:01	0.071

****

Table C.100. Outfall Loading Summary: Tide Focus Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	99.96	0.185	0.412	15.250	2.534	0.015
Outfall4	100.00	0.109	0.335	8.952	0.140	0.000
Outfall6	100.00	0.050	0.138	4.110	14.663	0.004
Outfall1	100.00	0.033	0.096	2.743	4.980	0.001
Outfall2	99.96	0.066	0.236	5.452	4.916	0.001
Outfall3	100.00	0.079	0.390	6.558	1.982	0.000
Outfall5	100.00	0.086	1.001	7.154	3.122	0.001
System	99.99	0.608	1.883	50.220	32.337	0.022

****

Table C.101. Link Flow Summary: Tide Focus Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.001	0 23:00	0.65	0.04	0.14
c6	CONDUIT	0.006	0 23:00	0.06	0.00	0.51
c9	CONDUIT	0.004	0 23:00	0.12	0.00	0.52
c16	CONDUIT	0.412	0 22:09	2.16	11.41	0.93
c5	CONDUIT	0.003	0 23:00	0.01	0.00	0.51
c11	CONDUIT	0.030	0 12:36	0.72	0.53	1.00
c13	CONDUIT	0.031	0 23:00	0.70	0.69	1.00
c14	CONDUIT	0.010	0 22:12	0.30	0.19	0.65
c2	CONDUIT	0.000	0 23:00	0.22	0.03	0.13
c4	CONDUIT	0.000	0 23:00	0.21	0.02	0.12
c8	CONDUIT	0.005	0 23:00	0.03	0.00	0.51
c10	CONDUIT	0.000	0 00:00	0.00	0.00	0.50
c15	CONDUIT	0.002	0 20:37	0.01	0.00	0.51
c12	CONDUIT	0.044	0 12:33	0.96	0.83	1.00
c3	CONDUIT	0.001	0 23:00	0.44	0.04	0.14
c7	CONDUIT	0.006	0 23:00	0.03	0.00	0.51
P2	PUMP	0.031	0 00:46		1.00	
p3	PUMP	0.043	0 01:35		1.00	
P5	PUMP	0.000	0 00:00		0.00	
P1	PUMP	0.010	0 19:27		1.00	
P4	PUMP	0.029	0 09:22		1.00	
R4	WEIR	0.335	0 21:59			0.83
R6	WEIR	0.138	0 00:00			0.92
R1	WEIR	0.096	0 20:00			0.69
R2	WEIR	0.236	0 00:00			1.00
R3	WEIR	0.390	0 00:00			1.00
R5	WEIR	1.001	0 00:00			1.00

****

Table C.102. Flow Classification Summary: Tide Focus Scenario

****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
		--- Fraction of Time in Flow Class ----								
c1	1.00	0.00	0.00	0.00	0.02	0.98	0.00	0.00	1.67	0.0000
c6	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.02	0.0000
c9	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.05	0.0000
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.57	0.0077
c5	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.01	0.0000
c11	1.00	0.00	0.41	0.00	0.59	0.00	0.00	0.00	0.25	0.0002
c13	1.00	0.00	0.03	0.00	0.97	0.00	0.00	0.00	0.43	0.0003
c14	1.00	0.00	0.84	0.00	0.16	0.00	0.00	0.00	0.04	0.0001
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.60	0.0000
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.59	0.0000
c8	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.01	0.0000
c10	1.00	0.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0000
c15	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.01	0.0000
c12	1.00	0.00	0.07	0.00	0.93	0.00	0.00	0.00	0.47	0.0004
c3	1.00	0.00	0.00	0.00	0.04	0.95	0.00	0.00	1.14	0.0000
c7	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.02	0.0000

****

Table C.103. Conduit Surcharge Summary: Tide Focus Scenario

****

Conduit	------------ Hours Full ------------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c16	0.01	0.01	0.01	22.80	0.01
c11	1.96	1.96	1.96	0.01	0.01
c13	1.24	1.24	1.24	0.01	0.01
c12	3.28	3.28	3.28	0.01	0.01

****
Table C.104. Pumping Summary: Tide Focus Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Low	% Time Off Curve High
P2	96.67	1	0.00	0.03	0.03	2.465	7.29	0.0	100.0
p3	93.15	1	0.00	0.04	0.04	3.348	7.14	0.0	100.0
P5	0.00	0	0.00	0.00	0.00	0.000	0.00	0.0	0.0
P1	15.48	1	0.00	0.01	0.01	0.133	0.20	0.0	100.0
P4	59.32	1	0.00	0.03	0.03	1.429	4.42	0.0	100.0

Analysis begun on: Sat May 04 13:29:00 2013
Analysis ended on: Sat May 04 13:29:00 2013
Total elapsed time: < 1 sec

C.6. Tide and Initial Depth Focus Scenario Simulation Status Report

****
Table C.105. Analysis Options: Tide and Initial Depth Scenario

****
Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method .... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****
Table C.106. Runoff Quantity Continuity : Tide and Initial Depth Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-------------	-------------
Total Precipitation	17.038	85.450
Evaporation Loss	0.000	0.000
Infiltration Loss	1.578	7.915
Surface Runoff	12.984	65.118
Final Surface Storage	2.509	12.586
Continuity Error (%)	-0.197	

****
Table C.107. Runoff Quality Continuity: Tide and Initial Depth Scenario
****

	TSS kg	Lead kg
	----------	----------
Initial Buildup	6111.392	0.000
Surface Buildup	0.822	1.382
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	5515.999	1.379
Remaining Buildup	583.621	0.000
Continuity Error (%)	0.206	0.228

****

Table C.108. Flow Routing Continuity: Tide and Initial Depth Scenario

****

	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	12.883	128.834
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.173	1.727
External Outflow	12.935	129.354
Internal Outflow	0.000	0.001
Storage Losses	0.000	0.000
Initial Stored Volume	1.347	13.472
Final Stored Volume	1.462	14.621
Continuity Error (%)	0.039	

****

Table C.109. Quality Routing Continuity: Tide and Initial Depth Scenario

****

	TSS kg	Lead kg
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	5489.545	1.372
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	0.000	0.000
External Outflow	3772.253	1.209
Mass Reacted	1061.953	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	614.220	0.154
Continuity Error (%)	0.749	0.749

****

Table C.110. Highest Continuity Errors: Tide and Initial Depth Scenario

****

Node j14 (-2.14%)
Node j7 (-1.20%)

****
Table C.111. Time-Step Critical Elements: Tide and Initial DepthScenario

****
Link c16 (46.60%)

****
Table C.112. Highest Flow Instability Indexes: Tide and Initial DepthScenario

****
Link R4 (3)
Link R5 (1)
Link c16 (1)

****
Table C.113. Routing Time Step Summary: Tide and Initial DepthScenario

****
Minimum Time Step : 23.42 sec
Average Time Step : 28.84 sec
Maximum Time Step : 30.00 sec
Percent in Steady State : 0.00
Average Iterations per Step: 2.43

****
Table C.114. Subcatchment Runoff Summary: Tide and Initial Depth Scenario

****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	85.45	0.00	0.00	8.87	60.37	15.70	0.39	0.707
S2	85.45	0.00	0.00	8.87	61.75	13.64	0.35	0.723
S13	85.45	0.00	0.00	5.91	69.49	15.48	0.45	0.813
S11	85.45	0.00	0.00	5.91	75.47	3.19	0.11	0.883
S10	85.45	0.00	0.00	5.91	75.33	3.38	0.11	0.882
S9	85.45	0.00	0.00	5.91	73.37	6.58	0.21	0.859
S8	85.45	0.00	0.00	5.91	73.07	7.15	0.23	0.855
S7	85.45	0.00	0.00	5.91	71.56	10.36	0.32	0.837
S6	85.45	0.00	0.00	8.87	67.58	5.77	0.17	0.791
S3	85.45	0.00	0.00	8.87	60.85	14.99	0.37	0.712
S4	85.45	0.00	0.00	8.87	59.53	16.99	0.41	0.697
S5	85.45	0.00	0.00	8.87	64.90	9.18	0.25	0.759
S12	85.45	0.00	0.00	8.87	66.22	7.44	0.21	0.775

****

Table C.115. Subcatchment Washoff Summary: Tide and Initial Depth Scenario

****

Subcatchment	TSS kg	Lead kg
S1	156.620	0.039
S2	544.254	0.136
S13	1157.629	0.289
S11	252.086	0.063
S10	267.572	0.067
S9	492.315	0.123
S8	534.616	0.134
S7	309.850	0.077
S6	172.718	0.043
S3	373.710	0.093
S4	423.624	0.106
S5	274.626	0.069
S12	556.379	0.139
System	5515.999	1.379

****
Table C.116. Node Depth Summary: Tide and Initial DepthScenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.09	0.14	17.82	0	07:59
j4	JUNCTION	0.11	0.16	20.27	0	08:00
j1	JUNCTION	0.12	0.18	22.74	0	08:00
j5	JUNCTION	0.07	0.12	22.07	0	07:59
j16	JUNCTION	0.07	0.11	23.11	0	07:59
j3	JUNCTION	0.10	0.15	16.30	0	07:59
Treatmentplant	JUNCTION	1.35	2.59	2.60	0	00:00
j11	JUNCTION	0.82	5.96	7.60	0	00:00
j9	JUNCTION	0.20	8.14	10.53	0	00:00
j7	JUNCTION	0.09	9.12	12.71	0	00:00
j12	JUNCTION	0.41	8.11	10.17	0	00:00
j14	JUNCTION	0.08	9.89	12.32	0	00:00
TreatmentPlantOutfall	OUTFALL	0.39	0.45	0.45	0	22:05
Outfall4	OUTFALL	1.17	2.16	2.16	0	21:59
Outfall6	OUTFALL	1.40	2.39	2.16	0	21:59
Outfall1	OUTFALL	0.81	1.80	2.16	0	21:59
Outfall2	OUTFALL	1.72	2.71	2.16	0	21:59
Outfall3	OUTFALL	2.09	3.08	2.16	0	21:59
Outfall5	OUTFALL	2.10	3.09	2.16	0	21:59
j6	STORAGE	1.83	2.59	2.95	0	00:00
j8	STORAGE	2.06	2.96	2.41	0	23:00
j10	STORAGE	2.21	3.20	2.28	0	23:00
j13	STORAGE	2.14	3.11	2.18	0	22:16
j15	STORAGE	1.92	2.59	2.36	0	00:00

****

Table C.117. Node Inflow Summary: Tide and Initial Depth Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	0.346	0.368	0	07:59	13.541	15.016
j4	JUNCTION	0.409	0.422	0	07:59	16.862	17.396
j1	JUNCTION	0.385	0.385	0	07:59	15.586	15.582
j5	JUNCTION	0.254	0.254	0	07:59	9.111	9.108
j16	JUNCTION	0.174	0.174	0	07:59	5.730	5.728
j3	JUNCTION	0.372	0.398	0	07:59	14.876	16.484
Treatmentplant	JUNCTION	0.321	0.800	0	07:59	9.886	34.058
j11	JUNCTION	0.000	0.043	0	00:00	0.000	3.593
j9	JUNCTION	0.000	0.029	0	00:00	0.000	2.409
j7	JUNCTION	0.000	0.020	0	00:00	0.000	1.345
j12	JUNCTION	0.000	0.031	0	00:00	0.000	2.550
j14	JUNCTION	0.000	0.010	0	00:00	0.000	0.858
TreatmentPlantOutfall	OUTFALL	0.000	0.478	0	22:05	0.000	28.183
Outfall4	OUTFALL	0.000	0.698	0	00:00	0.000	7.647
Outfall6	OUTFALL	0.000	0.680	0	00:00	0.000	20.274
Outfall1	OUTFALL	0.000	0.709	0	00:00	0.000	23.406
Outfall2	OUTFALL	0.000	0.609	0	00:00	0.000	20.085
Outfall3	OUTFALL	0.000	0.517	0	00:00	0.000	14.619
Outfall5	OUTFALL	0.000	0.915	0	00:00	0.000	16.867
j6	STORAGE	0.318	0.680	0	07:59	10.275	26.939
j8	STORAGE	0.227	0.605	0	07:59	7.091	25.502
j10	STORAGE	0.000	0.427	0	08:00	0.000	21.467
j13	STORAGE	0.320	0.571	0	07:59	10.551	22.579
j15	STORAGE	0.454	0.627	0	07:59	15.358	23.665

****

Table C.118. Node Surcharge Summary: Tide and Initial Depth Scenario

****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
j11	JUNCTION	18.60	5.703	0.000
j9	JUNCTION	2.16	7.889	0.000
j7	JUNCTION	0.05	8.871	0.000
j12	JUNCTION	10.21	7.851	0.000
j14	JUNCTION	0.05	9.631	0.000
j8	STORAGE	1.63	0.370	1.729
j10	STORAGE	4.28	0.609	1.147
j13	STORAGE	3.44	0.523	1.786

****

Table C.119. Node Flooding Summary: Tide and Initial Depth Scenario

****

Flooding refers to all water that overflows a node, whether it ponds or not.

Node	Hours Flooded	Maximum Rate CMS	Time of Max Occurrence days hr:min	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
j11	0.01	0.019	0 00:00	0.000	5.96
j9	0.01	0.011	0 00:00	0.000	8.14
j7	0.01	0.006	0 00:00	0.000	9.12
j12	0.01	0.005	0 00:00	0.000	8.11
j14	0.01	0.001	0 00:00	0.000	9.89

****

Table C.120. Storage Volume Summary: Tide and Initial DepthScenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	1.829	38	0	2.591	54	0 00:00	0.724
j8	2.063	44	0	2.961	63	0 23:00	0.633
j10	2.212	51	0	3.200	74	0 23:00	0.561
j13	2.139	44	0	3.114	64	0 22:16	0.926
j15	1.916	43	0	2.591	58	0 00:00	0.685

****

Table C.121. Outfall Loading Summary: Tide and Initial Depth Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	100.00	0.344	0.478	28.183	27.476	0.272
Outfall4	100.00	0.095	0.698	7.647	257.391	0.064
Outfall6	100.00	0.247	0.680	20.274	1110.925	0.278
Outfall1	100.00	0.285	0.709	23.406	383.864	0.096
Outfall2	100.00	0.245	0.609	20.085	846.998	0.212
Outfall3	100.00	0.178	0.517	14.619	339.895	0.085
Outfall5	100.00	0.206	0.915	16.867	806.692	0.202
System	100.00	1.600	4.299	131.081	3773.241	1.209

****

Table C.122. Link Flow Summary: Tide and Initial DepthScenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.023	0 07:59	1.44	1.04	0.96
c6	CONDUIT	0.405	0 08:00	1.58	0.12	0.62
c9	CONDUIT	0.362	0 08:00	1.38	0.15	0.63
c16	CONDUIT	0.478	0 22:05	2.48	13.26	0.95
c5	CONDUIT	0.241	0 08:00	1.01	0.06	0.58
c11	CONDUIT	0.036	0 00:11	0.78	0.63	1.00
c13	CONDUIT	0.040	0 00:12	0.86	0.90	1.00
c14	CONDUIT	0.053	0 00:00	1.04	0.95	1.00
c2	CONDUIT	0.010	0 07:57	0.57	1.07	0.95
c4	CONDUIT	0.012	0 08:00	0.73	0.93	0.88
c8	CONDUIT	0.358	0 08:00	1.44	0.09	0.60
c10	CONDUIT	0.051	0 00:00	1.01	0.87	1.00
c15	CONDUIT	0.174	0 08:00	0.73	0.05	0.58
c12	CONDUIT	0.049	0 00:17	1.02	0.92	1.00
c3	CONDUIT	0.017	0 07:12	1.03	1.07	0.99
c7	CONDUIT	0.398	0 08:00	1.59	0.10	0.60
P2	PUMP	0.031	0 00:00		1.00	
p3	PUMP	0.043	0 00:00		1.00	
P5	PUMP	0.020	0 00:00		1.00	
P1	PUMP	0.010	0 00:00		1.00	
P4	PUMP	0.029	0 00:00		1.00	
R4	WEIR	0.698	0 00:00			1.00
R6	WEIR	0.680	0 00:00			1.00
R1	WEIR	0.709	0 00:00			1.00
R2	WEIR	0.609	0 00:00			1.00
R3	WEIR	0.517	0 00:00			1.00
R5	WEIR	0.915	0 00:00			1.00

****

Table C.123. Flow Classification Summary: Tide and Initial DepthScenario

****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
		--- Fraction of Time in Flow Class ----								
c1	1.00	0.00	0.00	0.00	0.26	0.73	0.00	0.00	1.29	0.0013
c6	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.45	0.0002
c9	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.37	0.0002
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.73	0.0195
c5	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.26	0.0001
c11	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.44	0.0003
c13	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.25	0.0006
c14	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.25	0.0006
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.54	0.0013
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.61	0.0012
c8	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.41	0.0001
c10	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.34	0.0008
c15	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.17	0.0001
c12	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.11	0.0006
c3	1.00	0.00	0.00	0.00	0.49	0.50	0.00	0.00	0.93	0.0012
c7	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.46	0.0001

****

Table C.124. Conduit Surcharge Summary: Tide and Initial DepthScenario

****

Conduit	Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
	------------ Hours Full -------------				
c1	0.01	0.01	0.01	6.39	0.01
c16	0.01	0.01	0.01	23.00	0.01
c11	2.16	2.16	2.16	0.01	0.01
c13	10.21	10.21	10.23	0.01	0.01
c14	0.01	0.01	0.02	0.01	0.01
c2	0.01	0.01	0.01	3.08	0.01
c10	0.01	0.01	0.02	0.01	0.01
c12	18.60	18.60	18.62	0.01	0.01
c3	0.01	0.01	0.01	7.20	0.01

****

Table C.125. Pumping Summary: Tide and Initial DepthScenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Low	% Time Off Curve High
P2	100.00	1	0.00	0.03	0.03	2.550	8.75	0.0	100.0
P3	100.00	1	0.00	0.04	0.04	3.593	11.41	0.0	100.0
P5	81.21	2	0.00	0.02	0.02	1.345	5.25	0.0	100.0
P1	100.00	1	0.00	0.01	0.01	0.859	1.95	0.0	100.0
P4	100.00	1	0.00	0.03	0.03	2.409	7.10	0.0	100.0

Analysis begun on: Sat May 04 15:28:45 2013
Analysis ended on: Sat May 04 15:28:45 2013
Total elapsed time: < 1 sec

C.7. Regulators and Precipitation Focus Scenario Simulation Status Report

****

Table C.126. Analysis Options: Regulators and Precipitation Focus Scenario

****

Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method .... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****
Table C.127. Runoff Quantity Continuity: Regulators and Precipitation Focus Scenario
****

	Volume hectare-m	Volume 10^6 ltr
Total Precipitation	17.148	86.000
Evaporation Loss	0.000	0.000
Infiltration Loss	1.436	7.200
Surface Runoff	15.615	78.312
Final Surface Storage	0.103	0.517
Continuity Error (%)	-0.033	

****
Table C.128. Runoff Quality Continuity: Regulators and Precipitation Focus Scenario
****

	TSS kg	Lead kg
Initial Buildup	6111.392	0.000
Surface Buildup	1329.555	1.528
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	6111.506	1.528
Remaining Buildup	1329.555	0.000
Continuity Error (%)	-0.002	0.000

****
Table C.129. Flow Routing Continuity: Regulators and Precipitation Focus Scenario
****

	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	15.615	156.148
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
External Outflow	3.379	33.794
Internal Outflow	9.791	97.907
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	2.245	22.451
Continuity Error (%)	1.278	

****

Table C.130. Quality Routing Continuity: Regulators and Precipitation Focus Scenario

****

	TSS kg	Lead kg
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	6109.955	1.527
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	4050.742	1.013
External Outflow	118.232	0.327
Mass Reacted	1188.055	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	646.248	0.162
Continuity Error (%)	1.746	1.746

****

Table C.131. Highest Continuity Errors: Regulators and Precipitation Focus Scenario

****

Node j14 (2.18%)
Node j6 (2.05%)
Node j13 (2.01%)
Node j8 (1.78%)
Node j10 (1.71%)

****

Table C.132. Time-Step Critical Elements: Regulators and Precipitation Focus Scenario

****

Link c7 (27.87%)
Link c16 (6.11%)

****

Table C.133. Highest Flow Instability Indexes: Regulators and Precipitation Focus Scenario

****

Link c12 (2)

****

Table C.134. Routing Time Step Summary: Regulators and Precipitation Focus Scenario

****

Minimum Time Step : 5.64 sec
Average Time Step : 27.22 sec
Maximum Time Step : 30.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.56

****

Table C.135. Subcatchment Runoff Summary: Regulators and Precipitation Focus Scenario

****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	86.00	0.00	0.00	9.12	75.82	19.72	1.34	0.882
S2	86.00	0.00	0.00	9.12	76.58	16.92	1.20	0.891
S13	86.00	0.00	0.00	5.25	80.71	17.97	1.45	0.938
S11	86.00	0.00	0.00	2.73	83.32	3.52	0.31	0.969
S10	86.00	0.00	0.00	2.78	83.26	3.74	0.33	0.968
S9	86.00	0.00	0.00	3.58	82.44	7.39	0.64	0.959
S8	86.00	0.00	0.00	3.70	82.31	8.05	0.69	0.957
S7	86.00	0.00	0.00	4.35	81.64	11.81	1.00	0.949
S6	86.00	0.00	0.00	6.20	79.83	6.82	0.58	0.928
S3	86.00	0.00	0.00	9.12	76.09	18.74	1.29	0.885
S4	86.00	0.00	0.00	9.12	75.34	21.50	1.42	0.876
S5	86.00	0.00	0.00	7.70	78.31	11.07	0.87	0.911
S12	86.00	0.00	0.00	6.96	79.06	8.88	0.73	0.919

****

Table C.136. Subcatchment Washoff Summary: Regulators and Precipitation Focus Scenario

****

Subcatchment	TSS kg	Lead kg
S1	193.765	0.048
S2	658.248	0.165
S13	1244.271	0.311
S11	252.094	0.063
S10	267.589	0.067
S9	501.170	0.125
S8	546.426	0.137
S7	323.387	0.081
S6	190.859	0.048
S3	458.710	0.115
S4	531.530	0.133
S5	316.013	0.079
S12	627.445	0.157
System	6111.506	1.528

****
Table C.137. Node Depth Summary: Regulators and Precipitation Focus Scenario
****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.09	0.26	17.94	0	02:59
j4	JUNCTION	0.23	2.00	22.11	0	02:31
j1	JUNCTION	0.13	0.37	22.93	0	02:59
j5	JUNCTION	0.07	0.22	22.17	0	02:59
j16	JUNCTION	0.06	0.20	23.20	0	02:59
j3	JUNCTION	0.10	0.28	16.43	0	02:59
Treatmentplant	JUNCTION	4.43	20.62	20.63	0	03:01
j11	JUNCTION	1.85	5.96	7.60	0	00:30
j9	JUNCTION	1.74	2.26	4.65	0	01:48
j7	JUNCTION	0.94	3.71	7.30	0	01:36
j12	JUNCTION	2.24	8.11	10.17	0	00:35
j14	JUNCTION	1.42	2.75	5.18	0	01:30
TreatmentPlantOutfall	OUTFALL	0.33	0.50	0.50	0	00:29
Outfall4	OUTFALL	0.85	1.33	1.33	0	18:59
Outfall6	OUTFALL	1.08	1.56	1.33	0	18:59
Outfall1	OUTFALL	0.49	0.97	1.33	0	18:59
Outfall2	OUTFALL	1.40	1.88	1.33	0	18:59
Outfall3	OUTFALL	1.77	2.25	1.33	0	18:59
Outfall5	OUTFALL	1.78	2.26	1.33	0	18:59
j6	STORAGE	4.58	4.84	5.20	0	01:31
j8	STORAGE	4.45	4.69	4.14	0	01:36
j10	STORAGE	4.05	4.35	3.43	0	02:04
j13	STORAGE	4.54	4.90	3.97	0	01:39
j15	STORAGE	4.25	4.50	4.27	0	01:28

****

Table C.138. Node Inflow Summary: Regulators and Precipitation Focus Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	1.202	1.224	0	02:59	16.917	17.851
j4	JUNCTION	1.424	1.426	0	02:59	21.501	21.811
j1	JUNCTION	1.340	1.340	0	02:59	19.721	19.722
j5	JUNCTION	0.872	0.872	0	02:59	11.073	11.073
j16	JUNCTION	0.578	0.578	0	02:59	6.818	6.817
j3	JUNCTION	1.293	1.322	0	02:59	18.740	19.780
Treatmentplant	JUNCTION	0.965	2.238	0	02:35	11.135	36.446
j11	JUNCTION	0.000	0.194	0	03:01	0.000	5.021
j9	JUNCTION	0.000	0.029	0	00:54	0.000	2.315
j7	JUNCTION	0.000	0.036	0	01:36	0.000	1.595
j12	JUNCTION	0.000	0.122	0	03:01	0.000	3.212
j14	JUNCTION	0.000	0.034	0	01:29	0.000	0.833
TreatmentPlantOutfall	OUTFALL	0.000	1.618	0	03:01	0.000	33.794
Outfall4	OUTFALL	0.000	0.000	0	00:00	0.000	0.000
Outfall6	OUTFALL	0.000	0.000	0	00:00	0.000	0.000
Outfall1	OUTFALL	0.000	0.000	0	00:00	0.000	0.000
Outfall2	OUTFALL	0.000	0.000	0	00:00	0.000	0.000
Outfall3	OUTFALL	0.000	0.000	0	00:00	0.000	0.000
Outfall5	OUTFALL	0.000	0.000	0	00:00	0.000	0.000
j6	STORAGE	0.995	2.312	0	02:59	11.814	30.608
j8	STORAGE	0.694	1.928	0	02:59	8.051	27.138
j10	STORAGE	0.000	1.350	0	02:59	0.000	22.079
j13	STORAGE	1.034	1.914	0	02:59	12.403	23.984
j15	STORAGE	1.449	2.027	0	02:59	17.975	24.796

****

Table C.139. Node Surcharge Summary: Regulators and Precipitation Focus Scenario

****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
j4	JUNCTION	0.96	1.289	0.000
Treatmentplant	JUNCTION	5.13	18.024	0.000
j11	JUNCTION	8.32	5.703	0.000
j9	JUNCTION	21.21	2.004	3.109
j7	JUNCTION	21.40	3.451	2.153
j12	JUNCTION	6.57	7.851	0.000
j14	JUNCTION	21.51	2.493	3.491
j6	STORAGE	21.96	2.249	0.000
j8	STORAGE	21.90	2.099	0.000
j10	STORAGE	21.49	1.756	0.000
j13	STORAGE	21.89	2.309	0.000
j15	STORAGE	21.94	1.909	0.000

****

Table C.140. Node Flooding Summary: Regulators and Precipitation Focus Scenario

****

Flooding refers to all water that overflows a node, whether it ponds or not.

Node	Hours Flooded	Maximum Rate CMS	Time of Max Occurrence days hr:min		Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
j4	0.67	0.459	0	02:59	0.626	2.00
Treatmentplant	5.07	0.472	0	01:15	3.480	20.62
j11	4.41	0.194	0	03:01	2.228	5.96
j12	3.55	0.122	0	03:01	1.161	8.11
j6	14.34	2.291	0	02:59	23.797	4.84
j8	16.53	1.898	0	02:59	19.729	4.69
j10	15.75	1.306	0	03:00	14.073	4.35
j13	10.38	1.893	0	02:59	16.904	4.90
j15	12.33	2.017	0	02:59	19.374	4.50

****

Table C.141. Storage Volume Summary: Regulators and Precipitation Focus Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	4.578	95	0	4.840	100	0 01:31	0.020
j8	4.449	95	0	4.690	100	0 01:36	0.045
j10	4.047	93	0	4.347	100	0 02:04	0.043
j13	4.538	93	0	4.900	100	0 01:39	0.041
j15	4.252	94	0	4.500	100	0 01:28	0.010

****

Table C.142. Outfall Loading Summary: Regulators and Precipitation Focus Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	99.93	0.487	1.618	33.794	118.214	0.327
Outfall4	0.00	0.000	0.000	0.000	0.000	0.000
Outfall6	0.00	0.000	0.000	0.000	0.000	0.000
Outfall1	0.00	0.000	0.000	0.000	0.000	0.000
Outfall2	0.00	0.000	0.000	0.000	0.000	0.000
Outfall3	0.00	0.000	0.000	0.000	0.000	0.000
Outfall5	0.00	0.000	0.000	0.000	0.000	0.000
System	14.28	0.487	1.618	33.794	118.214	0.327

****
Table C.143. Link Flow Summary: Regulators and Precipitation Focus Scenario
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.023	0 05:03	2.25	1.05	1.00
c6	CONDUIT	1.284	0 02:35	3.44	0.37	1.00
c9	CONDUIT	1.317	0 03:00	4.07	0.53	0.76
c16	CONDUIT	1.618	0 03:01	8.24	44.86	1.00
c5	CONDUIT	0.870	0 02:59	5.00	0.21	0.66
c11	CONDUIT	0.030	0 01:48	0.71	0.54	1.00
c13	CONDUIT	0.101	0 03:01	1.99	2.26	1.00
c14	CONDUIT	0.024	0 01:29	0.47	0.43	1.00
c2	CONDUIT	0.010	0 00:26	0.57	1.07	1.00
c4	CONDUIT	0.014	0 00:38	0.79	1.06	1.00
c8	CONDUIT	1.214	0 02:59	6.04	0.29	0.68
c10	CONDUIT	0.026	0 01:36	0.53	0.44	1.00
c15	CONDUIT	0.578	0 02:59	4.76	0.17	0.64
c12	CONDUIT	0.150	0 03:01	2.97	2.83	1.00
c3	CONDUIT	0.019	0 03:15	1.06	1.20	1.00
c7	CONDUIT	1.321	0 02:59	7.45	0.3 2	0.69
P2	PUMP	0.031	0 05:16		1.00	
p3	PUMP	0.043	0 02:04		1.00	
P5	PUMP	0.020	0 00:52		1.00	
P1	PUMP	0.010	0 00:53		1.00	
P4	PUMP	0.029	0 00:54		1.00	
R4	WEIR	0.000	0 00:00			0.00
R6	WEIR	0.000	0 00:00			0.00
R1	WEIR	0.000	0 00:00			0.00
R2	WEIR	0.000	0 00:00			0.00
R3	WEIR	0.000	0 00:00			0.00
R5	WEIR	0.000	0 00:00			0.00

****

Table C.144. Flow Classification Summary: Regulators and Precipitation Focus Scenario

****

Conduit	Adjusted /Actual Length	--- Fraction of Time in Flow Class ---- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
c1	1.00	0.00	0.00	0.00	0.33	0.67	0.00	0.00	1.30	0.0008
c6	1.00	0.00	0.00	0.00	0.81	0.19	0.00	0.00	0.48	0.0003
c9	1.00	0.00	0.00	0.00	0.75	0.25	0.00	0.00	0.57	0.0003
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.40	0.0288
c5	1.00	0.00	0.00	0.00	0.81	0.19	0.00	0.00	0.45	0.0001
c11	1.00	0.00	0.04	0.00	0.96	0.00	0.00	0.00	0.02	0.0004
c13	1.00	0.00	0.02	0.00	0.98	0.00	0.00	0.00	0.30	0.0030
c14	1.00	0.00	0.04	0.00	0.96	0.00	0.00	0.00	0.01	0.0005
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.42	0.0008
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.24	0.0013
c8	1.00	0.00	0.00	0.00	0.74	0.26	0.00	0.00	0.64	0.0002
c10	1.00	0.00	0.04	0.00	0.96	0.00	0.00	0.00	0.01	0.0006
c15	1.00	0.00	0.00	0.00	0.91	0.09	0.00	0.00	0.34	0.0001
c12	1.00	0.00	0.02	0.00	0.98	0.00	0.00	0.00	0.34	0.0031
c3	1.00	0.00	0.00	0.00	0.38	0.62	0.00	0.00	0.89	0.0009
c7	1.00	0.00	0.00	0.00	0.72	0.28	0.00	0.00	0.76	0.0002

****

Table C.145. Conduit Surcharge Summary: Regulators and Precipitation Focus Scenario

****

Conduit	------------ Hours Full ------------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c1	4.35	4.35	4.36	6.24	4.35
c6	0.96	0.96	0.96	0.01	0.01
c16	5.10	5.10	5.10	22.86	5.10
c11	21.21	21.21	21.21	0.01	0.01
c13	6.57	6.57	6.58	2.95	0.01
c14	21.51	21.51	21.51	0.01	0.01
c2	4.35	4.35	4.36	1.15	0.01
c4	3.15	3.15	3.16	0.85	0.01
c10	21.40	21.40	21.40	0.01	0.01
c12	8.32	8.32	8.33	3.71	0.01
c3	4.78	4.78	4.79	6.63	4.78

****

Table C.146. Pumping Summary: Regulators and Precipitation Focus Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Low	% Time Off Curve High
P2	96.07	1	0.00	0.03	0.03	2.290	13.75	0.0	80.2
p3	95.18	1	0.00	0.04	0.04	3.409	19.16	0.0	95.6
P5	96.22	1	0.00	0.02	0.02	1.594	2.45	0.0	100.0
P1	96.16	1	0.00	0.01	0.01	0.826	0.64	0.0	100.0
P4	96.07	1	0.00	0.03	0.03	2.315	1.30	0.0	100.0

Analysis begun on: Sat May 04 16:56:28 2013
Analysis ended on: Sat May 04 16:56:28 2013
Total elapsed time: < 1 sec

C.8. Tide and Impervious Focus Scenario Simulation Status Report

****

Table C.147. Analysis Options: Tide and Impervious Focus Scenario

****

Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method .... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****

Table C.148. Runoff Quantity Continuity: Tide and Impervious Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
Total Precipitation	16.523	82.870
Evaporation Loss	0.000	0.000
Infiltration Loss	0.984	4.933
Surface Runoff	14.035	70.390
Final Surface Storage	1.533	7.688
Continuity Error (%)	-0.170	

****

Table C.149. Runoff Quality Continuity: Tide and Impervious Focus Scenario

****

	TSS kg	Lead kg
Initial Buildup	6111.392	0.000
Surface Buildup	0.073	1.470
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	5873.368	1.468
Remaining Buildup	231.284	0.000
Continuity Error (%)	0.111	0.116

****

Table C.150. Flow Routing Continuity: Tide and Impervious Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-----------	-----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	13.951	139.514
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.567	5.674
External Outflow	12.941	129.416
Internal Outflow	0.000	0.000
Storage Losses	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	1.437	14.366
Continuity Error (%)	0.968	

****

Table C.151. Quality Routing Continuity: Tide and Impervious Focus Scenario

****

	TSS kg	Lead kg
	----------	----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	5859.851	1.465
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	0.000	0.000
External Outflow	4079.861	1.311
Mass Reacted	1162.651	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	579.223	0.145
Continuity Error (%)	0.650	0.650

****

Table C.152. Highest Continuity Errors: Tide and Impervious Focus Scenario

****

Node j13 (1.49%)
Node j10 (1.40%)
Node j8 (1.25%)
Node j6 (1.23%)

****

Table C.153. Time-Step Critical Elements: Tide and Impervious Focus Scenario

****

Link c16 (35.82%)

****

Table C.154. Highest Flow Instability Indexes: Tide and Impervious Focus Scenario

****

Link R4 (18)
Link c16 (7)
Link c12 (1)

****

Table C.155. Routing Time Step Summary: Tide and Impervious Focus Scenario

****

Minimum Time Step	:	18.45 sec
Average Time Step	:	29.22 sec
Maximum Time Step	:	30.00 sec
Percent in Steady State	:	0.00
Average Iterations per Step	:	2.11

****

Table C.156. Subcatchment Runoff Summary: Tide and Impervious Focus Scenario

****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	82.87	0.00	0.00	6.08	66.59	17.32	0.24	0.804
S2	82.87	0.00	0.00	6.08	67.55	14.92	0.20	0.815
S13	82.87	0.00	0.00	3.04	73.63	16.40	0.21	0.889
S11	82.87	0.00	0.00	3.04	77.61	3.28	0.04	0.936
S10	82.87	0.00	0.00	3.04	77.52	3.48	0.04	0.935
S9	82.87	0.00	0.00	3.04	76.28	6.84	0.09	0.920
S8	82.87	0.00	0.00	3.04	76.09	7.44	0.09	0.918
S7	82.87	0.00	0.00	3.04	75.07	10.86	0.14	0.906
S6	82.87	0.00	0.00	6.08	71.60	6.11	0.08	0.864
S3	82.87	0.00	0.00	6.08	66.92	16.48	0.23	0.808
S4	82.87	0.00	0.00	6.08	66.00	18.84	0.26	0.796
S5	82.87	0.00	0.00	6.08	69.74	9.86	0.13	0.842
S12	82.87	0.00	0.00	3.04	75.76	8.51	0.11	0.914

****

Table C.157. Subcatchment Washoff Summary: Tide and Impervious Focus Scenario

****

Subcatchment	TSS kg	Lead kg
S1	172.835	0.043
S2	595.60	0.149
S13	1227.423	0.307
S11	252.091	0.063
S10	267.586	0.067
S9	501.165	0.125
S8	546.421	0.137
S7	323.359	0.081
S6	183.088	0.046
S3	411.191	0.103
S4	469.944	0.117
S5	295.232	0.074
S12	627.434	0.157
System	5873.368	1.468

****

Table C.158. Node Depth Summary: Tide and Impervious Focus Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.10	0.11	17.79	0	23:00
j4	JUNCTION	0.12	0.13	20.24	0	23:00
j1	JUNCTION	0.13	0.14	22.70	0	23 :00
j5	JUNCTION	0.08	0.08	22.03	0	23:00
j16	JUNCTION	0.07	0.08	23.08	0	23:00
j3	JUNCTION	0.11	0.12	16.27	0	23:00
Treatmentplant	JUNCTION	1.37	2.15	2.16	0	22:01
j11	JUNCTION	0.82	1.60	3.24	0	22:02
j9	JUNCTION	0.17	0.77	3.16	0	22:26
j7	JUNCTION	0.08	0.10	3.69	0	08:13
j12	JUNCTION	0.34	1.07	3.13	0	22:02
j14	JUNCTION	0.05	0.07	2.50	0	09:48
TreatmentPlantOutfall	OUTFALL	0.40	0.46	0.46	0	22:01
Outfall4	OUTFALL	1.17	2.16	2.16	0	21:59
Outfall6	OUTFALL	1.40	2.39	2.16	0	21:59
Outfall1	OUTFALL	0.81	1.80	2.16	0	21:59
Outfall2	OUTFALL	1.72	2.71	2.16	0	21:59
Outfall3	OUTFALL	2.09	3.08	2.16	0	21:59
Outfall5	OUTFALL	2.10	3.09	2.16	0	21:59
j6	STORAGE	1.75	2.19	2.55	0	22:43
j8	STORAGE	2.03	2.90	2.35	0	22:38
j10	STORAGE	2.16	3.19	2.27	0	22:27
j13	STORAGE	2.11	3.12	2.19	0	22:07
j15	STORAGE	1.85	2.49	2.26	0	22:21

****

Table C.159. Node Inflow Summary: Tide and Impervious Focus Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	0.205	0.227	0 23:00	14.833	16.535
j4	JUNCTION	0.264	0.272	0 23:00	18.723	19.312
j1	JUNCTION	0.241	0.241	0 23:00	17.216	17.212
j5	JUNCTION	0.131	0.131	0 23:00	9.804	9.802
j16	JUNCTION	0.079	0.079	0 23:00	6.081	6.079
j3	JUNCTION	0.228	0.253	0 23:00	16.384	18.205
Treatmentplant	JUNCTION	0.130	0.501	0 22:02	10.267	34.558
j11	JUNCTION	0.000	0.043	0 01:19	0.000	3.387
j9	JUNCTION	0.000	0.029	0 05:56	0.000	1.788
j7	JUNCTION	0.000	0.020	0 05:53	0.000	1.232
j12	JUNCTION	0.000	0.031	0 00:43	0.000	2.470
j14	JUNCTION	0.000	0.010	0 07:03	0.000	0.596
TreatmentPlantOutfall	OUTFALL	0.000	0.479	0 22:01	0.000	28.845
Outfall4	OUTFALL	0.000	0.111	0 16:00	0.000	5.756
Outfall6	OUTFALL	0.000	0.306	0 13:31	0.000	20.256
Outfall1	OUTFALL	0.000	0.342	0 13:54	0.000	22.757
Outfall2	OUTFALL	0.000	0.334	0 13:28	0.000	21.504
Outfall3	OUTFALL	0.000	0.390	0 00:00	0.000	16.333
Outfall5	OUTFALL	0.000	1.001	0 00:00	0.000	19.638
j6	STORAGE	0.139	0.358	0 23:00	10.802	26.392
j8	STORAGE	0.094	0.333	0 23:00	7.401	25.521
j10	STORAGE	0.000	0.390	0 00:00	0.000	21.579
j13	STORAGE	0.149	1.001	0 00:00	11.726	23.522
j15	STORAGE	0.214	0.294	0 23:00	16.306	22.953

****

Table C.160. Node Surcharge Summary: Tide and Impervious Focus Scenario

****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
j11	JUNCTION	20.62	1.350	2.670
j9	JUNCTION	2.42	0.520	4.593
j12	JUNCTION	9.41	0.815	4.279
j8	STORAGE	1.93	0.309	1.790
j10	STORAGE	3.46	0.601	1.155
j13	STORAGE	3.46	0.529	1.780

****

Table C.161. Node Flooding Summary: Tide and Impervious Focus Scenario

****

No nodes were flooded.

****

Table C.162. Storage Volume Summary: Tide and Impervious Focus Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	1.750	36	0	2.188	45	0 22:43	0.362
j8	2.029	43	0	2.900	62	0 22:38	0.363
j10	2.159	50	0	3.192	73	0 22:27	0.328
j13	2.110	43	0	3.120	64	0 22:07	0.358
j15	1.847	41	0	2.486	55	0 22:21	0.316

****
Table C.163. Outfall Loading Summary: Tide and Impervious Focus Scenario
****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	99.96	0.351	0.479	28.845	28.983	0.298
Outfall4	100.00	0.068	0.111	5.756	226.279	0.057
Outfall6	100.00	0.245	0.306	20.256	1205.035	0.301
Outfall1	100.00	0.276	0.342	22.757	412.139	0.103
Outfall2	100.00	0.260	0.334	21.504	936.839	0.234
Outfall3	100.00	0.197	0.390	16.333	371.068	0.093
Outfall5	100.00	0.237	1.001	19.638	900.380	0.225
System	99.99	1.634	2.042	135.089	4080.723	1.311

****

Table C.164. Link Flow Summary: Tide and Impervious Focus Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.022	0 23:00	1.43	1.03	0.83
c6	CONDUIT	0.256	0 23:00	1.05	0.07	0.59
c9	CONDUIT	0.219	0 23:00	1.20	0.09	0.60
c16	CONDUIT	0.479	0 22:01	2.48	13.30	0.96
c5	CONDUIT	0.123	0 23:00	0.54	0.03	0.56
c11	CONDUIT	0.030	0 12:53	0.72	0.53	1.00
c13	CONDUIT	0.033	0 16:02	0.70	0.74	1.00
c14	CONDUIT	0.010	0 09:49	0.30	0.19	0.65
c2	CONDUIT	0.008	0 23:00	0.55	0.87	0.75
c4	CONDUIT	0.008	0 23:00	0.58	0.59	0.70
c8	CONDUIT	0.219	0 23:00	0.92	0.05	0.58
c10	CONDUIT	0.020	0 08:13	0.53	0.34	0.70
c15	CONDUIT	0.079	0 23:00	0.35	0.02	0.55
c12	CONDUIT	0.043	0 22:07	0.91	0.82	1.00
c3	CONDUIT	0.016	0 23:00	1.02	1.01	0.82
c7	CONDUIT	0.253	0 23:00	1.05	0.06	0.58
P2	PUMP	0.031	0 00:43		1.00	
p3	PUMP	0.043	0 01:19		1.00	
P5	PUMP	0.020	0 05:53		1.00	
P1	PUMP	0.010	0 07:03		1.00	
P4	PUMP	0.029	0 05:56		1.00	
R4	WEIR	0.111	0 16:00			0.83
R6	WEIR	0.306	0 13:31			0.96
R1	WEIR	0.342	0 13:54			0.84
R2	WEIR	0.334	0 13:28			1.00
R3	WEIR	0.390	0 00:00			1.00
R5	WEIR	1.001	0 00:00			1.00

****
Table C.165. Flow Classification Summary: Tide and Impervious Focus Scenario
****

Conduit	Adjusted /Actual Length	--- Fraction of Time in Flow Class ---- Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
c1	1.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00	1.28	0.0004
c6	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.49	0.0000
c9	1.00	0.00	0.00	0.00	0.96	0.04	0.00	0.00	0.48	0.0000
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.74	0.0059
c5	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.28	0.0000
c11	1.00	0.00	0.25	0.00	0.75	0.00	0.00	0.00	0.31	0.0002
c13	1.00	0.00	0.03	0.00	0.97	0.00	0.00	0.00	0.24	0.0004
c14	1.00	0.00	0.30	0.00	0.70	0.00	0.00	0.00	0.18	0.0001
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.55	0.0003
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.61	0.0002
c8	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.45	0.0000
c10	1.00	0.00	0.25	0.00	0.75	0.00	0.00	0.00	0.31	0.0001
c15	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.20	0.0000
c12	1.00	0.00	0.06	0.00	0.94	0.00	0.00	0.00	0.02	0.0003
c3	1.00	0.00	0.00	0.00	0.79	0.21	0.00	0.00	0.94	0.0004
c7	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.51	0.0000

****
Table C.166. Conduit Surcharge Summary: Tide and Impervious Focus Scenario
****

Conduit	------------ Hours Full ------------- Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c1	0.01	0.01	0.01	16.07	0.01
c16	0.01	0.01	0.01	22.81	0.01
c11	2.42	2.42	2.42	0.01	0.01
c13	9.41	9.41	9.42	0.01	0.01
c12	20.62	20.62	20.62	0.01	0.01
c3	0.01	0.01	0.01	11.43	0.01

****

Table C.167. Pumping Summary: Tide and Impervious Focus Scenario

****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Low	Curve High
P2	96.88	1	0.00	0.03	0.03	2.471	8.11	0.0	100.0
p3	94.28	1	0.00	0.04	0.04	3.388	11.02	0.0	100.0
P5	74.42	1	0.00	0.02	0.02	1.232	4.57	0.0	100.0
P1	69.38	1	0.00	0.01	0.01	0.596	1.10	0.0	100.0
P4	74.24	1	0.00	0.03	0.03	1.789	4.52	0.0	100.0

Analysis begun on: Sat May 04 19:44:54 2013
Analysis ended on: Sat May 04 19:44:54 2013
Total elapsed time: < 1 sec

C.9. Depth and Precipitation Focus Scenario Simulation Status Report

****

Table C.168. Analysis Options: Depth and Precipitation Focus Scenario

****

Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method .... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****

Table C.169. Runoff Quantity Continuity: Depth and Precipitation Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-------------	-------------
Total Precipitation	17.148	86.000
Evaporation Loss	0.000	0.000
Infiltration Loss	1.436	7.200
Surface Runoff	15.615	78.312
Final Surface Storage	0.103	0.517
Continuity Error (%)	-0.033	

****

Table C.170. Runoff Quality Continuity: Depth and Precipitation Focus Scenario

****

	TSS kg	Lead kg
	----------	----------
Initial Buildup	6111.392	0.000
Surface Buildup	1329.555	1.528
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	6111.506	1.528
Remaining Buildup	1329.555	0.000
Continuity Error (%)	-0.002	0.000

****

Table C.171. Flow Routing Continuity: Depth and Precipitation Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
	-----------	-----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	15.615	156.147
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.630	6.304
External Outflow	16.008	160.087
Internal Outflow	1.073	10.726
Storage Losses	0.000	0.000
Initial Stored Volume	1.347	13.472
Final Stored Volume	0.616	6.163
Continuity Error (%)	-0.598	

****

Table C.172. Quality Routing Continuity: Depth and Precipitation Focus Scenario

****

	TSS kg	Lead kg
	----------	----------
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	6109.954	1.527
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	430.286	0.108
External Outflow	4927.432	1.405
Mass Reacted	691.675	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	75.596	0.019
Continuity Error (%)	-0.246	-0.246

****

Table C.173. Highest Continuity Errors: Depth and Precipitation Focus Scenario

****

Node j14 (-6.39%)
Node j7 (-3.33%)

****
Table C.174. Time-Step Critical Elements: Depth and Precipitation Focus Scenario
****

Link c7 (20.29%)
Link c16 (10.18%)

****
Table C.175. Highest Flow Instability Indexes: Depth and Precipitation Focus Scenario
****

Link R5 (23)
Link R3 (6)

****
Table C.176. Routing Time Step Summary: Depth and Precipitation Focus Scenario
****

Minimum Time Step : 16.34 sec
Average Time Step : 27.45 sec
Maximum Time Step : 30.00 sec
Percent in Steady State : 0.00
Average Iterations per Step: 3.01

****
Table C.177. Subcatchment Runoff Summary: Depth and Precipitation Focus Scenario
****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	86.00	0.00	0.00	9.12	75.82	19.72	1.34	0.882
S2	86.00	0.00	0.00	9.12	76.58	16.92	1.20	0.891
S13	86.00	0.00	0.00	5.25	80.71	17.97	1.45	0.938
S11	86.00	0.00	0.00	2.73	83.32	3.52	0.31	0.969
S10	86.00	0.00	0.00	2.78	83.26	3.74	0.33	0.968
S9	86.00	0.00	0.00	3.58	82.44	7.39	0.64	0.959
S8	86.00	0.00	0.00	3.70	82.31	8.05	0.69	0.957
S7	86.00	0.00	0.00	4.35	81.64	11.81	1.00	0.949
S6	86.00	0.00	0.00	6.20	79.83	6.82	0.58	0.928
S3	86.00	0.00	0.00	9.12	76.09	18.74	1.29	0.885
S4	86.00	0.00	0.00	9.12	75.34	21.50	1.42	0.876
S5	86.00	0.00	0.00	7.70	78.31	11.07	0.87	0.911
S12	86.00	0.00	0.00	6.96	79.06	8.88	0.73	0.919

****
Table C.178. Subcatchment Washoff Summary: Depth and Precipitation Focus Scenario
****

Subcatchment	TSS kg	Lead kg
S1	193.765	0.048
S2	658.248	0.165
S13	1244.271	0.311
S11	252.094	0.063
S10	267.589	0.067
S9	501.170	0.125
S8	546.426	0.137
S7	323.387	0.081
S6	190.859	0.048
S3	458.710	0.115
S4	531.530	0.133
S5	316.013	0.079
S12	627.445	0.157
System	6111.506	1.528

****

Table C.179. Node Depth Summary: Depth and Precipitation Focus Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.09	0.26	17.94	0	02:59
j4	JUNCTION	0.12	0.32	20.43	0	02:59
j1	JUNCTION	0.13	0.37	22.93	0	02:59
j5	JUNCTION	0.07	0.22	22.17	0	02:59
j16	JUNCTION	0.06	0.20	23.20	0	02:59
j3	JUNCTION	0.10	0.28	16.43	0	02:59
Treatmentplant	JUNCTION	1.65	6.70	6.71	0	03:01
j11	JUNCTION	1.19	5.96	7.60	0	00:00
j9	JUNCTION	0.31	8.14	10.53	0	00:00
j7	JUNCTION	0.19	9.12	12.71	0	00:00
j12	JUNCTION	0.94	8.11	10.17	0	00:00
j14	JUNCTION	0.04	9.89	12.32	0	00:00
TreatmentPlantOutfall	OUTFALL	0.35	0.50	0.50	0	00:53
Outfall4	OUTFALL	0.85	1.33	1.33	0	19:00
Outfall6	OUTFALL	1.08	1.56	1.33	0	19:00
Outfall1	OUTFALL	0.49	0.97	1.33	0	19:00
Outfall2	OUTFALL	1.40	1.88	1.33	0	19:00
Outfall3	OUTFALL	1.77	2.25	1.33	0	19:00
Outfall5	OUTFALL	1.78	2.26	1.33	0	19:00
j6	STORAGE	2.08	4.84	5.20	0	01:57
j8	STORAGE	2.31	4.69	4.14	0	02:15
j10	STORAGE	2.31	4.09	3.17	0	03:19
j13	STORAGE	2.10	3.40	2.47	0	03:03
j15	STORAGE	2.08	4.50	4.27	0	01:52

****

Table C.180. Node Inflow Summary: Depth and Precipitation Focus Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	1.202	1.224	0	02:59	16.917	17.849
j4	JUNCTION	1.424	1.437	0	02:59	21.501	21.846
j1	JUNCTION	1.341	1.341	0	02:59	19.721	19.722
j5	JUNCTION	0.872	0.872	0	02:59	11.074	11.073
j16	JUNCTION	0.578	0.578	0	02:59	6.818	6.817
j3	JUNCTION	1.293	1.319	0	02:59	18.740	19.768
Treatmentplant	JUNCTION	0.965	2.450	0	02:59	11.136	41.107
j11	JUNCTION	0.000	0.043	0	00:00	0.000	3.590
j9	JUNCTION	0.000	0.029	0	00:00	0.000	1.786
j7	JUNCTION	0.000	0.020	0	00:00	0.000	0.760
j12	JUNCTION	0.000	0.031	0	00:00	0.000	2.530
j14	JUNCTION	0.000	0.010	0	00:00	0.000	0.374
TreatmentPlantOutfall	OUTFALL	0.000	0.899	0	03:01	0.000	24.569
Outfall4	OUTFALL	0.000	1.461	0	03:01	0.000	19.560
Outfall6	OUTFALL	0.000	1.125	0	01:53	0.000	23.601
Outfall1	OUTFALL	0.000	1.183	0	01:58	0.000	27.517
Outfall2	OUTFALL	0.000	1.158	0	02:15	0.000	24.927
Outfall3	OUTFALL	0.000	0.985	0	03:17	0.000	21.243
Outfall5	OUTFALL	0.000	1.673	0	03:03	0.000	24.972
j6	STORAGE	0.995	2.312	0	02:59	11.815	33.268
j8	STORAGE	0.694	1.928	0	02:59	8.051	29.449
j10	STORAGE	0.000	1.347	0	02:59	0.000	25.188
j13	STORAGE	1.034	1.901	0	02:59	12.404	27.507
j15	STORAGE	1.450	2.027	0	02:59	17.976	27.986

****

Table C.181. Node Surcharge Summary: Depth and Precipitation Focus Scenario

****

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
Treatmentplant	JUNCTION	3.36	4.111	0.000
j11	JUNCTION	9.40	5.703	0.000
j9	JUNCTION	3.59	7.889	0.000
j7	JUNCTION	2.62	8.871	0.000
j12	JUNCTION	5.88	7.851	0.000
j14	JUNCTION	0.05	9.631	0.000
j6	STORAGE	5.24	2.249	0.000
j8	STORAGE	4.96	2.099	0.000
j10	STORAGE	4.33	1.501	0.255
j13	STORAGE	2.36	0.805	1.504
j15	STORAGE	4.55	1.909	0.000

****

Table C.182. Node Flooding Summary: Depth and Precipitation Focus Scenario

****

Flooding refers to all water that overflows a node, whether it ponds or not.

Node	Hours Flooded	Maximum Rate CMS	Time of Max Occurrence days hr:min	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
Treatmentplant	1.38	0.099	0 02:45	0.313	6.70
j11	0.19	0.019	0 00:00	0.001	5.96
j9	0.01	0.011	0 00:00	0.000	8.14
j7	0.01	0.006	0 00:00	0.000	9.12
j12	0.01	0.005	0 00:00	0.000	8.11
j14	0.01	0.001	0 00:00	0.000	9.89
j6	2.05	1.108	0 02:59	4.681	4.84
j8	1.29	0.741	0 02:59	2.239	4.69
j15	1.73	0.892	0 02:59	3.804	4.50

****
Table C.183. Storage Volume Summary: Depth and Precipitation Focus Scenario
****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	2.077	43	0	4.840	100	0 01:57	1.203
j8	2.309	49	0	4.690	100	0 02:15	1.187
j10	2.311	53	0	4.092	94	0 03:19	1.029
j13	2.096	43	0	3.396	69	0 03:03	1.699
j15	2.075	46	0	4.500	100	0 01:52	1.135

****
Table C.184. Outfall Loading Summary: Depth and Precipitation Focus Scenario
****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	100.00	0.332	0.899	24.569	30.286	0.181
Outfall4	100.00	0.308	1.461	19.560	718.760	0.180
Outfall6	100.00	0.349	1.125	23.601	1185.633	0.296
Outfall1	100.00	0.397	1.183	27.517	408.570	0.102
Outfall2	100.00	0.364	1.158	24.927	1023.624	0.256
Outfall3	100.00	0.303	0.985	21.243	450.868	0.113
Outfall5	99.97	0.386	1.673	24.972	1109.183	0.277
System	100.00	2.439	8.463	166.389	4926.923	1.405

****
Table C.185. Link Flow Summary: Depth and Precipitation Focus Scenario
****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.023	0 05:03	1.45	1.05	1.00
c6	CONDUIT	1.419	0 02:59	4.62	0.41	0.72
c9	CONDUIT	1.317	0 03:00	4.07	0.53	0.76
c16	CONDUIT	0.899	0 03:01	4.58	24.92	1.00
c5	CONDUIT	0.859	0 02:59	3.12	0.21	0.65
c11	CONDUIT	0.036	0 00:10	0.78	0.64	1.00
c13	CONDUIT	0.037	0 00:10	0.76	0.83	1.00
c14	CONDUIT	0.053	0 00:00	1.04	0.95	1.00
c2	CONDUIT	0.010	0 00:26	0.56	1.07	1.00
c4	CONDUIT	0.014	0 00:38	0.79	1.06	1.00
c8	CONDUIT	1.215	0 02:59	4.19	0.29	0.68
c10	CONDUIT	0.051	0 00:00	1.01	0.87	1.00
c15	CONDUIT	0.578	0 02:59	2.15	0.17	0.64
c12	CONDUIT	0.047	0 09:31	0.96	0.88	1.00
c3	CONDUIT	0.017	0 00:22	1.03	1.07	1.00
c7	CONDUIT	1.318	0 02:59	4.48	0.32	0.69
P2	PUMP	0.031	0 00:00		1.00	
p3	PUMP	0.043	0 00:00		1.00	
P5	PUMP	0.020	0 00:00		1.00	
P1	PUMP	0.010	0 00:00		1.00	
P4	PUMP	0.029	0 00:00		1.00	
R4	WEIR	1.461	0 03:01			1.00
R6	WEIR	1.125	0 01:53			1.00
R1	WEIR	1.183	0 01:58			1.00
R2	WEIR	1.158	0 02:15			1.00
R3	WEIR	0.985	0 03:17			1.00
R5	WEIR	1.673	0 03:03			1.00

****

Table C.186. Flow Classification Summary: Depth and Precipitation Focus Scenario

****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
		--- Fraction of Time in Flow Class ----								
c1	1.00	0.00	0.00	0.00	0.33	0.67	0.00	0.00	1.29	0.0008
c6	1.00	0.00	0.00	0.00	0.74	0.25	0.00	0.00	0.60	0.0003
c9	1.00	0.00	0.00	0.00	0.78	0.22	0.00	0.00	0.48	0.0004
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.50	0.0203
c5	1.00	0.00	0.00	0.00	0.83	0.16	0.00	0.00	0.35	0.0001
c11	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.29	0.0009
c13	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.32	0.0006
c14	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.12	0.0008
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.40	0.0008
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.23	0.0008
c8	1.00	0.00	0.00	0.00	0.77	0.23	0.00	0.00	0.52	0.0002
c10	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.14	0.0009
c15	1.00	0.00	0.00	0.00	0.93	0.06	0.00	0.00	0.23	0.0001
c12	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.33	0.0004
c3	1.00	0.00	0.00	0.00	0.38	0.62	0.00	0.00	0.87	0.0008
c7	1.00	0.00	0.00	0.00	0.75	0.25	0.00	0.00	0.57	0.0002

****

Table C.187. Conduit Surcharge Summary: Depth and Precipitation Focus Scenario

****

Conduit	Both Ends	Upstream	Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
	------------ Hours Full -------------				
c1	4.35	4.35	4.36	6.23	4.35
c16	2.67	2.67	2.67	23.00	2.67
c11	3.59	3.59	3.60	0.01	0.01
c13	5.88	5.88	5.90	0.01	0.01
c14	0.01	0.01	0.02	0.01	0.01
c2	4.35	4.35	4.36	1.15	0.01
c4	3.15	3.15	3.15	0.86	0.01
c10	2.58	2.58	2.59	0.01	0.01
c12	9.40	9.40	9.41	0.01	0.01
c3	4.77	4.77	4.78	6.63	4.77

****
Table C.188. Pumping Summary: Depth and Precipitation Focus Scenario
****

Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Low	% Time Off Curve High
P2	100.00	1	0.00	0.03	0.03	2.530	11.34	0.0	91.5
p3	100.00	1	0.00	0.04	0.04	3.590	12.36	0.0	93.6
P5	45.87	1	0.00	0.02	0.02	0.760	2.25	0.0	99.9
P1	43.59	1	0.00	0.01	0.01	0.374	1.12	0.0	99.9
P4	74.10	2	0.00	0.03	0.03	1.785	5.13	0.0	100.0

Analysis begun on: Sun May 05 12:35:51 2013
Analysis ended on: Sun May 05 12:35:51 2013
Total elapsed time: < 1 sec

C.10. Initial Depth and Impervious Focus Scenario Simulation Status Report

****
Table C.189. Analysis Options: Initial Depth and Impervious Focus Scenario
****

Flow Units .... CMS
Process Models:
Rainfall/Runoff .... YES
Snowmelt .... NO
Groundwater .... NO
Flow Routing .... YES
Ponding Allowed ..... YES
Water Quality .... YES
Infiltration Method .... HORTON
Flow Routing Method ...... DYNWAVE
Starting Date DEC-14-2010 00:00:00
Ending Date DEC-14-2010 23:00:00
Antecedent Dry Days 5.0
Report Time Step 00:05:00
Wet Time Step 00:05:01
Dry Time Step 00:05:01
Routing Time Step .... 30.00 sec

****

Table C.190. Runoff Quantity Continuity: Initial Depth and Impervious Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
Total Precipitation	17.148	86.000
Evaporation Loss	0.000	0.000
Infiltration Loss	0.776	3.891
Surface Runoff	16.362	82.061
Final Surface Storage	0.016	0.082
Continuity Error (%)	-0.039	

****

Table C.191. Runoff Quality Continuity: Initial Depth and Impervious Focus Scenario

****

	TSS kg	Lead kg
Initial Buildup	6111.392	0.000
Surface Buildup	1768.215	1.528
Wet Deposition	0.000	0.000
Sweeping Removal	0.000	0.000
Infiltration Loss	0.000	0.000
BMP Removal	0.000	0.000
Surface Runoff	6111.496	1.528
Remaining Buildup	1768.215	0.000
Continuity Error (%)	-0.001	0.000

****

Table C.192. Flow Routing Continuity: Initial Depth and Impervious Focus Scenario

****

	Volume hectare-m	Volume 10^6 ltr
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	16.362	163.625
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.645	6.449
External Outflow	15.617	156.173
Internal Outflow	2.236	22.362
Storage Losses	0.000	0.000
Initial Stored Volume	1.347	13.472
Final Stored Volume	0.608	6.080
Continuity Error (%)	-0.582	

****

Table C.193. Quality Routing Continuity: Initial Depth and Impervious Focus Scenario

****

	TSS kg	Lead kg
Dry Weather Inflow	0.000	0.000
Wet Weather Inflow	6109.401	1.527
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	0.000	0.000
Internal Flooding	850.378	0.213
External Outflow	4668.206	1.310
Mass Reacted	571.599	0.000
Initial Stored Mass	0.000	0.000
Final Stored Mass	36.950	0.009
Continuity Error (%)	-0.290	-0.290

****

Table C.194. Highest Continuity Errors: Initial Depth and Impervious Focus Scenario

****

Node j14 (-8.33%)
Node j7 (-4.02%)
Node j9 (-1.01%)

****

Table C.195. Time-Step Critical Elements: Initial Depth and Impervious Focus Scenario

****

Link c7 (23.64%)
Link c16 (6.13%)

****

Table C.196. Highest Flow Instability Indexes: Initial Depth and Impervious Focus Scenario

****

Link R5 (23)
Link R3 (2)

****

Table C.197. Routing Time Step Summary: Initial Depth and Impervious Focus Scenario

****

Minimum Time Step	:	14.57 sec
Average Time Step	:	26.74 sec
Maximum Time Step	:	30.00 sec
Percent in Steady State	:	0.00
Average Iterations per Step	:	2.93

****
Table C.198. Subcatchment Runoff Summary: Initial Depth and Impervious Focus Scenario
****

Subcatchment	Total Precip	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff mm	Peak Runoff 10^6 ltr	Runoff Coeff CMS
S1	86.00	0.00	0.00	5.63	80.32	20.89	1.65	0.934
S2	86.00	0.00	0.00	5.23	80.73	17.83	1.44	0.939
S13	86.00	0.00	0.00	1.95	83.92	18.69	1.60	0.976
S11	86.00	0.00	0.00	1.12	84.92	3.59	0.31	0.987
S10	86.00	0.00	0.00	1.13	84.90	3.81	0.33	0.987
S9	86.00	0.00	0.00	1.39	84.60	7.59	0.65	0.984
S8	86.00	0.00	0.00	1.43	84.55	8.27	0.71	0.983
S7	86.00	0.00	0.00	1.65	84.29	12.20	1.05	0.980
S6	86.00	0.00	0.00	3.51	82.51	7.05	0.61	0.959
S3	86.00	0.00	0.00	5.49	80.46	19.82	1.58	0.936
S4	86.00	0.00	0.00	5.87	80.07	22.85	1.78	0.931
S5	86.00	0.00	0.00	4.31	81.68	11.55	0.97	0.950
S12	86.00	0.00	0.00	1.50	84.47	9.49	0.82	0.982

****
Table C.199. Subcatchment Washoff Summary: Initial Depth and Impervious Focus Scenario
****

Subcatchment	TSS kg	Lead kg
S1	193.765	0.048
S2	658.247	0.165
S13	1244.27	0.311
S11	252.091	0.063
S10	267.586	0.067
S9	501.170	0.125
S8	546.426	0.137
S7	323.388	0.081
S6	190.858	0.048
S3	458.710	0.115
S4	531.530	0.133
S5	316.011	0.079
S12	627.442	0.157
System	6111.496	1.528

****

Table C.200. Node Depth Summary: Initial Depth and Impervious Focus Scenario

****

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days	hr:min
j2	JUNCTION	0.09	0.29	17.97	0	03:00
j4	JUNCTION	0.12	0.37	20.48	0	03:00
j1	JUNCTION	0.13	0.42	22.98	0	03:00
j5	JUNCTION	0.07	0.23	22.18	0	03:00
j16	JUNCTION	0.06	0.21	23.21	0	03:00
j3	JUNCTION	0.10	0.31	16.46	0	03:00
Treatmentplant	JUNCTION	1.98	8.39	8.40	0	03:00
j11	JUNCTION	1.45	5.96	7.60	0	00:00
j9	JUNCTION	0.44	8.14	10.53	0	00:00
j7	JUNCTION	0.23	9.12	12.71	0	00:00
j12	JUNCTION	1.36	8.11	10.17	0	00:00
j14	JUNCTION	0.19	9.89	12.32	0	00:00
TreatmentPlantOutfall	OUTFALL	0.33	0.50	0.50	0	00:33
Outfall4	OUTFALL	0.87	1.28	1.28	0	01:59
Outfall6	OUTFALL	1.10	1.51	1.28	0	01:59
Outfall1	OUTFALL	0.51	0.92	1.28	0	01:59
Outfall2	OUTFALL	1.39	1.88	1.33	0	18:59
Outfall3	OUTFALL	1.76	2.25	1.33	0	18:59
Outfall5	OUTFALL	1.80	2.21	1.28	0	01:59
j6	STORAGE	2.01	4.84	5.20	0	01:25
j8	STORAGE	2.36	4.69	4.14	0	01:35
j10	STORAGE	2.43	4.35	3.43	0	02:20
j13	STORAGE	2.24	4.66	3.73	0	03:01
j15	STORAGE	2.02	4.50	4.27	0	01:24

****

Table C.201. Node Inflow Summary: Initial Depth and Impervious Focus Scenario

****

Node	Type	Maximum Lateral Inflow CMS	Maximum Total Inflow CMS	Time of Max Occurrence days	Time of Max Occurrence hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr
j2	JUNCTION	1.440	1.462	0	03:00	17.834	18.57
j4	JUNCTION	1.782	1.795	0	03:00	22.851	23.136
j1	JUNCTION	1.651	1.651	0	03:00	20.891	20.891
j5	JUNCTION	0.975	0.975	0	03:00	11.551	11.550
j16	JUNCTION	0.610	0.610	0	03:00	7.047	7.046
j3	JUNCTION	1.577	1.603	0	03:00	19.817	20.636
Treatmentplant	JUNCTION	0.982	2.783	0	03:00	11.404	41.712
j11	JUNCTION	0.000	0.081	0	03:01	0.000	3.674
j9	JUNCTION	0.000	0.029	0	00:00	0.000	1.674
j7	JUNCTION	0.000	0.024	0	01:30	0.000	0.629
j12	JUNCTION	0.000	0.031	0	00:00	0.000	2.518
j14	JUNCTION	0.000	0.020	0	01:25	0.000	0.294
TreatmentPlantOutfall	OUTFALL	0.000	1.013	0	03:00	0.000	22.699
Outfall4	OUTFALL	0.000	1.673	0	03:00	0.000	21.059
Outfall6	OUTFALL	0.000	1.116	0	03:37	0.000	21.637
Outfall1	OUTFALL	0.000	1.183	0	01:25	0.000	25.307
Outfall2	OUTFALL	0.000	1.158	0	01:35	0.000	23.569
Outfall3	OUTFALL	0.000	1.066	0	02:20	0.000	21.385
Outfall5	OUTFALL	0.000	1.984	0	03:02	0.000	26.964
j6	STORAGE	1.050	2.678	0	03:00	12.199	34.937
j8	STORAGE	0.713	2.185	0	03:00	8.271	30.595
j10	STORAGE	0.000	1.632	0	03:00	0.000	26.378
j13	STORAGE	1.126	2.099	0	03:00	13.081	29.161
j15	STORAGE	1.598	2.208	0	03:00	18.690	28.828

****
Table C.202. Node Surcharge Summary: Initial Depth and Impervious Focus Scenario
****
Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
Treatmentplant	JUNCTION	3.52	5.798	0.000
j11	JUNCTION	6.72	5.703	0.000
j9	JUNCTION	4.04	7.889	0.000
j7	JUNCTION	3.05	8.871	0.000
j12	JUNCTION	5.11	7.851	0.000
j14	JUNCTION	2.29	9.631	0.000
j6	STORAGE	5.01	2.249	0.000
j8	STORAGE	4.77	2.099	0.000
j10	STORAGE	4.53	1.756	0.000
j13	STORAGE	3.61	2.064	0.245
j15	STORAGE	4.48	1.909	0.000

****
Table C.203. Node Flooding Summary: Initial Depth and Impervious Focus Scenario
****
Flooding refers to all water that overflows a node, whether it ponds or not.

Node	Hours Flooded	Maximum Rate CMS	Time of Max Occurrence days hr:min	Total Flood Volume 10^6 ltr	Maximum Ponded Depth Meters
Treatmentplant	2.49	0.140	0 01:29	0.699	8.39
j11	1.65	0.081	0 03:01	0.273	5.96
j9	0.01	0.011	0 00:00	0.000	8.14
j7	0.01	0.006	0 00:00	0.000	9.12
j12	0.01	0.005	0 00:00	0.000	8.11
j14	0.01	0.001	0 00:00	0.000	9.89
j6	2.48	1.474	0 03:00	8.866	4.84
j8	1.93	0.998	0 03:00	5.118	4.69
j10	1.07	0.548	0 03:00	1.477	4.35
j15	2.14	1.104	0 03:00	6.626	4.50

****

Table C.204. Storage Volume Summary: Initial Depth and Impervious Focus Scenario

****

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	E&I Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow CMS
j6	2.010	42	0	4.840	100	0 01:25	1.203
j8	2.357	50	0	4.690	100	0 01:35	1.187
j10	2.434	56	0	4.347	100	0 02:20	1.110
j13	2.238	46	0	4.655	95	0 03:01	2.008
j15	2.022	45	0	4.500	100	0 01:24	1.126

****

Table C.205. Outfall Loading Summary: Initial Depth and Impervious Focus Scenario

****

Outfall Node	Flow Freq. Pcnt.	Avg. Flow CMS	Max. Flow CMS	Total Volume 10^6 ltr	Total TSS kg	Total Lead kg
TreatmentPlantOutfall	100.00	0.336	1.013	22.699	34.931	0.152
Outfall4	100.00	0.373	1.673	21.059	774.330	0.194
Outfall6	100.00	0.346	1.116	21.637	1025.849	0.256
Outfall1	100.00	0.395	1.183	25.307	346.655	0.087
Outfall2	100.00	0.372	1.158	23.569	908.229	0.227
Outfall3	100.00	0.331	1.066	21.385	425.167	0.106
Outfall5	100.00	0.465	1.984	26.964	1152.315	0.288
System	100.00	2.618	9.144	162.621	4667.476	1.310

****

Table C.206. Link Flow Summary: Initial Depth and Impervious Focus Scenario

****

Link	Type	Maximum |Flow| CMS	Time of Max Occurrence days hr:min	Maximum |Veloc| m/sec	Max/ Full Flow	Max/ Full Depth
c1	CONDUIT	0.023	0 04:36	1.46	1.05	1.00
c6	CONDUIT	1.777	0 03:00	5.49	0.51	0.76
c9	CONDUIT	1.627	0 03:00	4.80	0.66	0.80
c16	CONDUIT	1.013	0 03:00	5.16	28.10	1.00
c5	CONDUIT	0.962	0 03:00	3.44	0.23	0.66
c11	CONDUIT	0.036	0 00:10	0.78	0.64	1.00
c13	CONDUIT	0.035	0 00:10	0.75	0.78	1.00
c14	CONDUIT	0.053	0 00:00	1.04	0.95	1.00
c2	CONDUIT	0.010	0 00:21	0.57	1.07	1.00
c4	CONDUIT	0.014	0 00:24	0.79	1.06	1.00
c8	CONDUIT	1.452	0 03:00	4.87	0.35	0.70
c10	CONDUIT	0.051	0 00:00	1.01	0.87	1.00
c15	CONDUIT	0.610	0 03:00	2.25	0.18	0.65
c12	CONDUIT	0.047	0 06:49	0.96	0.88	1.00
c3	CONDUIT	0.017	0 00:18	1.04	1.08	1.00
c7	CONDUIT	1.603	0 03:00	5.27	0.38	0.72
P2	PUMP	0.031	0 00:00		1.00	
p3	PUMP	0.043	0 00:00		1.00	
P5	PUMP	0.020	0 00:00		1.00	
P1	PUMP	0.010	0 00:00		1.00	
P4	PUMP	0.029	0 00:00		1.00	
R4	WEIR	1.673	0 03:00			1.00
R6	WEIR	1.116	0 03:37			1.00
R1	WEIR	1.183	0 01:25			1.00
R2	WEIR	1.158	0 01:35			1.00
R3	WEIR	1.066	0 02:20			1.00
R5	WEIR	1.984	0 03:02			1.00

****
Table C.207. Flow Classification Summary: Initial Depth and Impervious Focus Scenario
****

Conduit	Adjusted /Actual Length	Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Avg. Froude Number	Avg. Flow Change
		--- Fraction of Time in Flow Class ----								
c1	1.00	0.00	0.00	0.00	0.31	0.69	0.00	0.00	1.29	0.0008
c6	1.00	0.00	0.00	0.00	0.73	0.27	0.00	0.00	0.66	0.0003
c9	1.00	0.00	0.00	0.00	0.75	0.25	0.00	0.00	0.52	0.0004
c16	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.44	0.0209
c5	1.00	0.00	0.00	0.00	0.79	0.21	0.00	0.00	0.39	0.0001
c11	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.25	0.0009
c13	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.33	0.0006
c14	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.06	0.0011
c2	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.38	0.0007
c4	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.19	0.0007
c8	1.00	0.00	0.00	0.00	0.74	0.25	0.00	0.00	0.57	0.0002
c10	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.10	0.0010
c15	1.00	0.00	0.00	0.00	0.87	0.13	0.00	0.00	0.25	0.0001
c12	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.38	0.0013
c3	1.00	0.00	0.00	0.00	0.34	0.66	0.00	0.00	0.88	0.0007
c7	1.00	0.00	0.00	0.00	0.73	0.26	0.00	0.00	0.62	0.0002

****
Table C.208. Conduit Surcharge Summary: Initial Depth and Impervious Focus Scenario
****

Conduit	Hours Full Both Ends	Hours Full Upstream	Hours Full Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
c1	4.10	4.10	4.11	5.26	4.10
c16	3.04	3.04	3.06	23.00	3.04
c11	4.04	4.04	4.05	0.01	0.01
c13	5.11	5.11	5.13	0.01	0.01
c14	2.23	2.23	2.25	0.01	0.01
c2	4.10	4.10	4.11	0.69	0.01
c4	3.38	3.38	3.39	0.45	0.01
c10	3.00	3.00	3.01	0.01	0.01
c12	6.72	6.72	6.73	0.01	0.01
c3	4.38	4.38	4.39	5.56	4.38

****
Table C.209. Pumping Summary: Initial Depth and Impervious Focus Scenario
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Pump	Percent Utilized	Number of Start-Ups	Min Flow CMS	Avg Flow CMS	Max Flow CMS	Total Volume 10^6 ltr	Power Usage Kw-hr	% Time Off Pump Low	% Time Off Curve High
P2	100.00	1	0.00	0.03	0.03	2.518	12.52	0.0	88.8
P3	100.00	1	0.00	0.04	0.04	3.588	12.64	0.0	93.5
P5	37.98	1	0.00	0.02	0.02	0.629	1.73	0.0	99.9
P1	33.81	1	0.00	0.01	0.01	0.290	0.71	0.0	99.9
P4	69.46	2	0.00	0.03	0.03	1.674	4.80	0.0	100.0

Analysis begun on: Sun May 05 16:13:52 2013
Analysis ended on: Sun May 05 16:13:53 2013
Total elapsed time: 00:00:01