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Prepared for:
City of Nashua, New Hampshire

Long-Term Water Quality and Infrastructure Control Plan

Report on Baseline Conditions Update and Development and Evaluation of Alternatives to the City's Current CSO Control Plan

January 2003

M&E
Metcalf & Eddy

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City of Nashua

Public Works Division
165 Ledge Street
Nashua, NH 03060

Public Works
Division Director
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January 3, 2002

Ms. Joy Hilton
United States Environmental Protection Agency
Region 1
1 Congress Street
Suite 1100
Boston, MA 02114-2023

Mr. George Berlandi
New Hampshire Department of Environmental Services
6 Hazen Drive
Concord, NH 03302

Dear Ms. Hilton and Mr. Berlandi:

The city of Nashua, New Hampshire is pleased to submit its *Draft Report on Baseline Conditions Update and Development and Evaluation of Alternatives to the City's Current CSO Control Plan* to the United States Environmental Protection Agency (EPA), Region 1 and the New Hampshire Department of Environmental Services (DES) for review and comment. The report presents a comprehensive discussion of baseline conditions, the development and evaluation of alternatives for combined sewer overflow (CSO) control, and a comparison to the current recommended plan (CRP) of complete sewer separation, mandated by Administrative Order (A.O.) No. 99-09.

In order to comply with the A.O., the city has begun to separate combined sewers in parts of the city. Based on the work completed, the city has estimated the cost for separation to be approximately $2.3 million per mile. Based on this unit cost, the total cost to separation the 100 miles of combined sewers in Nashua will exceed $200 million. Furthermore, sewer separation significantly increases the volume of stormwater runoff, which carries with it a significant load of constituents that degrade receiving water quality. The significant cost of sewer separation, coupled with limited water quality benefits, supported a re-evaluation of the CRP. This report presents the findings of the re-evaluation and recommends a new plan for CSO control that is consistent with federal and state CSO policies.

The plan presented in this report maximizes the use of existing infrastructure to retain combined sewage in the collection system for conveyance to the Nashua Wastewater Treatment Facility (NWTF) during storm events. Flows that exceed the capacity of the NWTF will be treated just upstream of the NWTF. Excess wet weather flow will also be treated just downstream of the East Hollis Street and Nashua River CSOs. In addition, small storage basins will be provided to store excess wet weather flow at the Farmington Road and Burke Street CSOs. This plan will achieve an extremely high level of CSO control – zero overflows in a typical rainfall year.

The plan presented in this report offers several advantages over the CRP:

- High level of CSO control at a fraction of the cost
- No increase in pollutant-laden stormwater runoff
- Greater net benefit to receiving waters
- Shorter implementation period, so benefits to receiving waters can be realized sooner

In addition, Nashua remains committed to improvement of aging infrastructure throughout the city, and is committed to continuing to invest in stormwater control measures. These measures will augment the benefits to be attained by the CSO control plan presented in this report, further improving water quality in the Merrimack and Nashua Rivers.

It is also important to note that the CRP will not result in the elimination of CSOs in Nashua. The findings of this study show that even with complete sewer separation, storm induced inflow would cause widespread flooding of homes and businesses, as well as overflows from manholes along the river banks if the CSOs were sealed.

We appreciate the assistance of the regulatory agencies in developing this project. We look forward to a meeting at your earliest convenience initiate the necessary actions to halt the implementation of the CRP and immediately begin implementation of the plan presented in this report.

In the meantime, if you have any questions or required additional information, please contact me or Mr. Gregory R. Heath of Metcalf & Eddy.

Sincerely,

George Crombie RE

George Crombie
Director, Division of Public Works

cc. M. Wagner, EPA
G. Heath, Metcalf & Eddy
Mayor Bernard A. Streeter

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CHAPTER 1

INTRODUCTION

Like many older municipalities throughout New England and beyond, the city of Nashua, New Hampshire is served by a system of combined sewers. These sewers were originally designed to convey both sanitary and drainage flows directly into receiving waters for disposal. This practice resulted in pollution of the receiving waters. Later, steps were taken to capture dry weather sanitary flows from these combined sewers in intercepting sewers, for conveyance to wastewater treatment facilities. Meanwhile excess wet weather flows continued to discharge to the receiving waters, untreated. Referred to as combined sewer overflows (CSOs), these wet weather discharges to the Merrimack and Nashua Rivers have continued. The purpose of this study was to reevaluate the current approach for CSO control, which is complete sewer separation, to determine if another, better approach exists – an approach that would provide a similar or higher level of wet weather pollution (CSO plus stormwater) control at lower cost.

BACKGROUND

Incorporated in 1853, the city of Nashua is located along the western bank of the Merrimack River, where it meets the Nashua River, and is the second largest city in the state of New Hampshire (Figure 1-1). The city encompasses an area of 14,834 acres and has an estimated population of 86,605 (U.S. Census, 2000). The downtown district, located along both banks of the Nashua River, near the confluence with the Merrimack River, is more densely populated than the outlying residential areas. Most of the downtown district is relatively flat, though topography does slope upward toward the north, south, and west of the city. The three main watersheds in Nashua are the Merrimack River, the Nashua River, and Salmon Brook. All runoff from within the city of Nashua enters one of these three basins.

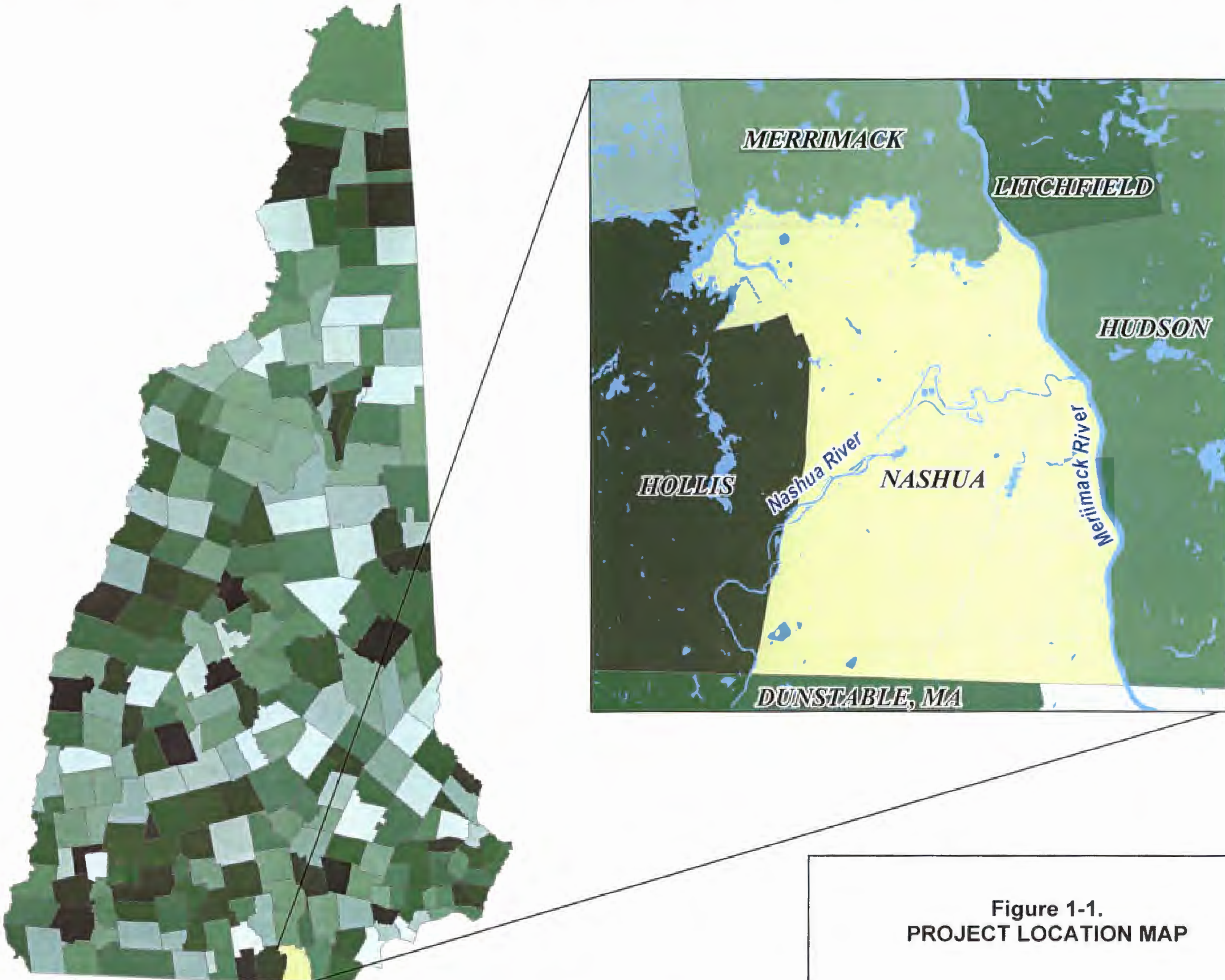

Figure 1-1.
PROJECT LOCATION MAP

g:\projects\newhampshire\nashua\project location map.mxd

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The existing wastewater collection system serves approximately 96 percent of the population, and approximately 78 percent of the total land area. A major portion of the system is composed of combined sewers, primarily in the older portions of the city. Separate sanitary and storm sewers have been constructed to serve the newer, outlying areas of the city. These separate, sanitary sewers discharge to the combined sewer system located downstream.

Role of the Department of Public Works

The Nashua Department of Public Works is responsible for operating and maintaining the Nashua Wastewater Treatment Facility (NWTF), as well as the collection system and associated pump stations. The collection and treatment system include 12 pump stations, 330 miles of sewer pipe, and a secondary wastewater treatment facility capable of treating a peak flow of 50 mgd.

Prior Efforts to Control CSOs

In November 1995, the city of Nashua retained the consulting engineering firm of Camp Dresser & McKee, Inc. (CDM) to prepare a Draft Combined Sewer (CSO) Facilities Plan/Long Term Control Plan, to evaluate the CSO abatement alternatives for various levels of control, and to recommend an implementation plan. The recommended plan, presented in the *Long-Term CSO Control Plan, September 1997*, included sewer separation, regulator modifications, system optimization, and a CSO storage facility.

On April 20, 1999, the United States Environmental Protection Agency, Region 1 (EPA) issued Nashua, New Hampshire an Administrative Order (A.O. No. 99-09) which requires the City to separate its combined sewer system by December 31, 2019, in order to mitigate its CSO problem and related water quality impacts as noted in the *Long Term CSO Control Plan*. As a provision of the Order, the City must submit to the EPA and to the New Hampshire Department of Environmental Services (DES), sewer separation progress reports and monthly progress projections by January 15 and July 15 each year.

The Order also requires the City to provide secondary treatment and disinfection to the maximum treatable flows during wet weather, and it includes wet weather related monitoring requirements.

In order to comply with the Order, the City has begun to separate the combined sewers in parts of the city. Based on the work completed on five past projects, the city has estimated the cost for separation to be approximately $2.3 million per mile. This unit cost includes the costs associated with the sewers and appurtenant structures, as well as the costs to reconstruct curbs and sidewalks, repave streets, and re-align other adjacent utilities disrupted during the work. Based on this unit cost, the total cost to separate the 110 miles of combined sewers in Nashua would exceed $250 million in today's dollars.

The EPA and NHDES have approved the current plan based on information they were provided at the time as the means to eliminate the city's CSOs. However, there are a number of issues that support a reconsideration of this plan.

Sewer separation is extremely disruptive to homes and businesses. The $250 million cost estimate does not reflect the economic impacts to the community caused by having the streets torn up for a significant amount of time, nor does it address the logistical problems that can arise when major thoroughfares are blocked.

Preliminary assessments suggest that even after complete sewer separation, it will not be feasible to eliminate all the CSO discharges. Experience from separation projects in other large communities indicates that often it is not possible to remove enough stormwater from the combined sewer system to allow the regulators to be blocked off without creating an unacceptable risk of flooding to homes and businesses during very large storms. Therefore, the premise that sewer separation would eliminate all CSOs does not appear to be valid.

Finally, sewer separation would create a new source of pollution to the receiving waters. The sewer separation project currently proposed for Nashua would reduce the discharge volume and activation frequency at the nine permitted CSOs by reducing the volume of stormwater runoff entering the combined sewer system. With the existing combined sewer system in Nashua, urban stormwater, which contains a variety of pollutants that can degrade water quality and cause water quality standards to be violated, is captured in the combined sewers during the smaller storm events, and receives secondary treatment and disinfection at Nashua's wastewater treatment facility. Only during larger, less frequently occurring storms is the capacity of the combined sewer system exceeded, resulting in CSO discharges. However, with complete sewer separation, all urban stormwater runoff would be discharged directly to the Nashua and Merrimack Rivers, untreated. Untreated stormwater would be discharged to the rivers during nearly every rainstorm, with the degree of water quality impact and water quality standards violations depending on the size of the storm.

Under baseline conditions, the NWTF treats approximately 312 mgal of stormwater on an annual basis. Under a completely separated scenario approximately 250 mgal of this water would be released directly to the receiving waters, without secondary treatment and disinfection. Preliminary assessments indicate that over the 20-year period of the Administrative Order, the reductions in bacterial pollution from CSOs to the Nashua and Merrimack Rivers, which would be achieved by sewer separation, would nearly be offset by the increase in bacteria loads associated with the increase in stormwater discharge resulting from the sewer separation project. In the end, after 20 years and over $250 million in investment, the total annual bacteria load to the Nashua and Merrimack Rivers would be reduced by approximately 10 percent. Furthermore, the benefits in terms of CSO reduction would be realized gradually, as the separation work proceeded through the city on the 20-year schedule.

RECOMMENDED PLAN

The recommended CSO control plan developed and presented in this report provides several advantages over the sewer separation plan currently required under the Administrative Order. These advantages include:

- The recommended plan will result in greater water quality benefits, specifically less bacterial pollution of the Nashua and Merrimack Rivers, as compared to the CRP. With complete sewer separation, stormwater volume, and hence, stormwater pollutant loading to the Nashua and Merrimack Rivers would increase. The recommended plan will reduce annual bacterial loading to the rivers from CSOs without increasing stormwater discharges. Furthermore, the recommended plan will result in better receiving water quality as compared to the CRP, based on the aerial extent, magnitude, and duration of predicted violations of water quality standards.

- The recommended plan will achieve environmental benefits much more quickly than the CRP. The recommended plan can be implemented in less than half the time it would take to separate all of the sewers in Nashua.

- The recommended plan offers flexibility to achieve higher degrees of CSO control in the future, to control untreated overflows during extreme storm events, if this ever is deemed necessary. With a program based solely on complete sewer separation, it will not be feasible to control CSO discharges during extreme storm events. If the existing CSO outfalls were sealed following complete sewer separation, flooding of homes and businesses and the overtopping of manholes along the receiving waters would result during these extreme events.

- The recommended plan maximizes the use of existing infrastructure by maximizing system storage through the implementation of system optimization measures. This is consistent with State and National CSO control policies.

- The recommended plan is much more cost effective in reducing pollution and can be implemented at significantly lower capital cost than complete sewer separation. The estimated cost of the recommended plan is $38 million, compared to over $250 million for the CRP.

These benefits of the recommended plan as compared to the CRP are discussed in more detail in the chapters that follow.

REPORT ORGANIZATION

This report summarizes the findings of the reevaluation and has been prepared to facilitate a change to the current Administrative Order. The document is organized as follows:

- **Chapter 1 – Introduction**

- **Chapter 2 – Collection and Treatment System Description.** This chapter presents an overview of the interceptor network serving the combined sewer areas of Nashua, summarizes the key features and available capacity of the NWTF and provides details of the regulators at nine CSOs.

- **Chapter 3 – Dry Weather Collection System Flows**. This chapter describes the flow metering program conducted during 2001, and the dry weather flows measured. Specifically, this chapter discusses infiltration and sanitary components of the dry weather flow, and diurnal flow variation.

- **Chapter 4 – Wet Weather Collection System Flows.** This chapter describes the wet weather flow and CSO/stormwater quality sampling program, including: rainfall monitoring, rainfall data analysis, and CSO flow monitoring.

- **Chapter 5 – Collection System Model Development.** This chapter describes the development of the collection system model, as well as dry weather and wet weather model calibration.

- **Chapter 6 – Development of Baseline Flows and Loads.** This chapter discusses development of the typical year rainfall record and design storms, along with the expected performance of the combined sewer system under these precipitation events. Results of the model output are combined with the water quality data, presented in Chapter 4, to give estimates of baseline bacterial loading.

- **Chapter 7 – Receiving Water Modeling and Baseline Water Quality Impacts.** This chapter discusses development and calibration of the receiving water model. Baseline water quality, based on the annual loading presented in chapter 6 is also discussed in this chapter.

- **Chapter 8 – System Optimization Measures.** This chapter presents the methodology for developing and evaluating system optimization measures (SOMs). The recommended site-specific system optimization measures are also presented.

- **Chapter 9 – Methodology for Developing CSO Control Alternatives.** This chapter presents the methodology for developing CSO control alternatives, including a discussion of how project costs and non-monetary factors were developed and used to compare alternatives.

- **Chapter 10 – Development and Evaluation of CSO Alternatives.** This chapter discusses and evaluates various CSO control technologies, to determine which were appropriate for consideration as components of the recommended CSO control plan. Evaluation was based on cost, performance, and non-monetary factors.

- **Chapter 11 – Recommended Plan.** This chapter presents the recommended plan and compares it to the Current Recommended Plan (CRP) of complete sewer separation.

- Appendices

CHAPTER 2

COLLECTION AND TREATMENT SYSTEM DESCRIPTION

This chapter describes the interceptor network serving the combined sewer areas of Nashua, including physical system features and system performance. It also summarizes the operation and hydraulic capabilities of the Nashua Wastewater Treatment Facility (NWTF), which receives flow from areas served by both separate and combined sewers.

COLLECTION SYSTEM

Nashua's wastewater collection system serves approximately 96 percent of the city's population, inhabiting 78 percent (11,600 acres) of the total land area within the city (14,836 acres). The remaining 4 percent of the population is served by subsurface disposal systems.

Approximately 25 percent of the collection system service area (2,900 acres) is served by combined sewers. Most of this combined sewer service area is located in the central portion of the city, along the Nashua and Merrimack Rivers. This is the older, more densely developed section of Nashua. Areas of the city that developed later, to the north, west, and south of the city, are served by separate sanitary sewers. Figure 2-1 shows the extent of these two service areas.

Intercepting Sewer System

Local sanitary and combined sewage flow collected in service lines and trunk sewers is delivered to the NWTF through a series of intercepting sewers. In the outer portions of the city, only sanitary flow is conveyed to these interceptors while in the older, downtown district, serviced by combined sewer pipes, both sanitary flow and stormwater runoff are conveyed to the intercepting sewer pipes. Pertinent characteristics of the major interceptors serving combined sewer areas are presented in Table 2-1.

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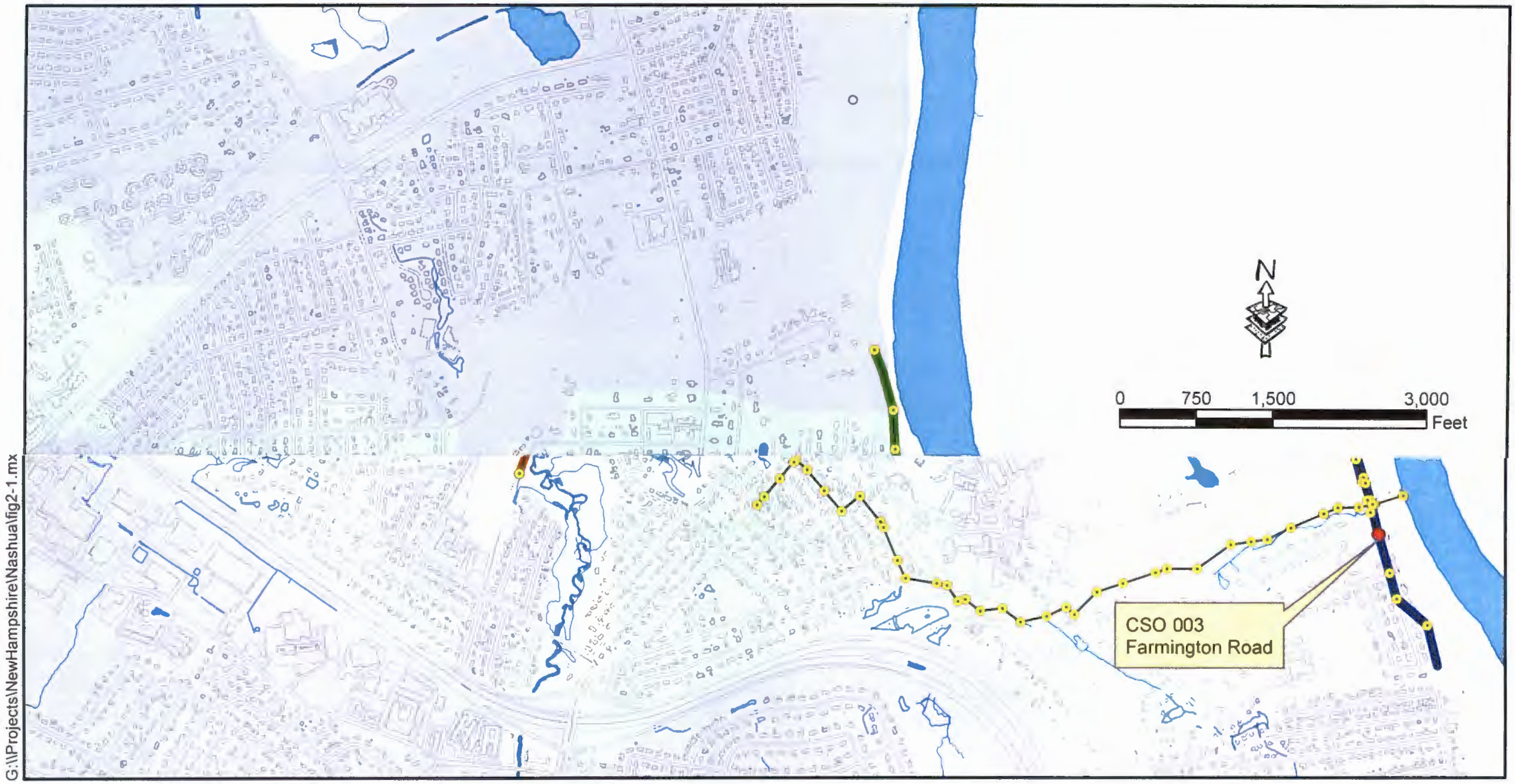

Legend

- Modeled Manholes
- Modeled Pipe Segments
- Nashua River Interceptor (NRI)
- North Merrimack River Interceptor (NMRI)
- South Merrimack Interceptor (SMI)
- Salmon Brook Interceptor (SBI)
- NWTF
- CSO Location
- Rivers, Streams, & Ponds
- Combined Model Areas
- Separate Model Areas

Data Sources:
Data provided by City of Nashua, New Hampshire - 2001

Coordinate System:
Projection: Stateplane
Datum: NAD83
Fipszone: 2800
Units: Feet

Figure 2-1.
Separate and Combined Sewer Service Areas with Major Sewer Interceptors

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TABLE 2-1. SUMMARY OF MAJOR COMBINED SEWER INTERCEPTORS

Interceptor	Date	Length of pipe (ft.)	Range of diameters (in.)
North Merrimack River Interceptor	1975	20,000	36-108
Nashua River Interceptor	1975	21,000	18-108
Salmon Brook Interceptor	1960	13,000	48
South Merrimack Interceptor*	1962/1992	17,000/2,700	16-30/36

*In 1992 a 36-inch diameter pipe was added to the SMI system. The two pipes run parallel to one another and are described in more detail below.

North Merrimack River Interceptor (NMRI) – The North Merrimack River Interceptor has two sections, the NMRI-1 and NMRI-2. NMRI-1 originates at the Nashua River CSO structure (006) as a 108-inch diameter RCP and ends as a 72-inch diameter pipe at the NWTF. NMRI-1 collects flow from the NRI-1, the NMRI-2, as well as flow diverted through the dry-weather connections at the Burke Street CSO (004) and the East Hollis Street CSO (005).

NMRI-2 originates in north Nashua near the border with Merrimack as a 30-inch diameter pipe, and ends at the entrance to the Nashua River CSO structure (006), as a triple barreled inverted siphon (18, 48, and 60-inch diameter pipes). The NMRI-2 also receives dry weather flow from the Lock Street regulator (009).

Nashua River Interceptor (NRI) – The Nashua River Interceptor is also comprised of two separate sections. NRI-1 originates as an 84-inch diameter pipe extending from a siphon structure under the Nashua River, connecting NRI-2 to NRI-1 and ends as a 108-inch pipe at the Nashua River CSO (006). NRI-1 collects flow from NRI-2, as well as dry weather flow from the Jackson/Beaucher CSO structure (012).

NRI-2 originates as a 48-inch diameter pipe at the convergence of three sanitary interceptors near Route 3 and ends as a 54-inch diamter pipe, leading to an inverted siphon under the Nashua River. In addition to flow from the three upstream sanitary interceptors, the NRI-2 collects dry weather flow from the Tampa Street CSO regulator (007) and the Broad Street CSO regulator (008).

Salmon Brook Interceptor (SBI) - The Salmon Brook Interceptor originates as a 30-inch diameter RCP at the F.E. Everett Highway and ends as a 48-inch diameter RCP at the NWTF. The SBI captures flow from the Hassells Brook Interceptor. There is a interconnection at Allds Street that allows flow in SBI to overflow into the Burke Street CSO (004) system.

South Merrimack Interceptor (SMI) – The original South Merrimack Interceptor (SMI-1) originates as an 18-inch diameter RCP at Spit Brook Road and ends as a 36-inch RCP at the NWTF. SMI-1 receives flow diverted through the dry-weather connection at the Farmington Road CSO (003).

The SMI-2, sometimes referred to as the South Merrimack Relief Interceptor was constructed in 1992 to provide additional hydraulic capacity for the south system. The SMI-2 originates as a 36-inch diameter pipe, upstream of the Farmington Road CSO structure (003) and receives flow from the SMI-1. The SMI-2 ties back into the SMI-1 just upstream of the NWTF and only one 36-inch diameter sewer enters the NWTF from the south. Though dry weather flow from the Farmington Road CSO structure (003) enters the SMI-1, a connection between the SMI-1 and SMI-2 exists just downstream of the dry weather connection allowing for flow from Farmington Road to be conveyed to the NWTF through both pipes.

Combined Sewer Overflows (CSOs)

Sanitary flow from residential, commercial, and industrial sources, as well as stormwater runoff are conveyed to the major sewer interceptors through a series of pipes, that grow in size until flow reaches the interceptor. Combined sewer overflow structures are located at many of these junctions and are designed to regulate the portion of flow that enters the interceptor from the local combined sewer system. During dry weather conditions, all of the flow from the tributary system is conveyed to the interceptor. However, during wet weather conditions, the portion of flow not conveyed to the interceptor overflows to a receiving water body.

Since flow to these structures can originate from both sanitary sources and runoff sources, the tributary area to CSO regulators can be separated into two components – a sewershed and a watershed.

Sewershed. The sewershed of a CSO regulator is defined by the extent of the pipe network that is tributary the regulator. In some cases the network of pipes that transports flow to a particular CSO structure can be quite large, encompassing many buildings and many acres of land. Pipes in these large sewersheds start as small-diameter pipes in the outlying areas and eventually build to large-diameter pipes that enter the CSO regulators. In older communities like Nashua, there are often interconnections between adjacent sewersheds. These interconnections are high points in the pipe network where flow in one direction leads to one CSO regulator, while flow in the other leads to a different CSO regulator. In some cases backwater from one sewershed can build up and overflow to an adjacent sewershed.

Watershed. The watershed of a CSO regulator is defined by topography and the extent of pipes. While the sewershed ends at the most upstream service point in the system, the watershed extends to the furthest point of land that slopes to the most upstream catch basins. The watershed of a CSO regulator includes the entire area in which runoff from wet weather events can get into the network of pipes tributary to a given CSO regulator. The watershed is usually larger than the sewershed. Together, these areas are known as the tributary area to a CSO regulator.

There are currently nine combined sewer overflows (CSOs), each of which has been assigned a NPDES discharge number. These CSOs are shown in Figure 2-2 and are described in Table 2-2.

TABLE 2-2. SUMMARY OF PERMITTED NASHUA CSOs

NPDES Discharge Number	CSO Name	Interceptor Sub-system	Regulator Type	Overflow Size (in.)	Receiving River
002	Salmon Brook	SBI	Diversion Weir	60	Merrimack
003	Farmington Road	SMI-1	Diversion Weir	36	Merrimack
004	Burke Street	NMRI	Diversion Weir	21	Merrimack
005	E. Hollis St.	NMRI	Diversion Weir	54	Merrimack
006	Nashua River	NMRI	High Outlet	120 x 72	Nashua
007	Tampa Street	NRI	Diversion Weir	48	Nashua
008	Broad Street	NRI	Diversion Weir	30	Nashua
009	Lock Street	NMRI	Diversion Weir	36	Nashua
012*	Jackson/Beaucher	NRI	Diversion Weir	(2) 36 x 60	Nashua

* Jackson/Beaucher overflow is bolted shut and cannot activate. As a result, this CSO regulator was not evaluated in this reassessment.

Salmon Brook Overflow (CSO 002) – Located on the Salmon Brook interceptor, this overflow is located on the NWTF property, upstream from where the SBI and SMI join and enter the NWTF. Flow from the SBI enters the east chamber of the diversion structure through a 48-inch diameter pipe. Dry-weather flow is diverted through one of two 16-inch connections into the west chamber and leaves the west chamber through a 24-inch diameter pipe. This 24-inch diameter pipe then combines with the 36-inch SBI and enters the headworks of the NWTF.

Overflows to the Merrimack River occur when the water surface elevation (WSEL.) in the east chamber reaches el. 23.37, Nashua City Datum (NCD). The overflow pipe from the Salmon Brook CSO regulator is a 60-inch diameter pipe that combines with the 60-inch NWTF outfall. Flow from the NWTF and Salmon Brook Overflow are discharged to the Merrimack River from the same pipe.

Farmington Road Overflow (CSO 003) - The Farmington Road overflow is a conventional diversion weir regulator. Combined flow enters the structure through a 36-inch RCP inlet and dry weather flow is diverted to the SMI through a 10-inch connection that increases to a 15-inch connection before dropping into the SMI. The overflow weir crest is at el. 20.31 (NCD) and flows that top the weir are discharged to the Merrimack River through a 36-inch diameter pipe.

An abandoned sluice gate, shown in Figure 2-3 can control dry weather flow entering the SMI. However, field verification of this structure shows it to be stuck in the open position.

Burke Street Overflow (CSO 004) - Flow enters the Burke Street regulator through a 24-inch diameter pipe. Dry-weather flow is diverted to the interceptor through a 10-inch pipe to the NMRI. Sewage that overflows the weir at el. 11.19 (NCD) discharges to the Merrimack River through a 21-inch diameter pipe. The weir wall and overflow pipe in the Burke Street is shown in Figure 2-4.

East Hollis Street Overflow (CSO 005) - This overflow structure is located at the end of East Hollis Street near the Bridge Street ramp. Flow enters the CSO structure through a 54-inch diameter pipe. Dry weather flow is diverted to the NMRI through a 24-inch diameter pipe, controlled by a sluice gate in the CSO regulator structure. Based on discussions with the NWTF staff, this sluice gate is in its full-open position and not regularly changed. Combined sewage flow that tops the weir crest at el. 8.60 (NCD), is discharged through a 54-inch overflow pipe.

The 54-inch outfall passes through the nearby Nashua pumping station before discharging to the Merrimack River. At the pump station there are two methods by which the overflow can be discharged into the river:

1. When the Merrimack River is below an elevation of 11.7, the overflow is bypassed around the pump station and channeled to the river.
2. When the river is above elevation 11.7, the bypass is closed to prevent the river from backing into the sewer system, and the overflow is pumped into the river by means of a 12-inch volute pump. If the pump cannot handle the flow, an adjacent wet well fills and is pumped out by two 30-inch propeller pumps.

An emergency overflow swale is located in a field adjacent to the pump station in the event the pumps cannot keep up with the flow. Combined sewage captured by this swale is returned to the pump station after the storm, and discharged to the river.

Figure 2-5 shows the overflow weir and 54-inch overflow pipe and Figure 2-6 shows the sluice gate on the 24-inch dry weather connection.

Figure 2-3. Sluice gate at Farmington Road CSO Regulator (003)

Figure 2-4. Weir and overflow pipe at Burke Street CSO Regulator (004)

Figure 2-5. Sluice gate at East Hollis Street CSO Structure (005)

Figure 2-6. Weir and overflow pipe at East Hollis Street CSO Regulator (005)

Nashua River Overflow (CSO 006) – The Nashua River Overflow is designed to provide hydraulic relief to the NRI and NMRI. The structure is located on the south side of the Nashua River, near the confluence with the Merrimack River; at the point where the NRI and NMRI combine. When the water surface in this CSO structure rises to el. 9.03 (NCD), sewage overflows from this structure and is discharged to the Nashua River. Since overflows from this structure can be sizable and quite turbulent, energy dissipaters have been installed on the spillway. Features of the Nashua River CSO structure are shown in Figures 2-7 and 2-8.

Tampa Street Overflow (CSO 007) - The Tampa Street regulator controls flow to the NRI. Flow enters the regulator through a 48-inch diameter pipe and dry weather flow is diverted through a 10-inch diameter connection. The crest of the overflow weir is set at el. 28.60 (NCD) and there is a brick cap over the interceptor. Flow that tops the weir is released to the Nashua River through a 48-inch diameter pipe.

Broad Street Overflow (CSO 008) – Flow enters the Broad Street Regulator through a 24-inch diameter pipe. Dry weather flow is diverted to the NRI through a 10-inch connection. Flow overtopping the brick weir at el. 39.07 discharges though a 30-inch diameter pipe to the Nashua River. As noted during flow meter installation, the 24-inch influent pipe is very steep and flow velocities entering the regulator are very high. The Broad Street CSO regulator is shown in Figure 2-9.

Lock Street Overflow (CSO 009) - Flow enters the Lock Street regulator through a 48-inch diameter pipe and dry weather flow is channeled to the NMRI through a 12-inch diameter pipe that increases to a 40-inch diameter pipe. Flow entering the NMRI is restricted by a metal plate (shown in Figure 2-10), obstructing a portion of the 12-inch opening to the dry weather connection. When flow conditions exceed the capacity of the opening and the water level in the regulator exceeds the weir crest at el. 29.97, combined sewage discharges to the Nashua River via a 36-inch diameter pipe.

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Figure 2-7. Spillway and energy dissipaters from Nashua River Overflow (006)

Figure 2-8. Access points for Nashua River Overflow (006)

Figure 2-9. High velocity influent flow and overflow weir at Broad Street CSO structure (008)

Figure 2-10. Plate on dry weather connection at Lock Street CSO (009)

Jackson/Beaucher Street Overflow (CSO 012) - Flow enters the Jackson/Beaucher Street regulator through a 30-inch diameter brick pipe. Dry-weather flow is diverted to the NRI through a 30-inch diameter RCP. Though a weir with a crest elevation at 19.97 (NCD) is in the CSO structure, the regulator cannot activate because the twin overflow gates on the downstream side of the weir are bolted shut. Since this CSO regulator cannot activate, it was not evaluated in this assessment.

NASHUA WASTEWATER TREATMENT FACILITY (NWTF)

Treatment Process

At the NWTF, the wastewater undergoes screening, aerated grit removal, and primary sedimentation, followed by activated sludge secondary treatment. After secondary treatment, the effluent is chlorinated and dechlorinated prior to being discharged to the Merrimack River. Sludge generated from the primary and secondary processes is thickened and anaerobically digested on site. Digested sludge is dewatered and sent off site for final disposal.

Treatment Capacity

The NWTF was initially constructed as a primary treatment facility in 1960. In 1975, the plant was expanded to provide an average daily capacity of 21.5 mgd and a peak capacity of 50 mgd. Activated sludge secondary treatment was added in 1989 to treat average daily flows of 16 mgd and peak flows of 38 mgd.

Secondary Bypass

During the upgrade to secondary treatment in 1989, a bypass pipe and diversion structure were added to the plant to allow primary effluent to bypass secondary treatment, and be conveyed directly to the chlorine contact chamber. This bypass allows the plant to handle up to 50 mgd during wet weather events: full secondary treatment for 38 mgd, with primary treatment and disinfection for an additional 12 mgd.

The city has found that running 38 mgd through secondary treatment for extended periods of time can "wash out" secondary solids, damaging the overall treatment process. As a result, the city has modified its procedures for manually bypassing secondary treatment as presented in Table 2-3.

TABLE 2-3. NWTF BYPASS OPERATION

Duration of high flow	Provide full secondary treatment	Provide primary treatment and disinfection only	Total wastewater flow receiving treatment
0 – 6 hours	38 mgd	12 mgd	50 mgd
6 – 12 hours	32 mgd	18 mgd	50 mgd
More than 12 hours	24 mgd	26 mgd	50 mgd

Bypass events usually result from increased flows in response to wet weather events. Activation of the bypass is not completely automated and the timing and duration of the bypass is up to the discretion of the operator.

In anticipation of a bypass event, the NWTF operator will ramp up chlorine dosage at the head of the chlorine contact chamber to a residual of 7.0 mg/L. This way the first flush of a bypass will receive full disinfection, prior to discharge. Dechlorination is controlled automatically and occurs in the outfall pipe.

Decisions on the wet weather operation of the NWTF are made based on the rising water levels at the headworks. Activation of lag and emergency pumps, as well as the activation of the bypass are determined from wet well elevations and not the hydraulic grade in the upstream interceptors.

CHAPTER 3

DRY WEATHER COLLECTION SYSTEM FLOWS

Both dry weather and wet weather flows contain domestic and commercial wastewater, industrial process wastewater, and groundwater infiltration; wet weather flow also includes stormwater runoff. Although the dry weather component accounts for only a small portion of the total combined sewage flow during wet weather periods, the dry weather component is significant due to its pollutant load. The parameters generally used to characterize this polluting characteristic of domestic sewage are *Escherichia coli* bacteria, 5-day biochemical oxygen demand (BOD_5), and total suspended solids (TSS).

To quantify dry weather wastewater flows, a flow monitoring program was conducted. The results of this program and discussion of each of the dry weather flow components are presented in this section.

FLOW MONITORING PROGRAM

That data collected from the flow monitoring program were used to determine typical dry weather flows, diurnal variations, per capita allowances, and infiltration/inflow allowances for different land use types. In addition, the measured dry weather flow characteristics were used to calibrate and verify the MOUSE model as described in Chapter 5.

Temporary and permanent flow meters were installed in the Nashua collection system. To meet NPDES permit requirements, the city of Nashua contracted Severn Trent Pipeline Services (STPS) of Auburn, NH to install and maintain seventeen flow meters within the combined system. These permanent flow meters were installed on overflow weirs or in overflow pipes at all nine CSO regulators, and on one influent line for eight of the CSO regulators. A meter was not installed on the influent line to the Broad Street CSO regulator (CSO 008) due to the hydraulic limitations of the site. At this location, the slope of the influent line is very steep and as a result the flow velocity is too high for accurate measurements.

Five temporary meters were deployed in the combined sewer system between May 17, 2001 and July 11, 2001. The temporary meters also were installed and maintained by STPS and were used to quantify flows in the major sewer interceptors. An additional meter was placed on a weir at the Salmon Brook Spillway to quantify flow from Salmon Brook entering the Merrimack River. Table 3-1 describes the location and each meter deployed as part of the monitoring program. Locations are also shown on Figure 3-1.

Locations for temporary meters were selected based on the physical characteristics of the sewer system, the delineation of the sewer system tributary areas, data needs for collection system model calibration and verification, and accessibility. All sites were field checked by M&E and STPS staff and evaluated for suitability prior to meter installation.

TABLE 3-1. FLOW METER INSTALLATIONS

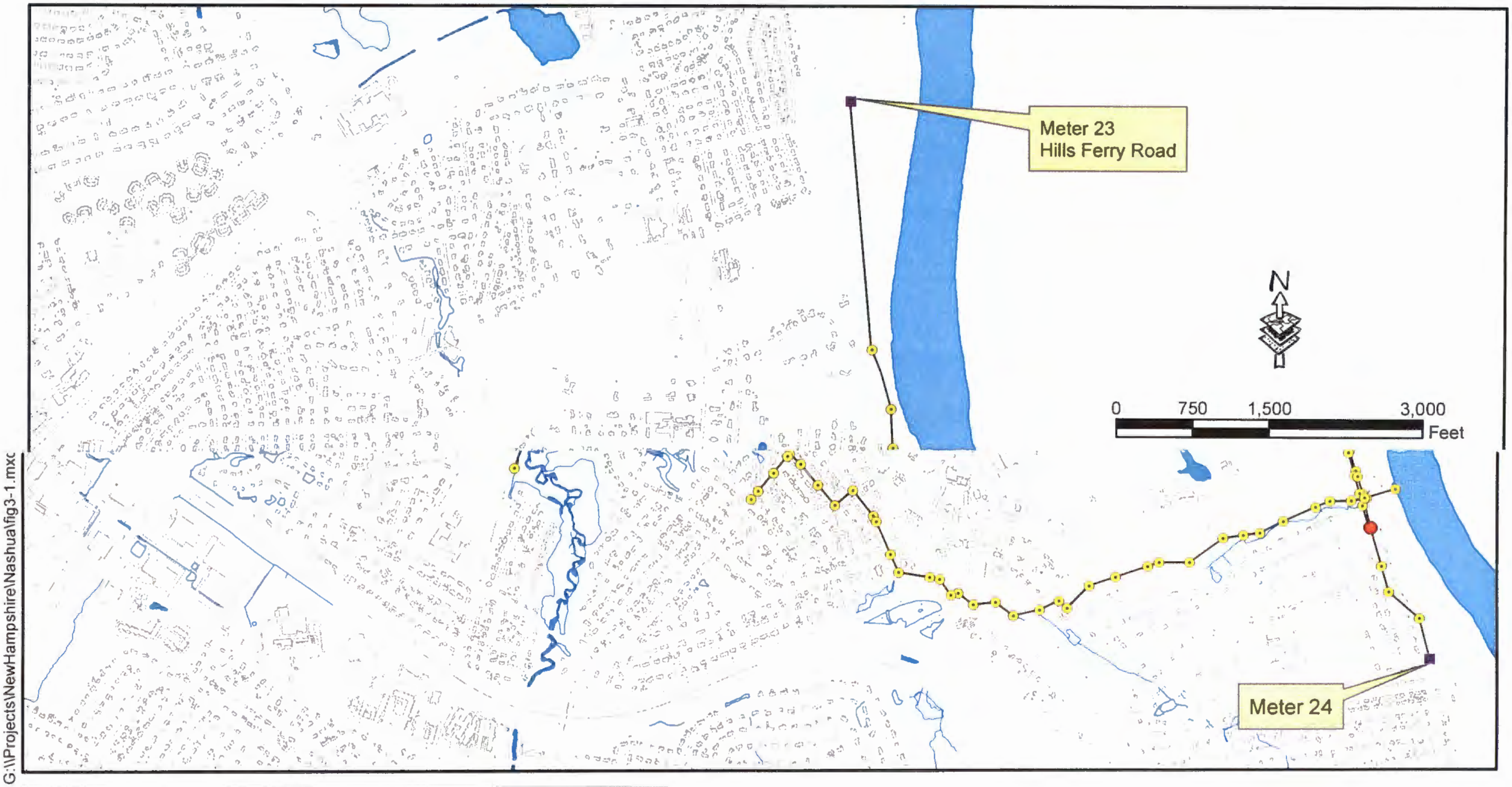
Meter ID	Regulator or Structure	Location	Pipe Size and Shape	Dates of Operation
M-2	CSO 002	In influent line to Salmon Brook regulator (002)	48-inch diameter	Permanent
M-2 OF	CSO 002	On weir crest in Salmon Brook regulator (002)	108-inch wide weir	Permanent
M-3	CSO 003	In influent line to Farmington Road regulator (003), approximately 3 feet up the pipe	36-inch diameter	Permanent
M-3 OF	CSO 003	On the overflow side of the weir in the Farmington Road regulator (003)	36-inch diameter	Permanent
M-4	CSO 004	In influent line to Burke Street regulator (004)	24-inch diameter	Permanent
M-4 OF	CSO 004	On overflow weir in Burke Street regulator (004)	96-inch wide weir	Permanent
M-5	CSO 005	In influent line to E. Hollis Street regulator (005)	54-inch diameter	Permanent
M-5 OF	CSO 005	On the overflow weir in E. Hollis Street regulator (005)	119-inch wide weir	Permanent
M-6	CSO 006	In influent line to Nashua River Overflow regulator (006)	108-inch diameter	Permanent
M-6 OF	CSO 006	On the overflow weir in Nashua River Overflow regulator (006)	120-inch wide by 72-inch high overflow conduit	Permanent

TABLE 3-1 CONTINUED. FLOW METER INSTALLATIONS

Meter ID	Regulator or Structure	Location	Pipe Size and Shape	Dates of Operation
M-7	CSO 007	In influent line to Tampa Street regulator (007)	48-inch diameter	Permanent
M-7 OF	CSO 007	On overflow weir in Tampa Street regulator (007)	182-inch wide weir	Permanent
M-8 OF	CSO 008	On overflow weir in Broad Street regulator (008)	124-inch wide weir	Permanent
M-9	CSO 009	In influent line to Lock Street regulator (009)	48-inch diameter	Permanent
M-9 OF	CSO 009	On the overflow side of the weir in the Lock Street regulator (009)	36-inch diameter	Permanent
M-12	CSO 012	In influent line to Jackson Street regulator (012)	42-inch diameter	Permanent
M-12 OF	CSO 012	Jackson Street regulator (012)	2- 60-inch by 36-inch overflow conduits	Permanent
M-21	SBI	In SBI in a manhole at the end of Verona Street	30-inch diameter	5/17/01 – 7/11/01
M-22	NRI	In NRI, upstream of siphon structure in Mine Falls Park	48-inch diameter	5/17/01 – 7/11/01
M-23	NMRI	In NMRI off Hillsferry Road	36-inch diameter	5/17/01 – 7/11/01
M-24	SMI	In SMI, in a manhole at the end of the Linton Street Extension at Elgin Street	30-inch diameter	5/17/01 – 7/11/01
M-25	NRI	In Nashua River Interceptor-2 at a manhole in Front Street	54-inch diameter	5/17/01 – 7/11/01
M-26	Salmon Brook	On a weir at the Salmon Brook Spillway, on the Ingersoll-Rand Property, off Burke Street	241.25-inch wide weir	5/17/01 – 7/11/01

DRY WEATHER FLOWS

Dry weather flows are comprised of ground water infiltration, domestic and commercial wastewater, and industrial process wastewater. Wet weather flows are comprised of the dry weather flow components plus storm water runoff. When analyzing dry weather flow, it is broken down into infiltration and sanitary flow components. It is necessary to separate



Legend

- Modeled Manholes
- Modeled Pipe Segments
- CSO Location*
- Non-CSO Metering Location
- NWTF
- Island
- Rivers, Streams, & Ponds
- Dam

** Note:*
All CSOs were metered on both the influent and overflow lines, except CSO 008, which was only metered on the overflow line.

Data Sources:
Data provided by City of Nashua, New Hampshire - 2001

Coordinate System:
Projection: Stateplane
Datum: NAD83
Fipszone: 2800
Units: Feet

Figure 3-1.
Flow Meter Locations

the dry weather flow from the wet weather flow when analyzing combined sewer performance and for development of the collection system model. Dry weather flow is the day-to-day demand on the system. Dry weather flow should be compared to the capacity of the system to determine if the system capacity is sufficient to handle this flow. Also, by separating the dry and wet weather flow components, the volume of stormwater entering the system can be estimated.

It is easier to calibrate the model for dry weather flow first, because dry weather flow is relatively consistent, and then calibrate the model for wet weather flow. Wet weather flow is more difficult to model because it is affected by many variables such as rainfall depth, rainfall intensity, and surface runoff. Separating the dry weather flow into sanitary and infiltration components is necessary in order to accurately analyze and model the system. Sanitary flows are generated by the population and industrial users tributary to the collection system, while infiltration is groundwater leaking into the system. Identifying the sanitary flow volumes and patterns can demonstrate when the system is under the highest dry weather demand. Once these dry weather flow components have been established, the collection system model can be used to assess the effects of reduced infiltration or changes in the population. If the volume of infiltration is excessive, it may be cost-effective to rehabilitate the sewer system.

Flow meter data collected during the dry weather calibration period were used to estimate the sanitary and infiltration flow components at each meter location. The infiltration component was estimated as 20 percent of the lowest nighttime flow measured by the meter and was assumed to be constant for the entire dry weather period. The sanitary flow component was computed by subtracting the infiltration from the total dry weather flow.

A summary of metered dry weather flow, including total flow and the infiltration and sanitary components is presented in Table 3-2. This data represents the dry weather calibration period, June 4 through June 10, 2001.

Infiltration Flow Component

Infiltration is groundwater that enters the system through defective pipes, pipe joints, connections, or manhole walls that are below the groundwater table. Infiltration varies seasonally due to the rise and fall of the water table. However, it is important to design for this kind of flow, especially in older sewer systems. The amount of infiltration in a sewer system can be estimated by assuming a certain amount of flow for each length of pipe, dependant on the age of the sewer. When flow meter data are available, the infiltration can also be determined from meter data collected. Approximately 80% of the lowest night time flow is attributed to infiltration.

TABLE 3-2. DRY WEATHER FLOW CHARACTERISTICS

Meter	Total Dry Flow (cfs)	Sanitary Flow (cfs)	Infiltration (cfs)
M-2	5.01	2.92	2.08
M-3	2.38	1.36	1.02
M-4	0.23	0.17	0.06
M-5	1.63	1.30	0.33
M-6	7.05	4.12	2.93
M-7	0.23	0.14	0.09
M-9	0.19	0.16	0.03
M-12	0.45	0.35	0.10
M-21	4.00	2.56	1.44
M-22	4.62	3.15	1.47
M-23	0.59	0.52	0.07
M-24	1.87	1.32	0.54
M-25	5.05	3.59	1.46

Based on night time flow measurements, groundwater infiltration comprises between 12% and 43% of dry weather flow in the combined sewer service areas. However, this represents a very small fraction of the flow during an overflow event. Therefore, while reducing infiltration could improve wastewater treatment plant performance and increase available capacity during dry weather, it would not be an effective means to reduce combined sewer overflows.

An example will illustrate this point. For the collection system in the area tributary to the East Hollis Street CSO (005), the measured flow rate for infiltration was 0.33 cfs. The storm of June 2, 2001, which occurred during the wet weather flow monitoring period, was typical of a 3-

month storm in terms of its peak intensity. During that storm, CSO 005 began to overflow when flow in the pipe directly upstream of the regulator reached approximately 9 cfs. During that storm the flow reached a peak of approximately 70 cfs. Therefore, even if it was possible to remove all of the infiltration in the upstream collection system, that would have reduced the wet weather flow by only four percent at the start of the overflow episode, and by less than one percent at the peak of the storm.

Sanitary Flow Component

The sanitary flow contributed by residential, commercial, and industrial areas can be calculated based on average flows and populations. For domestic wastewater, the amount of flow entering the system can be approximated using typical per capita allowances, expressed in terms of gallons per capita per day (gpcd). Commercial and industrial flows can be estimated based on the type of business, number of employees, and total working hours. The sanitary flow component can also be determined from flow metering. The average sanitary flow can be determined from the difference between the infiltration and the average dry weather flow.

Sanitary flow is variable throughout the day; however the peaks and valleys in a plot of sanitary flow versus time generally fall at similar times every day. The differences in the diurnal pattern become greater when weekday flow is compared to weekend flow. Reviewing Figures 3-2 and 3-3, diurnal patterns for a weekend day and weekday, respectively, show the peak morning flow to occur later on the weekend than the weekday. Furthermore, the peak flows are greater during the weekday than the weekend day.

RIVER INFLOW

The elevation of the Merrimack River is controlled by a series of gates on the river in Lowell, Massachusetts, downstream of Nashua. As a result, river elevation can vary throughout the season. At the East Hollis Street CSO (CSO 005) which has the lowest overflow weir in the system, river water can flow back over the weir and into the combined sewer system. However, to minimize inflow at this location, the city closes a sluice gate between the regulator and the

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FIGURE 3-2. WEEKEND DIURNAL FLOW PATTERN
JUNE 9-10, 2001

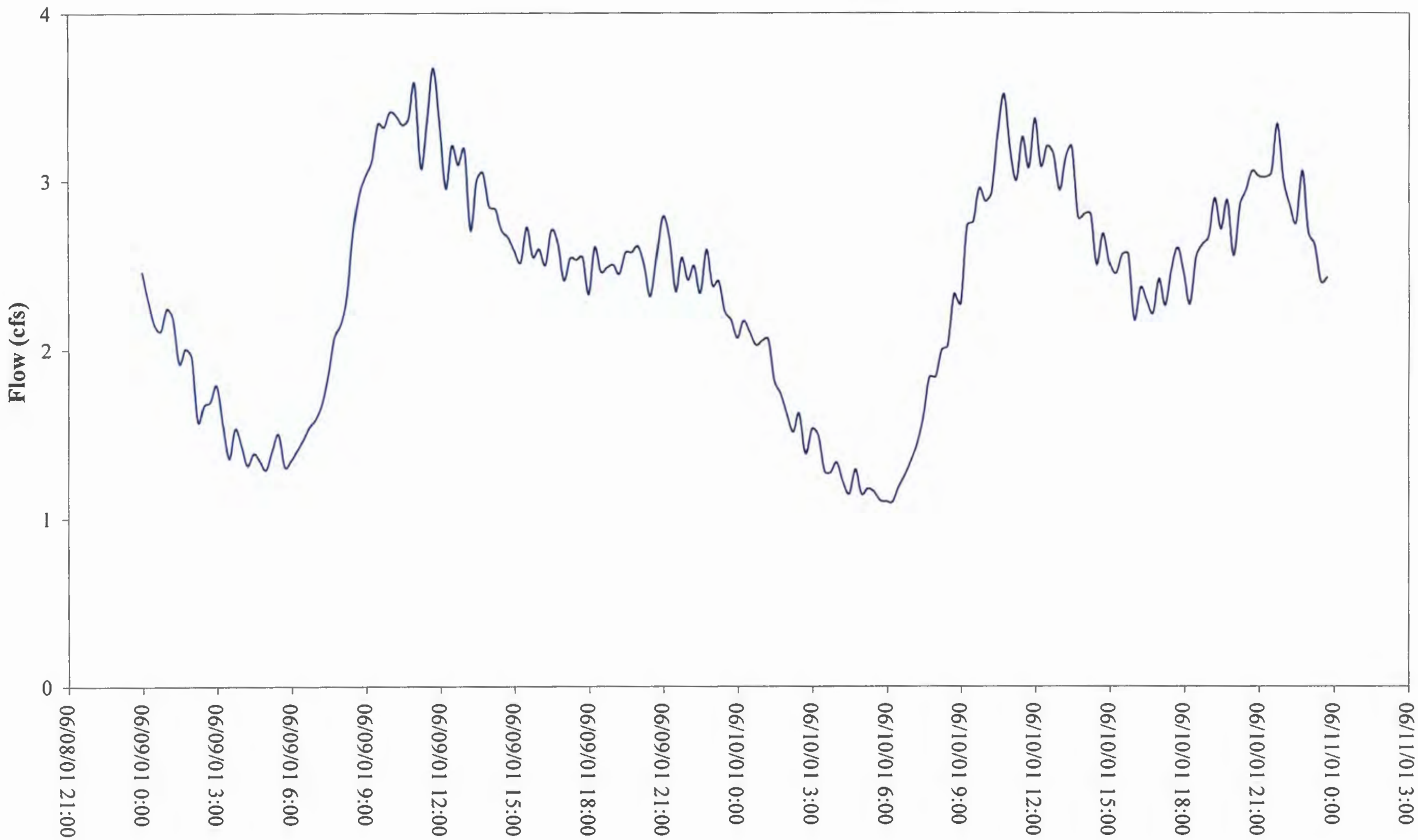

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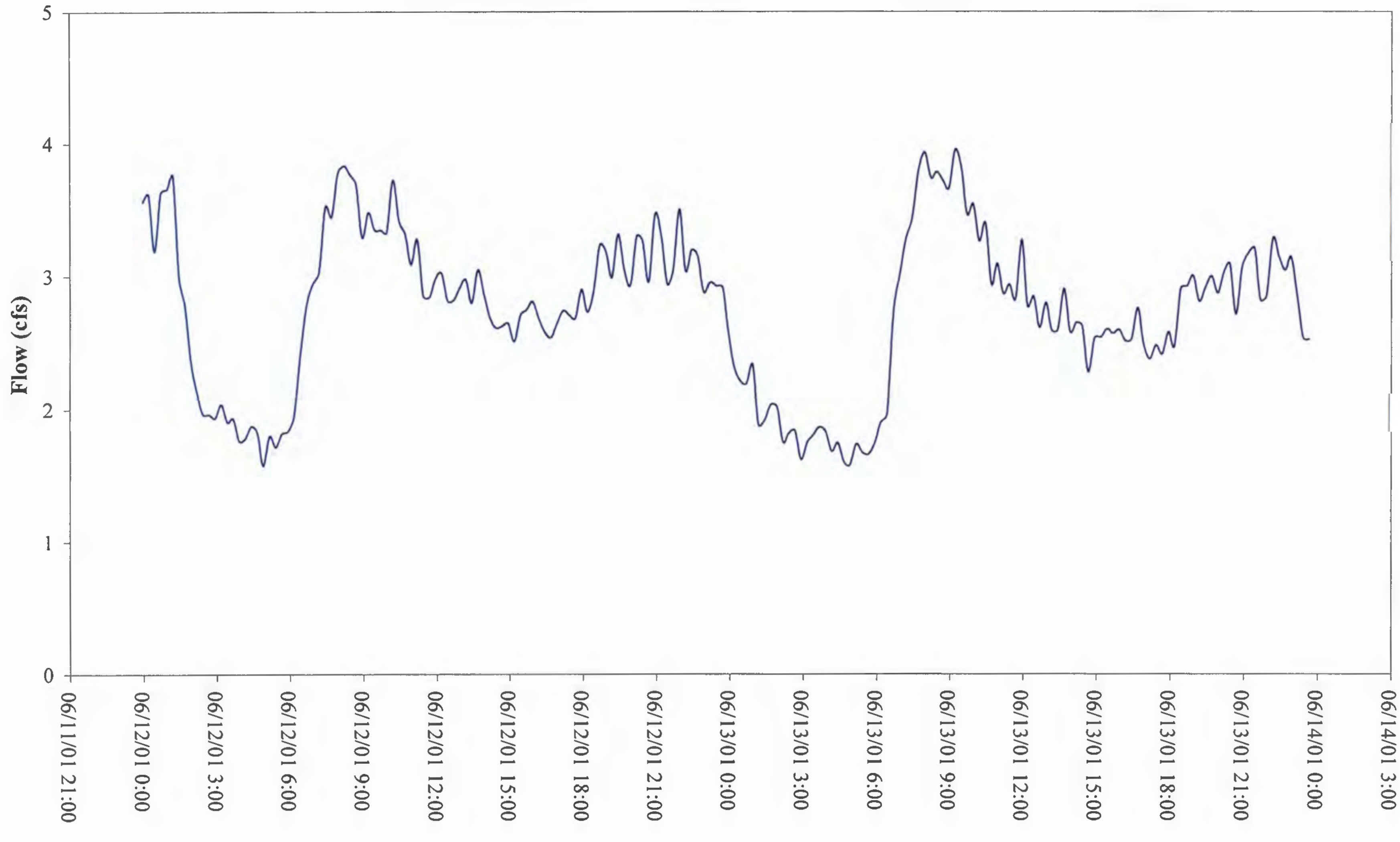
FIGURE 3-3. WEEKDAY DIURNAL FLOW PATTERN
JUNE 12-13, 2001



river when river elevation is high. At the Burke Street CSO structure (CSO 004), a "duckbill" style valve (Figure 3-4) has been installed at the headwall to limit the amount of river water backing into the CSO structure when the river elevation is high.

The elevation of the downstream reach of the Nashua River is dictated by the elevation of the Merrimack. Upstream water elevation is dictated by the operation of the Jacksonville Dam. The dam, owned and maintained by the Essex Power Company, is located just downstream of the Main Street bridge, near the Nashua Library.

The elevation of the Nashua River, upstream of the Jacksonville Dam, was shown to have flooded the Tampa Street CSO structure (CSO 007) during early spring of 2001 (Figure 3-5). Flow metering data from this regulator indicated there was standing water in the structure between March 22 and March 28, 2001. The City reports that these conditions may exist throughout the year. During this time the meter placed on the crest of the overflow weir recorded a maximum water depth of 26.45 inches, which corresponds to el. 30.80 (NCD). The city should consider installation of a "duckbill" type valve or flap gate in this structure to keep Nashua River water out of the collection system.

Figure 3-4. "Duckbill" type valve on overflow at Burke Street CSO (004)

Figure 3-5. Tampa Street CSO outfall partially submerged in Nashua River

CHAPTER 4

WET WEATHER COLLECTION SYSTEM FLOWS

Nashua's combined sewers were originally designed to convey stormwater runoff, plus dry weather flow to the nearby rivers. Many years later, interceptor sewers were constructed along the river to capture all of the dry weather flow and a portion of the wet weather flow from the combined sewers and to convey that flow to the NWTF for treatment.

Based on M&E's review of the collection system network, there are three general locations with the potential to act as "bottlenecks", or hydraulic constrictions in the collection system that prevent a portion of the flow from being treated at the NWTF. These bottlenecks cause combined sewage to overflow to the river. These three potential "bottlenecks" are:

1. At the NWTF, which cannot accept more than 50 mgd. When flow delivered to the treatment facility exceeds 50 mgd, it backs up from the headworks into the interceptors, ultimately contributing to overflows at upstream CSO regulators.

2. At the regulators where the combined sewers discharge into the interceptors. These regulators, which in some cases are small diameter pipes connecting the combined sewers to the interceptors, are designed to restrict the amount of wet weather flow that can enter an interceptor in order to protect downstream treatment works.

3. In the interceptors, which do not have the capacity to carry all of the wet weather flow from the combined sewers. When an interceptor's capacity is exceeded, the interceptor may be relieved through the CSO regulators.

The collection system model was developed as a tool to estimate the volume of sewage overflowing to the river under various storm conditions. The model also has made it possible to consider impacts of modifying the existing collection system to reduce overflows by providing separate sewers in combined areas, by allowing more flow to be treated at the NWTF, or by providing treatment of wet weather flows at alternate locations.

WET WEATHER MONITORING PROGRAM

The wet weather monitoring program provided a basis for calibrating the collection system model, for determining typical pollutant levels in CSO and stormwater discharges, and for calibrating the river water quality model.

The wet weather and dry weather flow monitoring programs were both conducted during the spring and summer of 2001, utilizing the same flow meters (refer to Chapter 3). The program included monitoring flows at CSOs, interceptors, and regulators; gauging rainfall; and sampling CSOs, stormwater, and river water. Details of the wet weather sample collection can be found in the *Final Quality Assurance Project Plan for Receiving Water, Combined Sewer Overflow and Stormwater Quality Sampling, August 2002* (QAPP).

RAINFALL MONITORING

Rainfall data were collected and used to calibrate the computer models. All rainfall data were collected from one continuously recording tipping bucket type rain gauge located at the NWTF. The rain gauge was installed and maintained by STPS as part of the permanent flow monitoring program.

A total of 22 storms of various duration and intensity occurred during the monitoring period. The distribution of these storms, by total rainfall, is presented in Table 4-1. Table 4-2 gives the characteristics of each storm that occurred during the monitoring period, in terms of total rainfall, duration, and intensity. A storm was considered to be a single event only if it was preceded by a period of at least 12 hours without precipitation.

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TABLE 4-1 STORM EVENTS MAY 5 – JULY 11, 2001
DISTRIBUTION BY TOTAL RAINFALL

Total Rainfall (inches)	No. of Storms
>2.0	2
1.51-2.00	0
1.01-1.50	1
0.76-1.00	1
0.51-0.75	2
0.26-0.50	3
0.11-0.25	5
<0.10	8
TOTAL	**22**

The largest storm event occurred on June 2, 2001, where the rain gauge recorded 2.50 inches of rainfall. This storm was equivalent to a 2-year storm in terms of total rainfall. Other storms with significant total rainfall recorded during the monitoring period include: May 26, 2001 (0.96 inches); June 11, 2002 2001 (1.06 inches); June 17, 2001 (2.36 inches); July 1, 2002 2001 (0.71 inches); and July 5, 2001 (0.64 inches). These and some of the smaller events were used to analyze wet weather flows and collection system hydraulic response, and to calibrate the model. ✓

TABLE 4-2. SUMMARY OF STORM EVENTS FOR
MAY 5 – JULY 11, 2001

Storm	Start Date	Start Hour	Duration (hrs)	Rainfall (in)	Intensity (in/hr)		Interevent hours
					Average	Maximum	
1	5/5/2001	8	2	0.03	0.02	0.02	--
2	5/12/2001	17	2	0.16	0.08	0.13	175
3	5/14/2001	16	1	0.01	0.01	0.01	45
4	5/15/2001	11	22	0.12	0.01	0.07	18
5	5/22/2001	7	13	0.26	0.02	0.05	142
6	5/23/2001	23	8	0.16	0.02	0.11	27
7	5/26/2001	22	27	0.96	0.04	0.20	63
8	5/28/2001	18	2	0.04	0.02	0.03	17
9	5/29/2001	10	1	0.01	0.01	0.01	14
10	5/30/2001	17	1	0.02	0.02	0.02	30
11	6/2/2001	3	39	2.50	0.06	0.39	57
12	6/10/2001	20	1	0.03	0.03	0.03	170
13	6/11/2001	14	15	1.06	0.07	0.26	17
14	6/17/2001	11	18	2.36	0.13	1.39	126
15	6/20/2001	19	4	0.26	0.07	0.13	62
16	6/23/2001	13	25	0.16	0.01	0.12	62

TABLE 4-2 CONTINUED. SUMMARY OF STORM EVENTS FOR MAY 5 – JULY 11, 2001

Storm	Start Date	Start Hour	Duration (hrs)	Rainfall (in)	Intensity (in/hr)	Interevent hours	Storm
17	6/30/2001	17	4	0.31	0.08	0.30	147
18	7/1/2001	9	10	0.71	0.07	0.35	12
19	7/5/2001	2	15	0.64	0.04	0.57	79
20	7/8/2001	4	16	0.12	0.01	0.06	59
21	7/9/2001	18	2	0.04	0.02	0.03	22
22	7/11/2001	10	1	0.01	0.01	0.01	38

FLOW MONITORING

The flow meters described in Chapter 3 were also configured to provide flow data at CSO regulator and interceptor locations during storm events. Flow meters placed on overflow weirs or in overflow pipes quantified the overflow volumes for the storms listed in Table 4-2. Data were analyzed with respect to overflow volumes for the storms recorded during the monitoring period, the characteristics of overflows at CSO locations, and the major factors (i.e. drainage size area, interceptor hydraulics) that affect these characteristics.

CSO Volumes

The total volume of overflow recorded at each CSO during the temporary flow monitoring period is presented in Table 4-3. This data shows the East Hollis Street CSO structure (005) was the most active with 14 overflow events. Farmington Road (003), Burke Street (004), and Lock Street (009) also had 10 or more activations during the 10-week program. Although the Nashua River CSO structure (006) activated only six times, it released 9.52 MG, which was the largest volume. The Nashua River CSO structure is extremely large, and serves as the major relief point in the combined system during large storm events. The Tampa Street and Salmon Brook CSOs were the least active, each with only 1 activation of 0.09 MG each.

TABLE 4-3. SUMMARY OF CSO ACTIVATION MAY 5 – JUNE 11, 2001

CSO Regulator	No. of Activations	Total Volume (MG)
CSO 002 Salmon Brook	1	0.09
CSO 003 Farmington Road	10	1.28
CSO 004 Burke Street	11	4.06
CSO 005 E. Hollis Street	14	2.75
CSO 006 Nashua River	6	9.52
CSO 007 Tampa Street	1	0.09
CSO 008 Broad Street	8	0.68
CSO 009 Lock Street	11	1.02

CSO WET WEATHER SAMPLING

During two wet weather events in the fall of 2002, samples were taken at CSO, stormwater, and river transect locations in accordance with the approved QAPP. Samples collected at these locations on October 16, 2002 (1.19 inches of rainfall) and November 6, 2002 (0.93 inches) were analyzed for *E. coli*. Grab samples from stormwater outlets were collected, composited, and analyzed for *E. coli* as well as other constituents, including: BOD_5, TSS, total phosphorus, TKN, lead, copper, zinc, settleable solids, pH and temperature.

River transect samples were collected at specific time intervals after the peak of the storm, and were used to calibrate the receiving water model, as discussed in Chapter 7. At the CSO and stormwater outlets, up to six rounds of grab samples were collected, depending on the duration of the discharges. Laboratory results of *E. coli* from these samples were used to estimate average bacteria concentrations discharged from CSOs and stormdrains. Those data, including the results of QA/QC samples, can be found in Appendix A.

CSO *E. coli* Concentrations

Representative CSO samples for E. Coli were collected from two locations, East Hollis Street (CSO 005) and Burke Street (CSO 004). The model indicates that the East Hollis Street CSO should have the most frequent activations of all the CSO regulators. It was active during both sampling events. Since the regulator was overflowing for the duration of the events, the full six

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rounds of grab samples were collected from this CSO. The Burke Street CSO regulator discharged for only a short period of time during the October 16, 2002 sample event. Therefore, only one CSO sample was collected at this location. The Burke Street regulator was not observed to activate during the November 6, 2002 event. A summary of the concentrations from the samples collected can be found on Table 4-4. As noted in Table 4-4, an average CSO *E. coli* concentration was computed. This concentration is used to calculate loading values for the receiving water model, as discussed later in this report.

TABLE 4-4. SUMMARY OF *E. coli* CONCENTRATIONS AT CSO LOCATIONS

	October 16, 2002 Sampling Event		**November 6, 2002 Sampling Event**	
Grab Sample	East Hollis St. (CSO 005) col/100mL	Burke St. (CSO 004) col/100mL	East Hollis St. (CSO 005) col/100mL	Burke St. (CSO 004) col/100mL
Round 1	TNTC	TNTC	308,000	No Flow
Round 2	230,000	No Flow	220,000	No Flow
Round 3	226,000	No Flow	198,000	No Flow
Round 4	304,000	No Flow	5,500	No Flow
Round 5	108,000	No Flow	1,300	No Flow
Round 6	760,000	No Flow	2,100	No Flow
arithmetic **Average**	**325,600**	No Flow	**122,483**	No Flow
AVERAGE FOR ALL CSO SAMPLES: 212,000 col/100 mL				

TNTC=colonies on plate were Too Numerous To Count

One of the samples collected at the East Hollis Street CSO and the one sample collected from the Burke Street CSO were reported as TNTC. Otherwise, bacteria concentrations enumerated from samples collected during the October 16, 2002 event were all of the same order of magnitude, and in the range expected for CSO. Samples for the November 6, 2002 event were of similar magnitude for rounds 1 through 3. The concentrations did drop significantly in rounds 4 through 6.

Based on the data generated, the average *E. coli* concentration for CSOs was computed to be 212,000 col/100 mL.

Stormwater *E. coli* Concentrations

Samples were collected from a storm drain on Seminole Drive (SD-1) and a storm drain on Celeste Street (SD-2). Both of these drains were observed to be dry under dry weather conditions with no visual or olfactory evidence of illicit connections. The Seminole Drive drain discharges to the Nashua River and serves a residential area west of Route 3. The Celeste Street drain is a smaller diameter drain and services a residential area in the northern part of Nashua, near the border with Merrimack. Due to the small area tributary to SD-2, the drain will often stop flowing shortly after the rain stops. As a result, not all six rounds were collected at this location during the two sampling events. Six rounds were, however, collected at SD-1. Data collected are presented in Table 4-5.

TABLE 4-5. SUMMARY OF *E. coli* CONCENTRATIONS AT STORMWATER OUTLET LOCATIONS

	October 16, 2002 Sampling Event		**November 6, 2002 Sampling Event**	
Grab Sample	Seminole Drive (SD-1) col/100mL	Celeste St. (SD-2) col/100mL	Seminole Drive (SD-1) col/100mL	Celeste St. (SD-2) col/100mL
Round 1	2,600	780	220	980
Round 2	5,200	1,800	640	11,000
Round 3	6,800	2,420	4,200	1,320
Round 4	44,000	No Flow	3,800	1,660
Round 5	4,600	No Flow	1,580	No Flow
Round 6	800	No Flow	940	No Flow
Average	**10,667**	**1,667**	**1,897**	**3,740**
AVERAGE FOR ALL SW SAMPLES: 5,000 col/100 mL				

Review of the October 16, 2002 data from SD-1 show the concentrations to be relatively constant, with the exception of round 4, where concentrations were slightly higher than previous rounds. During rounds 5 and 6, the measured concentration of *E. coli* dropped, possibly in response to the sustained flow stage of the storm. Average concentrations from the two drains are slightly higher from SD-1 than from SD-2.

Samples collected during the November 6, 2002 data show a different trend – average concentrations were somewhat higher in SD-2 than SD-1. As with the October 16, 2002 event, the drain stopped flowing shortly after the cessation of the rain and not all six rounds of samples were collected. All six rounds were collected at SD-1. Review of the data show the concentrations to rise during rounds 3 and 4 and taper off in rounds 4 and 5 (sustained flow).

Based on the data collected from the two drains during the two events, an average *E. coli* concentration for stormwater of 5,000 col/100 mL was computed.

Other Stormwater Constituient Concentrations

An additional set of grab samples was collected at each storm drain outlet, composited at the laboratory, and analyzed for several constituents. These analyses obtained data which could be used as a comparison to analyses for the same constituents, conducted on selected CSOs during the 1997 LTCP. Data collected from composite stormwater samples in this study is presented in Table 4-6 and CSO data presented in the 1997 LTCP is summarized in Table 4-7.

TABLE 4-6. SUMMARY OF CONSTITUIENT CONCENTRATIONS AT STORMWATER OUTLET LOCATIONS

Parameter	October 16, 2002 Sampling Event		November 6, 2002 Sampling Event		Average
	Seminole Drive	Celeste Street	Seminole Drive	Celeste Street	
TSS (mg/L)	8	15	8	<4	8
Settleable Solids (ml/L)	<0.20	<0.20	<0.20	<0.20	<0.20
TKN (mg/L)	0.820	0.966	0.281	0.860	0.732
Total Phos. (mg/L)	0.150	N/A	0.126	0.374	0.217
BOD_5 (mg/L)	6	6	126	218	89
Copper (mg/L)	<0.010	0.021	<0.010	<0.010	0.009
Zinc (mg/L)	0.050	0.042	0.026	0.103	0.055
Lead (mg/L)	<0.005	0.019	<0.005	<0.005	0.007

As expected, the average constituent concentrations, presented in Tables 4-6 and 4-7, were greater for the CSO samples than for the stormwater samples, except for BOD_5. The average stormwater concentration for BOD_5 was significantly greater than the average CSO concentration because of the results of the samples collected during the November 6, 2002 sampling event. Data collected during this second event were two orders of magnitude higher than the samples collected at the same locations three weeks earlier. The higher concentrations measured during the second sampling event significantly raised the average concentration but

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TABLE 4-7. CONSTITUENT CONCENTRATIONS AT CSO LOCATIONS, from 1997 LTCP

Parameter	CSO 004 Burke Street		CSO 005 E. Hollis Street		CSO 006 Nashua River		CSO 009 Lock Street		**Average**
	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2	Event 1	Event 2	
TSS (mg/L)	17	45	29	52	44	74	27	69	**45**
Settleable Solids (ml/L)	BDL	BDL	1	0.6	1	1.3	BDL	0.6	**0.9**
TKN (mg/L)	1.8	0.5	2.6	0.5	4.5	0.5	1.3	0.4	**1.5**
Total Phos. (mg/L)	0.28	0.57	0.37	1.40	0.68	1.30	0.24	0.70	**0.69**
BOD_5 (mg/L)	7	11	13	49	16	33	7	11	**18**
Copper (mg/L)	BDL	0.02	0.034	0.05	BDL	0.05	BDL	0.02	**0.03**
Zinc (mg/L)	0.061	0.07	0.099	0.15	0.074	0.09	0.049	0.08	**0.08**
Lead (mg/L)	0.011	0.015	0.017	0.029	0.022	0.027	0.021	0.06	**0.025**

BDL = Below Detection Limit

(what [illegible] the [illegible] levels?

were not eliminated since review of the field duplicate and equipment blank samples show no indication of contamination.

The results of the BOD_5 analysis aside, constituent concentrations were higher for CSO than stormwater, but not significantly. This analysis shows that stormwater runoff carries with it, a number of pollutants in concentrations that can be just as harmful to the receiving waters as the CSO. It is important to note that the loading of these constituents from stormwater runoff occurs every time it rains, whereas CSO discharges only in events large enough to cause the CSO to activate.

CHAPTER 5

COLLECTION SYSTEM MODEL DEVELOPMENT

A detailed hydrologic and hydraulic model of the Nashua combined sewer system has been developed. This model was created in order to provide a reliable tool that could accurately simulate flows in the combined sewer system, and thus predict CSO activation frequency and overflow volumes. The calibrated model was then used to predict how proposed modifications to the collection system would impact activation frequency and volume. This section of the report describes the hydrologic and hydraulic model developed for this project including the assumptions made during model development.

BACKGROUND

The city's previous Long-Term CSO Control Plan, dated September 1997, was based on a prior model of the collection system. That model was used to evaluate a variety of alternatives for CSO control in Nashua, including the recommended plan. It contained approximately 120 nodes and 130 conduits, covering the main interceptors in the study area. A schematic of the extent of the hydraulic model used in the 1997 analysis is shown in Figure 5-1.

MODEL SOFTWARE

Modeling efforts in Nashua used two different modeling programs; one to predict runoff volumes entering the network during rainstorm events, and the other to simulate flow in the pipe network.

Hydrologic Modeling

The hydrologic model used to simulate stormwater runoff was the U.S. EPA Stormwater Management Model (SWMM) RUNOFF block. The RUNOFF block of SWMM simulates the

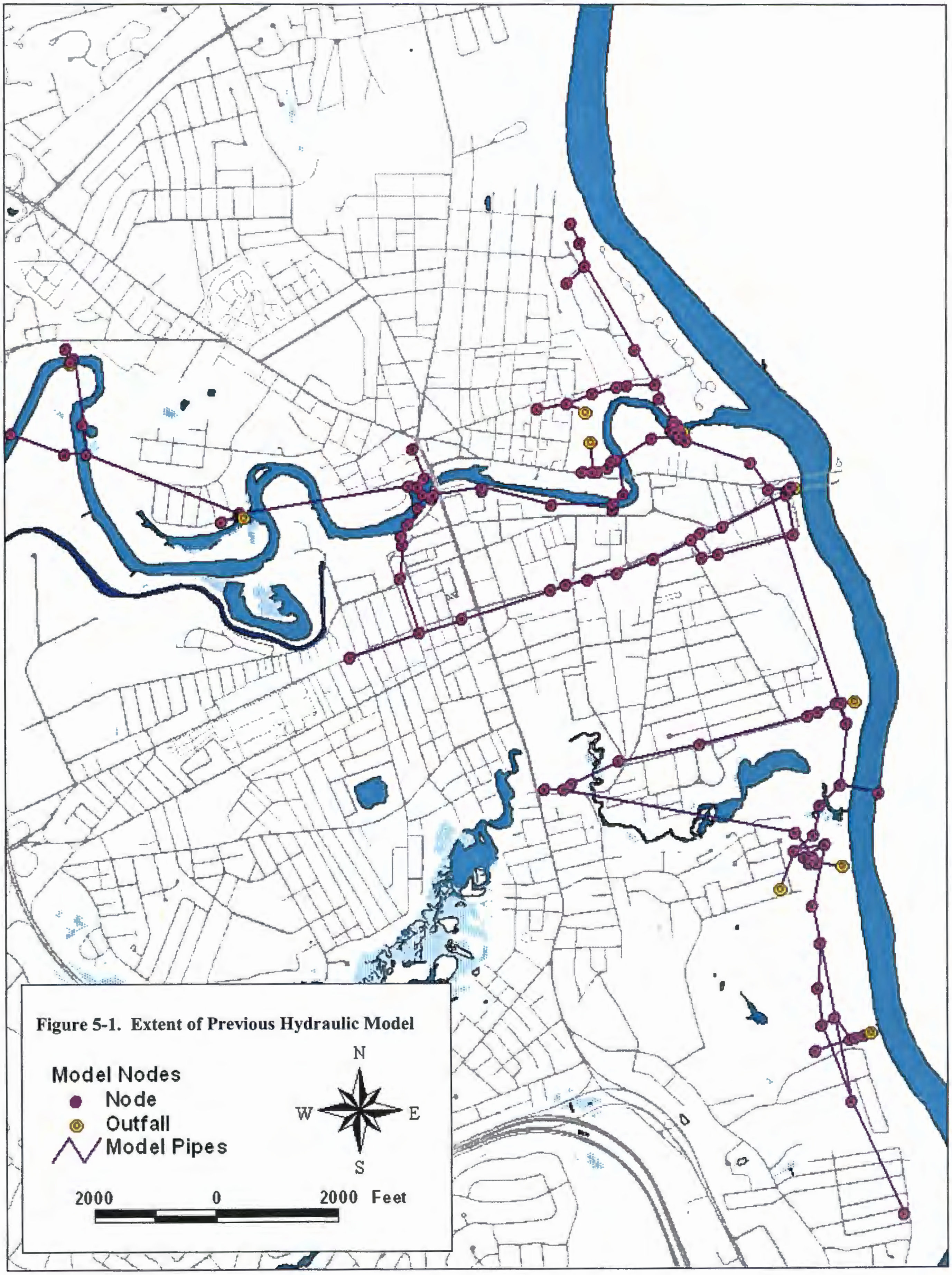

Figure 5-1. Extent of Previous Hydraulic Model

stormwater runoff in drainage basins discharging to the sewers and interceptors modeled with MOUSE. More specifically, RUNOFF calculates the inflow hydrographs (flow versus time) at the various inflow points of the MOUSE model as a function of user-specified rainfall hyetographs (rainfall versus time). RUNOFF simulates overland flow in pervious and impervious areas, infiltration losses in pervious areas, surface detention and channel flow in smaller drains. The primary parameters characterizing the different subcatchments are their surface areas, widths, percent imperviousness, depression storage depths, roughness coefficients (Manning's n), and infiltration parameters.

Hydraulic Modeling

The hydraulic model used to simulate flow in the pipe network in Nashua was the Danish Hydraulic Institute (DHI) MOUSE modeling system. The MOUSE model simulates routing of flows in the sewer system by solving the one-dimensional hydrodynamic flow equations of continuity and momentum. In the model, the sewer system is divided into conduits and nodes. The conduits represent pipes, which are characterized by their cross-section shape, dimensions, length, slope and friction factor (Manning's coefficient). The nodes are located at each end of the conduits to simulate headlosses associated with various hydraulic structures, including manholes, pump stations, and regulators.

The MOUSE model can calculate backwater effects due to downstream flow restrictions and surcharged conditions (sewers flowing full). All the hydraulic elements present in sewer systems can be simulated in MOUSE, including conduits with various cross sections, circular manholes, detention basins, weirs, pump stations with a variety of operational models, flow regulators, and constant or time variable outlet water levels. Modeling results can be viewed on the monitor screen, printed in color, or exported to a spreadsheet. Also, water levels in conduits can be animated, to show flow condition as a function of time.

MODEL CONSTRUCTION

Drainage Basin Characteristics

A drainage basin characterization developed during the 1997 LTCP was used in the development of the new model. The characterization was modified based on field inspections and a review of local topology. In many cases, the existing basins were subdivided to provide more detail for the new hydrologic model. A summary of the basin characteristics used for the model can be found in Table 5-1, and the corresponding basin delineation is presented in Figure 5-2. The parameters listed in the table are described in the following sections.

Percent Impervious.

The amount of runoff entering a combined system is primarily a function of the percent of directly connected impervious area. Impervious area is defined as the portion of the drainage basin in which runoff never infiltrates into the ground. In urban areas, impervious area is mostly comprised of pavement and roof area; undeveloped areas are generally assigned a small percent impervious value as well.

The percent of directly connected impervious area in each basin was estimated through a combination of topographic data, GIS data, and meter data. Topographic information was used to delineate the individual areas tributary to each flow meter. From the flow meter data, runoff coefficients were estimated for each basin as a preliminary calibration assessment. Each of these large metering basins, were then subdivided into several sub basins.

Percent impervious values were then assigned to each of the sub basins, computed through an assessment of land use types, generated from output of GIS data. The GIS data provided a distribution of impervious area for each basin, e.g. roads, parking lots, buildings (with unconnected roof drains), and other paved surfaces, which could be expressed as a percentage of the total sub basin area. The percentages were then scaled up or down so that the average percent impervious of the sub basins roughly equaled the value computed from metering data.

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TABLE 5-1. SUMMARY OF RUNOFF BASIN CHARACTERISTICS

								Manning's		Dep. Stor.		Green-Ampt		
Meter	C or S	Catchment ID	Input Location	Width (ft)	Area (acres)	Percent Imp.	Slope (ft/ft)	Imp.	Perv.	Imp.	Perv.	Suct.	Hyd.Con.	Air/Void
2/2OF	Combined	54662	5466	211	19.2	1.93	0.1274	0.04	0.2	0.05	0.2	8	0.3	0.33
		42251	4225	1056	96	8.79	0.0257	0.04	0.2	0.05	0.2	8	0.3	0.33
		4347	4347	69	6.3	6.46	0.0483	0.04	0.2	0.05	0.2	8	0.3	0.33
		44112	4411	253	23	5.93	0.0758	0.04	0.2	0.05	0.2	8	0.3	0.33
		42252	4225	1115	101.4	7.27	0.0720	0.04	0.2	0.05	0.2	8	0.3	0.33
	Separated	4327	4327	529	48.1	1.17	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		4502	4502	370	33.7	0.88	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		44111	4411	141	12.8	0.99	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
3/3OF	Combined	55651	5565	324	46.3	7.6	0.1006	0.04	0.2	0.05	0.2	8	0.3	0.33
		5289	5289	568	81.1	9.13	0.0354	0.04	0.2	0.05	0.2	8	0.3	0.33
		5310	5310	210	30	10.1	0.0405	0.04	0.2	0.05	0.2	8	0.3	0.33
		5268	5268	95	13.6	7.95	0.0499	0.04	0.2	0.05	0.2	8	0.3	0.33
		54661	5466	513	73.3	4.04	0.0637	0.04	0.2	0.05	0.2	8	0.3	0.33
		5392	5392	229	32.7	3.14	0.0638	0.04	0.2	0.05	0.2	8	0.3	0.33
		5378	5378	84	12	5.13	0.0847	0.04	0.2	0.05	0.2	8	0.3	0.33
		5482	5482	222	31.8	7.97	0.0526	0.04	0.2	0.05	0.2	8	0.3	0.33
		5194	5194	202	28.9	7.71	0.0445	0.04	0.2	0.05	0.2	8	0.3	0.33
	Separated	55652	5565	239	34.1	1.71	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
4/4OF	Combined	4183	4183	102	6.8	16.74	0.0318	0.04	0.2	0.05	0.2	8	0.3	0.33
		3975	3975	110	7.3	9.04	0.0245	0.04	0.2	0.05	0.2	8	0.3	0.33
		4035	4035	38	2.5	24.42	0.0247	0.04	0.2	0.05	0.2	8	0.3	0.33
		3969	3969	304	20.3	17.23	0.0150	0.04	0.2	0.05	0.2	8	0.3	0.33
		4044	4044	584	39	16.14	0.0100	0.04	0.2	0.05	0.2	8	0.3	0.33
		3936	3936	208	13.9	16.75	0.0269	0.04	0.2	0.05	0.2	8	0.3	0.33
		3838	3838	561	37.4	14.95	0.0563	0.04	0.2	0.05	0.2	8	0.3	0.33
		3917	3917	131	8.8	27.35	0.0225	0.04	0.2	0.05	0.2	8	0.3	0.33
5/5OF	Combined	2756	2756	792	13.2	24.973	0.0093	0.04	0.2	0.05	0.2	8	0.3	0.33
		26691	2669	916	15.3	20.927	0.0090	0.04	0.2	0.05	0.2	8	0.3	0.33
		26692	2669	336	5.6	20.587	0.0188	0.04	0.2	0.05	0.2	8	0.3	0.33
		2541	2541	1386	23.1	19.3545	0.0410	0.04	0.2	0.05	0.2	8	0.3	0.33
		2343	2343	1618	27	18.581	0.0165	0.04	0.2	0.05	0.2	8	0.3	0.33
		2543	2543	969	16.2	24.1315	0.0261	0.04	0.2	0.05	0.2	8	0.3	0.33
		3590	3590	959	16	16.8895	0.0217	0.04	0.2	0.05	0.2	8	0.3	0.33
		37411	3741	1158	19.3	13.5575	0.0297	0.04	0.2	0.05	0.2	8	0.3	0.33
		37412	3741	1563	26.1	13.4555	0.0322	0.04	0.2	0.05	0.2	8	0.3	0.33
		6722	672	6905	115.1	11.3475	0.0333	0.04	0.2	0.05	0.2	8	0.3	0.33
		3627	3627	3939	65.6	18.9635	0.0221	0.04	0.2	0.05	0.2	8	0.3	0.33
		2497	2497	798	13.3	14.314	0.0595	0.04	0.2	0.05	0.2	8	0.3	0.33
		684	684	1491	24.8	15.2405	0.0382	0.04	0.2	0.05	0.2	8	0.3	0.33
		604	604	1855	30.9	22.1085	0.0107	0.04	0.2	0.05	0.2	8	0.3	0.33
		678	678	1641	27.4	21.4965	0.0175	0.04	0.2	0.05	0.2	8	0.3	0.33
		3529	3529	2337	39	15.283	0.0311	0.04	0.2	0.05	0.2	8	0.3	0.33
		3157	3157	4766	79.4	12.869	0.0284	0.04	0.2	0.05	0.2	8	0.3	0.33
		2390	2390	2850	47.5	15.521	0.0613	0.04	0.2	0.05	0.2	8	0.3	0.33
		3472	3472	498	8.3	23.8765	0.0900	0.04	0.2	0.05	0.2	8	0.3	0.33
		3206	3206	622	10.4	17.935	0.0342	0.04	0.2	0.05	0.2	8	0.3	0.33
	Separated	37413	3741	15533	258.9	1.69	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		6721	672	3151	52.5	1.33	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
6	Combined	1517	1517	1467	29.3	9.786	0.0475	0.04	0.2	0.05	0.2	8	0.3	0.33
		1602	1602	938	18.8	5.964	0.0475	0.04	0.2	0.05	0.2	8	0.3	0.33
		633	633	243	4.9	16.996	0.0185	0.04	0.2	0.05	0.2	8	0.3	0.33
		1997	1997	1306	26.1	14.532	0.0764	0.04	0.2	0.05	0.2	8	0.3	0.33
		639	639	1366	27.3	5.824	0.0250	0.04	0.2	0.05	0.2	8	0.3	0.33

EXHIBIT G
AR J.1

TABLE 5-1. SUMMARY OF RUNOFF BASIN CHARACTERISTICS

Meter	C or S	Catchment ID	Input Location	Width (ft)	Area (acres)	Percent Imp.	Slope (ft/ft)	Manning's Imp.	Manning's Perv.	Dep. Stor. Imp.	Dep. Stor. Perv.	Green-Ampt Suct.	Green-Ampt Hyd.Con.	Green-Ampt Air/Void
6	Combined	1412	1412	721	14.4	11.676	0.0312	0.04	0.2	0.05	0.2	8	0.3	0.33
		1555	1555	526	10.5	15.68	0.0370	0.04	0.2	0.05	0.2	8	0.3	0.33
		19361	1936	528	10.6	10.85	0.0449	0.04	0.2	0.05	0.2	8	0.3	0.33
		19362	1936	922	18.4	15.246	0.0421	0.04	0.2	0.05	0.2	8	0.3	0.33
		1960	1960	215	4.3	23.506	0.0289	0.04	0.2	0.05	0.2	8	0.3	0.33
		2151	2151	296	5.9	30.044	0.0230	0.04	0.2	0.05	0.2	8	0.3	0.33
		2410	2410	438	8.8	24.71	0.0219	0.04	0.2	0.05	0.2	8	0.3	0.33
		3123	3123	710	14.2	27.636	0.0043	0.04	0.2	0.05	0.2	8	0.3	0.33
		2624	2624	1084	21.7	31.248	0.0266	0.04	0.2	0.05	0.2	8	0.3	0.33
		2048	2048	353	7.1	20.468	0.0203	0.04	0.2	0.05	0.2	8	0.3	0.33
		2293	2293	605	12.1	27.342	0.0428	0.04	0.2	0.05	0.2	8	0.3	0.33
		2538	2538	245	4.9	25.088	0.1089	0.04	0.2	0.05	0.2	8	0.3	0.33
		2911	2911	495	9.9	30.31	0.0158	0.04	0.2	0.05	0.2	8	0.3	0.33
		30003	3000	1609	32.2	24.57	0.0227	0.04	0.2	0.05	0.2	8	0.3	0.33
		30004	3000	146	2.9	27.146	0.0171	0.04	0.2	0.05	0.2	8	0.3	0.33
		1391	1391	1803	36.1	12.516	0.0437	0.04	0.2	0.05	0.2	8	0.3	0.33
		1780	1780	1374	27.5	11.466	0.0194	0.04	0.2	0.05	0.2	8	0.3	0.33
		1670	1670	2591	51.8	15.4	0.0289	0.04	0.2	0.05	0.2	8	0.3	0.33
	Separated	30001	3000	423	8.5	2.814	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		601	601	354	7.1	3.836	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		30002	3000	128	2.6	4.466	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
6OF	Combined	1327	1327	1982	39.6	11.242	0.0605	0.04	0.2	0.05	0.2	8	0.3	0.33
		1187	1187	2104	42.1	9.688	0.0572	0.04	0.2	0.05	0.2	8	0.3	0.33
		1189	1189	598	12	12.096	0.0329	0.04	0.2	0.05	0.2	8	0.3	0.33
		1310	1310	603	12.1	12.782	0.0432	0.04	0.2	0.05	0.2	8	0.3	0.33
		1182	1182	1170	23.4	10.612	0.0290	0.04	0.2	0.05	0.2	8	0.3	0.33
		11982	1198	529	10.6	11.354	0.0395	0.04	0.2	0.05	0.2	8	0.3	0.33
		11981	1198	629	12.6	7.378	0.0498	0.04	0.2	0.05	0.2	8	0.3	0.33
		12081	1208	446	8.9	8.302	0.0532	0.04	0.2	0.05	0.2	8	0.3	0.33
		12082	1208	618	12.4	10.808	0.0424	0.04	0.2	0.05	0.2	8	0.3	0.33
		1226	1226	616	12.3	7.896	0.0836	0.04	0.2	0.05	0.2	8	0.3	0.33
		12171	1217	505	10.1	9.646	0.0668	0.04	0.2	0.05	0.2	8	0.3	0.33
		12172	1217	599	12	10.164	0.0487	0.04	0.2	0.05	0.2	8	0.3	0.33
		1223	1223	292	5.8	10.024	0.0649	0.04	0.2	0.05	0.2	8	0.3	0.33
		1439	1439	1616	32.3	7.126	0.0385	0.04	0.2	0.05	0.2	8	0.3	0.33
		1844	1844	1459	29.2	11.648	0.0286	0.04	0.2	0.05	0.2	8	0.3	0.33
		1787	1787	1005	20.1	5.376	0.0473	0.04	0.2	0.05	0.2	8	0.3	0.33
7/7OF	Combined	2235	2235	3100	20.9	7.22	0.0208	0.04	0.2	0.05	0.2	8	0.3	0.33
		2317	2317	2500	16.4	3.67	0.1274	0.04	0.2	0.05	0.2	8	0.3	0.33
		2355	2355	1100	7.3	5	0.1164	0.04	0.2	0.05	0.2	8	0.3	0.33
		2597	2597	1900	12.4	6.33	0.0528	0.04	0.2	0.05	0.2	8	0.3	0.33
		2485	2485	400	2.6	8.02	0.0208	0.04	0.2	0.05	0.2	8	0.3	0.33
		2590	2590	1600	10.8	4.91	0.0300	0.04	0.2	0.05	0.2	8	0.3	0.33
9/9OF	Combined	1982	1982	125	10.4	16.31	0.0873	0.04	0.2	0.05	0.2	8	0.3	0.33
		1947	1947	198	16.5	16.67	0.0671	0.04	0.2	0.05	0.2	8	0.3	0.33
		1925	1925	180	15	23.95	0.0312	0.04	0.2	0.05	0.2	8	0.3	0.33
		1881	1881	156	13	15.06	0.0892	0.04	0.2	0.05	0.2	8	0.3	0.33
12	Combined	1733	1733	292	9.7	10	0.0297	0.04	0.2	0.05	0.2	8	0.3	0.33
		1998	1998	275	9.2	15.99	0.0609	0.04	0.2	0.05	0.2	8	0.3	0.33
		1885	1885	198	6.6	13.33	0.0486	0.04	0.2	0.05	0.2	8	0.3	0.33
		1838	1838	164	5.5	10.36	0.0280	0.04	0.2	0.05	0.2	8	0.3	0.33
		2181	2181	604	20.1	21	0.0601	0.04	0.2	0.05	0.2	8	0.3	0.33
		2223	2223	304	10.1	20.11	0.0496	0.04	0.2	0.05	0.2	8	0.3	0.33

TABLE 5-1. SUMMARY OF RUNOFF BASIN CHARACTERISTICS

Meter	C or S	Catchment ID	Input Location	Width (ft)	Area (acres)	Percent Imp.	Slope (ft/ft)	Manning's Imp.	Manning's Perv.	Dep. Stor. Imp.	Dep. Stor. Perv.	Green-Ampt Suct.	Green-Ampt Hyd.Con.	Green-Ampt Air/Void
12	Combined	2202	2202	143	4.8	19.96	0.0815	0.04	0.2	0.05	0.2	8	0.3	0.33
		2233	2233	109	3.6	24.06	0.0161	0.04	0.2	0.05	0.2	8	0.3	0.33
		2200	2200	366	12.2	16.84	0.0697	0.04	0.2	0.05	0.2	8	0.3	0.33
		2285	2285	134	4.5	28.69	0.0150	0.04	0.2	0.05	0.2	8	0.3	0.33
		658	658	252	8.4	25.93	0.0390	0.04	0.2	0.05	0.2	8	0.3	0.33
21	Combined	52171	5217	23	14.9	1.83	0.0245	0.04	0.2	0.05	0.2	8	0.3	0.33
		52172	5217	15	9.8	2.05	0.0731	0.04	0.2	0.05	0.2	8	0.3	0.33
		50504	5050	108	71.6	5.08	0.0380	0.04	0.2	0.05	0.2	8	0.3	0.33
	Separated	50501	5050	79	52.1	0.24	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		50502	5050	1499	991.1	0.22	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		50503	5050	20	13.3	0.23	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		4919	4919	126	83.3	0.33	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		52173	5217	2994	1979.4	0.21	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
22	Separated	21771	2177	5857	2928.6	0.27	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		21772	2177	1867	933.4	0.23	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
23	Separated	849	849	379	946.7	0.11	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
24	Separated	5719	5719	522	1303.8	0.58	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
25	Combined	14011	1401	179	24.5	8.07	0.0267	0.04	0.2	0.05	0.2	8	0.3	0.33
		1671	1671	150	20.5	11.6	0.0676	0.04	0.2	0.05	0.2	8	0.3	0.33
		1674	1674	49	6.7	13.85	0.0365	0.04	0.2	0.05	0.2	8	0.3	0.33
		1471	1471	110	15.1	9.52	0.0265	0.04	0.2	0.05	0.2	8	0.3	0.33
		12831	1283	603	82.6	7.37	0.0598	0.04	0.2	0.05	0.2	8	0.3	0.33
		12832	1283	361	49.5	11.8	0.0427	0.04	0.2	0.05	0.2	8	0.3	0.33
		2399	2399	173	23.6	17.3	0.0843	0.04	0.2	0.05	0.2	8	0.3	0.33
		2377	2377	45	6.1	28.61	0.0467	0.04	0.2	0.05	0.2	8	0.3	0.33
	Separated	14012	1401	80	10.9	0.17	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		14013	1401	85	11.7	0.49	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33
		12833	1283	589	80.6	0.27	0.0001	0.04	0.2	0.05	0.2	8	0.3	0.33

Key to Table Headings:

C or S	Combined or separated tributary area
Catchment ID	Unique name for delineated basin
Input Location	MOUSE node runoff flow is directed to
Width (ft)	Width of basin
Area (acres)	Area of basin
Percent Imp.	Percent of basin area that is impervious
Slope (ft/ft)	Slope of the basin
Manning's Imp.	Manning's n value assigned to impervious surfaces
Manning's Perv.	Manning's n value assigned to pervious surfaces
Dep. Stor. Imp.	Measure of rainfall depth abosrbed or stored by impervious surfaces, before runoff can occur
Dep. Stor. Perv.	Measure of rainfall depth abosrbed or stored by pervious surfaces, before runoff can occur
Green-Ampt Suct.	Average capillary suction of water into soil
Green-Ampt Hyd.Con.	Hydraulic conductivity
Green-Ampt Air/Void	Porosity of the soil

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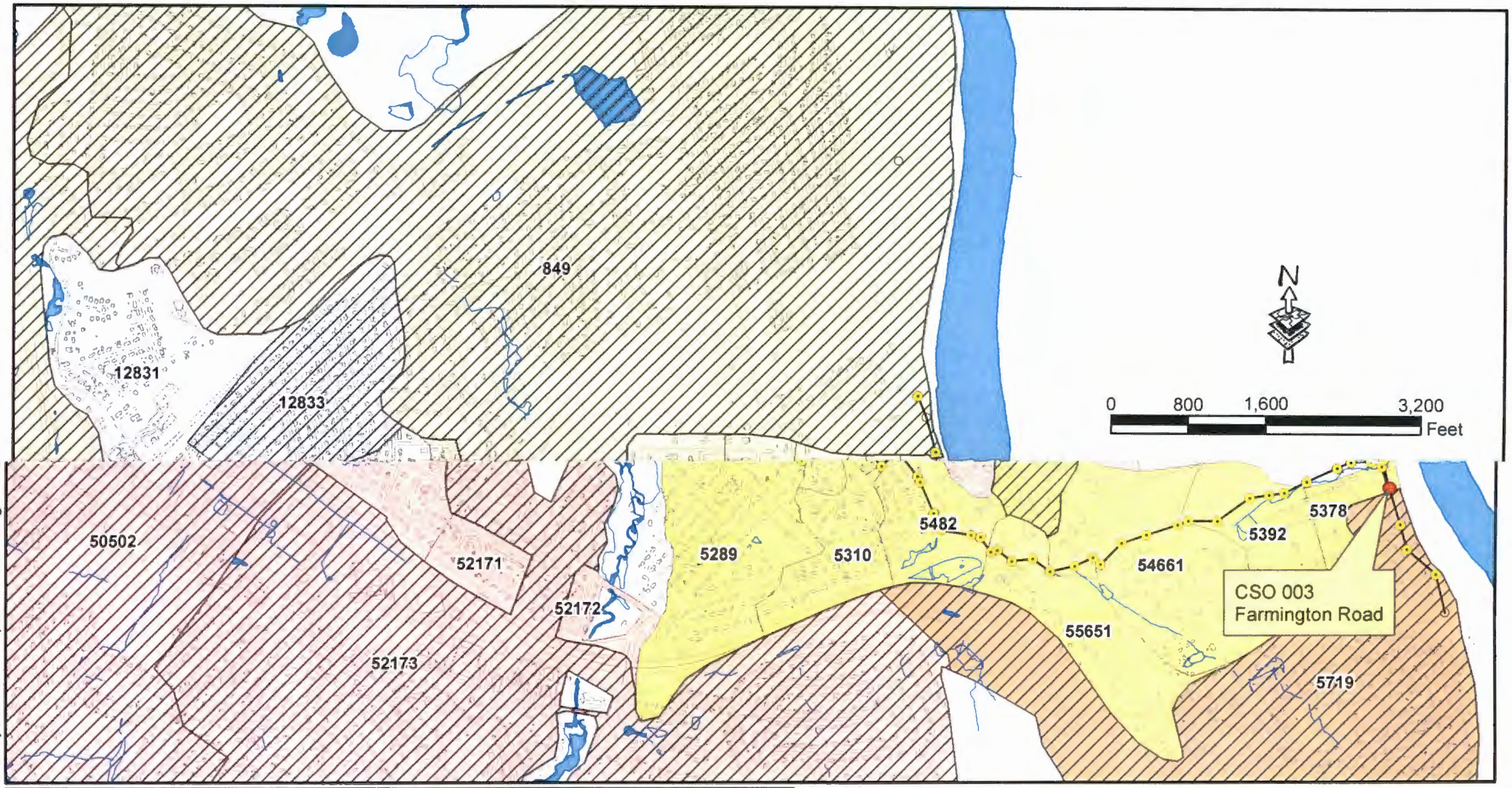

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Legend

- Modeled Manholes
- Modeled Pipe Segments
- CSO Location
- NWTF
- Areas Tributary to 002
- Areas Tributary to 003
- Areas Tributary to 004
- Areas Tributary to 005
- Areas Tributary to 006
- Areas Tributary to 007
- Areas Tributary to 008
- Areas Tributary to 009
- Areas Tributary to 012
- Areas Tributary to NWTF
- Separate Areas
- Rivers, Streams, & Ponds

Data Sources:
Data provided by City of Nashua, New Hampshire - 2001

Coordinate System:
Projection: Stateplane
Datum: NAD83
Fipszone: 2800
Units: Feet

Figure 5-2.
CSO Locations
with Tributary Basins

Pervious area parameters

Those portions of the basins not identified as impervious were considered to be pervious for modeling purposes. A small portion of this area modeled as pervious is actually impervious area not directly connected to the pipe system. For example, water landing on pavement completely surrounded by grass will eventually run off and infiltrate the ground. In the RUNOFF model, rainfall that lands on pervious areas infiltrates the soil. The rate at which this occurs depends on several factors including soil type and degree of saturation.

The Green-Ampt equation was used to simulate the rate at which rain infiltrates in the pervious areas. The Green-Ampt equation is a physically based model that describes the infiltration process on pervious surfaces (USEPA SWMM4 Users Manual, 2000). Since this equation was used in the previous modeling work, it was also used in the reevaluation.

Depression Storage

At the beginning of a storm, not all of the water landing on impervious areas will reach the drainage system. Depressions in the ground, such as potholes, will store a small amount of water before generating any runoff flows. Although a single depression may be insignificant, the cumulative impact of many such depressions can be noticeable.

For the Nashua model, depression storage values of 0.05 inches for impervious areas and 0.2 inches for pervious areas were used, based on prior modeling work. In other words, the first 0.05 inches of rain landing on the impervious areas of the basin will be stored in depressions. During a long rainfall period, water in the depressions evaporates. The evaporation rates used in the model came from previous modeling of the Nashua system, and are typical of evaporation rates used in New England.

Other Parameters

In SWMM RUNOFF, the basin width, land slope, and Manning's n of the pervious and impervious areas are used to calculate the flow entering the combined sewer system. SWMM treats each basin as a channel of a specified width, length, and area. Flow is then calculated as open channel flow using Manning's equation. These parameters were calculated as follows:

- Basin areas were calculated in GIS.
- Land slopes were also calculated in GIS using Spatial Analyst. Land slopes were derived using digital elevation model (DEM) provided by the USGS.
- Basin widths were estimated, then adjusted during model calibration until modeled peak flows matched metered flows.
- The Manning's n of the impervious area was assumed to be 0.04, and the Manning's n of the pervious area was assumed to be 0.2. These numbers were obtained from previous modeling and are similar to typical values used for urban areas such as Nashua.

Model Network

The MOUSE model constructed for the Nashua project included approximately 600 nodes and 625 pipes in the combined areas of the system. The downstream boundary condition was the Nashua NWTF, which was simulated as a series of pump stations discharging to outfalls as described later in this section. Pipes extended upstream of all regulators and included all major interceptors and most pipes 24-inch diameter and larger in the combined area. By including a significant portion of the pipe network upstream of the regulators, the runoff areas could be broken into smaller pieces, which allows for a mode accurate simulation of hydrology. A map of the model network is shown on Figure 5-3. As a comparison, the previous model included only 121 nodes and 129 pipes (refer to Figure 5-1).

In MOUSE, the modeler must define the pipe and manhole parameters. For pipes, this includes information on the connecting manholes, the upstream and downstream inverts (if not the same

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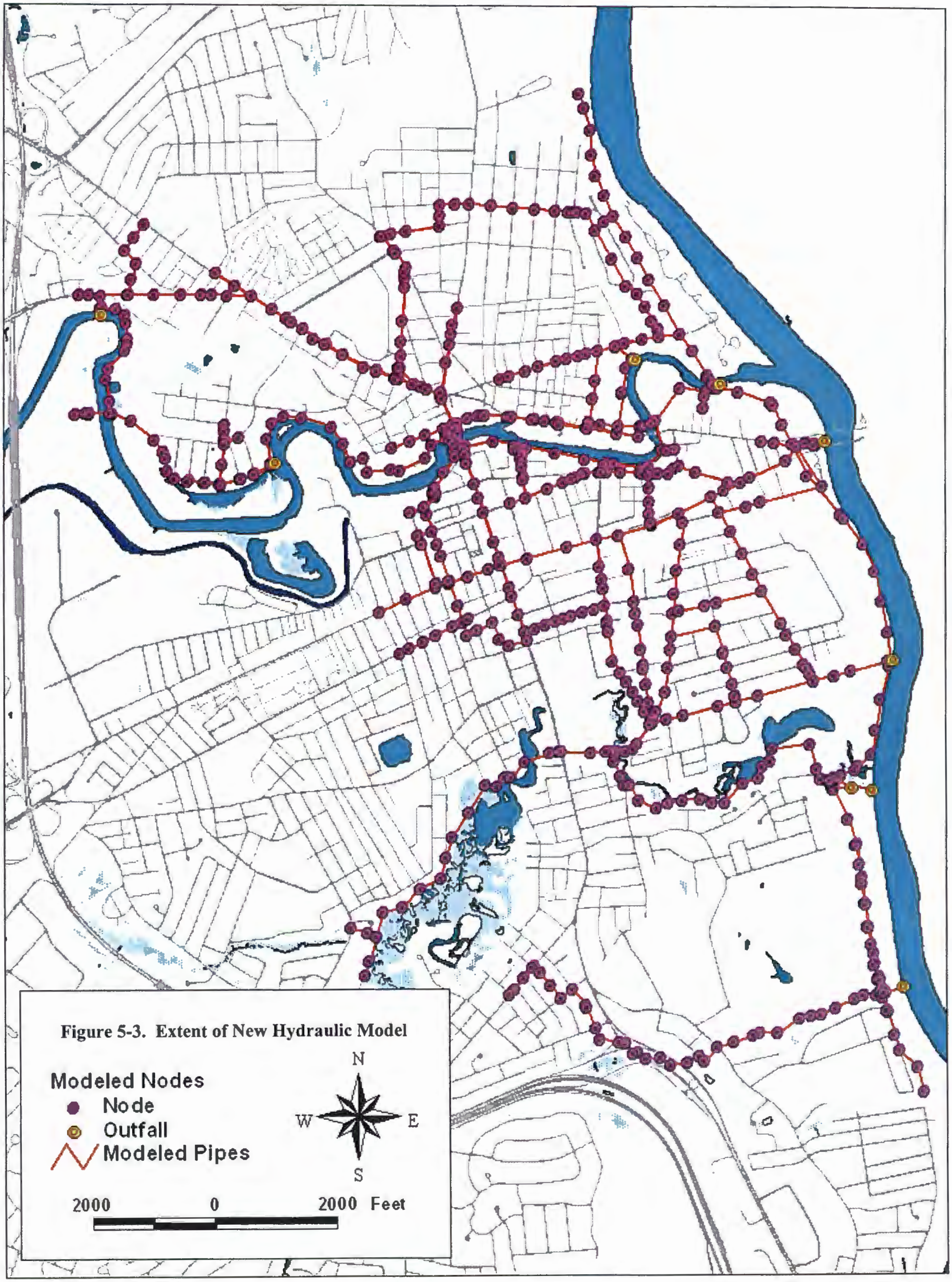


Figure 5-3. Extent of New Hydraulic Model

as the manhole inverts); as well as pipe shape, size, and material. For pipe manholes, the user needs to specify manhole locations, inverts, rim elevations, and manhole diameters in the model.

The information used in the model came from several sources. Data provided by the city of Nashua were analyzed and incorporated in the model. In addition, record drawings were used to obtain information on manhole locations, pipe inverts, sizes, shapes, and connectivity. It was assumed that a Manning's n value of 0.015 (rough concrete) would be suitable for most pipes in the system. This is a number typically used when modeling pipe systems, because the higher n value provides an allowance for headlosses due to manholes and aging pipes.

Elevated Manning's n values were also applied to the conduits representing dry weather connections. Elevated n values simulated the headlosses associated with the small diameter connection into the large diameter interceptor. Convergence to the calibrated n value was accomplished through iterations of the model calibration.

For modeling purposes, it was also assumed that the manhole diameters were equal to the diameter of the largest pipe connecting to the manhole, with a minimum manhole diameter of 4 feet.

Dry Weather Inflow

A dry weather flow analysis was performed on the existing meter data as described in Chapter 3. The sanitary flow, infiltration, and diurnal curve were developed for each metering location based on the meter data, and input into the model. Both sanitary flow and infiltration for each meter were proportioned to each tributary basin by area. For example, the portion of sanitary flow and infiltration contributed by one sub basin within a larger metering basin would be equal to the proportion of the sub basin area, compared to the total metering basin area – 20 percent of the land area contributes 20 percent of the flow.

Boundary Conditions

Boundary conditions were specified for both the upstream extents of the modeled sewer interceptors and for the NWTF, at the downstream end of the modeled extent.

Upstream Boundary Upstream boundary conditions were developed from flow data provided by meter M-21 on the SBI, meter M-22 on the NRI, meter M-23 on the NMRI, and meter M-24 on the SMI.

Downstream Boundary The NWTF was used as the downstream boundary for the model. A model schematic depicting the treatment facility as represented in MOUSE can be found in Figure 5-4. The primary pipe into the facility is the 72-inch diameter NMRI. Flow from the SBI and SMI combine just outside of the headworks and connect with the NMRI prior to screening. Once in the headworks, three pumps with a total capacity of 77.35 cfs (50 MGD) send flow to one of two modeled outfalls – NWTF outfall and the bypass outfall. On/Off elevations for the pumps were determined based on discussions with treatment facility operators, and are summarized in Table 5-2.

TABLE 5-2. MODEL CHARACTERISTICS OF NWTF

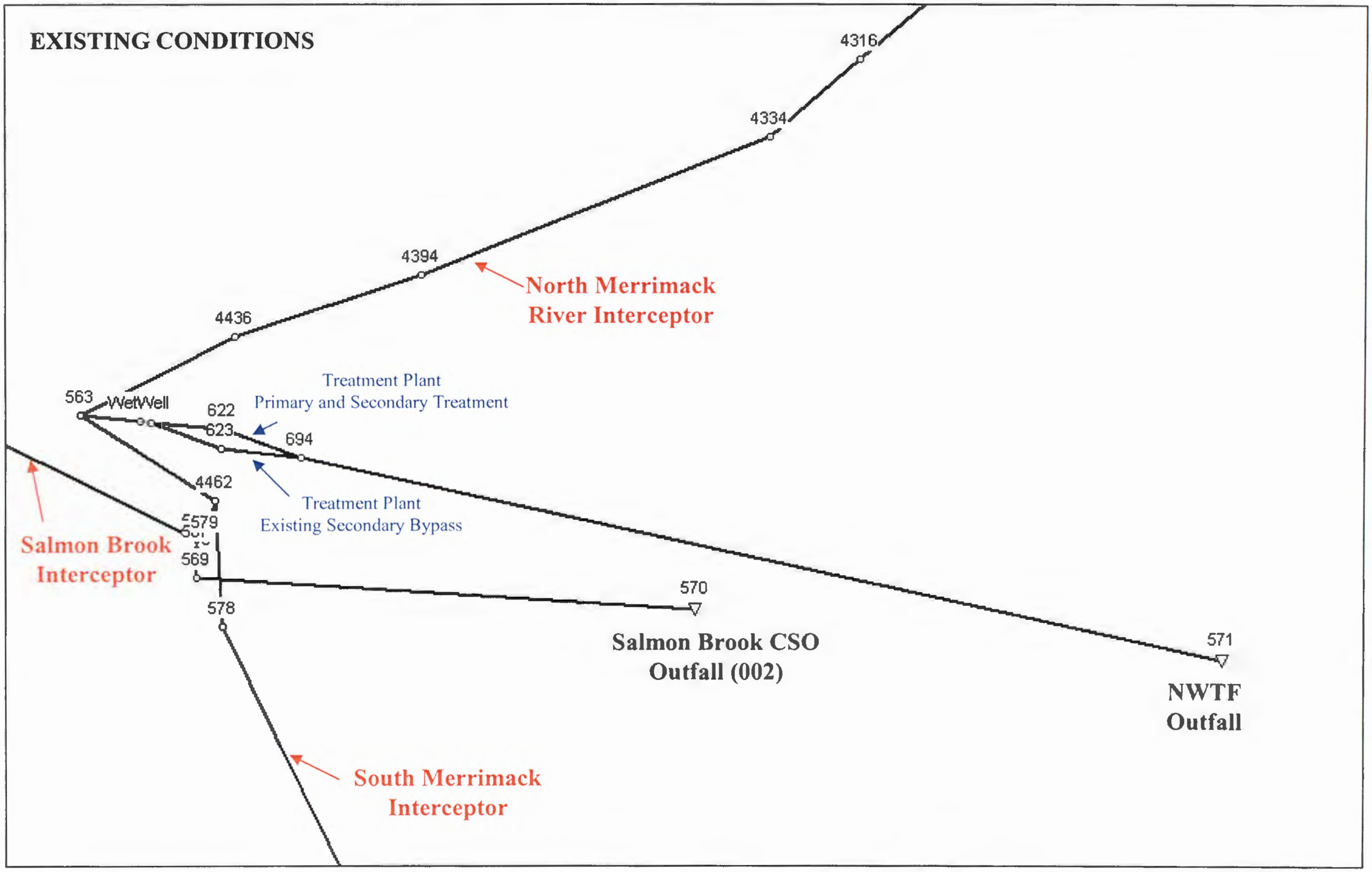
Pump	Capacity	On Elevation* (ft)	Off Elevation (ft)
1	30.94 cfs (20 MGD)	-0.1	-4.6
2	27.85 cfs (18 MGD)	1.4	-1.6
3	18.56 cfs (12 MGD)	2.9	-1.6

*Elevations based on NCD

MODEL CALIBRATION

The model was calibrated by comparing the computed results with flow measurements taken from all of the flowmeters used during the metering period. The model was adjusted until satisfactory agreement between computed and measured values were obtained.

FIGURE 5-4. MODEL SCHEMATIC OF NWTF



When discrepancies between measurements and calculations were observed, possible causes were investigated and the model was adjusted accordingly. In some cases, the discrepancies were due to incorrect regulator weir elevations, pipe sizes, connections or other system features, caused by inaccuracies in the database and drawings used to develop the model. These were investigated by contacting knowledgeable individuals, inspecting the site, and reviewing record drawings. In other cases, minor adjustments were made to model parameters, such as basin width, until the model results match the meter results.

Dry Weather Calibration

The model was first calibrated for dry weather conditions. This was accomplished by dividing the dry weather flow from the study area into two components; sanitary flow (which followed a diurnal pattern) and infiltration (which was constant), as described previously in Chapter 3. These flows were then proportioned to the model nodes based on the area tributary to each node. In a few cases, the Manning's n values for stretches of pipe and dry weather connections at regulators were increased to match measured head values.

Field observations indicate that during the spring, water in the Nashua and Merrimack Rivers rises high enough to cause the rivers to flow into the combined system. This phenomenon was not observed during the calibration period, and it was assumed that during simulations, the water elevation in the rivers did not get high enough to influence flow in the combined system.

Wet Weather Calibration

After the model was calibrated to dry weather conditions, it was calibrated for three discrete wet weather events that occurred in June of 2001. Though these wet weather events were modeled as discrete storms in the MOUSE model, the SWMM RUNOFF model was run as a continuous simulation for the entire month of June. This was done so that the antecedent conditions can be more accurately accounted for. In the SWMM RUNOFF model, the amount of runoff from pervious areas is dependent on how saturated the soils are. During the beginning of a storm event, the soil conditions may vary depending on how wet, or dry, the period before the storm event was. By simulating the entire monitoring period, this phenomenon is more accurately taken into account.

Calibration storms. As explained in Chapter 4, the output of SYNOP identified 22 different storm events during the 3-month metering period. During the month of June, seven storms occurred, ranging from 0.03 inches to 2.50 inches in volume as measured by the temporary rain gage installed for the calibration period. Table 5-3 lists the storms that were identified for model calibration.

TABLE 5-3. WET WEATHER CALIBRATION EVENT SUMMARY

	Storm 1 6/2/02	Storm 2 6/11/02	Storm 3 6/17/02
Volume (in.)	2.50	1.06	2.36
Peak Intensity (in/hr)	0.39	0.26	1.39
Average Intensity (in/hr)	0.06	0.07	0.13
Duration (hours)	39	15	18

2001

The model was adjusted in order to match computed flows and elevations to metered values. Minor adjustments to the widths and percent impervious values were made for several basins within the project area. The model also was adjusted to account for other discrepancies discovered during the wet weather calibration, including the Manning's n value of the dry weather connection in several regulators was adjusted as mentioned previously.

Model parameters were adjusted until computed flows and elevations reasonably matched metered values. Decisions on calibration were made after comparing modeled flow to metered flow, as shown in Table 5-4, and after reviewing calibration plots. Examples of calibration plots for both head and flow for meter M-4 are shown in Figure 5-5 and 5-6 and M-4OF are shown in Figures 5-7 and 5-8. A complete set of calibration plots can be found in Appendix B.

The calibrated model was then used to evaluate the performance of various control alternatives.

TABLE 5-4. COMPUTED VERSUS METERED VOLUMES

Meter Location	6/2/2001		6/11/2001		6/17/2001	
	Metered	Modeled	Metered	Modeled	Metered	Modeled
2	8.50	7.27	7.38	6.12	4.46	4.54
2OF	0.00	0.00	0.00	0.00	0.09	0.13
3	5.57	4.89	5.20	3.93	2.85	3.19
3OF	0.40	0.13	0.11	0.01	0.41	0.59
4	1.81	2.00	0.89	1.11	1.32	1.67
4OF	0.73	0.75	0.26	0.17	2.05	1.00
5	9.48	9.27	5.67	5.34	6.36	6.79
5OF	0.52	1.28	0.12	0.02	1.45	2.09
6	18.67	17.01	14.40	12.51	11.43	13.29
6OF	1.93	2.10	1.23	0.00	5.74	6.09
7	0.42	0.56	0.39	0.41	0.35	0.41
7OF	0.00	0.00	0.00	0.00	0.09	0.08
8	0.03	0.38	0.04	0.06	0.52	0.66
9	1.21	0.89	0.55	0.52	0.56	0.74
9OF	0.28	0.22	0.17	0.04	0.18	0.35
12	1.75	1.36	1.25	0.83	0.95	1.10
12OF	0.00	0.00	0.00	0.00	0.00	0.00
21	5.62	5.52	5.43	5.10	3.18	3.07
22	6.36	6.47	6.44	6.09	3.52	3.53
23	0.81	0.83	0.78	0.79	0.42	0.44
24	2.85	3.03	2.60	2.75	1.73	1.70
25	8.02	8.05	7.43	7.10	3.82	4.48

FIGURE 5-5. METERED VERSUS MODELED FLOW, METER M-4

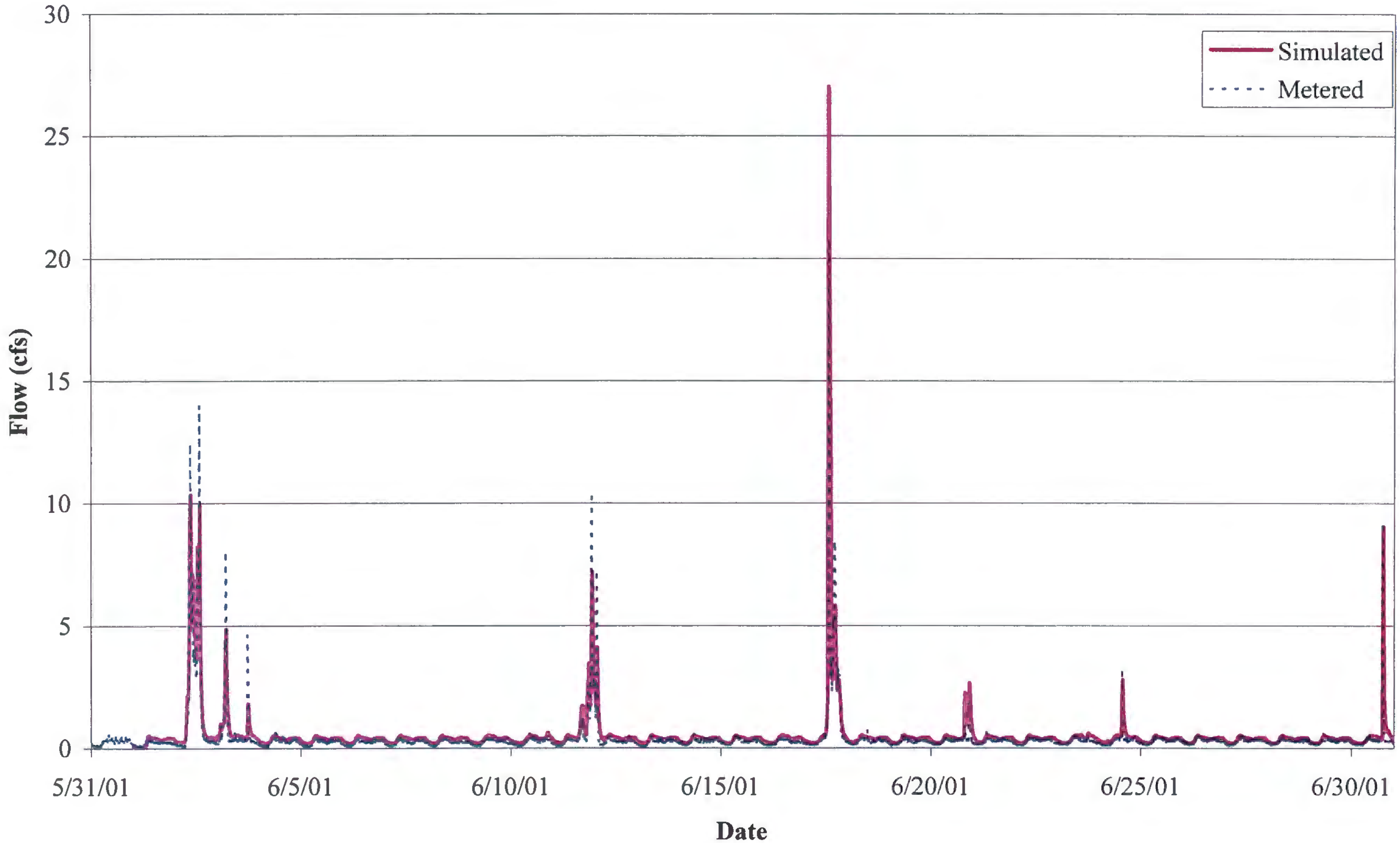


FIGURE 5-6. METERED VERSUS MODELED DEPTH, METER M-4

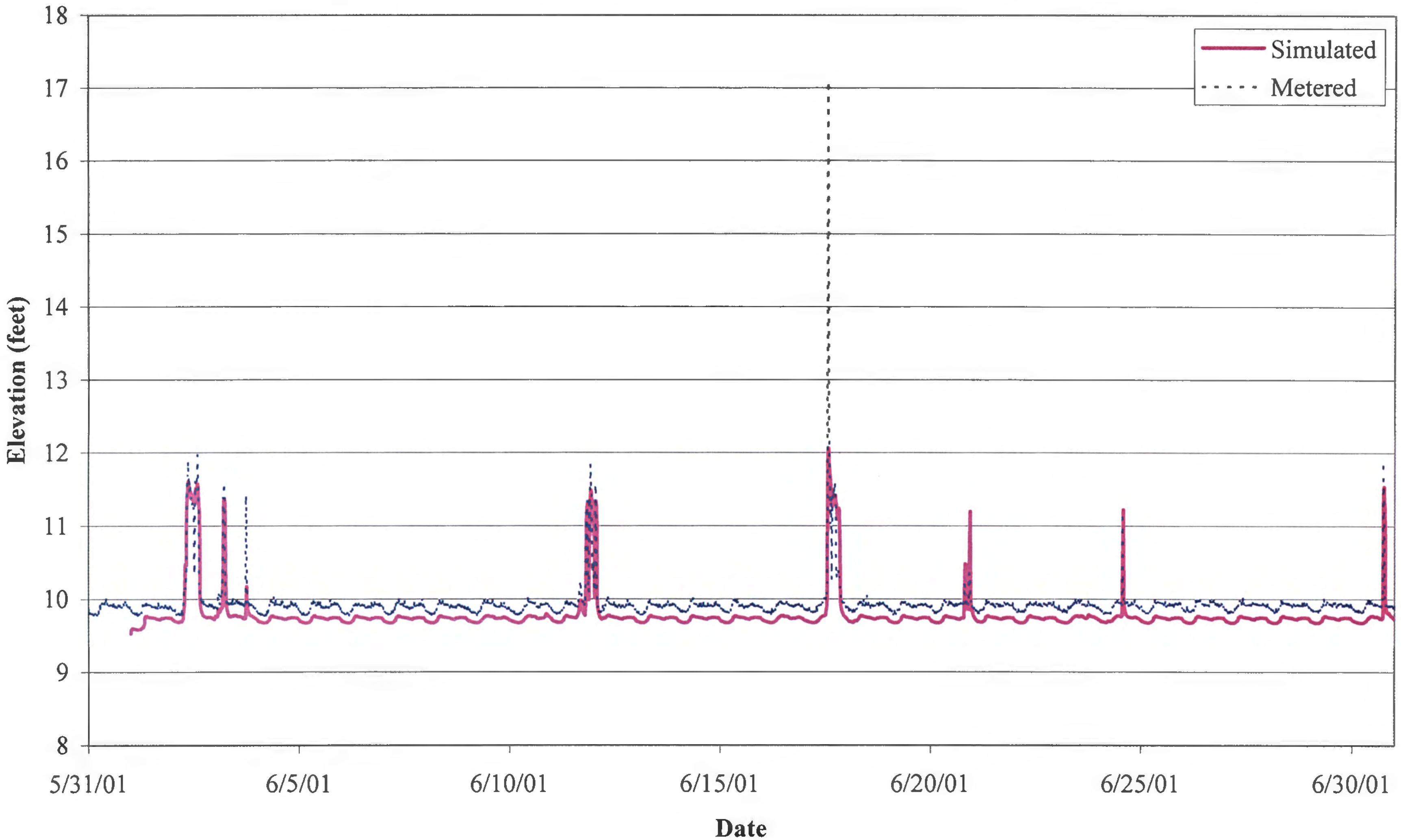


FIGURE 5-7. METERED VERSUS MODELED FLOW, METER M-4OF

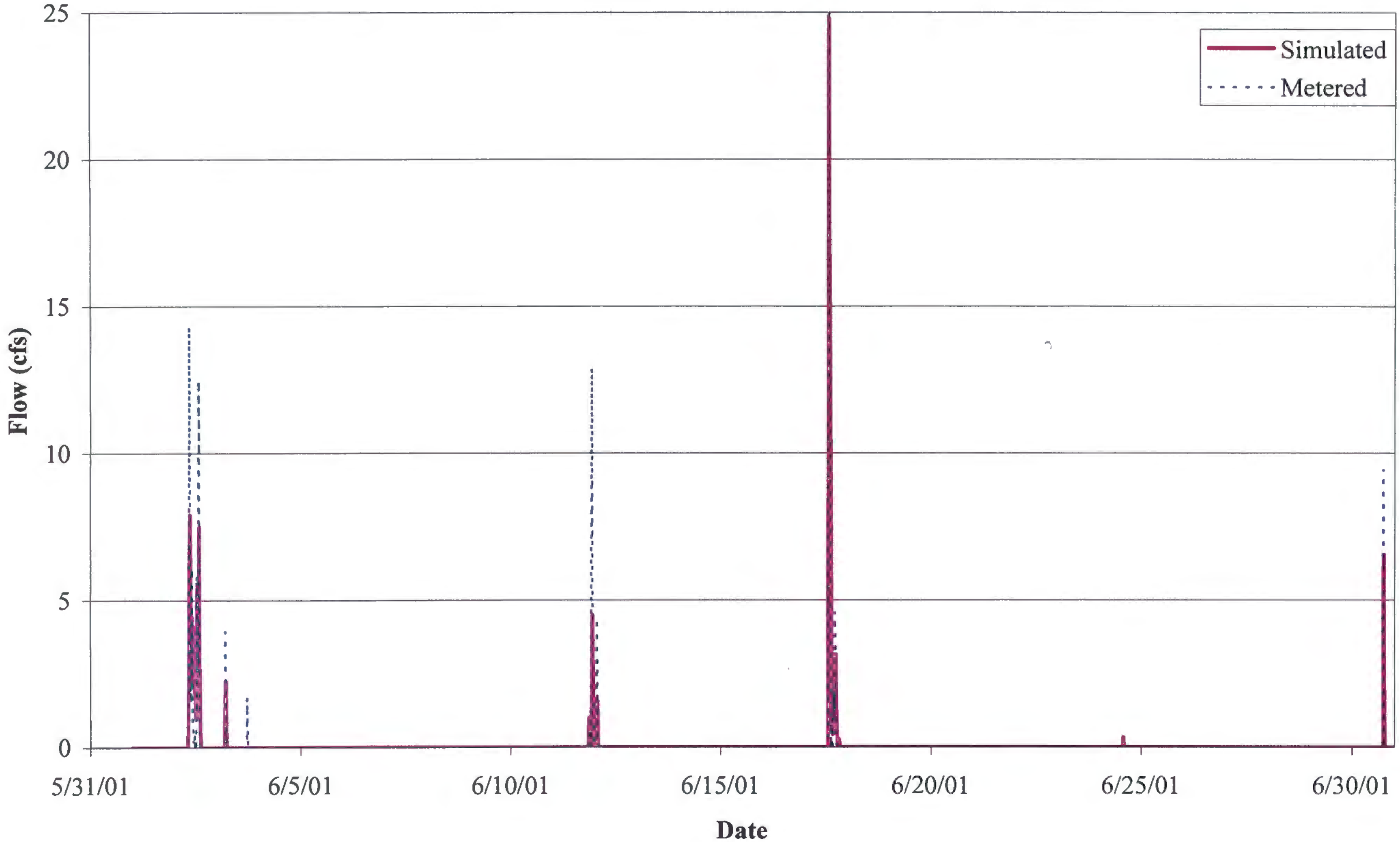
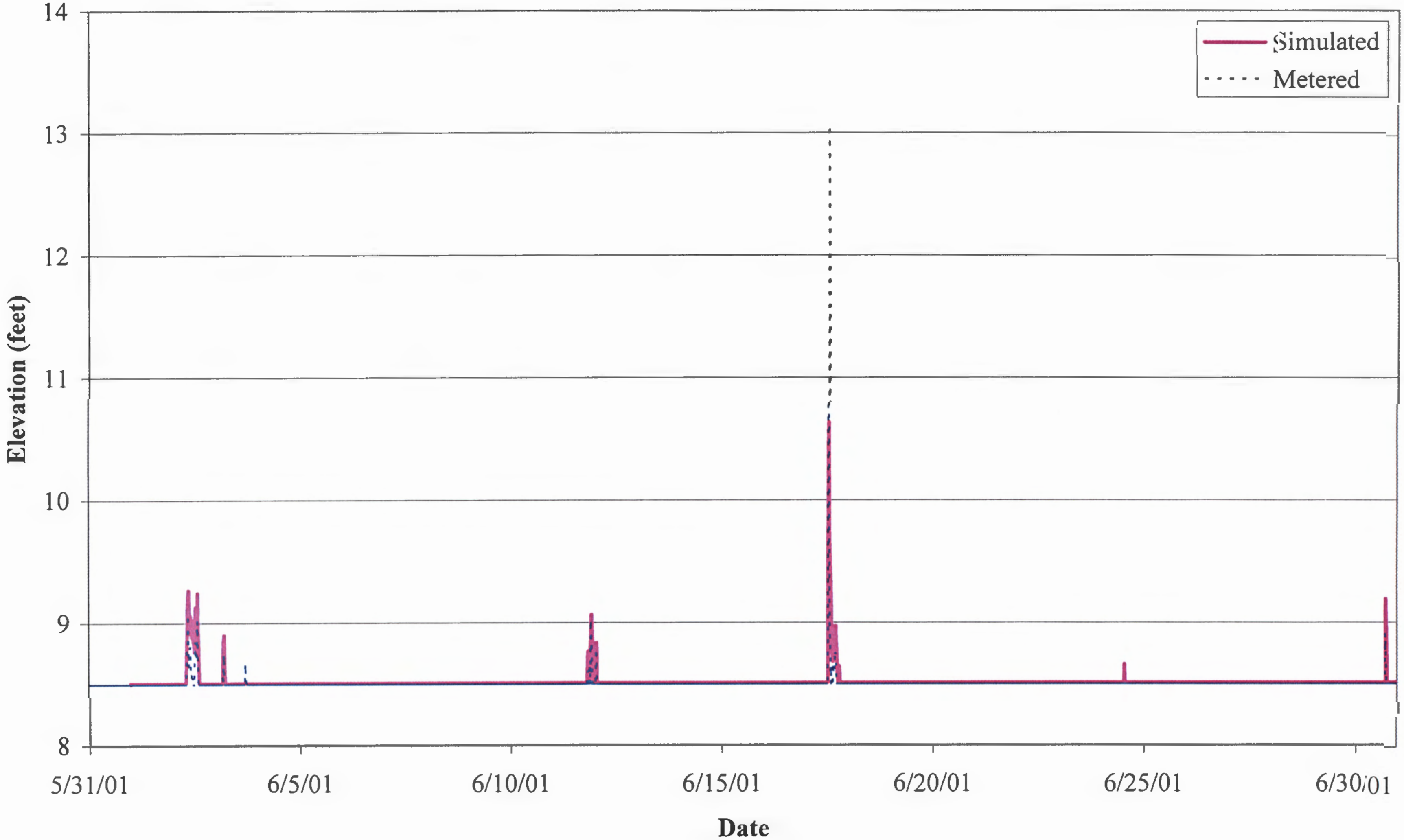


FIGURE 5-8. METERED VERSUS MODELED DEPTH, METER M-4OF



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CHAPTER 6

DEVELOPMENT OF BASELINE FLOWS AND LOADS

The baseline CSO flows and loads represent the existing conditions of the collection system, and provide a baseline. or reference point, from which to evaluate proposed CSO control alternatives. The baseline CSO flows were calculated from a calibrated hydraulic model of the collection system, using a typical year rainfall file and design storms. Baseline CSO loads were computed by multiplying these expected CSO volumes by average constituent concentrations found in samples taken during wet weather events.

DEVELOPMENT OF TYPICAL YEAR

The 1997 LTCP included a detailed analysis of long-term precipitation records collected from several rainfall monitoring stations in the Merrimack Valley. From this data, a single, long-term precipitation record was selected to represent typical annual rainfall.

The long-term precipitation record that was ultimately selected consisted of 5 sequential years of rainfall data (1973-1977) collected from NCDC Gauge No. 5705 located in Nashua. This five-year precipitation record closely reflected the long-term rainfall pattern observed at all of the rain gauges evaluated. Within this five-year precipitation record, each year contained an average of 83 storms, with an average depth of 43.7 inches of rainfall per year. The year 1975 had the most rain (50.7 inches), and 1976 had the least (35.3 inches).

The hydraulic model developed for this study is much more complex than the model used in the 1997 study. As a result, it would be very time consuming to run and evaluate a simulation using all five years of rainfall data. Therefore, a single year from the five year period was selected, and used to represent a typical year of rainfall for CSO modeling purposes.

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The rainfall record from 1974 was selected because the array of storm events within that year closely match the distribution of storm recurrence intervals for the five years of data. However,

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the 1974 precipitation record does not include any large storm where the total rainfall exceeds a 1-year recurrence storm event. Therefore, one large storm from 1973 that fit those characteristics (2.64 inches of rainfall) was added to the 1974 precipitation record. Some smaller sized storms were likewise eliminated from the 1974 precipitation record, so that the total rainfall for the year would not increase. A summary of the storms included in the modified 1974 rainfall file, used as the typical year in this assessment, can be found in Table 6-1.

CSO Performance in Typical Year Simulation

The calibrated model was run with the typical year rainfall record. Results of the simulation represent annual CSO performance, under baseline conditions, and are presented in Table 6-2.

Near Future Baseline Conditions

At the time of model calibration, the city of Nashua was nearing completion of several sewer separation projects within the city. Realizing that current conditions were subject to change in the near future, these sewer separation projects were incorporated into the baseline model configuration and run with the typical year rainfall file. The results of this simulation, coined the "Near Future Baseline Condition" (NFBC), are presented in Table 6-3. * *

The NFBC is the baseline condition against which all proposed CSO control alternatives have been compared. The three sewer separation projects included in the NFBC condition are described below.

1. **French Hill.** This project involved separating a combined area just north of the Nashua River. That area is bound on the east by Chandler Street, on the west by Concord Street, and on the north by Laton Street. Separation of this area effectively removed 66.34 acres of runoff from the Lock Street (CSO 009) tributary system. This project also included removing the plate from the Lock Street CSO regulator.

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TABLE 6-1. SUMMARY OF STORMS CONTAINED IN TYPICAL YEAR RAINFALL FILE (1974, MODIFIED)
(NCDC Gauge No. 5705, 1974)

Storm # No	Start Date	Start Hour	Duration (hours)	Rainfall (inches)	Intensity (in/hr) Average	Intensity (in/hr) Maximum	Interevent (hours)
1	01/01/1974	2	10	0.2	0.02	0.04	12
2	01/03/1974	21	5	0.06	0.01	0.02	57
3	01/09/1974	6	9	0.22	0.02	0.05	124
4	01/10/1974	7	30	0.59	0.02	0.06	16
5	01/16/1974	10	6	0.27	0.04	0.1	117
6	01/18/1974	21	10	0.32	0.03	0.05	53
7	01/21/1974	12	10	0.65	0.06	0.15	53
8	01/26/1974	22	8	0.35	0.04	0.1	120
9	01/28/1974	18	8	0.35	0.04	0.11	36
10	01/31/1974	17	6	0.09	0.01	0.03	63
11	02/02/1974	8	12	0.25	0.02	0.05	33
12	02/07/1974	0	9	0.32	0.04	0.06	100
13	02/19/1974	16	10	0.77	0.08	0.2	295
14	02/22/1974	8	19	0.58	0.03	0.14	54
15	03/03/1974	18	8	0.1	0.01	0.02	207
16	03/16/1974	6	23	2.34	0.1	0.66	292
17	03/19/1974	13	3	0.06	0.02	0.03	56
18	03/21/1974	11	25	1.75	0.07	0.43	43
19	03/24/1974	9	2	0.11	0.05	0.06	45
20	03/30/1974	0	31	0.47	0.02	0.12	133
21	04/01/1973	15	40	2.64	0.07	0.32	146
22	04/02/1974	11	5	0.2	0.04	0.11	52
23	04/04/1974	3	1	0.05	0.05	0.05	35
24	04/05/1974	12	5	0.27	0.05	0.1	32
25	04/08/1974	14	5	0.24	0.05	0.1	69
26	04/09/1974	8	2	0.32	0.16	0.19	13
27	04/12/1974	21	18	0.14	0.01	0.04	83
28	04/14/1974	22	5	0.75	0.15	0.4	31
29	04/23/1974	12	1	0.05	0.05	0.05	201
30	04/24/1974	4	4	0.16	0.04	0.06	15
31	04/29/1974	14	1	0.03	0.03	0.03	126
32	04/30/1974	20	9	0.95	0.11	0.28	29
33	05/03/1974	15	11	0.25	0.02	0.07	58
34	05/06/1974	19	7	0.17	0.02	0.09	65
35	05/09/1974	23	17	0.94	0.06	0.38	69
36	05/12/1974	11	13	0.9	0.07	0.43	43
37	05/17/1974	1	5	0.19	0.04	0.1	97
38	05/22/1974	23	19	0.71	0.04	0.18	137
39	05/24/1974	17	4	0.15	0.04	0.11	23
40	05/25/1974	13	20	0.35	0.02	0.1	16
41	05/28/1974	14	2	0.08	0.04	0.05	53
42	05/29/1974	6	7	0.1	0.01	0.03	14
43	06/01/1974	0	10	0.37	0.04	0.15	59
44	06/16/1974	12	17	1.3	0.08	0.28	362
45	06/21/1974	13	4	0.4	0.1	0.25	104
46	06/22/1974	13	1	0.05	0.05	0.05	20
47	06/25/1974	13	25	0.22	0.01	0.02	71
48	06/29/1974	0	2	0.05	0.02	0.03	58
49	07/03/1974	1	3	0.08	0.03	0.05	95
50	07/05/1974	14	7	0.49	0.07	0.2	58
51	07/07/1974	18	9	0.5	0.06	0.34	45
52	07/19/1974	0	1	0.07	0.07	0.07	261
53	07/19/1974	15	6	0.2	0.03	0.06	14
54	08/04/1974	13	6	0.26	0.04	0.16	376
55	08/08/1974	19	4	1.59	0.4	1.1	96
56	08/17/1974	19	3	0.12	0.04	0.05	212
57	08/23/1974	18	2	0.06	0.03	0.03	140
58	08/27/1974	20	7	0.8	0.11	0.55	96
59	08/29/1974	1	41	1.31	0.03	0.3	22
60	09/02/1974	7	3	0.2	0.07	0.12	61
61	09/03/1974	7	20	2.05	0.1	0.4	21
62	09/06/1974	22	12	0.72	0.06	0.14	67
63	09/13/1974	18	12	0.73	0.06	0.35	152
64	09/20/1974	18	3	0.21	0.07	0.16	156
65	09/21/1974	10	11	1	0.09	0.5	13
66	09/25/1974	21	2	0.05	0.02	0.03	96
67	09/28/1974	17	25	1.9	0.08	0.53	66
68	10/16/1974	0	27	1.55	0.06	0.12	390
69	10/31/1974	0	4	0.1	0.02	0.06	333
70	11/04/1974	0	5	0.2	0.04	0.09	92
71	11/04/1974	17	30	1.25	0.04	0.25	12
72	11/12/1974	5	26	0.28	0.01	0.07	150
73	11/14/1974	16	1	0.03	0.03	0.03	33
74	11/20/1974	10	35	1.01	0.03	0.14	137
75	11/25/1974	19	8	0.15	0.02	0.04	94
76	11/29/1974	6	1	0.02	0.02	0.02	75
77	12/02/1974	4	11	1.35	0.12	0.26	69
78	12/07/1974	19	3	0.15	0.05	0.12	124
79	12/08/1974	17	7	1.18	0.17	0.31	19
80	12/12/1974	11	4	0.07	0.02	0.02	83
81	12/14/1974	1	2	0.02	0.01	0.01	34
82	12/16/1974	16	12	1.15	0.1	0.18	61
83	12/21/1974	23	11	0.2	0.02	0.05	115
84	12/25/1974	5	12	0.6	0.05	0.15	67
85	12/31/1974	20	10	0.27	0.03	0.04	147

* Highlighted storm is 1973 event inserted into 1974 rainfall file.

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TABLE 6-2. CSO DISCHARGES PREDICTED FOR TYPICAL YEAR (1974, MODIFIED)

NPDES Discharge Number	CSO Name	No. of Activations During Year	Overflow volume during largest storm event (MG)	Total Annual Overflow Volume (MG)	
002	Salmon Brook	0	0.00	0.00	✓ M
003	Farmington Rd.	17	0.46	1.42	✓ M
004	Burke St.	30	0.72	5.53	✓ M
005	E. Hollis St.	17	1.58	6.32	✓ m
006	Nashua River	9	3.90	11.10	N
007	Tampa St.	2	0.07	0.09	N
008	Broad St.	23	0.50	2.89	N
009	Lock St.	26	0.25	1.53	N
		Total	**7.62**	**28.88**	

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TABLE 6-3. CSO DISCHARGES PREDICTED FOR TYPICAL YEAR NEAR FUTURE BASELINE CONDITION (1974, MODIFIED)

NPDES Discharge Number	CSO Name	No. of Activations During Year	Overflow volume during largest storm event (MG)	Total Annual Overflow Volume (MG)
002	Salmon Brook	0	0.00	0.00
003	Farmington Rd.	17	0.46	1.41
004	Burke St.	25	0.72	5.53
005	E. Hollis St.	17	1.54	5.76
006	Nashua River	10	3.47	9.59
007	Tampa St.	2	0.07	0.09
008	Broad St.	23	0.50	2.89
009	Lock St.	20	0.10	0.38
		Total	**6.86**	**25.65**

2. **South Main Street.** This project involved sewer separation work in the vicinity of South Main Street, Robinson Road, and Daniel Webster Highway. Separation of this area effectively removed 53.65 areas of surface runoff from the Farmington Road (CSO 003) tributary system.

3. **Temple/Pearl Street.** This project entailed separating an area in the vicinity of Temple and East Pearl Streets. The area is south of the Nashua River, bounded on the east by Spring Street, on the west by South Street and on the south by East Hollis Street. Separation of this area effectively removed 47.37 acres of runoff from the sewer system, resulting in a slight reduction in overflows at the East Hollis Street and Nashua River CSOs (CSO 005 and CSO 006).

For all three of these separation projects, removal of 80 percent of the storm induced inflow was considered complete separation. Considering the age of the infrastructure and the difficulty of disconnecting all roof drains, it has been assumed that 20 percent of the storm runoff will remain in the system. Details of sewer separation modeling are discussed in Chapter 10.

DEVELOPMENT OF DESIGN STORMS

In the 1997 LTCP, actual storm events from the long-term precipitation record that fit the criteria of recurring once every two weeks to 10 years were introduced as the design storms. These design storms, in addition to the annual simulation, were used in the LTCP to evaluate and size CSO control alternatives. To maintain consistency with the 1997 LTCP, the same design storms have been used in this project. A summary of the characteristics of the design storms used is presented in Table 6-4.

CSO Performance

The calibrated collection system model, configured for NFBC, was run with the design storms described above as inputs. It is generally expected that the greater the rainfall for the design storm, the greater the CSO discharge. However, review of Table 6-5 show the CSO volumes for the 2-year and 5-year design storms to be nearly equal. This anomaly was due to differences in timing of the peak rainfall intensity.

As observed in Figure 6-1, the peak intensity for the 10-year, 1-year and 6-month storms occurred near the middle of the rainfall event. However, the peak rainfall for the 5-year storm

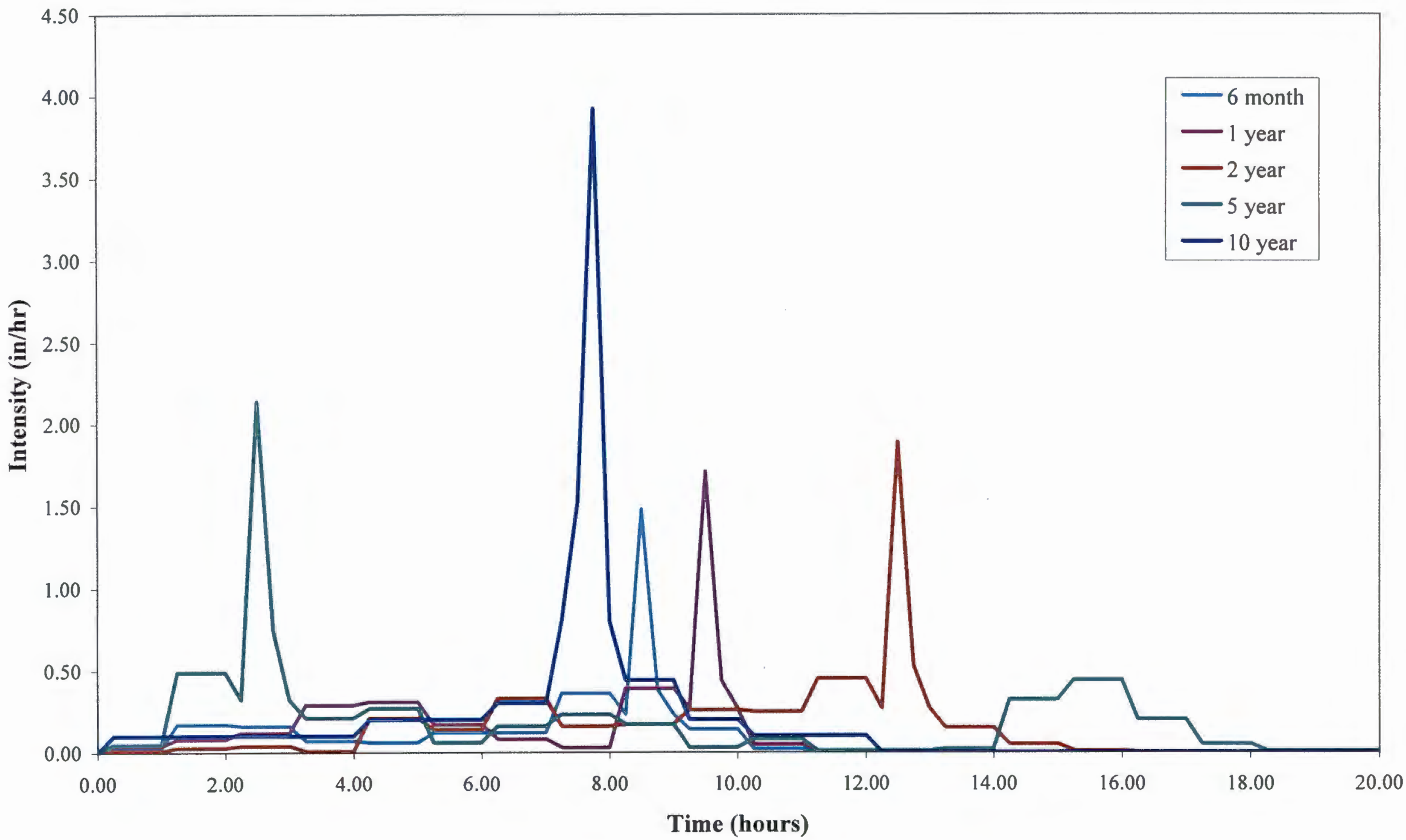
TABLE 6-4. CHARACTERISTICS OF DESIGN STORMS USED IN EVALUATION

Recurrence Interval	Total Depth (in.)	Peak Hour Intensity (in/hr)	Peak 15-Minute Intensity (in/hr)
2-week	0.55	0.22	0.56
1-month	0.91	0.30	0.77
3-month	1.44	0.46	1.17
6-month	1.82	0.58	1.48
1-year	2.23	0.67	1.71
2-year	2.99	0.59	1.89
2-year (TP-40)	2.50	1.19	2.65
5-year	3.71	0.88	2.14
5-year (TP-40)	3.20	1.52	3.39
10-year	3.70	1.76	3.92

occurred early in the event; and the peak of the 2-year storm occurred during the later part of the event. As rainfall is input to the hydrologic model, runoff is generated from both impervious (streets and sidewalks) and pervious (grass) areas.

The amount of runoff from the impervious surface remains constant over the course of the storm and is driven by intensity. There is no runoff from pervious surfaces until later in the storm, after the pervious surface has become saturated with water.

Figure 6-1. Rainfall Hydrographs for Design Storms



The peak of the 2-year storm occurred toward the end of the event, when the ground was already saturated with water. Therefore, the total runoff from the 2-year storm was greater than if the peak had occurred during the middle of the event.

In contrast, the peak for the 5-year storm occurred early in the storm, producing the opposite effect. Less runoff was generated since most of the rain which landed on pervious surfaces during the peak intensity of the storm infiltrated the ground and did not increase the runoff. The impacts of these changes were significant enough to reduce the overflow volume for the 5-year storm and increase the overflow volume for the 2-year storm, to the point where they were nearly equal.

Since the ambiguity in predicted CSO volumes between the two design storms had the potential to be problematic, two additional 2-year and 5-year storms were developed using Technical Paper 40 (Rainfall Frequency Atlas of the United States, United States Department of Agriculture, May 1961). Predicted CSO volumes for the 2-year and 5-year TP-40 storms are also presented in Table 6-5.

BASELINE LOADINGS

Wet weather sampling was conducted during two rain events in the fall 2002. During these events, discharges from CSO and stormwater outlets were sampled in order to determine typical pollutant concentrations specific to Nashua. As presented in Chapter 4, the average *E. coli* concentration for CSOs was computed to be 212,000 col/100mL. Baseline loadings were then determined by applying the average *E. coli* concentration to the predicted CSO discharge volumes, based on computer model predictions using design storms and the annual rainfall simulation. Loading projections based on the design storms and the annual simulation are presented in Tables 6-6 and 6-7, respectively.

TABLE 6-5. CSO VOLUMES FOR DESIGN STORMS UNDER NEAR FUTURE BASELINE CONDITIONS (NFBC)

NPDES Discharge Number	Location	Overflow Volume (in Mgal) for Modeled Storm Recurrence Intervals									
		2 week	1 month	3 month	6 month	1 year	2-Year		5-Year		10 year
							Actual	TP-40	Actual	TP-40	
CSO 002	Salmon Brook	0.00	0.00	0.00	0.00	0.00	0.00	0.76	0.00	2.26	3.65
CSO 003	Farmington Road	0.00	0.02	0.12	0.28	0.33	0.46	2.04	0.54	4.11	5.91
CSO 004	Burke Street	0.05	0.20	0.39	0.61	0.83	1.23	1.94	1.38	3.21	4.23
CSO 005	E. Hollis St.	0.01	0.07	0.57	1.27	1.33	2.46	4.89	2.49	7.74	10.13
CSO 006	Nashua River OF	0.00	0.00	1.10	2.81	2.56	5.94	17.09	5.12	29.78	42.53
CSO 007	Tampa Street	0.00	0.00	0.00	0.07	0.07	0.15	1.06	0.04	1.77	2.35
CSO 008	Broad Street	0.03	0.08	0.21	0.35	0.51	0.70	1.41	0.86	2.61	3.68
CSO 009	Lock Street	0.02	0.05	0.11	0.18	0.26	0.37	0.33	0.44	0.56	1.50
	Total	**0.11**	**0.42**	**2.50**	**5.58**	**5.89**	**11.30**	**29.52**	**10.86**	**52.04**	**73.99**

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TABLE 6-6. PROJECTED BACTERIAL LOADINGS FROM CSOs UNDER NEAR FUTURE BASELINE CONDITIONS (NFBC)

NPDES Discharge Number	Location	Colonies of *Escherichia Coli* Bacteria for Modeled Storm Recurrence Intervals									
		2 week	1 month	3 month	6 month	1 year	2-Year		5-Year		10 year
							Actual	TP-40	Actual	TP-40	
CSO 002	Salmon Brook	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	6.1E+06	0.0E+00	1.8E+07	2.9E+07
CSO 003	Farmington Road	0.0E+00	1.9E+05	9.7E+05	2.2E+06	2.6E+06	3.7E+06	1.6E+07	4.3E+06	3.3E+07	4.7E+07
CSO 004	Burke Street	4.4E+05	1.6E+06	3.1E+06	4.9E+06	6.7E+06	9.9E+06	1.6E+07	1.1E+07	2.6E+07	3.4E+07
CSO 005	E. Hollis St.	5.2E+04	5.2E+05	4.6E+06	1.0E+07	1.1E+07	2.0E+07	3.9E+07	2.0E+07	6.2E+07	8.1E+07
CSO 006	Nashua River OF	0.0E+00	0.0E+00	8.8E+06	2.3E+07	2.1E+07	4.8E+07	1.4E+08	4.1E+07	2.4E+08	3.4E+08
CSO 007	Tampa Street	0.0E+00	0.0E+00	0.0E+00	5.6E+05	6.0E+05	1.2E+06	8.5E+06	3.0E+05	1.4E+07	1.9E+07
CSO 008	Broad Street	2.6E+05	6.7E+05	1.7E+06	2.8E+06	4.1E+06	5.6E+06	1.1E+07	6.9E+06	2.1E+07	3.0E+07
CSO 009	Lock Street	1.2E+05	3.7E+05	9.2E+05	1.5E+06	2.1E+06	3.0E+06	2.6E+06	3.5E+06	4.5E+06	1.2E+07
	Total	**8.7E+05**	**3.3E+06**	**2.0E+07**	**4.5E+07**	**4.7E+07**	**9.1E+07**	**2.4E+08**	**8.7E+07**	**4.2E+08**	**5.9E+08**

Escherichia coli concentration used to compute loads is 212,000 col/100mL as presented in Chapter 4
CSO volumes for design storms are presented in Table 6-5

TABLE 6.7. PROJECTED BACTERIAL LOADING FROM CSOs TYPICAL YEAR SIMULATION, NFBC

NPDES Discharge Number	CSO Name	No. of Activations During Year	Total Annual Overflow Volume (MG)	Annual Load *E. coli* (colonies)
002	Salmon Brook	0	0.00	0
003	Farmington Rd.	17	1.41	1.1×10^{13} ✓
004	Burke St.	25	5.53	4.4×10^{13} ✓
005	E. Hollis St.	17	5.76	4.6×10^{13} ✓
006	Nashua River	10	9.59	7.7×10^{13} ✓
007	Tampa St.	2	0.09	7.2×10^{11} ✓
008	Broad St.	23	2.89	2.3×10^{13} ✓
009	Lock St.	20	0.38	3.0×10^{12} ✓
		Total	**25.65**	**2.1×10^{14}**

Escherichia coli concentration used to compute loads is 212,000 col/100mL as presented in Chapter 4.

3.7853 $\frac{\text{liters}}{\text{gal}}$

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CHAPTER 7

RECEIVING WATER MODELING AND BASELINE WATER QUALITY IMPACTS

There are 9 permitted CSOs in the Nashua system; 4 are tributary to the Merrimack River (002, 003, 004, and 005), and 4 are tributary to the Nashua River (006, 007, 008, 009). The ninth CSO regulator (Jackson/Beaucher, CSO 012) exists along the Nashua River; however, it is presumed inactive since gates on the discharge side of the weir are bolted shut. Any overflow discharging from this regulator would combine with overflow from the Lock Street CSO (009), and would discharge through that outfall to the Nashua River.

In addition to the pollutant loadings from Nashua CSOs, CSO discharges from upstream communities on the Merrimack River (such as Manchester, NH) contribute to the impairment of river water quality during wet weather events. Furthermore, stormwater discharges from non-CSO communities can also adversely impact water quality. Near the study area in Nashua, the communities of Nashua, Merrimack, Hudson, and Litchfield discharge stormwater runoff to the Merrimack River during wet weather events.

The receiving water analysis was conducted using a receiving water flow and quality model, with CSO discharge flow inputs taken from the collection system hydraulic model (MOUSE), and stormwater runoff flow inputs from separated areas taken from the hydrologic model (SWMM).

WATER QUALITY PARAMETERS

The water quality analysis was limited to *Escherichia coli* bacteria, an indicator of the presence of other pathogenic organisms associated with fecal pollution. The use of *E. coli* as the main parameter for CSO impact evaluation is based on the findings of a dilution analysis, conducted as part of the 1997 LCTP. The dilution study, conducted in accordance with methodologies developed by NHDES, shows that during wet weather events, the bacterial loadings from CSOs are not significantly diluted by the receiving water. Based on review of the dilution analysis, NHDES determined that CSO discharges along the Merrimack and Nashua Rivers in Nashua exceed water quality standards for bacteria. For this reason, *E. coli* bacteria was used as the parameter of interest in the 1997 LCTP and will be used as the parameter of interest in this assessment.

Receiving Water Criteria

The Merrimack and Nashua Rivers are both classified as Class B water bodies, the second highest rating among the State of New Hampshire classifications. According to state regulations (RSA 485-A:8), Class B waters "shall contain not more than either a geometric mean based on at least 3 samples obtained over a 60-day period of 126 *Escherichia coli* per 100 milliliters, or greater than 406 *Escherichia coli* per 100 milliliters for any one sample." Class B water criteria also state that "there shall be no disposal of sewage or waste into said waters except those which have received adequate treatment to prevent the lowering of biological, physical, chemical, or bacteriological characteristics below those given above."

In cases where it is demonstrated to the satisfaction of the NHDES that the Class B criteria cannot be reasonably met in certain surface waters at all times as a result of CSO events, a temporary partial use (TPU) area can be established. The TPU classification allows a relaxation of certain water quality classification criteria during, and up to 3 days following the cessation of a CSO activation. At all other times, the Class B water quality criteria must be met.

DESCRIPTION OF THE MODEL

To assess the receiving water quality impacts of the wet weather discharges, the two-dimensional MIKE 21 flow and transport model was used. Being two-dimensional, the model resolves lateral, as well as longitudinal, variations in flow and pollutant concentrations. Lateral resolution is warranted for discharges into wide rivers, such as the Merrimack River, where full mixing of the river flow requires significant distances.

The MIKE 21 model, developed by the Danish Hydraulic Institute, has been used in hundreds of applications worldwide and has been refined and improved over the last 20 years. MIKE 21 calculates depth-averaged values of the current speed and water quality parameter by solving the governing differential equations using the Alternating Direction Implicit (ADI) scheme, which is recognized as being one of the most accurate, reliable and efficient approaches. Values of the variables are calculated along a uniform rectangular grid at regular time intervals, starting from user-specified *initial conditions*. The solution is controlled by *boundary conditions* imposed on the periphery of the simulation domain. Examples of boundary conditions would be *no flow* through a land boundary, or *specified flow* at inflow points.

The advection/dispersion model solves the mass transport equation for constituents which can be conservative (no decay) or subject to linear first order decay. Dye is an example of a conservative substance, and first order decay can be used to simulate bacteria and carbonaceous biochemical oxygen demand (BOD). The mass transport equation is solved using an extension of the QUICKEST scheme, which avoids wiggle instabilities, yet does not result in damping. The model simulates *E. coli* using a first order die-off coefficient calculated as a function of temperature, salinity, and light. For this application, however, a constant die-off coefficient of 0.8/day was used. This value is an average value for freshwater (Mancini, 1978; Bowie et al, 1985).

MODEL SETUP

The model has two segments. The first segment is the Nashua River upstream of the Jacksonville Dam. The second segment includes both the Nashua River section downstream of the dam, as well as the Merrimack River. These segments are described below:

1) Nashua River, upstream boundary to Jackson Mill dam. This segment is approximately 3.5 river miles in length, and receives discharges from the Broad Street CSO (008) and the Tampa Street CSO (007). Storm drains serving separated areas of the downstream district also discharge to this segment of the Nashua River.

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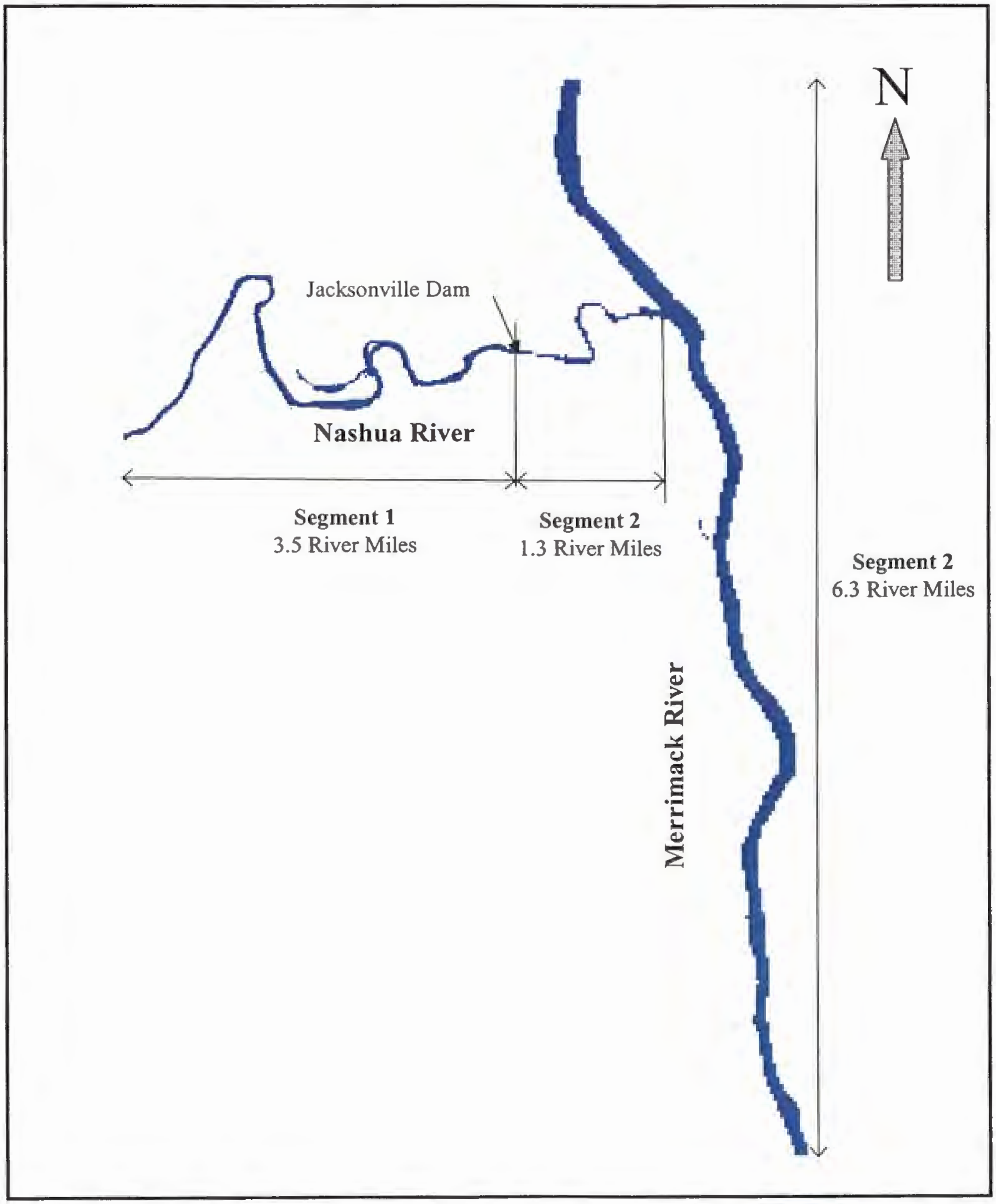
FIGURE 7-1. RECEIVING WATER MODEL SEGMENTS



2) Nashua River, Jacksonville dam to confluence with the Merrimack River, and the Merrimack River, upstream boundary to downstream boundary. The downstream section of the Nashua River is approximately 1.3 river miles in length, and receives discharges from the Lock Street CSO (009) and the Nashua River CSO (006). This segment also includes the entire modeled length of the Merrimack River, 6.3 river miles in length. CSO inputs to the Merrimack River include the East Hollis Street CSO (005), the Burke Street CSO (004), the Salmon Brook CSO (002), and the Farmington Road CSO (003). This segment also receives stormwater runoff from both the Nashua and Hudson sides of the Merrimack River, as well as pollutant loadings from the Nashua River and Salmon Brook.

The modeled grid for segment 1 was composed of 16.4 ft. by 16.4 ft. squares, and the modeled grid for segments 2 was composed of 32.8 ft. by 32.8 ft. squares. Therefore, the Merrimack River and downstream Nashua River grid was typically divided into 9 squares across its width, and the Nashua River grid was typically divided into 18 squares across its width.

The river bathymetry for both the Merrimack and Nashua Rivers was specified based on cross-section data measured for flood insurance studies. Depths along the cross sections were connected along each river, interpolating the bathymetry for the entire modeled length.

The transverse diffusion coefficient, which controls the lateral spreading of the effluent plumes, was calculated as $\varepsilon_l = 0.6\ d\ u^*$, where:

d = water depth
u^* = shear velocity = $(g\ d\ S)^{1/2}$
g= acceleration of gravity
S = slope of the water surface

The slope of the water surface can be calculated using Manning's equation (Fischer et al, 1979). The following values for ε_l were calculated and used in the model:

River Segment 1 – 0.82 ft^2/sec.
River Segment 2 – 0.75 ft^2/sec.
River Segment 3 – 0.86 ft^2/sec.

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For the longitudinal diffusion coefficient, which has a much smaller effect on the distribution of pollutant concentrations, the same value as the transverse diffusion coefficient was used for each river section.

Initial Conditions

Initial *E. coli* concentrations at sample transect points were specified in the model prior to the wet weather calibration. The initial *E. coli* concentrations were based on dry weather samples collected during the summers of 2001 and 2002. The samples were collected by NWTF personnel in accordance with the QAPP and established the background bacteria concentration at the sample points. Average background concentrations for each sample transect were computed by taking the arithmetic mean of all the data collected at one transect. The average *E. coli* concentration in the Merrimack River was 20 col/100 mL, and the average concentration of *E. coli* found in the Nashua River was 30 col/100 mL. Dry weather sampling data are contained in Appendix B. A

Merrimack R.?

Though Salmon Brook is tributary to the Merrimack River, it was not included in the MIKE 21 model, since there are no CSO discharges to Salmon Brook. However, dry weather flow from Salmon Brook was included as a point source input to the Nashua River. Samples taken at RIV-6 showed the dry weather bacterial concentration in Salmon Brook was approximately 130 col/100 mL. Flow data from meter M-26, which quantified the flow in the Salmon Brook, was used with this dry weather concentration to compute a dry weather load to the Merrimack River.

Boundary Conditions

The boundary conditions were specified at the upstream ends of the Merrimack and Nashua Rivers, RIV-1 and RIV-5, respectively. As noted above, the initial conditions were computed from dry weather data and the change in concentration across the boundary condition was based on the *E. coli* measurements collected during the wet weather sampling events. Round 1 sampling was intended to represent the peak of the storm, after the CSOs and storm drains were discharging. Therefore, the model gradually increased the receiving water bacteria concentrations from initial conditions to the concentrations observed during round 1 of sampling, to simulate the effects of wet weather inputs occurring between the beginning and the peak of the storm. Similarly, the concentrations were gradually decreased over time at the end of the event for the 12, 24, and 48-hour samples. Wet weather sampling data are contained in Appendix B. A

Source Characterization

The CSO discharge flows were taken from the output of the MOUSE model. The stormwater discharges from separated areas in Nashua and Hudson were calculated for 20 catchments using SWMM runoff model. For each catchment, a percent impervious was estimated based on the portion of the delineated basin where the ground surface was paved or covered. Where available, land use types were also factored into the development of percent impervious. For example, commercial surfaces were considered impervious while cropland or pasture was considered pervious. All numbers were scaled using a calibration factor obtained from the runoff model developed for the MOUSE model. The catchment characteristics are summarized in Table 7-1.

TABLE 7-1. CATCHMENT CHARACTERISTICS FOR SEPARATE STORMWATER BASINS

Basin ID	Receiving Water	Area (Acres)	Percent Impervious
1002	Nashua River	328.6	5
1003	Nashua River	270.3	5
1005	Nashua River	30.8	2
1006	Nashua River	125.1	21
1007	Nashua River	722.4	11
1008	Nashua River	434.4	12
1009	Nashua River	535.9	15
1010	Nashua River	50.2	16
1011	Merrimack River	1998.5	30
1012	Merrimack River	263.0	20

TABLE 7-1 CONTINUED. CATCHMENT CHARACTERISTICS FOR SEPARATE STORMWATER BASINS

Basin ID	Receiving Water	Area (Acres)	Percent Impervious
1013	Merrimack River	488.3	13
1014	Merrimack River	952.3	9
1015	Merrimack River	861.1	6
1016	Merrimack River	1152.3	14
1017	Merrimack River	3523.5	11
1018	Merrimack River	2752.7	3
1019	Merrimack River	692.2	7
1020	Merrimack River	317.9	8
1021	Nashua River	12.9	17
1022	Nashua River	50.8	27

212,000 ?

An *E. coli* concentration of 215,000 col/100 mL was specified for the CSO discharge, and a concentration of 5,000 col/100 mL was used for stormwater, based on the wet weather sampling data, as discussed in Chapter 4.

MODEL CALIBRATION

The model was calibrated using data collected from the October 16, 2002 storm event. Data from the November 6, 2002 event were used to verify calibration of the receiving water model. For calibration purposes, the NWTF effluent was assigned a bacterial concentration of 10 col/100 mL. NWTF records show that the secondary treatment bypass was active during these two events. However, since bacteria samples collected from the bypass indicate effective disinfection, no separate model adjustment was necessary to account for the secondary treatment bypass.

Figures 7-2, 7-3 and 7-4, show the calibration plots at RIV-2 (downstream boundary on the Merrimack River), RIV-3 (Merrimack River, downstream of confluence with Nashua River), and RIV-4 (downstream boundary on the Nashua River) for the October 16, 2002 event.

**FIGURE 7-2. CALIBRATION RESULTS FOR 10/16/02 STORM EVENT
MERRIMACK RIVER, RIV-2**

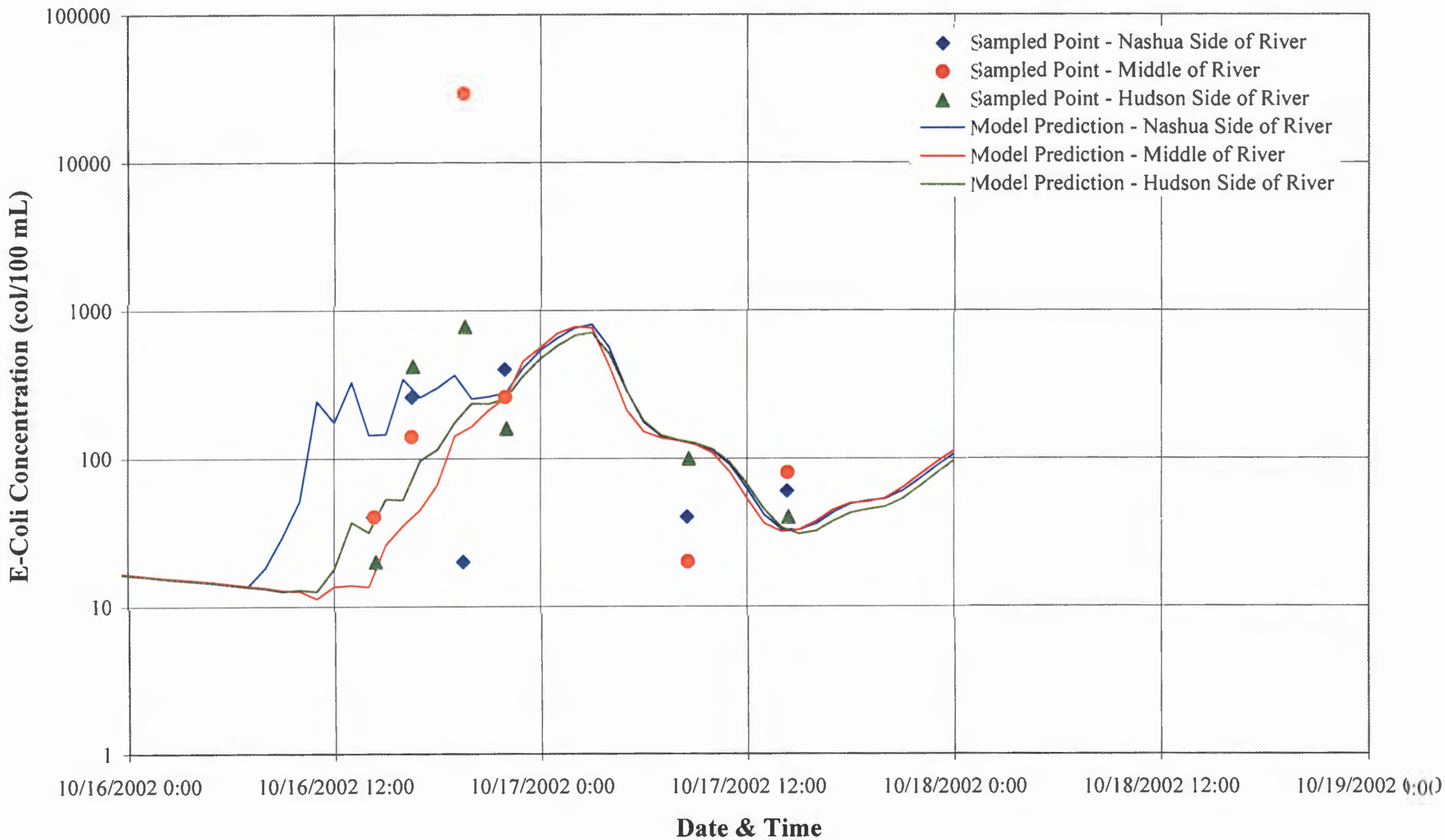

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FIGURE 7-3. CALIBRATION RESULTS FOR 10/16/02 STORM EVENT
MERRIMACK RIVER, RIV-3

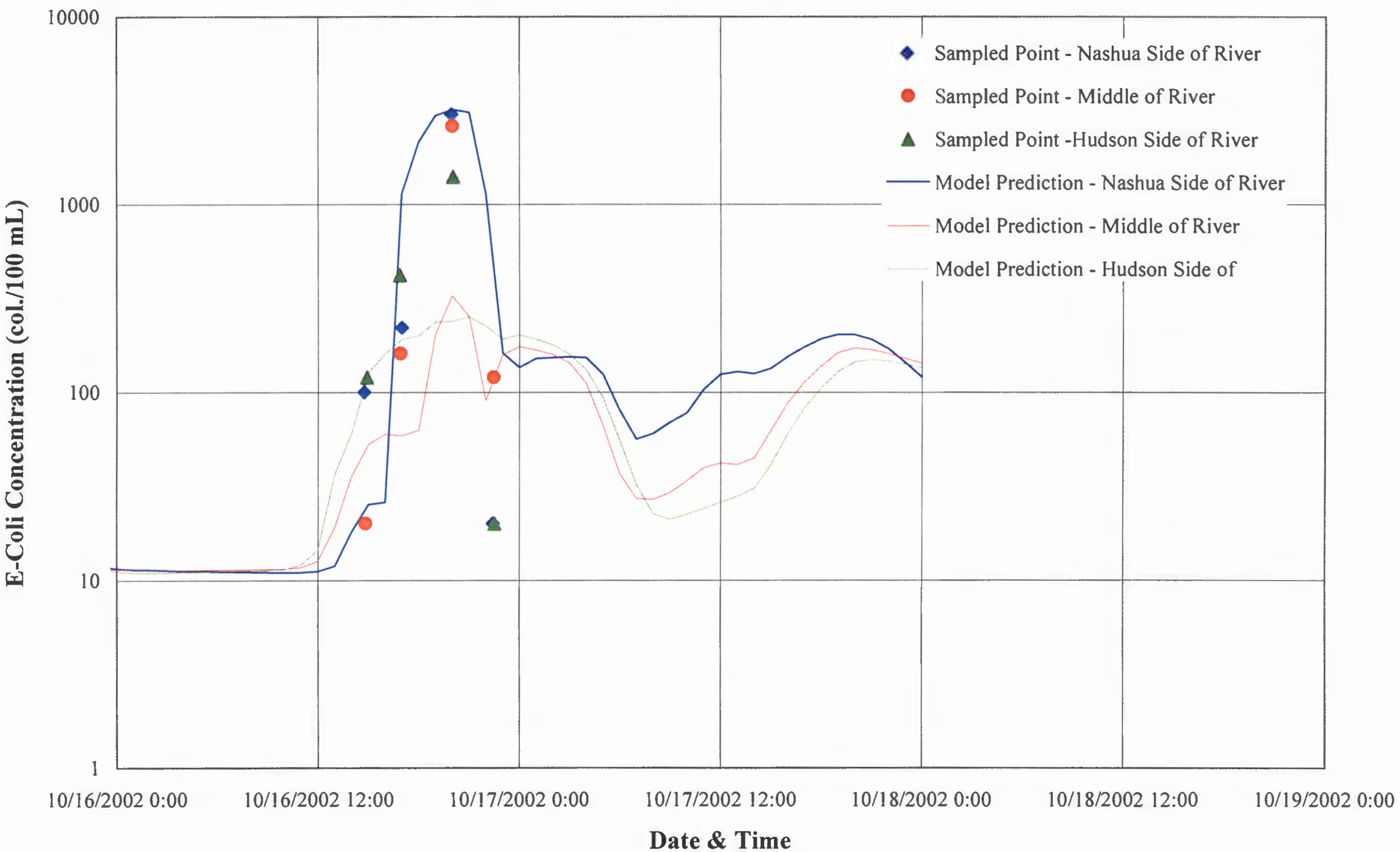

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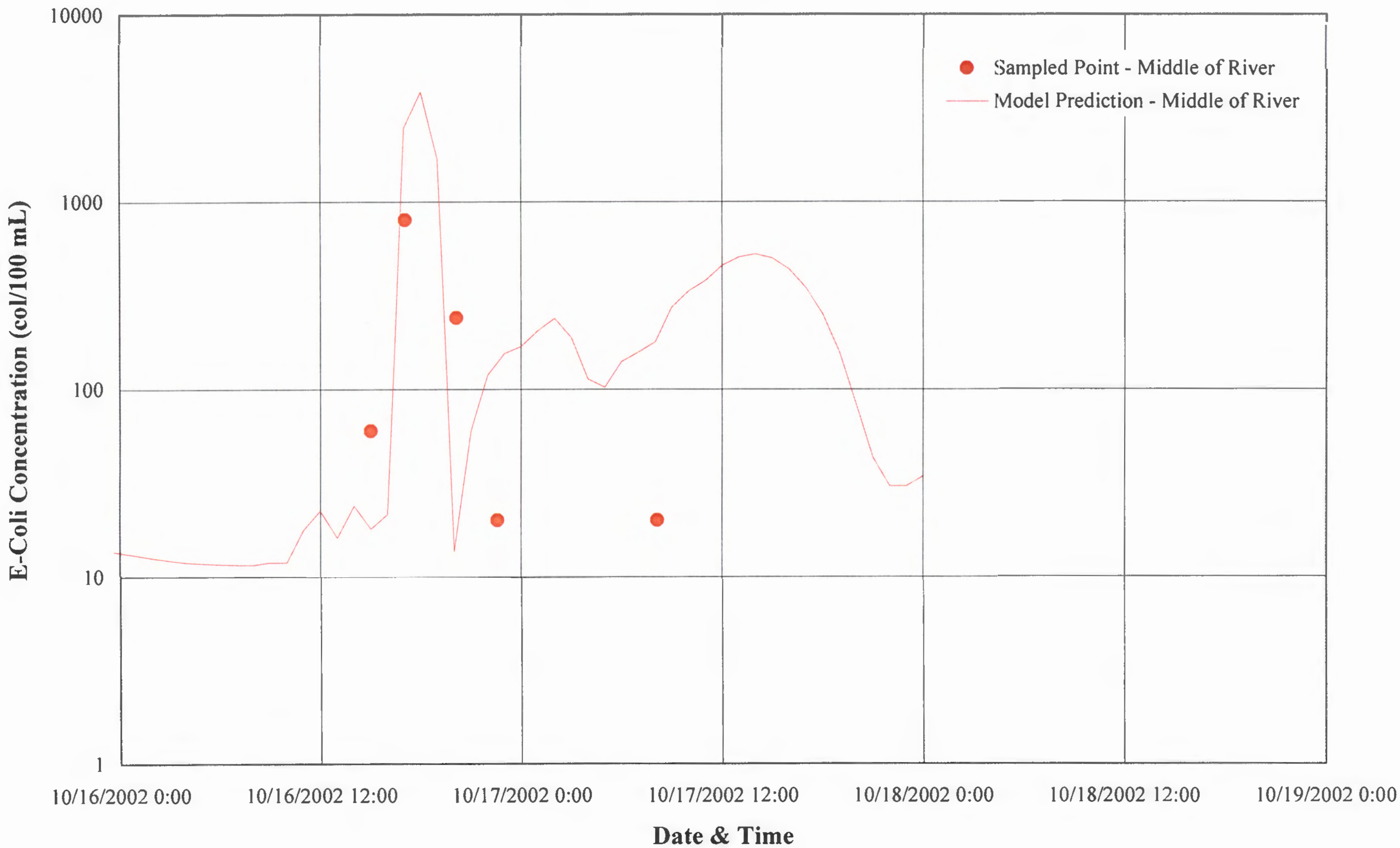
**FIGURE 7-4. CALIBRATION RESULTS FOR 10/16/02 STORM EVENT
NASHUA RIVER, RIV-4**



In general, the model matched the sample data well at RIV-2 and RIV-3. While in both cases, the model occasionally predicted values that were above or below what was observed in the sampling, in general the calibration was good.

In general, the calibration at RIV-4 also showed good agreement between model output and sample data. The model predicted concentrations that were higher than what was observed for rounds 4 and 5, but was able to capture the spike that occurred during rounds 2 and 3.

MODEL APPLICATION

To evaluate the impacts of the Nashua CSOs on the receiving water, model simulations were run based on the 3-month and 1-year design storms, as described in Chapter 6. The CSO flows calculated by the collection system model and stormwater flows computed from the runoff model from the Near Future Baseline Condition (NFBC), current recommended plan (CRP) and the Recommended Plan developed in this study (Refer to Chapter 11) were input to the receiving water model. The model computed the *E. coli* concentrations in the Merrimack and Nashua Rivers throughout the design storm event.

For all simulations, the following concentrations were used for CSO and stormwater discharges:

- Untreated CSO – 212,000 col/100mL
- Treated CSO – 126 col/100mL
- Separate stormwater – 5,000 col/100mL

It is important to note that the Recommended Plan developed in this study will result in zero discharges of untreated CSO in either the 3-month or 1-year storm events. Treated CSO/wet weather discharges would, however, occur. It is also important to note that the Recommended Plan developed in this study will not result in increased separate stormwater discharges.

RESULTS

The impacts on the receiving waters were evaluated using isopleths and time versus concentration plots. The isopleths are a graphical depiction of the concentration of bacteria in the entire portion of the receiving water covered by the receiving water model at the time of greatest impact. The coloration shown in the isopleths divides the impact into three categories: blue, which indicates "no impact" (concentrations between 0 and 126 col/100ml); yellow, which indicates "near violation" (concentrations 126 to 406 col/100ml); and red, which indicates "in violation" (concentrations above 406 col/100ml). These graduations were based on the water quality standards described above.

Graphs of time versus concentration show the variation in the bacterial concentration over the entire storm event at a particular transect point along the river. These plots can be used to evaluate the magnitude and duration of the water quality violation at these specific locations in the river.

3-Month Storm

Figure 7-5 shows the impact of CSO and stormwater runoff on the Merrimack and Nashua Rivers under NFBC. As shown in the figure, a considerable amount of the river within the study can be characterized as being "near violation" and "in violation" at the time of greatest impact during the simulation. This is due in part to the stormwater runoff from Nashua and neighboring communities, but is also due to the CSO input that occurs under NFBC. Figures 7-6 and 7-7 show the impacts of the CRP and the Recommended Plan, respectively. Review of these two figures show an improvement over the plot of the NFBC, but in both cases there are still areas "in violation". In both cases, these violations are due to discharges of separate stormwater (or treated CSO discharges) as untreated CSO discharges are not predicted to occur during the 3-month storm.

The model indicates that implementing the Recommended Plan (Figure 7-7) whould lead to better wet weather water quality than implementing the CRP. Bacteria concentrations along the Nashua River, and in the Merrimack River, near its confluence with the Nashua River, are shown to be less than 126 col/100mL under the Recommended Plan. Furthermore, the total area that exceeds the 406 col/100mL is only 73.2 acres, compared to 129.2 acres for NFBC and 88.8 acres for the CRP. Since no CSO discharges would occur under either the CRP or the Recommended

FIGURE 7-5. *E. coli* CONCENTRATION IN RIVERS, 3-MONTH DESIGN STORM, NFBC

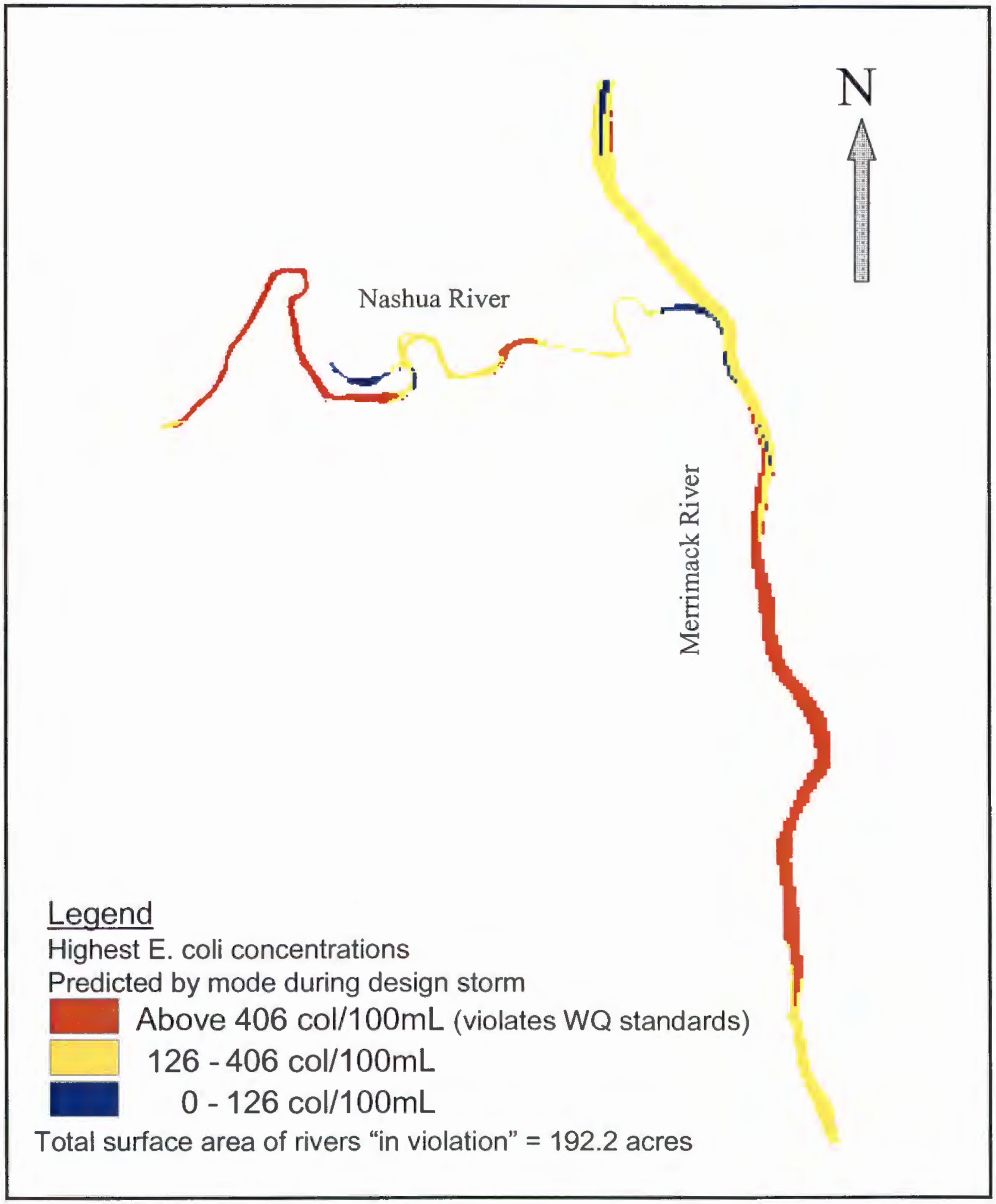


FIGURE 7-6. *E. coli* CONCENTRATION IN RIVERS, 3-MONTH DESIGN STORM, CRP

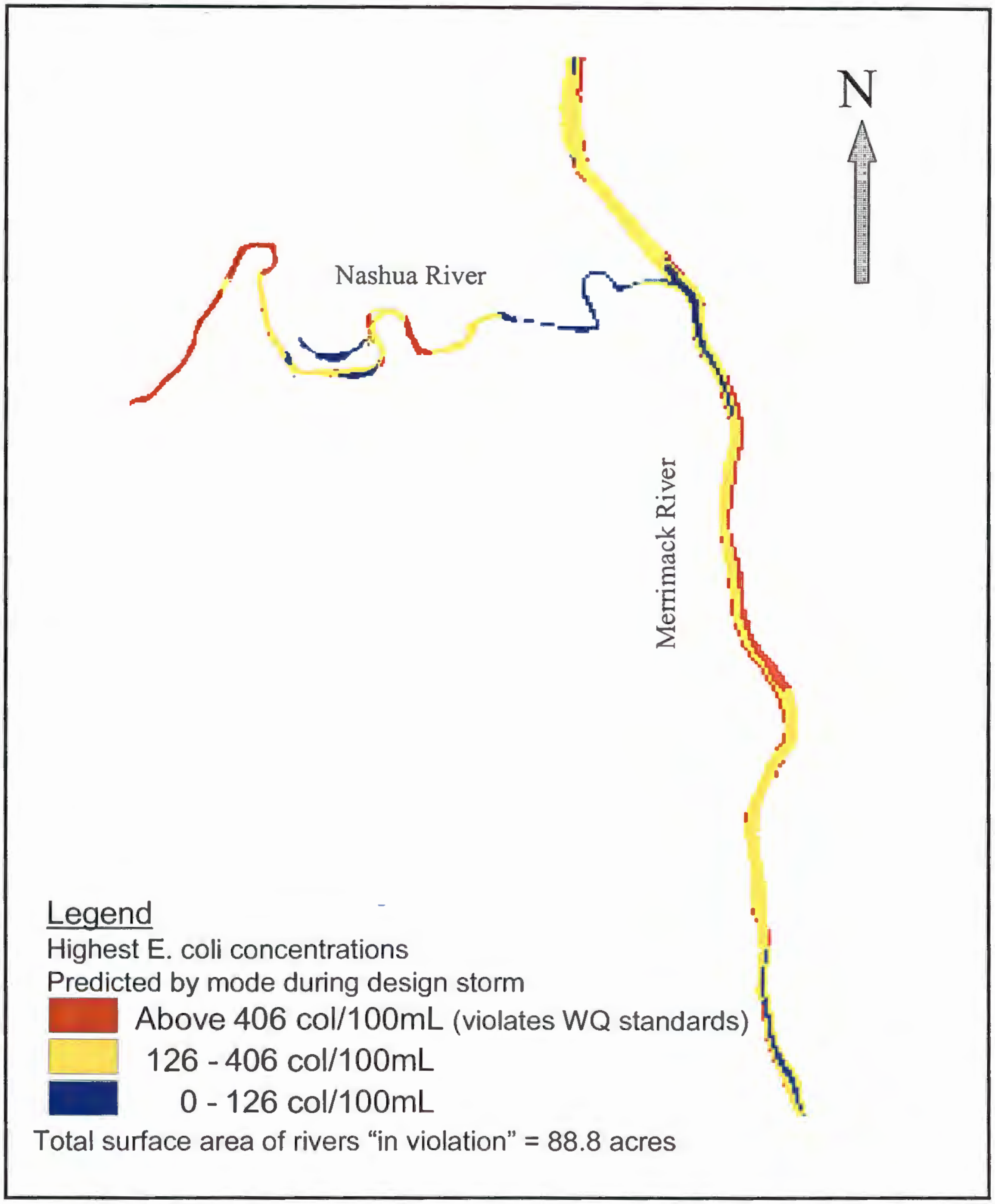


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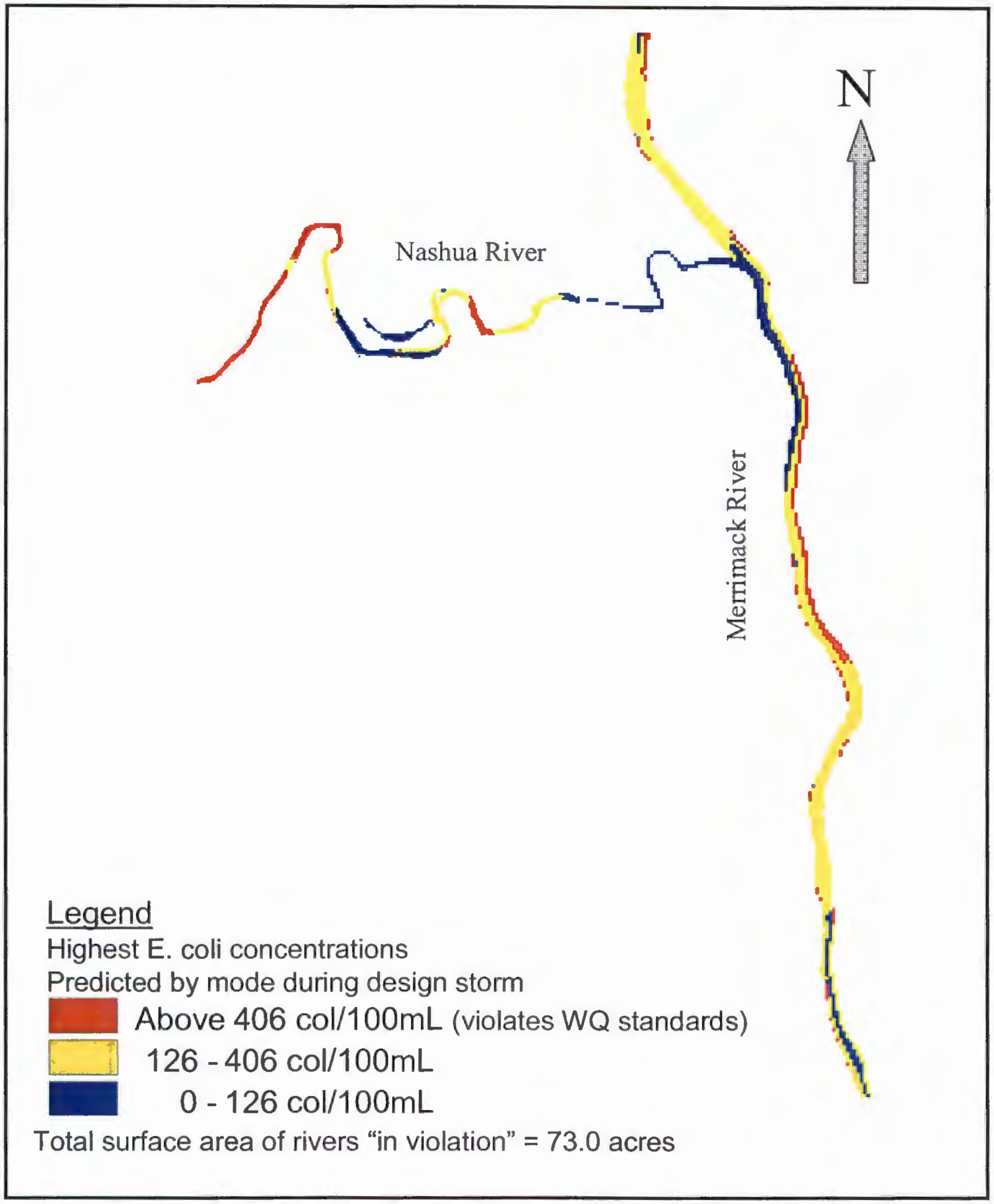
FIGURE 7-7. *E. coli* CONCENTRATION IN RIVERS, 3-MONTH DESIGN STORM, RECOMMENDED PLAN



Plan, the differences observed in river quality between these two conditions must be attributed to the increased stormwater loading under the CRP. The CRP results in increased quantities of stormwater as compared to NFBC while the Recommended Plan does not.

1-Year Storm

Figures 7-8, 7-9, and 7-10 show the impacts on the receiving waters for NFBC, CRP and the Recommended Plan, under a 1-year storm. As was observed for the 3-month storm, these figures indicate that implementing the Recommended Plan would result in better water quality than could be achieved under either the NFBC or the CRP, during a 1-year storm. For the 1-year storm, the total area "in violation" is 275.4 acres for the NFBC, 228.7 acres for the CRP, and 180.5 acres for the Recommended Plan.

Time versus concentration plots were developed for each condition at three transect points – RIV-2, downstream boundary on the Merrimack River; RIV-3, on the Merrimack River, downstream of the confluence with the Nashua River; and RIV-4, at the downstream boundary of the Nashua River. These transect points correspond to the locations where wet weather samples were collected. Figures 7-11, 7-12, and 7-13 show bacterial levels predicted by the model for these transects under NFBC; Figures 7-14, 7-15, and 7-16 show the predicted bacteria levels for the CRP; and Figures 7-17, 7-18, and 7-19 show the predicted bacterial levels for the Recommended Plan.

Figures 7-11, 7-12, and 7-13 show there would be a significant increase in bacteria concentrations in the rivers as a result of wet weather pollution during the 1-year storm under NFBC. As previously noted in this chapter, the elevated concentrations can be attributed to the stormwater runoff contributed by Nashua, Hudson, Merrimack, and Litchfield, and to the CSO load from Nashua.

In a 1-year storm, no untreated CSO would be discharged under either the CRP or the Recommended Plan. Consequently, both of these approaches show a significant improvement in water quality as compared to the NFBC, especially at RIV-3

FIGURE 7-8. *E. coli* CONCENTRATION IN RIVERS, 1-YEAR DESIGN STORM, NFBC

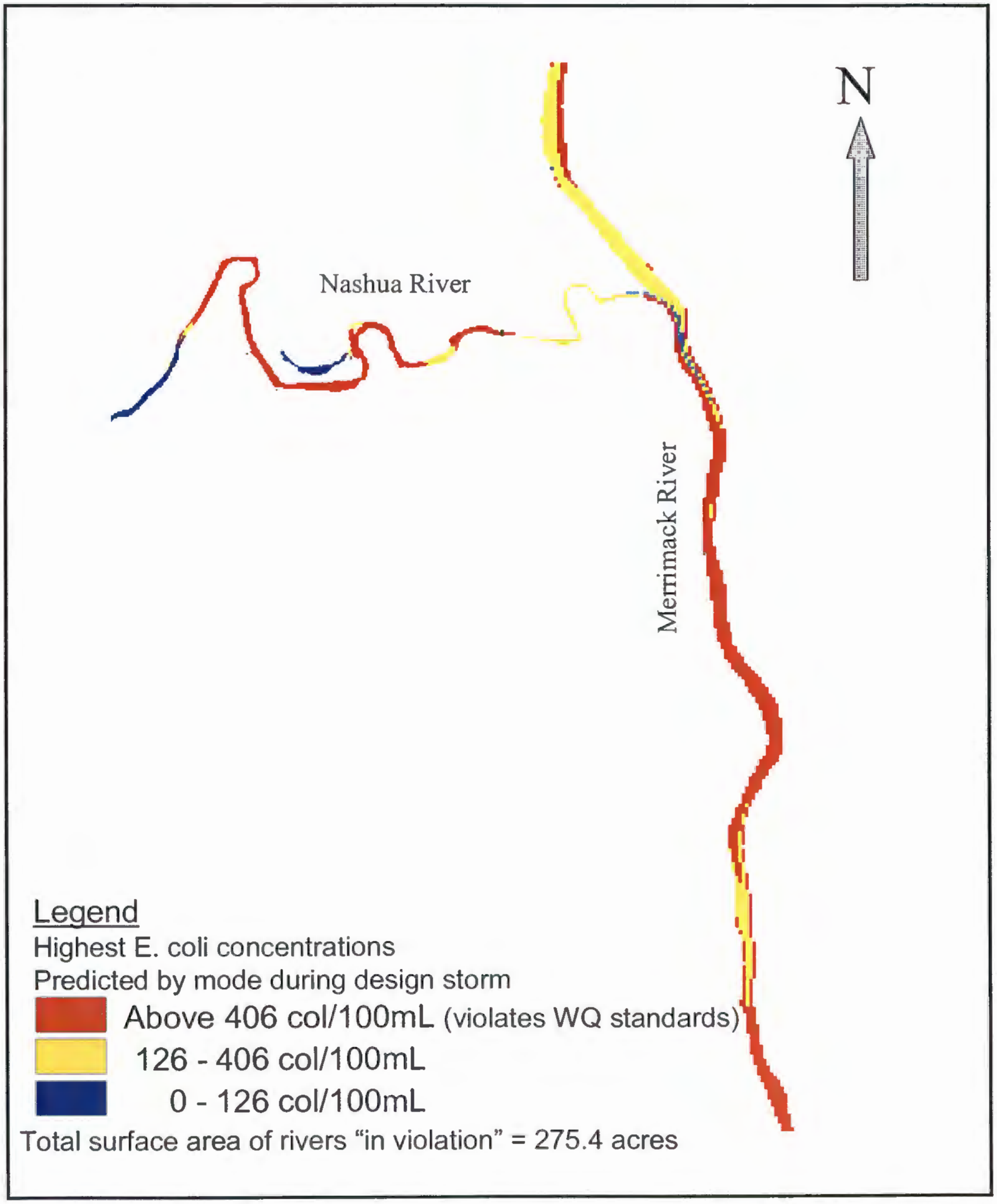


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FIGURE 7-9. *E. coli* CONCENTRATION IN RIVERS, 1-YEAR DESIGN STORM, CRP

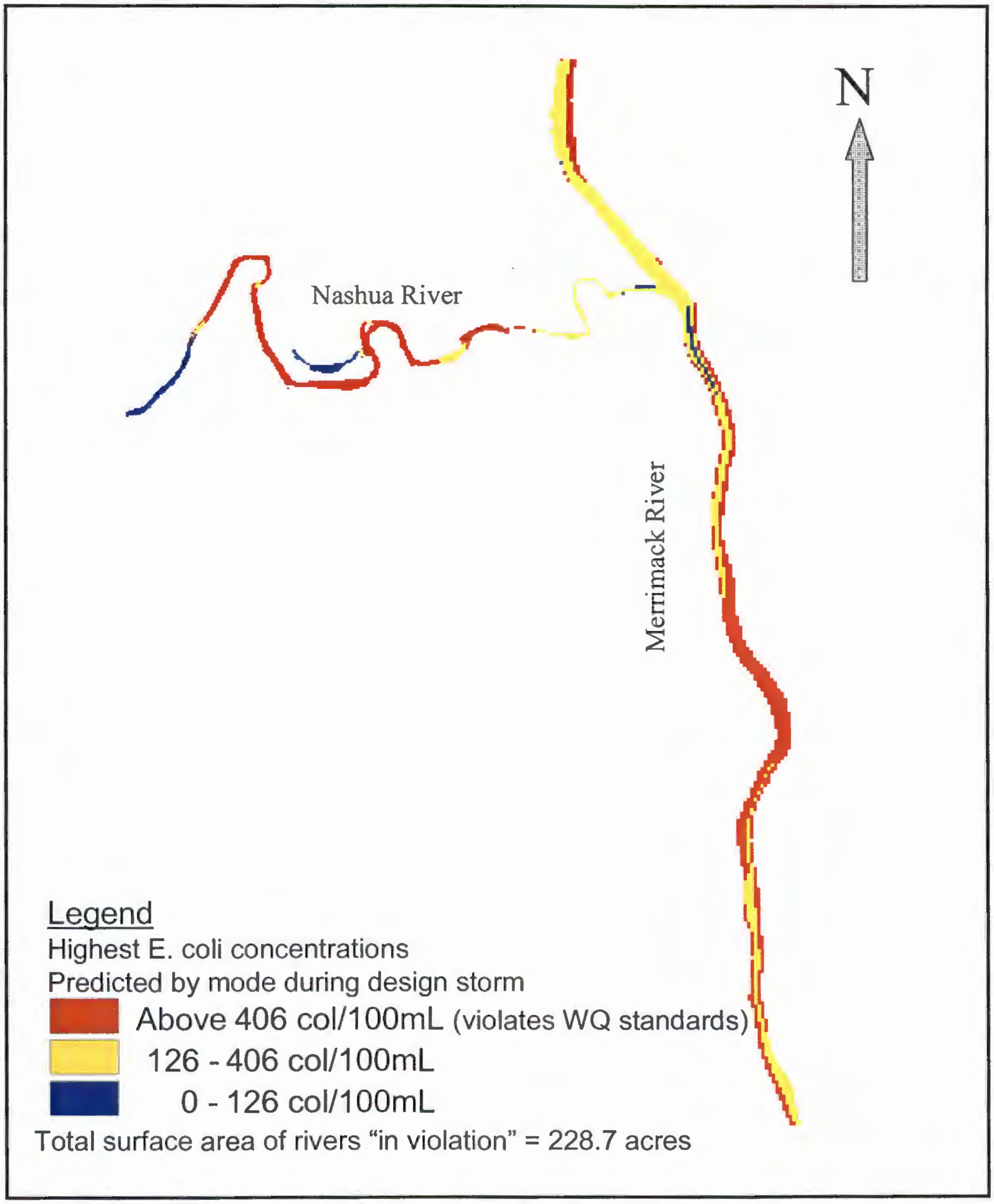

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FIGURE 7-10. *E. coli* CONCENTRATION IN RIVERS, 1-YEAR DESIGN STORM, RECOMMENDED PLAN

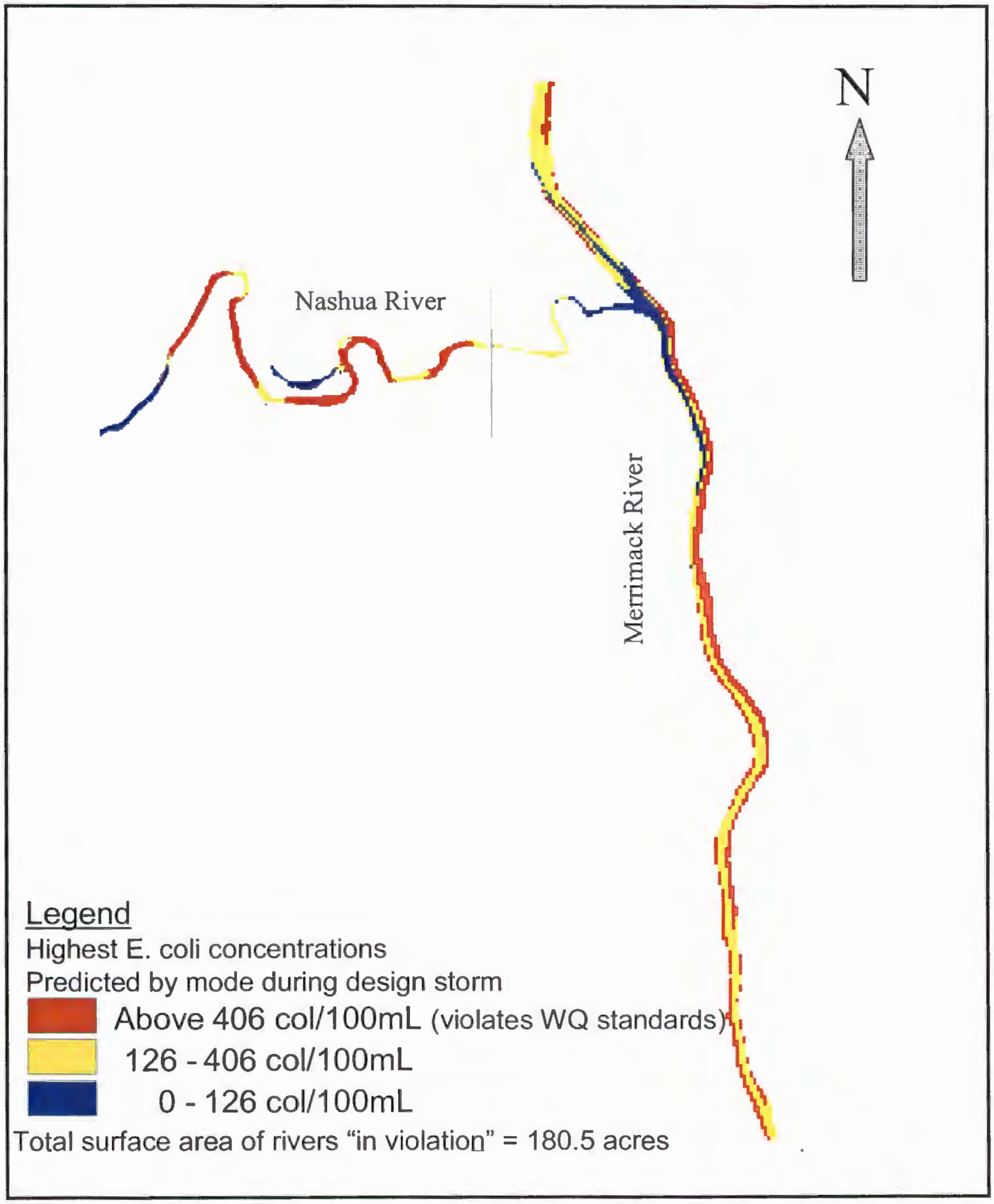

**FIGURE 7-11. TIME vs. *E coli* BACTERIA CONCENTRATION AT MERRIMACK RIVER, RIV-2
1-YEAR DESIGN STORM, NFBC**

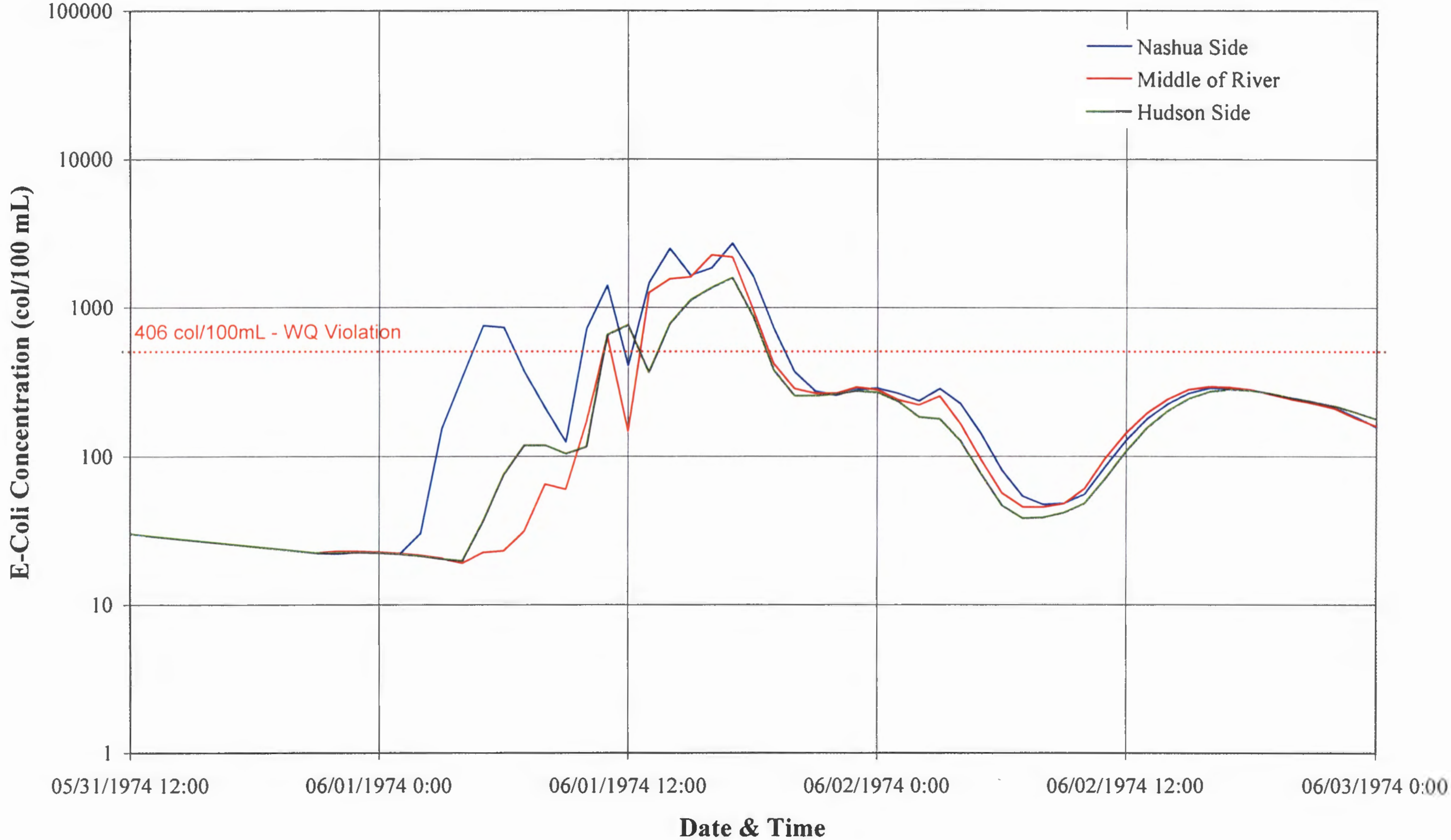

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**FIGURE 7-12. TIME vs. *E coli* BACTERIA CONCENTRATION AT MERRIMACK RIVER, RIV-3
1-YEAR DESIGN STORM, NFBC**

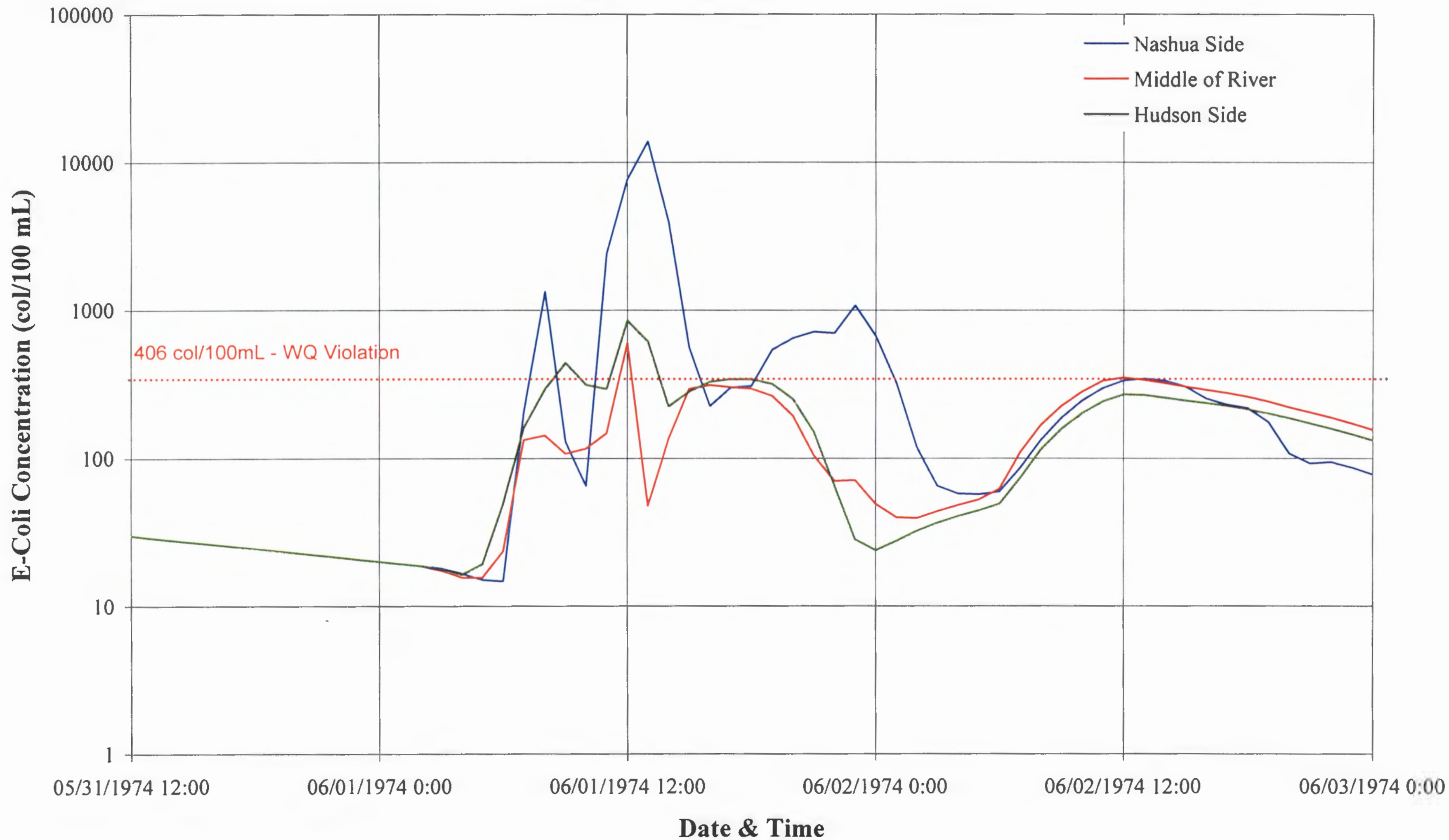

**FIGURE 7-13. TIME vs. *E. coli* BACTERIA CONCENTRATION AT NASHUA RIVER, RIV-4
1-YEAR DESIGN STORM, NFBC**

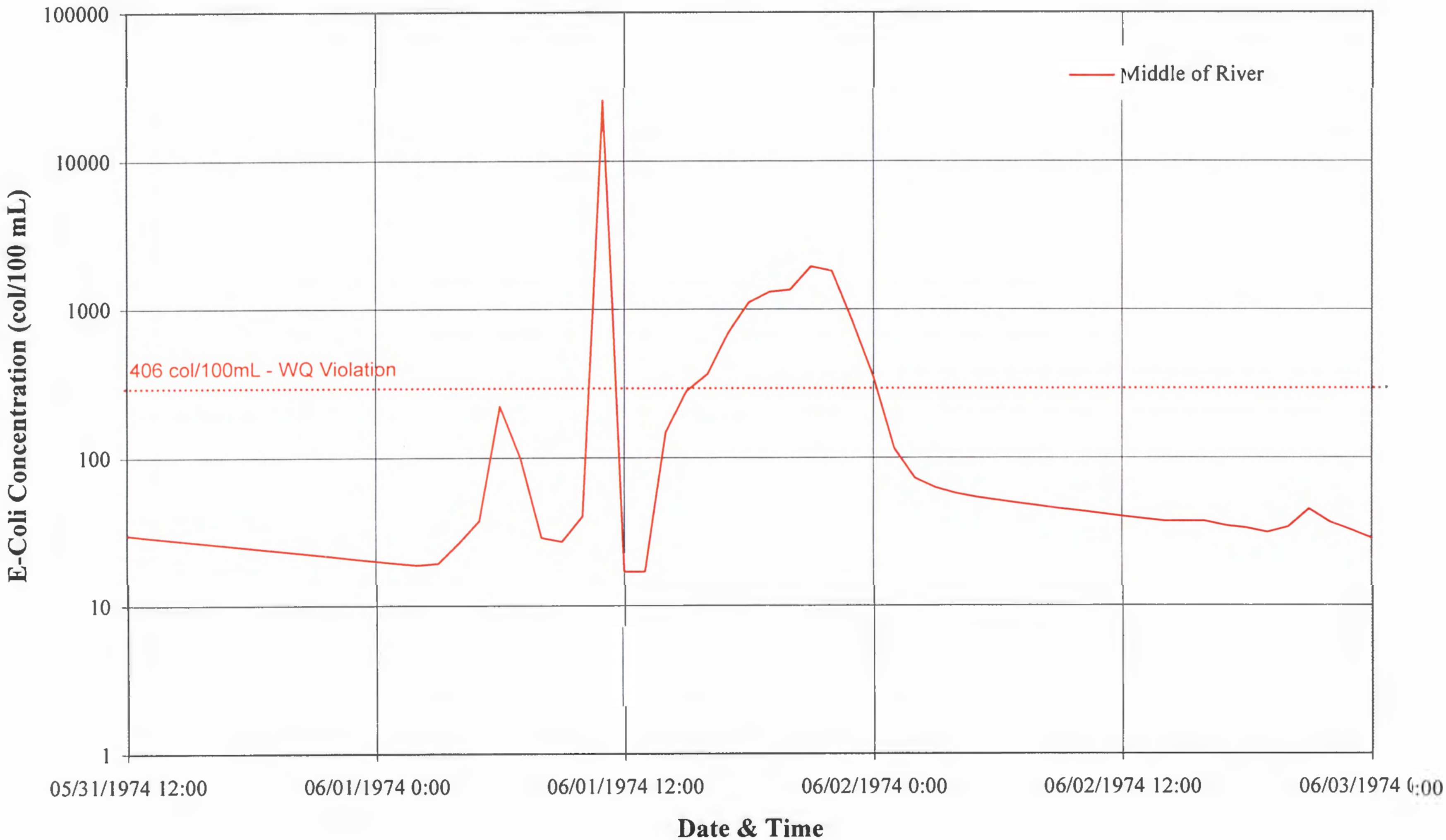

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FIGURE 7-14. TIME vs. *E coli* BACTERIA CONCENTRATION AT MERRIMACK RIVER, RIV-2
1-YEAR DESIGN STORM, CRP

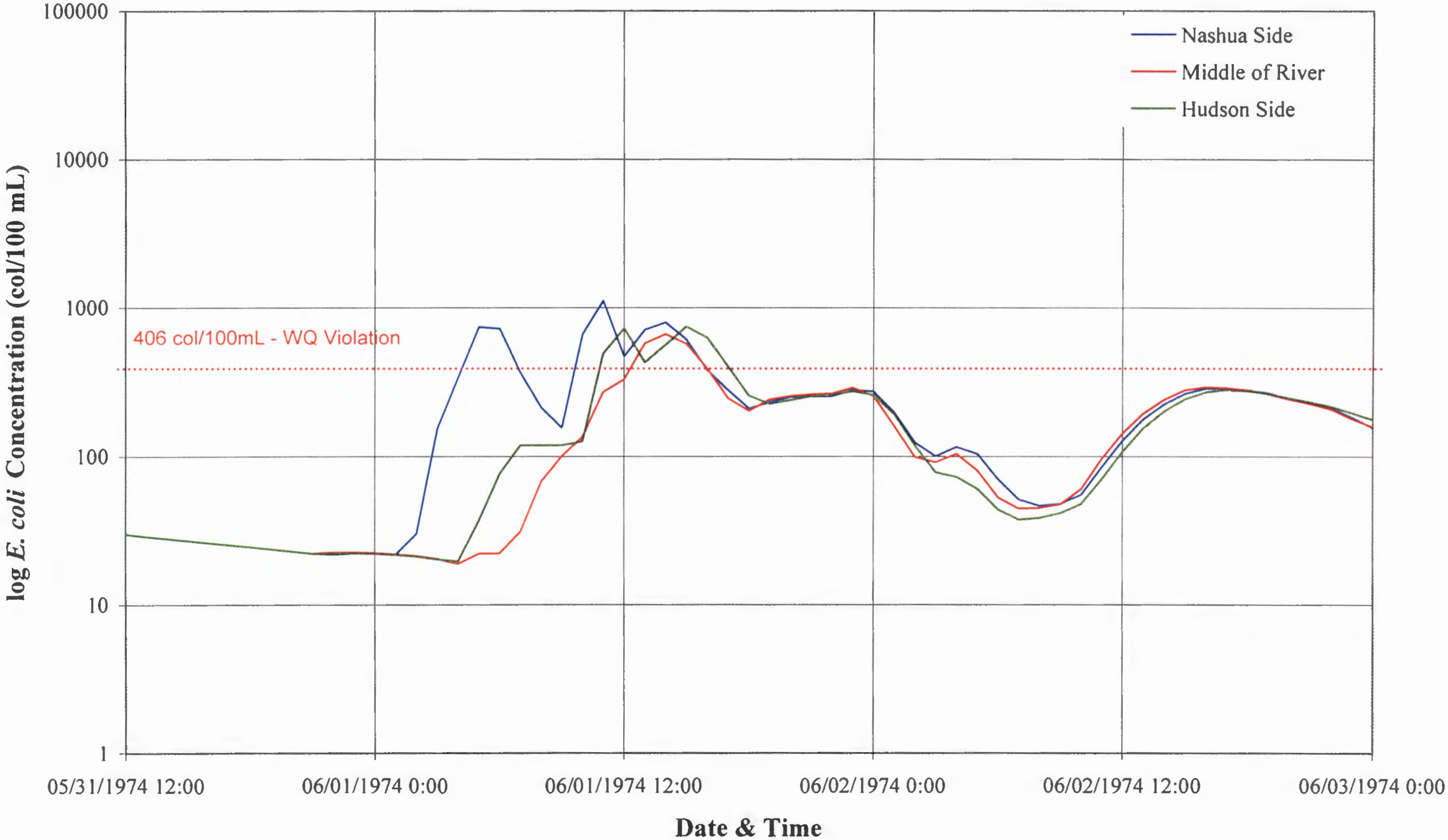

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FIGURE 7-15. TIME vs. *E coli* BACTERIA CONCENTRATION AT MERRIMACK RIVER, RIV-3
1-YEAR DESIGN STORM, CRP

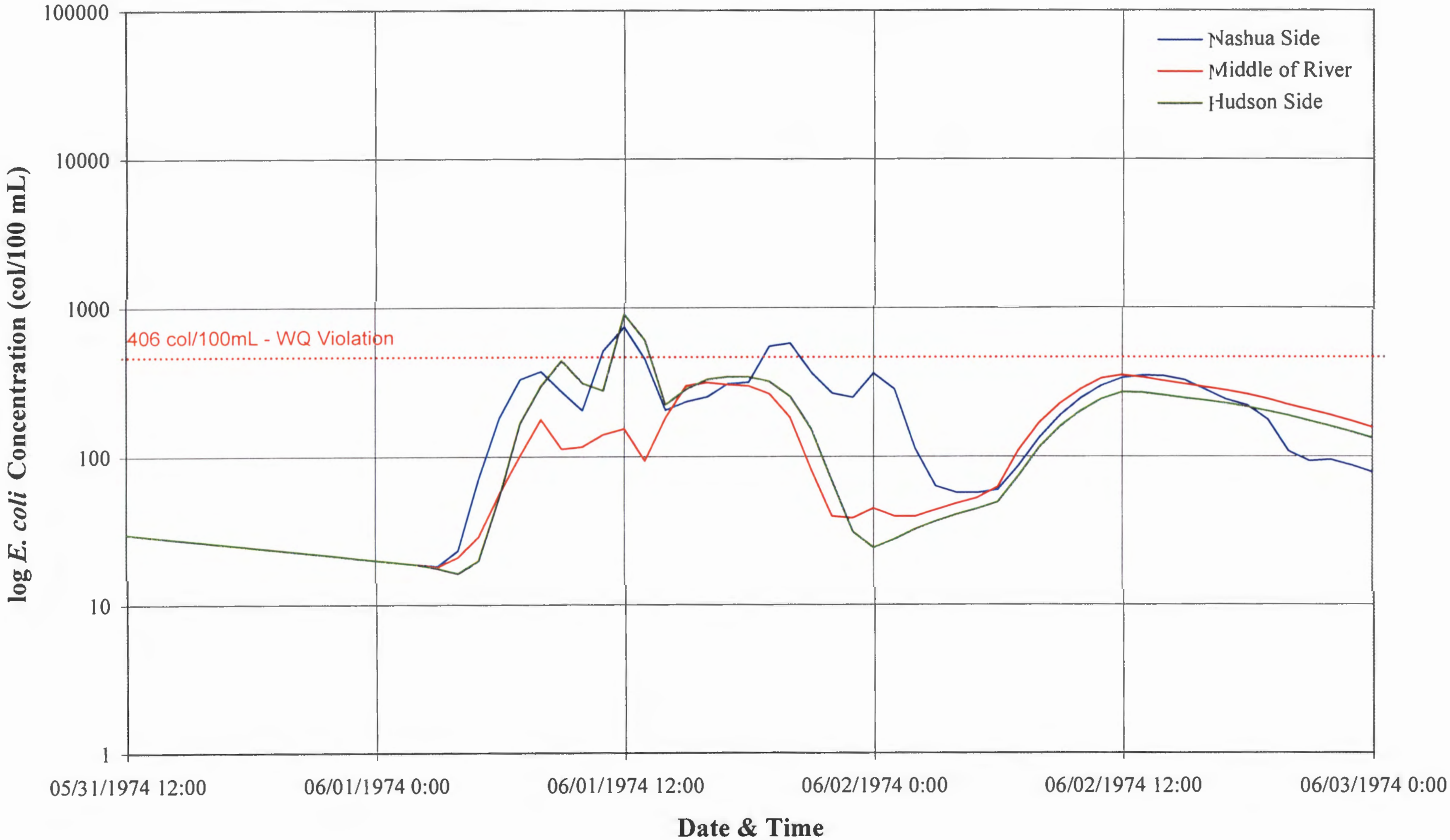


**FIGURE 7-16. TIME vs. *E coli* CONCENTRATION AT NASHUA RIVER, RIV-4
1-YEAR DESIGN STORM, CRP**

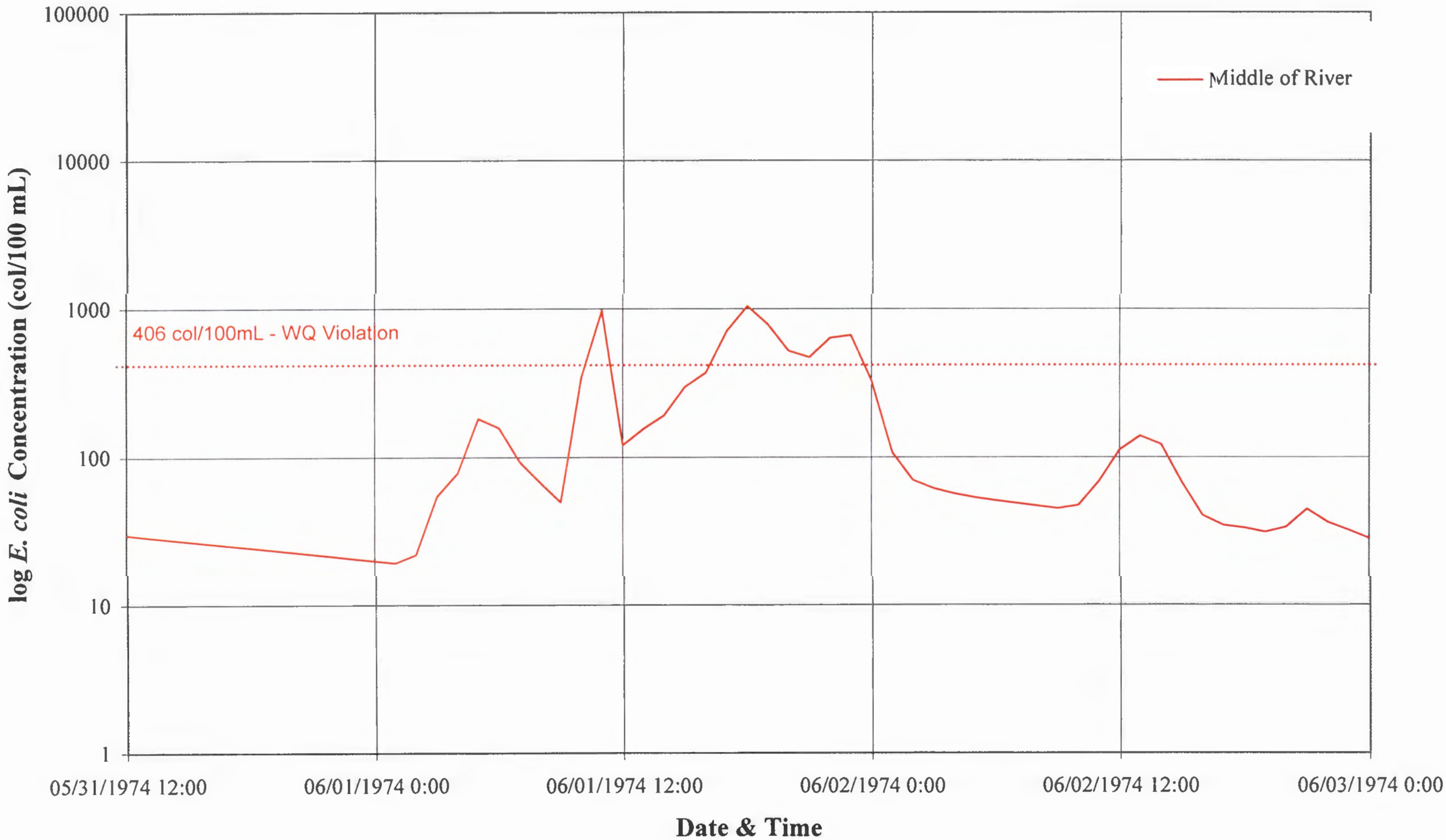


FIGURE 7-17. TIME vs. *E. coli* BACTERIA CONCENTRATION AT MERRIMACK RIVER, RIV-2
1-YEAR DESIGN STORM, RECOMMENDED PLAN

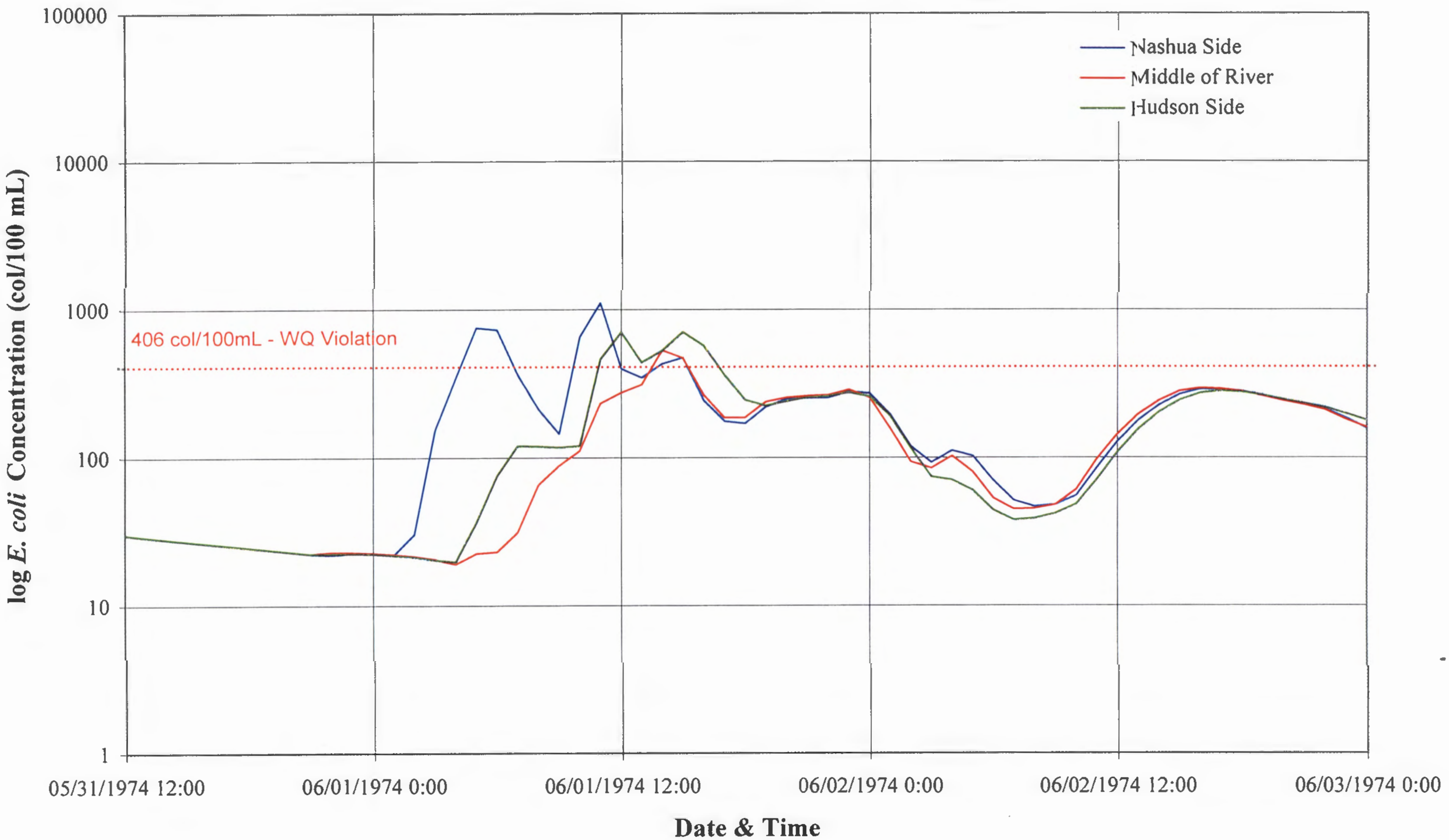


**FIGURE 7-18. TIME vs. *E. coli* BACTERIA CONCENTRATION MERRIMACK RIVER, RIV-3
1-YEAR DESIGN STORM, RECOMMENDED PLAN**

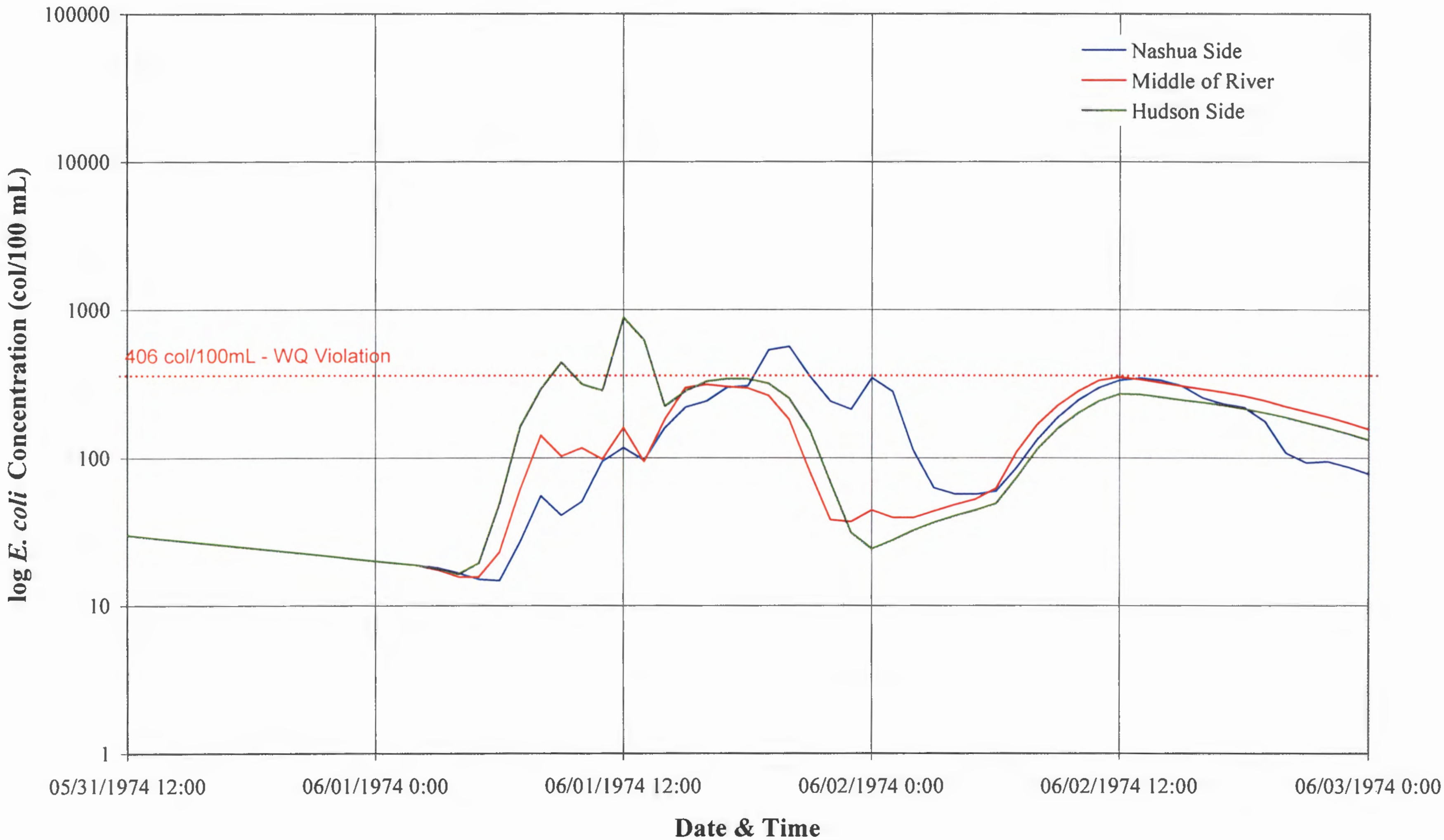
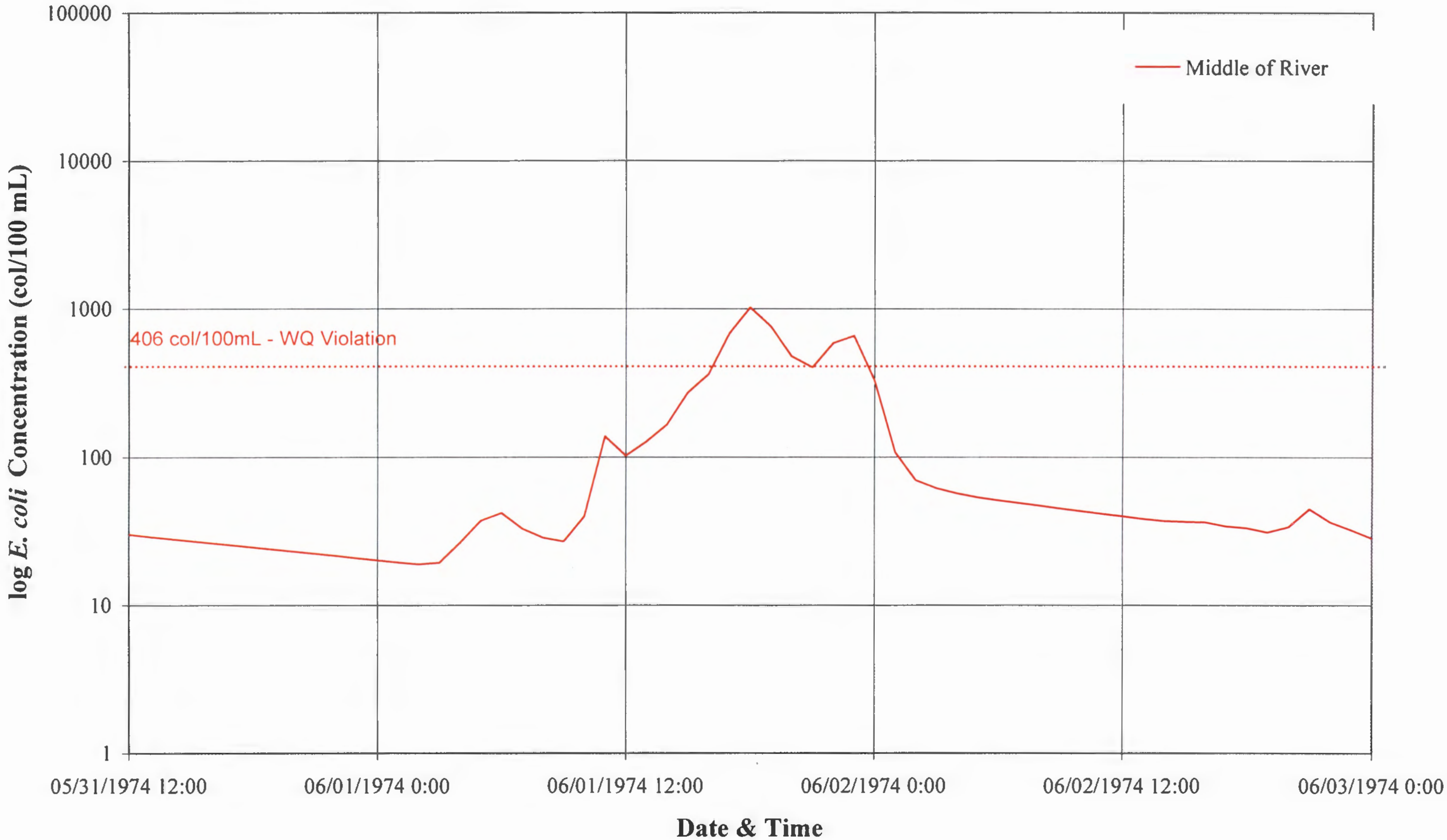


**FIGURE 7-19. TIME vs. *E. coli* BACTERIA CONCENTRATION AT NASHUA RIVER, RIV-4
1-YEAR DESIGN STORM, RECOMMENDED PLAN**



(Refer to Figures 7-12, 7-15, and 7-18). The significant improvement in water quality at this location can be attributed to the control of the East Hollis Street CSO, which is located near RIV-3.

For the 1-year storm, no untreated CSO will discharge under either the CRP or the Recommended Plan. Therefore, predicted elevated bacteria levels in the receiving waters can be attributed to the stormwater runoff and the impact of upstream sources. The volume of stormwater runoff would be greater under the CRP than under the Recommended Plan, due to sewer separation. As noted in the isopleths above, the increase in stormwater runoff associated with implementing the CRP would negatively impact water quality. Although both the CRP and the Recommended Plan significantly improve water quality compared to the NFBC, the benefits are greater under the Recommended Plan.

The benefit of the Recommended Plan over the CRP can be seen by comparing Figures 7-14 and 7-17 (RIV-2, CRP and Recommended Plan, respectively). The magnitude of the second peak is much less in Figure 7-17 (Recommended Plan) than in Figure 7-14 (CRP). Furthermore, the second peak observed in the time versus concentration plot in Figure 7-16 (RIV-4, CRP) has been reduced in Figure 7-19 (RIV-4, Recommended Plan).

7-2

The results depicted by Figure 7-11 to 7-19 are summarized in Table 7-1. This table presents the magnitude and total hours of violation of the water quality standard for each location (RIV-2, RIV-3, RIV-4) and conditions (NFBC, CRP, Recommended Plan) depicted by the figures. For locations RIV-2 and RIV-3, this table presents results for the Nashua side of the Merrimack River, only. Inspection of this table clearly shows the benefit of both the CRP and the Recommended Plan, in terms of magnitude and duration of water quality standard violation, at each location, as compared to NFBC. For example, at location RIV-2, both the hours and magnitude of violation are lower for the CRP and Recommended Plan as compared to NFBC. Inspection of this table also clearly shows that the Recommended Plan developed in this study is superior to the CRP in terms of magnitude and duration of water quality standard violation at locations RIV-1, RIV-3 and RIV-4.

TABLE 7-2. HOURS AND MAGNITUDE OF WATER QUALITY STANDARD VIOLATION FOR 1-YEAR STORM AT RIV-2, RIV-3, AND RIV-4

Figure	Location	Condition	Hours of Violation	Magnitude of Violation (col/100mL)
7-11	RIV-2	NFBC	11	2,700
7-14	RIV-2	CRP	8	1,100
7-17	RIV-2	Recommended Plan	7	1,000
7-12	RIV-3	NFBC	12	14,000
7-15	RIV-3	CRP	5	740
7-18	RIV-3	Recommended Plan	3	560
7-13	RIV-4	NFBC	9	25,000
7-16	RIV-4	CRP	8	1,100
7-19	RIV-4	Recommended Plan	6	1,000

Note: Hours and magnitude of violation based on model predictions for Nashua side of River for locations RIV-2 and RIV-3.

SUMMARY

Results of the water quality modeling show that both the CRP and the Recommended Plan have considerable benefit over the NFBC. Despite the elimination of CSO activations in the 3-month and 1-year design storm for both the CRP and Recommended Plan, the increase in stormwater runoff from complete sewer separation (CRP) does negatively impact water quality and as a result, the benefits of the CRP are not as great as the Recommended Plan.

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CHAPTER 8

SYSTEM OPTIMIZATION MEASURES

System optimization measures (SOMs) are relatively low cost, easy to implement modifications to the existing sewer system that reduce or eliminate CSOs. Optimizing the existing sewer system can reduce the extent of CSO control measures required to meet New Hampshire Department of Environmental Services (DES) and EPA CSO control requirements. The proposed system optimization measures would typically be implemented prior to more expensive CSO control alternatives.

System optimization measures that were considered include modifying or repairing regulators, replacing or repairing backwater gates, and increasing the size of regulator outlets (dry weather connections) to the interceptor system. These measures were identified after the existing combined sewer system in Nashua was evaluated with the calibrated hydraulic model. The system optimization analysis focused on system hydraulics at the NWTF and at upstream CSO regulators, assessing whether the storage and transport capacity of the system was being utilized effectively to minimize overflow volume and frequency.

This chapter describes the process used to develop and evaluate system optimization measures. This process includes:

- defining system optimization objectives
- outlining the types of projects to be considered for system optimization
- collecting information required to implement and evaluate the effectiveness of various system optimization measures
- evaluating SOM alternatives
- selecting the recommended system optimization measures

The recommended system optimization measures, and their effects on the CSO frequency and volume, are presented at the end of this chapter.

SYSTEM OPTIMIZATION OBJECTIVES

Specific objectives were developed to provide a benchmark for measuring the effectiveness of SOM alternatives. The objectives are presented below.

Reduce CSO Discharge Frequency and Volume

Achieving quantifiable reductions in CSO volumes during significant hydrologic events, even if modest in some cases, and reducing the annual frequency of CSO discharges through control of smaller, frequently occurring storm events are both system optimization objectives. Reductions in both total volume and frequency of discharge were considered in evaluating the effectiveness of optimization measures.

For this project, impacts were assessed at each regulator, and on a system-wide basis. This was necessary because in some cases an optimization measure could decrease CSO discharges at one regulator while causing discharges to increase at other regulators.

Eliminate CSO Discharges Where Possible

In some instances, it may be found that in-system modifications could make it possible to eliminate CSOs. Eliminating CSOs is a system optimization objective that is consistent with the state and national CSO policies. It reduces National Pollution Discharge Elimination System (NPDES) reporting requirements, and can provide additional benefits where a discharge to an environmentally sensitive area can be eliminated.

Reduce Operational Problems

Inspections of the Nashua sewer system and historical data identified some structures as being especially prone to operational problems such as blockages (plate in Lock Street CSO regulator) and river inflow (Tampa Street CSO regulator). An objective of system optimization measures is to reduce these and other operational problems in the sewer system.

Limit Increases to Hydraulic Grade Line

The hydraulic grade line (HGL) is an imaginary line connecting the water surface elevations from one manhole to the next in the sewer system. Where the HGL is below the crown (top) of the pipe, it coincides with the water surface in the pipe. When the HGL is above the crown of the pipe, that pipe is surcharged. This is a common occurrence in the vicinity of the CSO regulators during wet weather.

Certain optimization measures, such as raising regulator overflow weirs, can increase HGLs in upstream conduits. Increasing the HGL too much can cause basement and street flooding during heavy rainfall. Avoiding potential flooding problems was another objective of system optimization.

During the system optimization evaluation, an alternative was considered unacceptable if the peak HGL came within six feet of the ground surface, unless that represented an improvement over existing conditions. Based on knowledge of typical collection systems in the New England area, if the HGL is kept six or more feet below the ground surface, flow generally will not back up into building connections. If the existing peak hydraulic grade was already within six feet of the ground surface, no increases to the peak hydraulic grade were allowed.

Improve System Operational Efficiency and Flexibility

In many cases CSO regulators are impacted by how the downstream facilities are operated. Operating procedures at the NWTF were reviewed to confirm that they were being simulated correctly in the MOUSE model (refer to Chapter 5), and to determine if changes at the plant or in downstream reaches of the interceptors could be made that would reduce the overflows at upstream CSO regulators.

SYSTEM OPTIMIZATION PROJECT TYPES

Several types of system optimization projects were identified to attain the objectives described above. These types of projects are discussed below.

Downstream Improvements Impacting Multiple Regulators

In some instances, hydraulic restrictions were identified that, if improved, might impact multiple upstream regulators. In these cases, improving the downstream condition was considered prior to evaluating system optimization measures at the upstream regulators. One example of this type of project would be the wet weather bypass at the NWTF.

Maximizing Use of Interceptors with Available Capacity

In some cases, interceptor reaches were identified as having additional, unused capacity for conveying and/or storing flows. Where these sections of interceptor are downstream of a regulator, modifications were considered to increase flow from the regulator to the interceptor. This could be done by increasing the diameter, slope, or head (hydraulic gradient) on the existing connector pipe.

In-System Storage Opportunities

On this project, unused interceptor capacity provided the opportunity for in-line flow storage. In that case, diverting more water from the regulators to the interceptors would reduce CSO volumes. This could be accomplished by raising weirs and increasing the diameter of the dry weather connections. Storage capacity was also available, to a lesser extent, in the pipe system tributary to the CSO regulators. By raising weir elevations, additional combined flow could be stored behind the weirs. That stored wastewater would then drain into the downstream interceptor network by gravity when interceptor capacity became available.

ASSEMBLE EXISTING SYSTEM INFORMATION

The current configuration of CSO regulators, and the operation of the collection system, were evaluated in order to assess the effectiveness of potential system optimization measures.

Collect information on CSO Regulators

Information gathered regarding physical constraints at regulators was used to determine whether individual system optimization alternatives were feasible. Hydraulic information on each regulator was used further in the hydraulic evaluation process, discussed later in this chapter. A summary of the regulator information is presented in Table 8-1.

Identify Hydraulically Related Subsystems

Interconnections between adjacent CSO tributary systems provide opportunities for flow from one system to be diverted to another. This can be beneficial where one CSO system is at hydraulic capacity under peak flow conditions, while the adjacent system still has capacity available. Through minor adjustments in pipe configuration or weir elevations, flow can sometimes be diverted to an adjacent system in an effort to reduce overflows. Interconnections between adjacent tributary systems exist in Nashua, and were included in the MOUSE model. However, using the interconnections as an SOM tool was deemed infeasible since these interconnections were at high points within the pipe network. Building the hydraulic grade to an elevation high enough to utilize these interconnections would cause flooding in the downstream system. As a result, system optimization through the use of interconnections was not evaluated in this project.

Perform Baseline Hydraulic Evaluations

The hydraulic model described in Chapter 5 was used to simulate flows in the combined sewer system under a variety of wet weather conditions. For SOM evaluations, the model was used to simulate the hydraulic response of the sewer system for the hydrological conditions experienced

TABLE 8-1. SUMMARY OF REGULATOR INFORMATION

NPDES Discharge Number	Location	Regulator		Primary Influent Line			Weir		Dry Weather Connection			Intercepting Sewer	
		Rim el. (feet)	Invert el. (feet)	Diam. (inches)	Crown el. (feet)	Invert el. (feet)	Length (feet)	Elevation (feet)	Diam. (inches)	U/S Invert el. (feet)	D/S Invert el. (feet)	U/S Diam. (inches)	D/S Diam. (inches)
CSO 002	Salmon Brook	28.32	17.50	48	25.00	21.00	8.75	23.37	16	18.01	18.00	30	24
CSO 003	Farmington Road	29.00	17.95	24	19.95	17.95	6.80	20.30	10	17.95	17.90	24	24
CSO 004	Burke Street	24.00	8.50	12	10.52	9.52	10.00	11.20	10	8.50	8.25	66	66
CSO 005	East Hollis Street	23.68	5.80	54	10.30	5.80	8.00	8.30	24	6.35	6.30	66	66
CSO 006	Nashua River	27.69	3.90	-	-	-	6.00	9.03	-	-	-	108	54
CSO 007	Tampa Street	32.87	24.53	36	27.53	24.53	14.00	28.60	10	24.53	24.30	48	54
CSO 008	Broad Street	46.10	37.84	24	39.84	37.84	10.40	39.07	18	37.84	37.40	-	18
CSO 009	Lock Street	49.80	28.25	48	33.05	29.05	1.00	29.79	10	28.25	28.20	48	72

Elevations are in feet based on Nashua City Datum (NCD)
Nashua River CSO structure is on the interceptor and is therefore not configured like a conventional CSO regulator

during the 3-month, 1-year, and 2-year storms. Results of the 3-month and 1-year storm were used to assess CSO volume and HGL elevations, while results of the 2-year storm were only used to evaluate HGL elevations since controlling the overflow of a 2-year storm was out of the scope of SOM objectives, but checking hydraulic grades in this larger-sized event was important.

Evaluation of Treatment Plant Performance

The collection system model predicts that during the 3-month and 1-year design storm events, flow in the interceptors tributary to the NWTF will exceed the peak wet weather capacity of the NWTF. When this occurs, the water level in the interceptors will rise until available storage capacity in the interceptors is consumed. Once the HGL in the interceptors reaches the lowest CSO discharge elevations, combined sewage will be released out the overflows. Hydraulic profiles of the NMI, from the NWTF through the Nashua River CSO structure, show this condition for both the 3-month storm (Figure 8-1) and the 1-year storm (Figure 8-2). These figures show that this interceptor will be surcharged during these design storms. As noted in Figures 8-1 and 8-2, the peak hydraulic grade rises above the overflow weir at the East Hollis Street CSO structure (005). When this occurs, water from the interceptor will overflow the weir and discharge into the Merrimack River.

Increasing the capacity of the NWTF could reduce the duration of surcharging during some storm events, and eliminate it during other, smaller events. This would increase available storage capacity in the interceptors and in the lower lying combined sewers tributary to the CSO regulators. System optimization measures at upstream CSOs could then be implemented that would to take advantage of this new in-system storage capacity, and reduce CSO discharges.

NWTF Bypass

Increasing the capacity of the existing NWTF would not be a simple undertaking. The 50 mgd wet weather capacity of the NWTF is established by the limiting hydraulic capabilities of the influent pumps. In order to increase the capacity of the facility, these pumps and associated

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FIGURE 8-1. PEAK HGL IN NORTH MERRIMACK INTERCEPTOR NEAR FUTURE BASELINE CONDITIONS, 3-MONTH DESIGN STORM

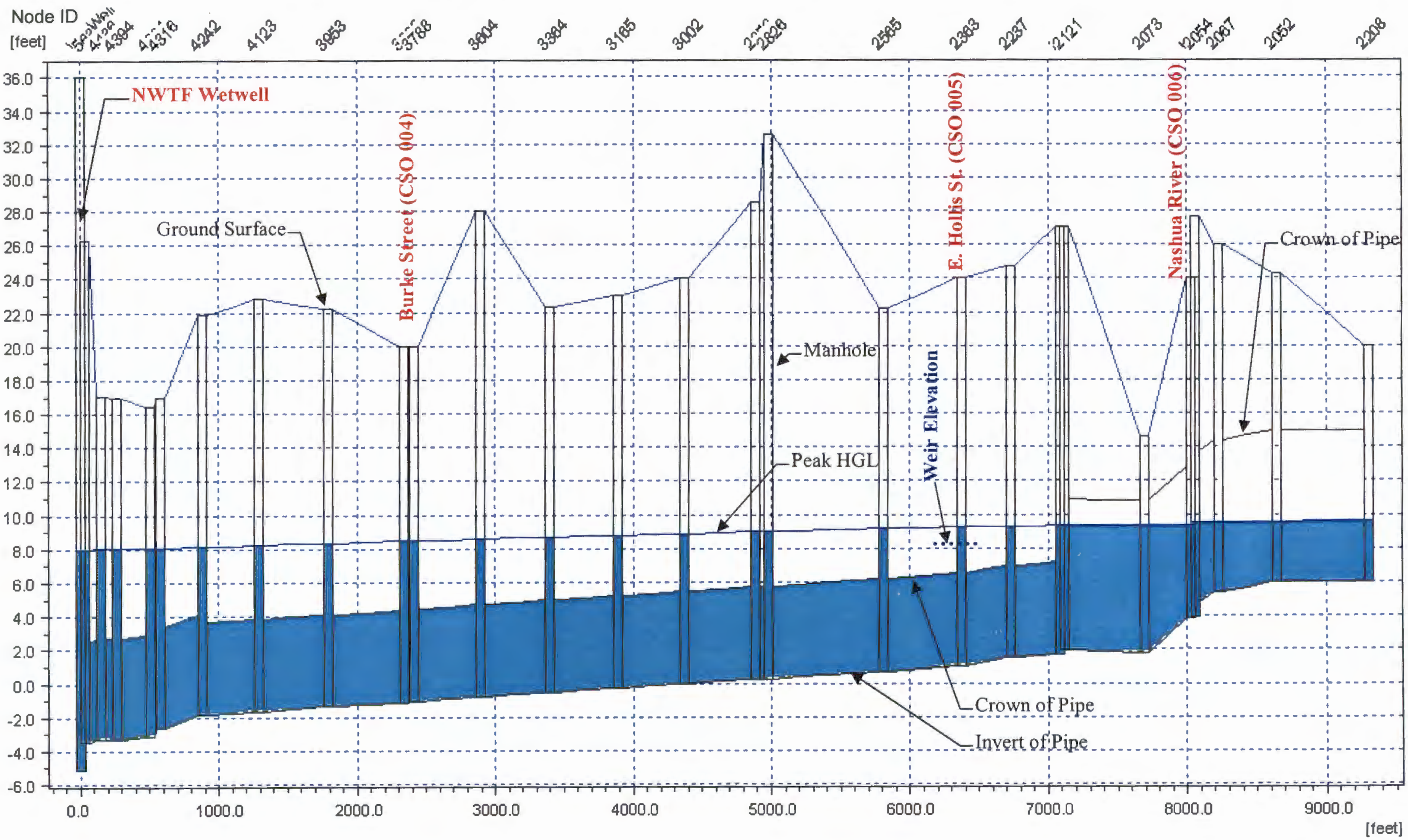

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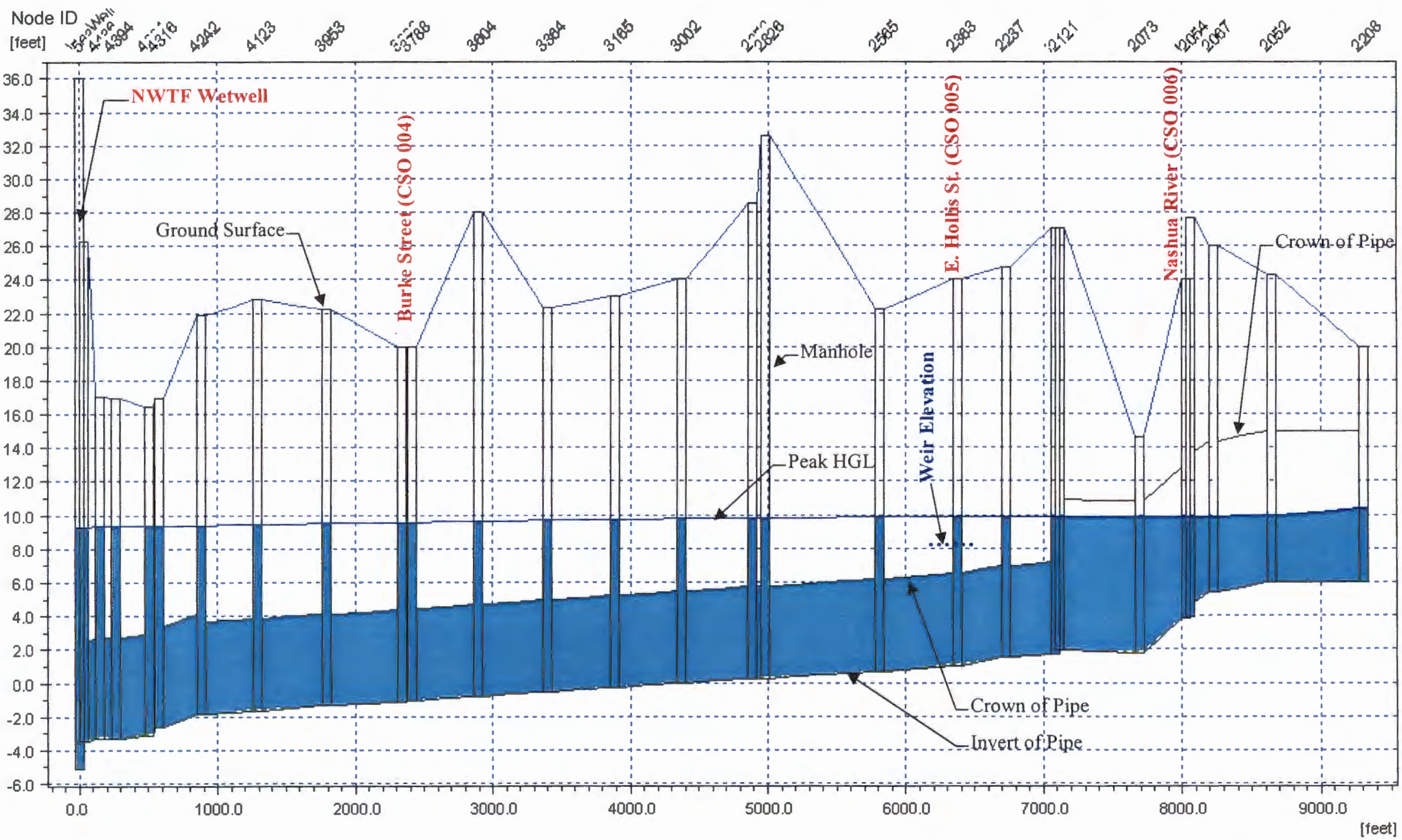
FIGURE 8-2. PEAK HGL IN NORTH MERRIMACK INTERCEPTOR NEAR FUTURE BASELINE CONDITIONS, 1-YEAR DESIGN STORM



structures (e.g. bar screens and influent channels) would also have to be modified to increase capacity. In addition, modifications would be required for existing unit processes to maintain existing levels of treatment at higher design flows. Increasing the maximum wet weather treatment capacity would increase the wet weather to dry weather flow operating ratio, which is currently 5:1. Increasing that ratio would likely make it extremely difficult to maintain consistent effluent quality at the treatment plant.

As an alterative to increasing the capacity of the treatment plant, a wet-weather bypass upstream of the NWTF was considered. This was modeled as a high weir, upstream of the headworks of the plant. In the model, all flow that exceeded a set elevation would overflow to a wet-weather bypass unit, parallel to the existing wastewater treatment train. The optimum weir elevation was determined through successive iterations, so that the first 50 mgd of flow would continue to the NWTF, and the excess would bypass to the wet weather unit.

then what

As shown in Figures 8-3 and 8-4, the optimum weir elevation would be between el. 2.0 and el. 3.0 (NCD). Figure 8-3 shows that the NWTF continues to peak at 50 mgd with the upstream bypass elevation set at el. 3.0. With the upstream bypass elevation set at el. 2.0, flow to the NWTF is reduced below 50 mgd. For purposes of this study, the weir elevation selected was el. 3.0 (NCD).

Providing additional flow capacity just upstream of the NWTF provided relief from the backwater condition that occurred in the main interceptors. Figures 8-5 and 8-6 show hydraulic profiles for the 3-month and 1-year storms, this time with relief just upstream of the NWTF. These figures should be compared to Figures 8-1 and 8-2, which show the same hydraulic profile but without relief just upstream of the NWTF. Note that relief just upstream of the NWTF decreases the HGL in the vicinity of the NWTF wetwell by over 4 ft. in the 3-month storm. At CSO 005 (E. Hollis St.) the HGL is lowered by over 1.5 ft., and is below the CSO weir elevation in the 3-month storm. In the 1-year storm, the HGL in the vicinity of the NWTF wetwell is lowered by over 5 ft. and the HGL at CSO 005 is lowered by almost 2 ft., nearly to the CSO weir elevation. Comparison of these profiles, with and without relief just upstream of the NWTF, demonstrates that relatively significant system benefits can be achieved by maximizing flow to

FIGURE 8-3. FLOW INTO NWTF WITH BYPASS SET AT ELEVATION 3.00 (NCD)
3-MONTH DESIGN STORM

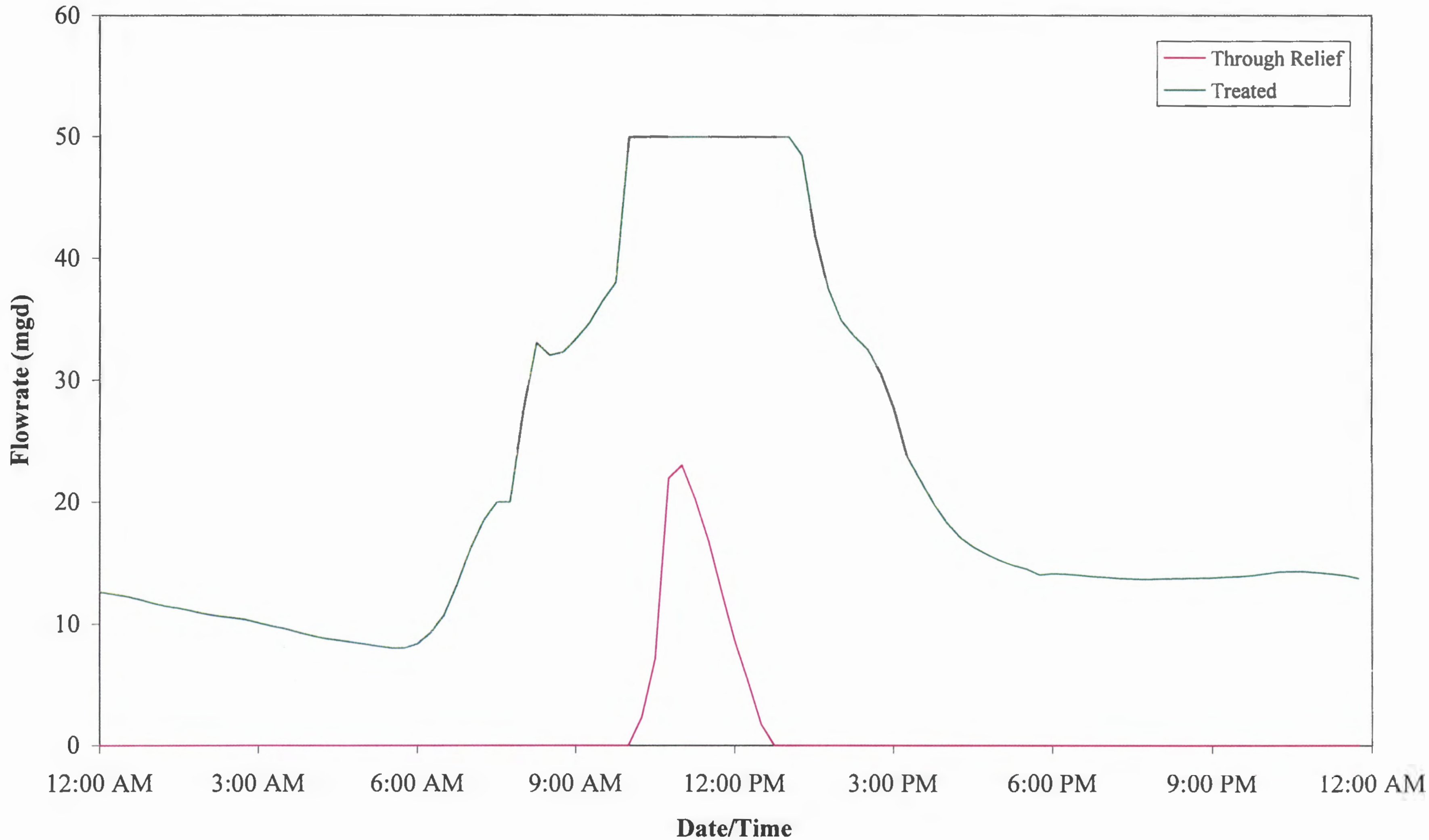


**FIGURE 8-4. FLOW INTO NWTF WITH BYPASS SET AT ELEVATION 2.00 (NCD)
3-MONTH DESIGN STORM**

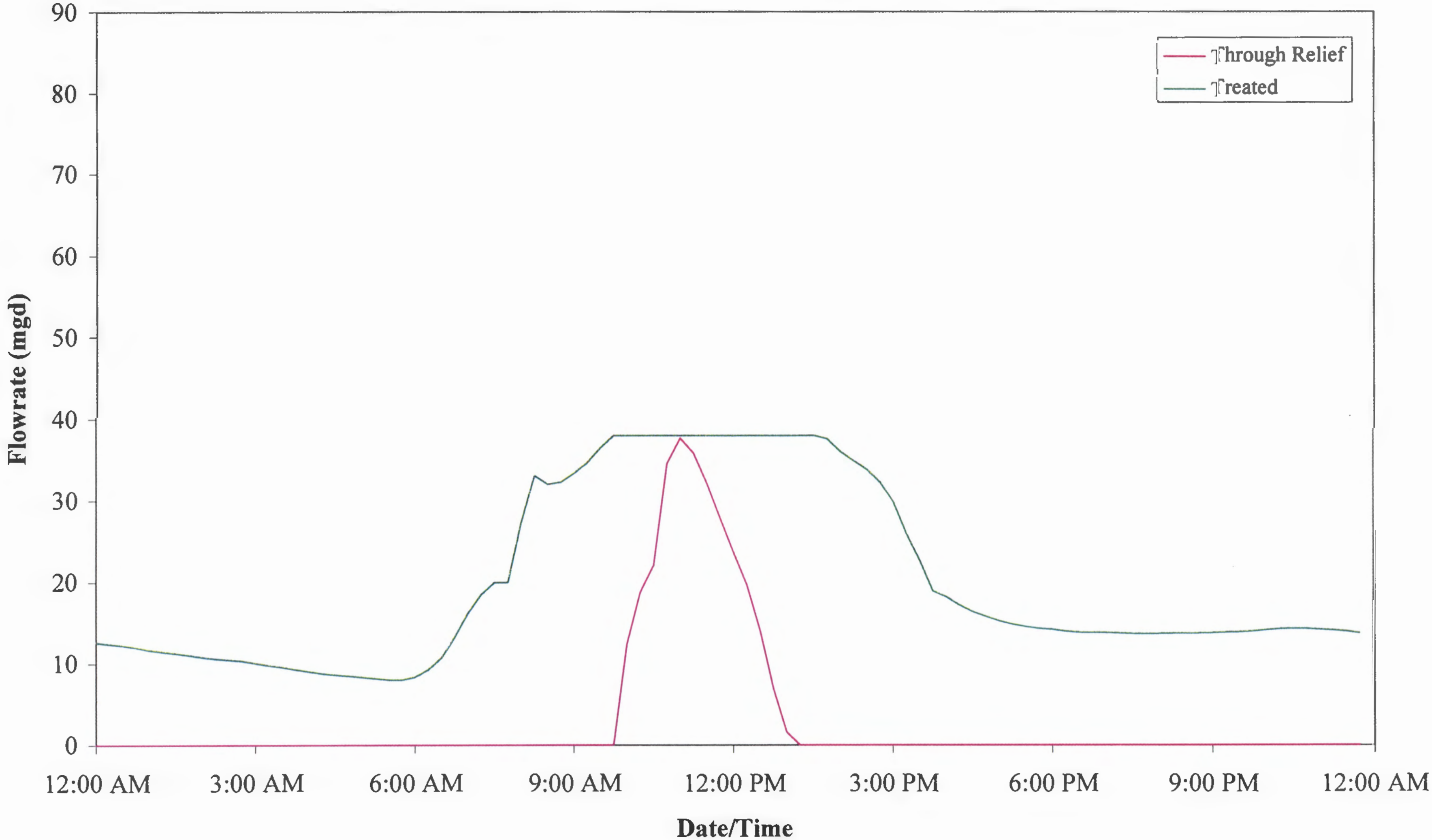


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FIGURE 8-5. PEAK HGL IN NORTH MERRIMACK INTERCEPTOR NEAR FUTURE BASELINE CONDITIONS WITH RELIEF AT NWTF, EL. 3.00 (NCD) 3-MONTH DESIGN STORM

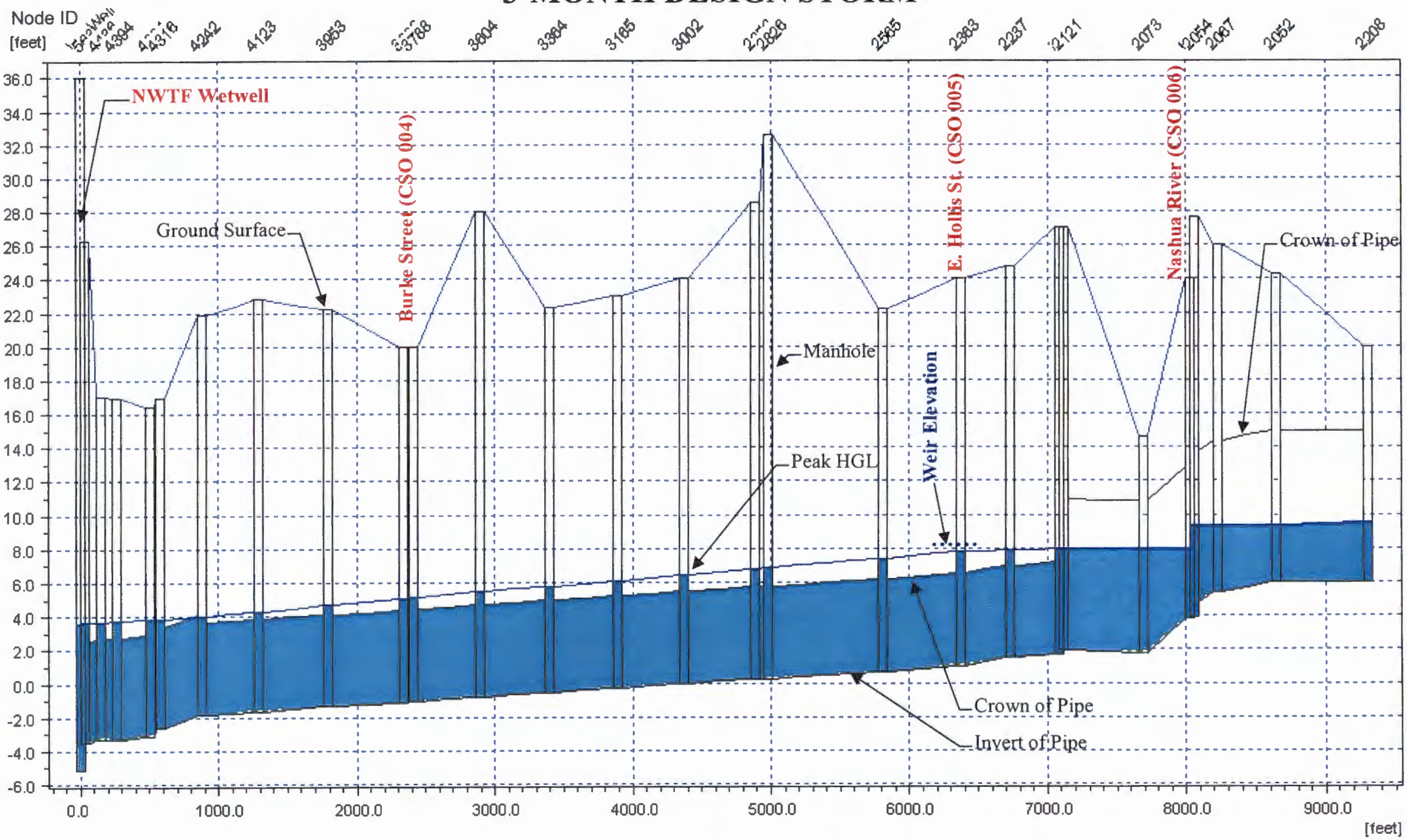

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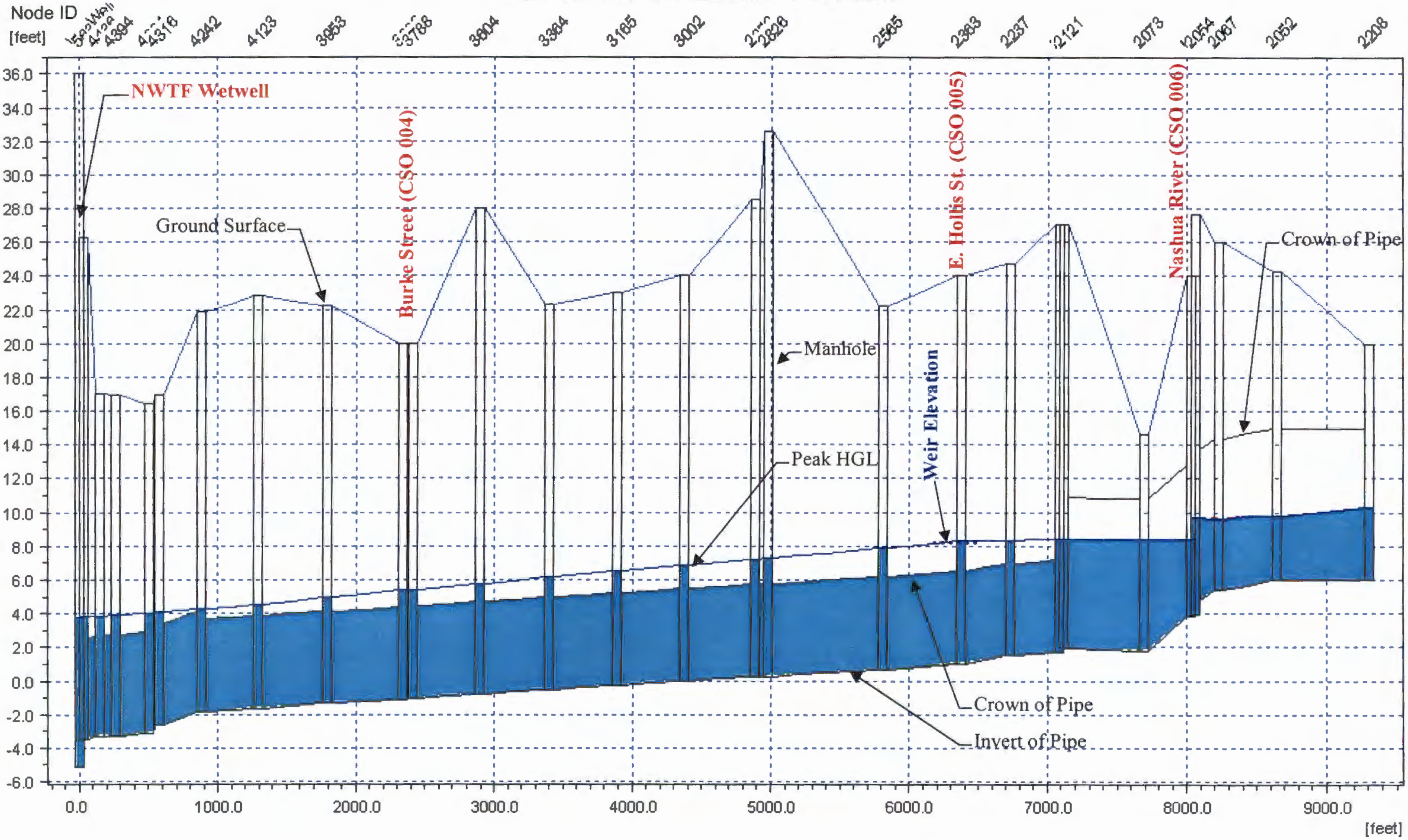
FIGURE 8-6. PEAK HGL IN NORTH MERRIMACK INTERCEPTOR NEAR FUTURE BASELINE CONDITIONS WITH RELIEF AT NWTF, EL. 3.00 (NCD) 1-YEAR DESIGN STORM



the vicinity of the NWTF. Table 8-2 shows the differences in CSO volumes at individual CSO regulators, and on a system-wide basis, after providing the wet-weather bypass just upstream of the NWTF.

TABLE 8-2. CSO VOLUMES WITH NWTF BYPASS

CSO Regulator	Overflow Volume in MG			
	3-Month Storm		1-Year Storm	
	Existing Conditions (NFBC)[1]	With NWTF Bypass	Existing Conditions (NFBC)	With NWTF Bypass
CSO 002 Salmon Brook	0.00	0.00	0.00	0.00
CSO 003 Farmington Rd.	0.12	0.12	0.32	0.32
CSO 004 Burke St.	0.39	0.39	0.83	0.84
CSO 005 E. Hollis St.	0.53	0.33	1.20	0.86
CSO 006 Nashua River	0.91	0.32	2.25	1.26
CSO 007 Tampa St.	0.00	0.00	0.05	0.02
CSO 008 Broad St	0.20	0.21	0.50	0.51
CSO 009 Lock St.	0.03	0.04	0.08	0.08
TOTAL	**2.18**	**1.41**	**5.23**	**3.89**

Note: [1] NFBC is near future baseline conditions (refer to Chapter 6).

With more capacity available in the interceptor network tributary to the NWTF, system optimization measures could more appropriately be evaluated.

Development of System Optimization Measures

Projected future hydraulic conditions in the interceptors served as a basis for evaluating potential system optimization measures at upstream CSOs. These proposed measures were evaluated on the basis of cost and amount of overflow reduction. Other non-monetary factors such as ease of construction and operation, and institutional/implementation constraints were also considered. Some typical system optimization measures considered for the Nashua collection system include:

- Increasing the hydraulic capacity of connecting pipes or short sections of interceptor, by increasing either the pipe diameter or the slope.
- Modifying regulator structures, for example by building or raising a weir.

EVALUATION OF ALTERNATIVES

The collection system model was used to predict overflow reductions for the optimization of alternatives. These were evaluated in terms of total overflow volume in millions of gallons for the 3-month and 1-year design storm, and peak HGLs for the 1-year and 2-year design storms. Starting with the near future baseline condition (NFBC) as a baseline, predicted CO volumes and HGLs for various SOM arrangements were compared to predicted CSO volumes and HGLs under NFBC to determine whether system optimization goals were achieved.

Since the CSO regulators are hydraulically interrelated within each subsystem (NRI and NMRI), modifications proposed for individual regulators were evaluated in terms of net overflow reduction on a subsystem and system-wide basis.

Iterative model simulations were run with slight modifications to weir elevations and dry weather connecting pipe diameters to minimize total overflow volumes without causing excessive surcharging in the sewers. The overflow volumes and peak HGLs for the 3-month and 1-year storms, comprising the NFBC and the recommended plan, are presented later in this chapter.

Tabular output was developed for each iteration of proposed SOMs, including predicted CSO volumes and peak water surface elevations at key points. An example of this output is presented in Table 8-3. Information in these tables was used to evaluate performance against the SOM objectives, and provided direction for further SOM iterations.

Profiles showing the peak hydraulic grade line in the interceptors and combined sewer lines into each regulator were also reviewed. These profiles made it possible to see the impacts to the hydraulic grade along the entire length of pipe. Profiles showed that in some cases, removing a hydraulic restriction could cause flooding upstream or downstream of the regulator if the system optimization measure were incorporated. An example SOM hydraulic profile is shown in Figure 8-7.

RESULTS OF THE SYSTEM OPTIMIZATION ANALYSES

System optimization measures were evaluated on an individual regulator basis and on a system-wide basis. Results of the iterative process for each regulator are presented in the discussion below.

The ability to bypass flow just upstream of the NWTF would provide considerable additional capacity in the large diameter interceptors. Therefore, one of the additional SOM goals was to divert as much flow from the Broad Street CSO structure (008) and the Tampa Street CSO structure (007) as possible into the NRI. Maximizing conveyance of flow from the upstream reaches of the collection system could eliminate the need for local treatment or storage adjacent to these most remote CSO regulators. Controlling CSO activation through SOMs was preferred at these remote locations since they are the most removed from the NWTF and would require NWTF personnel to travel a significant distance during wet weather operation. Controlling CSO through storage and treatment units was deemed more acceptable at CSOs in closer proximity to the NWTF, where plant personnel would not have to travel a significant distance to respond during wet weather facility activations.

Broad Street (CSO 008). The model indicated that CSO activations at this location are primarily caused by the hydraulic constriction at the dry weather connection. Flow diverted to the NRI passes through a 10-inch connection at the CSO structure, then continues to the NRI in an 18-inch pipe. Hydraulic profiles of the NRI show there is capacity available in this interceptor during peak flow conditions. However, increasing the diameter of the 10-inch connection alone would not provide adequate relief to convey more flow to the NRI. The model indicates that the 18-inch connecting pipe must also be replaced with a 30-inch diameter pipe. With that modification, overflows would be significantly reduced.

Further recommended adjustments to this structure include raising the weir by 0.5 feet from el. 39.07 (NCD) to el. 39.57 (NCD). Raising the weir in this regulator was not intended to increase in system storage. Since the slope of the influent combined sewer is very steep, this will not

TABLE 8-3. SYSTEM OPTIMIZATION MEASURE EVALUATION TABLE

CSO	System Optimization Measure (SOM)	Overflow Elevation (ft)		Diameter of Dry-Weather Connection (ft)		Ground Surface at Regulater (NCD)	1-year Overflow Volume (Mgal)		3-Month Peak Water Levels (ft)				1-Year Peak Water Levels (ft)				2-Year Peak Water Levels (ft)			
		Existing	SOM	Existing	SOM		w/o SOM	w/ SOM	Location	w/o SOM	w/ SOM	Change	Location	w/o SOM	w/ SOM	Change	Location	w/o SOM	w/ SOM	Change
002	No System Optimization Measures	23.37	23.37	1.33	1.33	28.32	0.00	0.00	Influent	23.75	23.75	0.0	Influent	24.00	24.00	0.0	Influent	24.3	24.27	0.0
									Regulator	21.20	21.20	0.0	Regulator	22.43	22.43	0.0	Regulator	23.2	23.23	0.0
									Int. Conn.	9.79	9.84	0.0	Int. Conn.	9.98	10.09	0.1	Int. Conn.	10.1	10.28	0.2
									Int. U/S	10.54	10.76	0.2	Int. U/S	10.62	11.01	0.4	Int. U/S	10.7	11.21	0.5
									Int. D/S	3.63	3.80	0.2	Int. D/S	3.82	4.04	0.2	Int. D/S	3.9	4.20	0.3
003	Increase diameter of DWC - 10" - 20"	20.30	20.30	0.83	1.67	29.00	0.32	0.00	Influent	24.35	24.30	0.0	Influent	24.62	24.53	-0.1	Influent	24.84	24.68	-0.2
									Regulator	20.67	19.61	-1.1	Regulator	21.08	19.69	-1.4	Regulator	21.88	20.45	-1.4
									Int. Conn.	14.05	14.50	0.4	Int. Conn.	14.18	16.33	2.1	Int. Conn.	14.33	18.38	4.0
									Int. U/S	14.06	14.46	0.4	Int. U/S	14.19	16.28	2.1	Int. U/S	14.33	18.32	4.0
									Int. D/S	13.86	14.21	0.4	Int. D/S	13.97	15.94	2.0	Int. D/S	14.10	17.86	3.8
									Int.R U/S	13.81	14.25	0.4	Int.R U/S	13.94	16.03	2.1	Int.R U/S	14.08	17.99	3.9
									Int.R D/S	13.95	14.37	0.4	Int.R D/S	14.09	16.19	2.1	Int.R D/S	14.23	18.20	4.0
004	Raise elevation of weir by 0.25 feet, Increase diameter of DWC - 10" - 24"	11.20	11.45	0.83	2.00	24.00	0.84	0.00	Influent	13.93	14.03	0.1	Influent	19.78	19.00	-0.8	Influent	21.09	20.93	-0.2
									Regulator	11.71	10.27	-1.4	Regulator	11.94	11.50	-0.4	Regulator	11.98	11.62	-0.4
									Int. Conn.	5.11	5.69	0.6	Int. Conn.	5.41	6.35	0.9	Int. Conn.	5.57	6.86	1.3
									Int. U/S	5.44	6.04	0.6	Int. U/S	5.76	6.75	1.0	Int. U/S	5.93	7.29	1.4
									Int. D/S	4.67	5.12	0.5	Int. D/S	4.94	5.66	0.7	Int. D/S	5.07	6.07	1
005	Raise elevation of weir by 3.0 feet	8.30	11.30	2.00	2.00	23.68	0.86	0.00	Influent	13.85	14.37	0.5	Influent	14.42	15.38	1.0	Influent	14.60	15.79	1.2
									Regulator	10.69	13.07	2.4	Regulator	12.01	14.50	2.5	Regulator	12.36	14.95	2.6
									Int. Conn.	7.80	8.57	0.8	Int. Conn.	8.29	9.60	1.3	Int. Conn.	8.57	10.47	1.9
									Int. U/S	7.89	8.64	0.8	Int. U/S	8.35	9.68	1.3	Int. U/S	8.63	10.57	1.9
									Int. D/S	7.41	8.16	0.7	Int. D/S	7.88	9.14	1.3	Int. D/S	8.14	9.96	1.8
006	Raise elevation of weir by 2.5 feet	9.03	11.53	4.5 x 4.5	4.5 x 4.5	27.69	1.26	0.00	Influent	9.35	9.90	0.6	Influent	9.70	11.27	1.6	Influent	9.98	12.18	2.2
									Regulator	9.35	9.90	0.6	Regulator	9.70	11.27	1.6	Regulator	9.98	12.18	2.2
									Int. Conn.	8.00	8.72	0.7	Int. Conn.	8.43	9.78	1.3	Int. Conn.	8.71	10.68	2.0
									Int. U/S	9.34	9.90	0.6	Int. U/S	9.69	11.27	1.6	Int. U/S	9.95	12.17	2.2
									Int. D/S	8.00	8.73	0.7	Int. D/S	8.44	9.78	1.3	Int. D/S	8.71	10.68	2.0
007	Increase diameter of DWC - 10" - 18"	28.60	28.60	0.83	1.50	32.87	0.02	0.00	Influent	27.47	27.21	-0.3	Influent	28.72	27.47	-1.2	Influent	29.11	27.96	-1.1
									Regulator	27.47	25.63	-1.8	Regulator	28.71	26.32	-2.4	Regulator	28.84	27.97	-0.9
									Int. Conn.	25.19	25.43	0.2	Int. Conn.	25.35	25.83	0.5	Int. Conn.	25.45	26.18	0.7
									Int. U/S	25.19	25.43	0.2	Int. U/S	25.34	25.83	0.5	Int. U/S	25.44	26.18	0.7
									Int. D/S	24.94	25.16	0.2	Int. D/S	25.08	25.55	0.5	Int. D/S	25.18	25.86	0.7
008	Raise elevation of weir by 0.43 feet, Increase diameter of DWC - 18" - 30"	39.07	39.50	1.50	2.50	46.10	0.51	0.00	Influent	53.63	53.63	0.0	Influent	53.85	53.85	0.0	Influent	53.93	53.93	0.0
									Regulator	39.42	38.98	-0.4	Regulator	39.57	39.35	-0.2	Regulator	39.63	39.55	-0.1
									Int. Conn.	34.55	35.31	0.8	Int. Conn.	34.59	37.23	2.6	Int. Conn.	34.60	39.22	4.6
									Int Con2	28.27	28.60	0.3	Int Con2	28.38	28.87	0.5	Int Con2	28.47	29.01	0.5
									Int. D/S	27.99	28.30	0.3	Int. D/S	28.09	28.58	0.5	Int. D/S	28.18	28.73	0.6
009	Increase diameter of DWC - 10" - 15", Remove diversion plate from DWC	29.79	29.79	0.83	1.25	49.80	0.08	0.00	Influent	32.87	32.81	-0.1	Influent	33.04	32.94	-0.1	Influent	33.10	32.98	-0.1
									Regulator	30.80	29.35	-1.4	Regulator	31.30	29.75	-1.5	Regulator	31.45	29.88	-1.6
									Int. Conn.	28.55	28.78	0.2	Int. Conn.	28.57	28.91	0.3	Int. Conn.	28.58	28.95	0.4
									Int Con2	11.01	11.11	0.1	Int Con2	11.32	11.48	0.2	Int Con2	11.44	12.18	0.7
									Int. U/S	15.70	15.71	0.0	Int. U/S	15.86	15.86	0.0	Int. U/S	15.95	15.96	0.0
									Int. D/S	10.19	10.28	1.0	Int. D/S	10.48	11.28	0.8	Int. D/S	10.61	12.18	1.6

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FIGURE 8-7. EXAMPLE OF PROFLIE USED TO EVALUATE SOMs
Farmington Road CSO (003) – Profile of Primary influent line, through DWC into SMI