A CASE STUDY OF WATERSHED-BASED POLLUTION TRADING IN THE ASSABET RIVER BASIN

By

STEVEN L. MCGINNIS

Bachelor of Science in Biological and Agricultural Engineering
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Kansas State University
Manhattan, Kansas

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Signature of Author
Steven L. McGinnis

Certified By
Dr. David H. Marks
James Mason Crafts Professor of Civil and Environmental Engineering
Thesis Supervisor

Approved By
Dr. Andrew J. Whittle
Chairman, Departmental Committee on Graduate Studies

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ABSTRACT

The Town of Acton, Massachusetts, is a community located in the Assabet River Basin that currently relies mostly upon individual sewage treatment via onsite septic systems. Due to frequent septic system failure in the regions of Acton with shallow groundwater levels, the Town of Acton has begun designing a wastewater treatment plant (WWTP) that will serve these regions of Acton. If approved by the United States Environmental Protection Agency (EPA), the Acton WWTP will discharge some of its effluent to the Assabet River, which is currently in a eutrophic state due to the phosphorus loading from five existing WWTPs upstream of Acton. As a result, the EPA has requested Acton to develop a point/nonpoint source phosphorus trading program to remove three times as much nonpoint source (NPS) phosphorus loading from Acton as will be discharged from the planned Acton WWTP. The watershed-based pollution trading that will be utilized in the proposed Acton Trading Program is a very recent water quality management development – preliminary guidance for its use was issued by the EPA in 1996 and final trading guidelines are expected to be published in late 1999. As innovative and flexible methods of maintaining water quality in unique watersheds, the few existing watershed-based pollution trading programs are extremely distinct in terms of development, implementation, and outcome. Although the use of watershed-based pollution trading is relatively unproven, observation of the existing trading programs indicates that trading has the potential to improve water quality in heavily impaired watersheds such as the Assabet River Basin. However, a comprehensive review and modeling analysis of the Acton Trading Program predicts that the planned use of point/nonpoint source trading will be ineffective in improving water quality in the Assabet River. This is due to a variety of implementation issues, including the trading location within the watershed and the small phosphorus loads targeted in the program. As a result, the study concluded that alternative point/point source trading arrangements involving the Acton WWTP and one of the other five Assabet River Basin WWTPs would provide better water quality impacts for the financial cost of improvement.

Thesis Supervisor:	Dr. David H. Marks
Title:	James Mason Crafts Professor of Civil and Environmental Engineering

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The Town of Acton, Massachusetts, is a community located in the Assabet River Basin that currently relies mostly upon individual sewage treatment via onsite septic systems. Due to frequent septic system failure in the regions of Acton with shallow groundwater levels, the Town of Acton has begun designing a wastewater treatment plant (WWTP) that will serve these regions of Acton. If approved by the United States Environmental Protection Agency (EPA), the Acton WWTP will discharge some of its effluent to the Assabet River, which is currently in a eutrophic state due to the phosphorus loading from five existing WWTPs upstream of Acton. As a result, the EPA has stated that in order to obtain a National Pollution Discharge Elimination System (NPDES) permit for the Acton WWTP, the Town of Acton must meet two conditions:

- the maximum allowable phosphorus concentration in the Acton WWTP must be less than 0.1 milligrams per liter (mg/L)
- a watershed-based point/nonpoint source phosphorus trading program should be implemented to reduce nonpoint source (NPS) phosphorus loading from the Town of Acton to the Assabet River by three times the amount that is discharged from the Acton WWTP.

Watershed-based pollution trading is a very recent water quality management development – preliminary guidance for its use was issued by the EPA in 1996 and final trading guidelines are expected to be published in late 1999. As innovative and flexible methods of maintaining water quality in unique watersheds, the few existing watershed-based pollution trading programs are extremely distinct in terms of development, implementation, and outcome. The diversity of the existing trading programs exemplifies the flexibility that exists to conform trading programs to manage nearly any site-specific watershed pollution problem. Although the use of watershed-based pollution trading is relatively unproven, observation of the existing trading programs indicates that trading has the potential to improve water quality in heavily impaired watersheds such as the Assabet River Basin.

Despite the ideal conditions for watershed-based phosphorus trading in the Assabet River Basin, the proposed Acton Trading Program is not likely to achieve its water quality objectives for the Assabet River. A comprehensive review of the Acton Trading Program and stream water quality modeling studies performed with the EPA stream water quality model, QUAL2E, indicate that the proposed Acton Trading Program will have little impact on water quality in the Assabet River. This conclusion was reached from the following determinations:

- The location of the proposed point/nonpoint source phosphorus trades is downstream of the significant phosphorus loading that impairs water quality in most of the Assabet River.
- The phosphorus loads targeted in the proposed Acton Trading Program are very small as compared to the municipality WWTP phosphorus discharges upstream of Acton.
- The nutrient catchment capability of Warner's Pond further reduces the impact of NPS phosphorus loading reductions on the Assabet River.

Further examination of the phosphorus loads involved in the Acton Trading Program indicates that the tributaries that drain Acton consistently contribute less phosphorus to the Assabet River than the proposed Acton WWTP would at the target 0.1 mg/L effluent phosphorus concentration.

Therefore, even if all of the phosphorus could be removed from these tributaries through implementation of Acton phosphorus reduction BMPs, the actual Acton NPS phosphorus loading reduction to Acton WWTP phosphorus discharge trading ratio would not even reach 1:1, let alone 3:1.

Due to the predicted minor impact of the proposed Acton Trading Program on water quality in the Assabet River, the conclusions of this study support the following recommendations:

- Alternative Acton WWTP point/point source trades with one of the five municipal WWTPs that discharge to the Assabet River upstream of Acton should be considered.
- The point/point trades could be achieved by utilizing efficient phosphorus removal via chemical addition to obtain moderate phosphorus discharge concentrations at both plants.

The combined phosphorus removal method is much more efficient than the method proposed in the Acton Trading Program, which pushes chemical phosphorus removal to its limits to obtain extremely low phosphorus effluent concentrations from only the Acton WWTP. Therefore, alternative point/point source trading arrangements should efficiently provide a better water quality benefit than the proposed Acton Trading Program.

2 INTRODUCTION TO THE ASSABET RIVER BASIN

The Assabet River Basin is located in east-central Massachusetts (See Figure 1). The basin drains approximately 135 square miles and contains nineteen small towns and one city. As can be seen in Figure 2, the Assabet River originates in an impounded swampy area located in

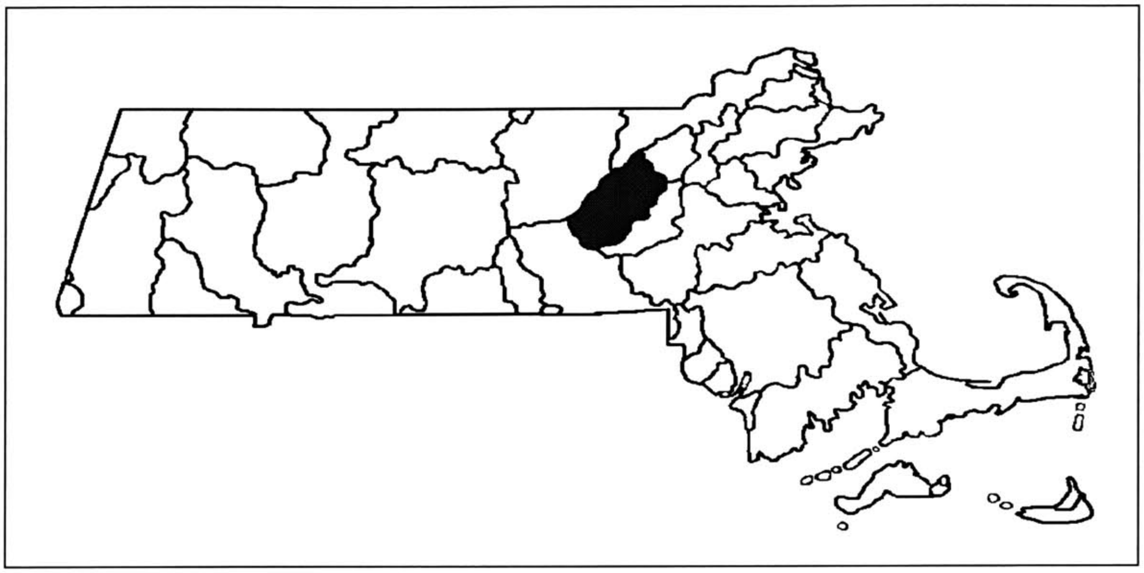

Figure 1. Location of Assabet River Basin
Source: U.S. EPA, 1999.

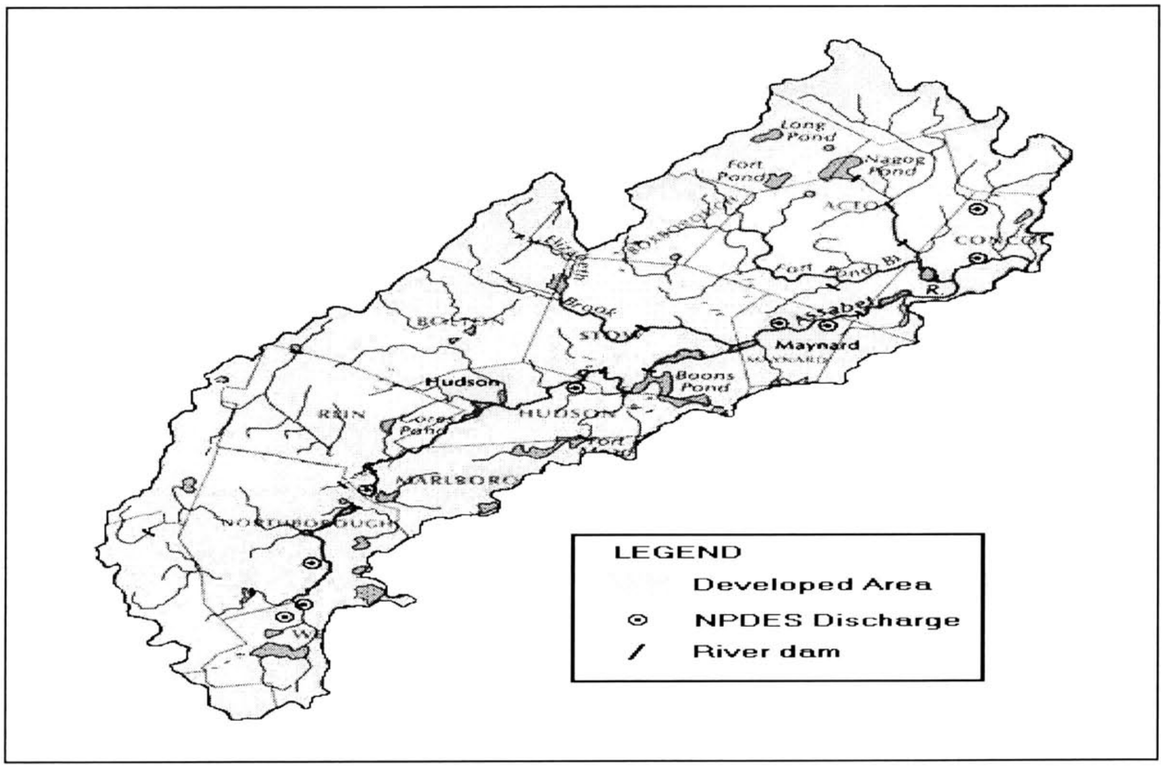

Figure 2. Map of Assabet River Basin
Source: U.S. EPA, 1999.

Westborough, Massachusetts, and stretches 31 miles through a number of highly populated areas. Just past the Town of Concord, the Assabet River merges with the Sudbury River to form the Concord River, which feeds the Merrimack River (Organization for the Assabet River, 1999).

WWTPs operated by the communities of Westborough (including Shrewsbury), Marlborough, Hudson, Maynard, and Concord discharge to the Assabet River. In addition, the Massachusetts Correctional Institute (MCI) at Concord also discharges minimal flows to the river. The Assabet River follows a pattern in which WWTP discharges are located just above a dam in an impoundment area as can be seen in Figure 3 (Hanley, 1989). On average, the WWTPs are located approximately every six miles along the river (Roy, 1998).

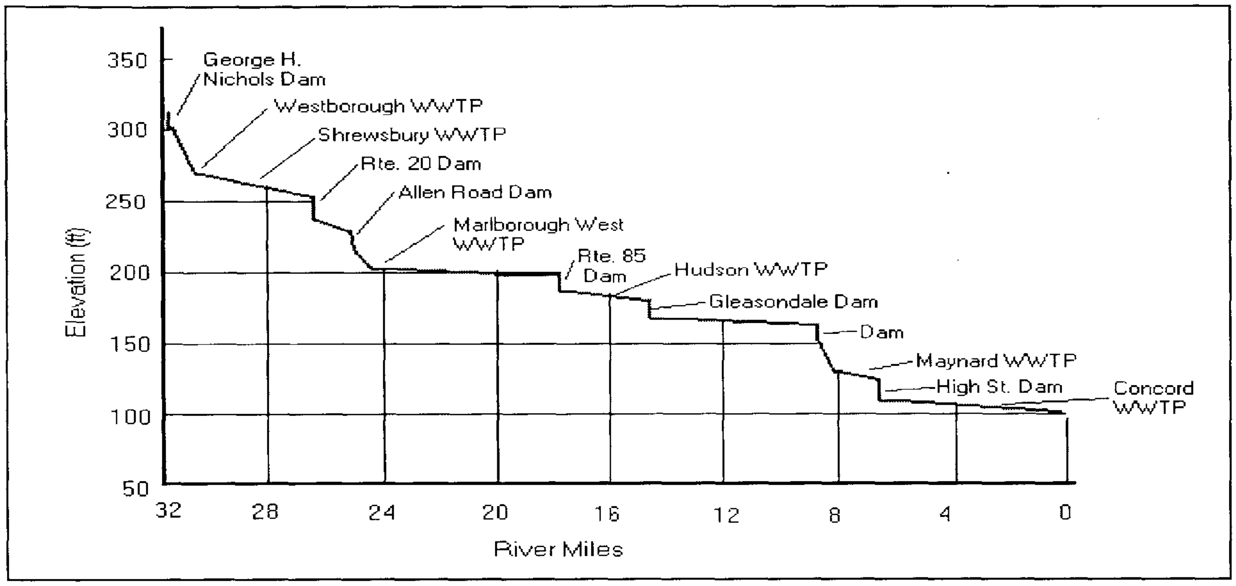

Figure 3. Dam and WWTP Locations along the Assabet River
Source: Adapted from Hanley, 1989.

The small natural gradient and numerous impoundments created by the periodic location of dams along the river produce an overall sluggish flow throughout most of the watershed, with the exception of some fast flowing sections located near Maynard and Hudson (Organization for the Assabet River, 1999). Although the dams create impoundments that suffer from worse water quality, the water is reaerated as the river flows over the dams, which periodically improves dissolved oxygen (DO) levels in the river (Hanley, 1989).

2.1 Assabet River Basin Water Quality History

The Assabet River has been laden with water quality and environmental problems for many years. Poor water quality in the river first prompted the Massachusetts Department of Environmental Protection Division of Water Pollution Control to undertake extensive water quality sampling in 1965. However, the primary emphasis of the sampling was to determine DO and biochemical oxygen demand (BOD) rather than nutrient concentrations. Subsequent sampling endeavors to assess the condition of water quality ensued in 1969, 1974, 1979, 1986, and 1987 (Hanley, 1989). A report on the pollution of the Assabet River issued in 1971 found that phosphates from WWTP discharges were resulting in an average river phosphate concentration 60 times the allowable limit. In addition, worse conditions were observed in the numerous impoundment areas. At the time, only the Shrewsbury, Hudson, and Maynard WWTPs were discharging to the Assabet River and WWTPs were being constructed at Westborough and Marlborough. As a result of its findings, the report strongly urged communities along the Assabet River to develop phosphate removal programs (Cooperman and Jobin, 1971). The poor Assabet River water quality conditions prevailed, despite the passage of the 1972 Federal Water Pollution Control Act (FWPCA) and subsequent assignment of NPDES permits to the discharging WWTPs. The 1979 sampling report also found that the Assabet River "impoundments are highly eutrophic with large amounts of aquatic growth, especially algal blooms during certain periods of the summer." The report also stated that all sections of the Assabet River were in violation of the Class B standard that had been assigned to the river in 1978. The entire river violated total phosphorus and fecal coliform standards, and only one section passed the DO standard for this classification (Massachusetts Department of Environmental Quality Engineering, 1981).

The poor water quality in the Assabet River prompted the Massachusetts Department of Environmental Quality Engineering (DEQE) to develop the first water quality management plan for the Assabet River in 1981. The plan noted the problems caused by NPSs, but maintained that the poor water quality in the Assabet River was largely due to excessive point source discharges from the WWTPs located along the river (MDEQE, 1981). The 1981 water quality management report was subsequently revised in 1989. The 1989 water quality management plan stressed increased nutrient studies and strict adherence to discharge limits to improve water quality in the Assabet River. Although $50 million in WWTP improvements from 1972-1989 increased overall DO levels, water quality studies of the Assabet River performed in 1989 indicated that WWTP nutrient loadings were still affecting the trophic state of the river (Hanley, 1989).

In 1986, the poor water quality conditions in the Assabet River spurred the development of the Organization for the Assabet River (OAR), a non-profit organization of local residents dedicated to improving the water quality in the Assabet River. The OAR maintains a substantial water quality monitoring program and sponsors related environmental protection programs. The group utilizes the water quality data to help enforce wastewater discharge regulations on the five WWTPs that discharge into the Assabet River (OAR, 1999).

2.2 Current Water Quality Conditions

The water quality problems suffered by the Assabet River have become commonplace in many areas of Massachusetts. In addition to continued water quality difficulties resulting from municipal WWTPs, industrial discharges have also increased as several computer technology companies have located within the Assabet River Basin. Steep growth rates throughout the Assabet River Basin have forced many communities to struggle with demanding periods of rapid residential development. The trophic state of the river has continued to worsen due to excessive nutrient loading (Hanley, 1989). During the summer of 1995, the flows in the Assabet River were recorded by the United States Geological Survey to be less than the sum of the WWTP discharges into the river (Roy, 1998). As a result, the entire stretch of the Assabet River was listed by the State of Massachusetts on its most recent "List of Impaired Waters in Massachusetts."

The Assabet River remains in a highly eutrophic state characterized by excessive algal blooms. Throughout the warm months, the river is covered by an algae mat (Figure 4).

Figure 4. Assabet River Algae Mat.
Source: Photo by Author.

During the summer, the layer of vegetation on the Assabet River often becomes thick enough to significantly impede canoeing through impoundment areas (Roy, 1998). The excessive algae growth in the river remains the direct result of the presence of the excessive nutrients required to support such growth, specifically the phosphorus and nitrogen inputs (Biswas, 1997).

3 OVERVIEW OF WATERSHED-BASED POLLUTION TRADING

As a fairly recent and relatively untested water quality management method, little is currently known about watershed-based pollution trading. Therefore, this literature review is intended to provide insight on the development and current use of watershed-based pollution trading.

3.1 Pollution Trading Overview

Although many variations of trading programs exist, the basic concept of pollution trading involves one pollutant source compensating another nearby source to decrease the discharge of a pollutant, whether present in air emissions or water effluent, rather than decrease its discharges of the pollutant to the same environmental medium. In either case, pollution trading utilizes economic incentives to more efficiently reduce overall pollutant discharges in a given area. As the alternative source (the seller) in a pollution trade can often reduce pollutant discharges more efficiently than the original source (the buyer), the compensation for pollutant discharge reduction is profitable above the incurred reduction costs, even at higher reduction ratios. Yet, the cost of compensation borne by the buyer is still less than the original cost of compliance. As such, pollution trading can provide a cost-effective pollution reduction alternative to expensive treatment upgrades for many sources (U.S. EPA Office of Water, 1996a).

Although air pollution trading was included in the 1990 Clean Air Act Amendments (CAAA) Acid Rain Program for sulfur dioxide (SO_2) emissions, watershed-based effluent trading has received significant attention from the EPA as an innovative water quality management policy only within the past three years. Spurred by the proclaimed but arguable success of SO_2 trading under the CAAA (Smith and Ellerman, 1998; Bohi and Burtraw, 1997), the EPA has heavily promoted effluent trading as a flexible and efficient approach to achieving overall water quality on a watershed basis. As such, the topic has provoked a flurry of interest and speculation, despite its limited use in water quality management at the state and local level.

3.1.1 Trading Arrangements

Pollutant trades involving discharges to water bodies are limited to the watershed level and can take many forms, including trading combinations between point sources and NPSs. Trades can also vary widely in terms of the number of participating sources. At the simplest level, a point source may propose a discharge permit that incorporates intraplant trading among its outfalls for a certain pollutant to increase flexibility and reduce the overall cost of compliance for that pollutant. Bilateral trades can also occur and can include both point/point source and point/nonpoint source arrangements (Downing and Sessions, 1985). In addition to bilateral trades, pollutant trading can occur within an organized watershed program or created market among many members. In such a system, the pollution credit prices may be negotiated among the watershed sources, and the credits may also be placed in a "bank" that is created by the program if not needed for immediate use (Pelley, 1996).

3.1.2 Types of Pollutants

The type of pollutants traded can include conventional pollutants (BOD, oils, greases), as well as nutrients (nitrogen and phosphorus) and metals (selenium and copper). As U.S. environmental regulations intend to eliminate the discharge of toxic pollutants into the environment, toxic pollutant trading is prohibited in the United States. A vast majority of the programs currently in existence or under development have traded nutrients, particularly phosphorus. Many others focus on the improving DO levels in water bodies by trading BOD loading among sources. Some unique programs have also focused on metals, such as selenium and copper, as well as reducing road-deicing chemicals to help meet sodium limits. Wetland mitigation has also been incorporated into a number of point/nonpoint source trading programs that attempt to reduce nutrient loading to receiving waters (U.S. EPA Office of Water, 1996a).

3.1.3 Trading Ratios

In order to ensure overall environmental protection and compensate for the difference in environmental impacts resulting from the trade pollutant source location changes, trading ratios may be applied to the pollutant reduction transaction. As defined by the EPA, trading ratios "reflect the relative environmental benefit of reducing a unit of pollution from one source compared to another (or, conversely, the relative harm of not reducing a unit compared to another)." In addition to addressing known environmental differences, trading ratios also provide a margin of safety for trades that include uncertain environmental impacts, such as those involving NPS pollution reductions that are difficult to quantify without extensive monitoring (U.S. EPA Office of Water, 1996a).

Also known as environmental bonuses or environmental premium requirements, trading ratios are essentially the equivalent of a discharge reduction tax borne by the original source when utilizing trades rather than additional upgrades to achieve compliance (Merrifield, 1998). Depending upon the site-specific factors, trading ratios may range from as little as 1.1:1 to as much as 3:1. Higher ratios often reflect both a margin of safety and an intended environmental benefit for an impaired water body. For example, of the reduction units required for a 3:1 trade, one unit may be intended to offset the buyer source increase, another to provide a margin of safety for uncertain reduction methods, and the final unit to provide an environmental benefit by reducing the net pollutant discharge (U.S. EPA Office of Water, 1996a).

3.2 History of Trading Program Development

During the early 1970s, a number of laws were passed by the United States Congress to protect the environment, beginning with the Clean Air Act of 1970 (CAA) designed to reduce air pollution. Soon after, the FWPCA was passed to achieve the "restoration and maintenance of the chemical, physical, and biological integrity of the Nation's waters" (West Group, 1998). In 1977, amendments to the FWPCA were passed, which have been collectively become known as the Clean Water Act (CWA). Despite the fact that many water pollutants have both anthropogenic and natural sources, national environmental command-and-control regulations (CCR) established under the CWA largely focused on improving overall water quality by reducing anthropogenic point source loads to water bodies (Zander, 1991b). The CCRs required end-of-pipe limits for point source discharges that consisted of secondary treatment for municipal point source discharges and best practicable control technology for industrial point source discharges (Shanahan, 1999). The national technology-based standards contained within CCRs, although expensive to maintain and inefficient in comparison to total-watershed pollution control approaches, were much easier for state water quality agencies to administer (Downing and Sessions, 1985). Due to the fact that the CCRs left NPS pollution virtually unregulated, increasingly stringent CWA technology-based standards still failed to achieve water quality objectives in many water bodies nationwide. In these impaired watersheds, the states were given substantial water quality management flexibility to meet ambient water quality standards (Shanahan, 1999). As a result, many water quality management officials began to search for comprehensive watershed pollution control methods to reduce the overall pollutant loading in impaired watersheds for the least economic cost (Caldart, 1998).

As compliance costs rose and problematic watersheds remained in nonattainment during the 1980s despite the full implementation of water CCR standards, the regulations became increasingly unpopular among both environmentalists and dischargers. As a result economic alternatives to environmental CCRs, especially pure market-based theories, were studied and proposed as pollution control strategies. Many of these strategies relied on previous theories, such as Hardin's oft-cited, "Tragedy of the Commons" natural resource privatization proposal to end environmental neglect as public goods (Stewart and Krier, 1978), and Ruff's (1970) optimum pollution control attained through the "invisible hand" of market mechanisms. Similarly, others merged biology and economics theories to search for methods that would naturally serve the collective interest of environmental protection by invoking selfishness to convince humans to rationally consider the environmental consequences of their actions (Ridley and Low, 1993).

Despite the substantial attention given to the notion of utilizing economic pollution control methods in lieu of environmental CCRs, many of the economic solutions remained largely theoretical, rendering them impractical and difficult to implement. However, studies of German water quality management systems in which water quality standards were attained through a system of charges reflecting the environmental impacts of the pollutant discharge showed promise for practical economic pollution controls. As such, when the similarly practical and innovative idea of small "artificial" or "created" pollution trading markets arose in the 1980s from these studies, the concept spawned considerable interest at the state and local levels (Downing and Sessions, 1985).

3.2.1 Initial State and Local Trading Case Studies

The economic incentives associated with pollution trading or credit systems prompted some states to study how to optimize pollution reduction among a group of sources through the use of trading mechanisms. A number of state and local studies estimated that trading programs could significantly reduce pollution control costs over conventional methods (David and David, 1983). The studies culminated in a few specialized trading programs designed to control water pollution to a few impaired and recreational water bodies. As can be seen below, the programs had varying levels of success, as some programs struggled due to poor design while others developed into case studies for national guidelines.

3.2.1.1 Fox River, Wisconsin

In 1981 the State of Wisconsin created a water pollution permit program that was the first trading program of its type in the United States. The program set up initial waste load allocations (WLAs) for 14 paper plants that discharged into the Fox River (Figure 5), and allowed them to trade pollution rights to meet these allocations through a transferable discharge permit (TDP) system (O'Neil, 1983). Initial studies indicated that abatement costs on the Fox River could be cut by at least half if the TDP system was implemented in place of conventional control methods (David and David, 1983). However, despite this potential, the trading program suffered from virtually nonexistent trading activity - only one trade was arranged throughout the first fifteen years of the program. The lone trade occurred when a paper mill shut down its own WWTP and requested that the state shift its allocation to a local WWTP.

Figure 5. Fort Howard Paper Plant along the Fox River, Wisconsin
Source: Wisconsin Division of Natural Resources, 1999.

The lack of trading activity in the Fox River TDP system has been attributed to competition between the program companies, which are mostly paper mills. Studies have suggested that the paper mills have been saving their allocations for future growth rather than trading them with competitors. Further evaluation of the economic incentives for trading in the paper industry have suggested that the initial savings estimates were substantially flawed, and that little economic incentive exists for paper companies to trade pollution credits.

In addition to the competition issues, trading is also limited due to the rigid program structure. Trades in which compliance cost reduction is the sole objective are prohibited through the program's demonstration of need requirements for trade approval. Existing point sources are only allowed to enter trades during expansion or if unable to meet the discharge limits with the requisite technologies. New plants are allowed to trade upon completion, but must also show a demonstration of need beyond the required treatment technologies (U.S. EPA Office of Water, 1996a).

3.2.1.2 Lake Dillon, Colorado

Phosphorus control measures undertaken in the Lake Dillon Watershed in Colorado inhibit algal growth and retain the mesotrophic state of the reservoir for both domestic water supply and recreational purposes (Morris and Lewis, 1992). Lake Dillon (Figure 6) is one of many Colorado reservoirs, including the Cherry Creek Reservoir noted below, in which officials strive for the highest water quality standards for these reasons. Lake Dillon is a staging reservoir for the drinking water requirements of the inhabitants of Denver, Colorado, but is also surrounded by a number of recreational communities and ski resorts. As such, the communities in the Lake Dillon Watershed have considerable interest in protecting the water quality in Lake Dillon. In 1984, the Colorado Water Quality Control Commission (CWQCC) established a maximum lake phosphorus concentration to ensure the quality of water in the reservoir. Shortly thereafter, a local group comprised of county, town, and ski resort representatives dubbed the "Phosphorus Group" began brainstorming ideas for a total watershed control approach. They came up with the "Dillon Bubble," a plan to utilize pollution trading to allow for further development in the watershed without compromising water quality in Lake Dillon (Zander, 1991b). Realizing the communities' collective interest was ideal for watershed management programs, the CWQCC has been allowing point/nonpoint source phosphorus trading in the watershed since 1984.

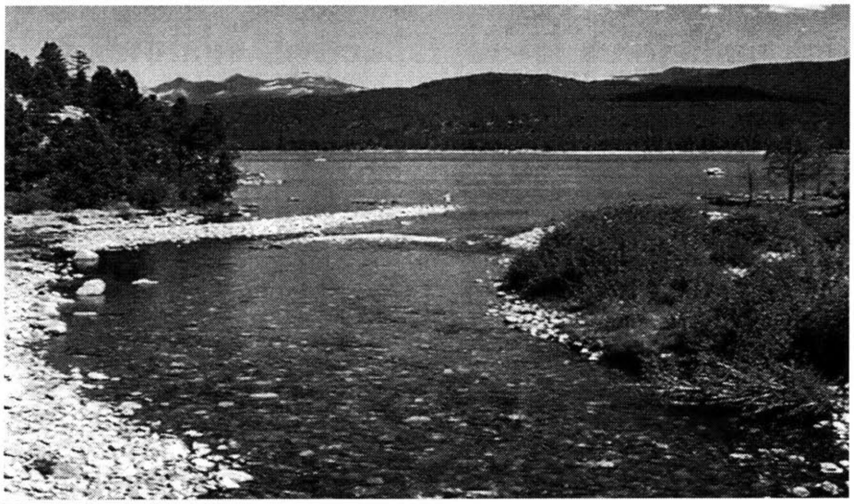

Figure 6. Lake Dillon
Source: USEPA, 1999.

However, similar to the Fox River trading program, few trades have occurred in the Lake Dillon trading program. Although the cooperative environment between municipalities in the Dillon Watershed is conducive to trading, point sources in the watershed have not needed trades to maintain water quality as they have already achieved some of the highest phosphorus removal capabilities in the nation (U.S. EPA Office of Water, 1996c). As a result, controlling the NPS loading to Lake Dillon from community and ski area runoff and failing community septic systems has become the primary focus of the trading program. For example, the Town of Frisco, Colorado plans to utilize stormwater controls to offset the phosphorus loading to Lake Dillon from a new golf course. Other communities and resorts are also searching for NPS load reductions to offset the increased phosphorus loading that would result from planned recreational additions and future resort developments (U.S. EPA Office of Water, 1996a).

3.2.1.3 Cherry Creek Reservoir, Colorado

Similar to Lake Dillon, Cherry Creek Reservoir is a popular recreational attraction that is visited annually by more than 1.5 million people. As a result, the Cherry Creek Basin Authority also established a total maximum daily load (TMDL) for the reservoir in 1984 to protect water quality for both drinking and aesthetic uses. However, it has been estimated that approximately 80% of the phosphorus loading in the watershed results from NPS pollution. As a result, shortly after the TMDL was established the 12 WWTPs that discharge to Cherry Creek were allowed to apply phosphorus trading credits to their discharge allowances from efforts to reduce NPS phosphorus loading in the watershed (U.S. EPA Office of Water, 1996b).

Despite the allowance of trading in the Cherry Creek Basin since 1985, formal guidelines for trading in the basin were not developed until shortly after the publication of the EPA's watershed-based trading guidelines in 1996. The guidelines allow point sources located within the Cherry Creek Basin to continue earning WLA credits by reducing NPS pollution. However, the guidelines require point sources to maintain phosphorus removal levels within designated effluent limits and demonstrate the need for an increased phosphorus discharge allowance. The Cherry Creek Basin Authority allows point sources to either contribute to authority-designated NPS control projects or devise their own NPS control project to earn discharge credits. The authority phosphorus loading reduction credits are placed in a bank to be purchased by point sources as increased discharge credits (U.S. EPA Office of Water, 1996b). In order to ensure reduction results from private NPS reduction projects, the project must be implemented and maintain NPS controls that reduce identified NPS loadings by half of the designated trade amount before any credits can be allocated to the source WLA (U.S. EPA Office of Water, 1996a).

3.2.2 Federal Watershed-Based Trading Emphasis

The watershed-based trading in the aforementioned state and local programs during the early 1980s did not go unnoticed by the EPA, which also began studying effluent trading as a method of watershed pollution control. However, the agency did not issue a formal pollution trading report for over 15 years (Pelley, 1996). A number of events in the early 1990s, including the reported success of sulfur dioxide trading under the CAAA, the Clinton administration's clean water campaign, and the EPA's move toward watershed-based water quality management, spurred widespread interest in the application of innovative trading programs to control water pollution sources. As a result, the EPA has recently made watershed-based trading one of the most encouraged approaches to watershed management.

3.2.2.1 Clean Air Act Sulfur Emission Trading

As the breakthrough that ended nearly a decade of legislative gridlock over policy solutions to the infamous acid rain problem, the Acid Rain Program under Title IV of the Clean Air Act Amendments of 1990 (CAAA) provoked considerable speculation (Ackerman and Moomaw, 1997). The unprecedented experiment in United States environmental regulation allowed electricity generation facilities to trade SO_2 emission allowances. A flurry of criticism about the expected outcome of the program ensued as economists projected poor program performance.

Indeed, the program did not initially live up to its proponents' expectations, as stated by Zorpette (1994), "Almost all involved agree that the rate of trading among utilities has fallen short of aspirations." However, the industry's cost of compliance soon sunk to very low levels and SO_2 emissions dropped rapidly, leaving many proponents to proclaim the program a success.

Subsequent studies performed by Burtraw (1996) indicated that the Acid Rain Program actually required little trading to significantly reduce SO_2 emissions. Despite the proclaimed success at low trading volumes, the number of allowance transactions steadily rose over the next three years (See Figure 7). Although only 215 transactions involving 9.2 million allowances occurred between utilities during 1994, 1429 transactions for approximately 15.2 million transactions were recorded in 1997 (U.S. EPA, 1999). Initial results of the program indicated that in both 1995 and 1996, the flexibility and trading allowed under Title IV reduced overall SO_2 emissions by 4 million tons annually, which resulted in emissions well below the required limits set by the program (Ellerman, Joskow, and Schmalensee, 1998). As a result, the concept of watershed-based effluent trading began to gain widespread attention and became the focus of the Clinton Administration's emphasis on reinventing environmental regulation (Clinton and Gore, 1995).

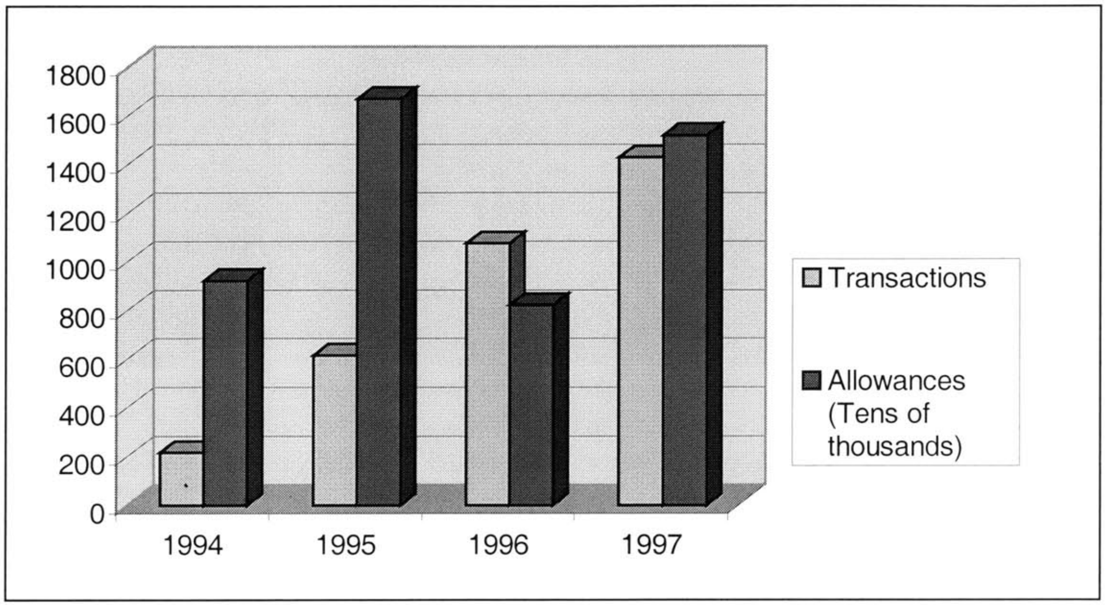

Figure 7. 1994-1997 SO_2 Emission Transactions and Allowances
Source: Adapted from U.S.EPA, 1999.

3.2.2.2 President Clinton's Clean Water Initiative

As part of an effort to reinvent environmental regulation, the Clinton Administration issued President Clinton's Clean Water Initiative in early 1994. One of the primary concerns denoted in the initiative was the development of NPS pollution control programs by states to finally bring NPS pollution under regulatory control. The initiative first advocated the concept of a watershed-level water quality control approach that would soon become the basis for all EPA management programs in an attempt to bolster Clean Water Act Section 319 NPS pollution control programs. Citing NPS pollution as the primary contributor to water quality impairment in the United States, the initiative proposed the implementation of NPS pollution control programs at the state level. Specifically, the initiative called for the identification of impaired watersheds and the implementation of innovative and flexible local control programs comprised of voluntary and regulatory pollution controls adapted to the unique watershed conditions (U.S. EPA Office of Water, 1994). As such, President Clinton's Clean Water Initiative was one of many developments that created an incentive to utilize watershed-based pollution trading to reduce pollution loading from both point and nonpoint pollutant sources in impaired watersheds.

3.2.2.3 Watershed Protection Approach to Water Quality Management

As mentioned previously, despite the water quality management flexibility provided within the CWA, many states have traditionally applied a relatively uniform state-wide approach to water quality management in accord with federal statutes. However, many factors incited the EPA to promote water quality management at the watershed level in the mid-1990s. Deemed the Watershed Protection Approach (WPA), the policy was intended to provide improved water quality management by strongly encouraging unique practices that obtain overall water quality goals within each watershed (Hall and Howett, 1994). The shift to the WPA began with internal EPA comments about breaches in water quality management resulting from the failure of traditional state-level approaches to identify the impact of unique NPS pollution on impaired watersheds (Brady, 1996). Further, a primary objective of the Clinton administration in its reinvention of environmental regulation was ecosystem protection and management, which incorporates both the ecological and social characteristics of a unique geographic area into its management strategy (Davenport, et al., 1996). As a result, the water quality management lapses and the need for site-specific controls evolved into the WPA water quality management policy that set the stage for trading practices.

3.2.2.4 EPA Draft Framework for Watershed-Based Trading

Shortly after the Clinton Administration emphasized ecosystem protection and water pollutant trading as a priority of its reinvention of environmental regulation and the switch to WPA policies, the EPA issued the Draft Framework for Watershed-Based Trading in 1996. In addition to the Clinton Administration objectives, the development of the document was also spurred by the strong belief of EPA Administrator William K. Reilly in the potential of economic incentives for environmental control. In addition, a 1992 trading initiative meeting of over 120 representatives and stakeholders also strongly emphasized the need for EPA watershed-based effluent trading guidelines (U.S. EPA Office of Water, 1992). In order to fulfill this need, the Draft Trading Framework was intended to serve as a guideline for communities and companies interested in implementing watershed-based pollution trading programs.

A primary goal of the Draft Trading Framework was to provide EPA support to trading programs that many potential trading parties had avoided due to fears of noncompliance if their trading partners failed to reduce pollution per the trading agreement. The document allays these fears by encouraging and supporting the incorporation of trading arrangements into each party's NPDES permit for point/point source trades and acknowledging the need for flexibility and cooperation for point/nonpoint and nonpoint/nonpoint source trades (Pelley, 1996). As a draft document announcing the future promulgation of trading policies by the EPA, the Trading Framework was subjected to the agency regulation review process and amassed a number of public comments. The comments are currently being incorporated into the final document that is expected to be finished in late 1999 (Canning, 1999).

3.3 Recent Trading Programs

Many trading programs have been implemented as a result of the EPA's recent support of trading programs for watershed management. After the release of the Draft Trading Framework, the initial trading programs received a renewed surge of interest as case studies for trading programs in development and a second wave of effluent trading programs emerged. These include the Tar-Pamlico River in North Carolina, the Minnesota River in Southwestern Minnesota, the Passaic River in New Jersey, and Boulder Creek in Colorado (U.S. EPA Office of Water, 1996a). In addition, a number of nonpoint/nonpoint wetland trading programs have also recently been developed on a statewide basis under the Draft Trading Framework guidelines.

3.3.1 Tar-Pamlico, North Carolina

The Tar-Pamlico River Basin shown in Figure 8 is a large river and estuarine system in eastern North Carolina that was designated by the North Carolina Environmental Management Commission (EMC) as a nutrient-sensitive water in 1989 as the result of numerous nutrient loading studies. The studies recommended that an extensive nutrient control plan be implemented in the Tar-Pamlico River Basin that reduced both point and NPS loadings in the watershed. As a state with a federally approved coastal zone management program, North Carolina was also required to submit NPS pollution control programs to the EPA under the 1990 Coastal Zone Management Act. Having hosted the EPA Administrator's 1992 Point/Nonpoint Trading Initiative Meeting that led to the development of the EPA Watershed-Based Trading Framework, North Carolina environmental officials were well-informed of the issues involved with trading programs. As a result, while developing a basinwide NPS control plan for the Tar-

Figure 8. Tar-Pamlico River
Source: Hall and Howett, 1994.

Pamlico River Basin, the North Carolina Division of Environmental Management (NCDEM) chose an innovative watershed management trading program in cooperation with the Environmental Defense Fund and other interested parties.

The Tar-Pamlico trading program was designed to obtain overall water quality in the watershed by allowing point sources to cooperate with other point sources and NPSs to reduce nutrient loading in the watershed over a five-year period. The flexibility of the program allows point sources to choose the most cost-effective method to fulfill their individual pollution control responsibilities. They can either reduce their own discharges or the discharges of another plant, or contribute $56 per kilogram of required reduction to a fund that is used to implement BMPs in the basin's agricultural areas to reduce nutrient loading to streams (Hall and Howett, 1994). This fee was determined as the average cost of reduction required to achieve a 3:1 nutrient reduction ratio for cropland BMPs and a 2:1 ratio for animal BMP reductions in the Tar-Pamlico River Basin (U.S. EPA Office of Water, 1996a).

Implemented during the final years of the Trading Framework development, the Tar-Pamlico trading program was given widespread attention by the EPA as one of the largest point/nonpoint source pollution trading programs. Engineering studies of the program predicted that the initial required nutrient reductions of 425,000 kg/yr would be obtained easily through trades and operational changes. In addition, the cost savings of the program were estimated to be approximately $60 million over conventional point source control upgrades that may have forced some companies out of business (Hall and Howett, 1994).

3.3.2 Minnesota River

The Minnesota River is one of the most polluted rivers in the State of Minnesota and has historically suffered from low DO levels. However, agricultural land comprises over 90 percent of land drained by the river in the southwestern part of the state. As a result of the impaired conditions of the Minnesota River, expanding industries in the watershed were denied NPDES permits to discharge to the river in 1994. Rather than continue to impede industrial development in the Minnesota River Basin, the Minnesota Pollution Control Agency (MPCA) decided to implement a trading strategy to offset industrial expansion discharges to the river with agricultural NPS loading reductions. For every pound of BOD discharge allowed for expansion or new plant construction in a NPDES permit, the discharging company would have to agree to coordinate the implementation of upstream BMPs that would counter the new point source discharge (Wallace, Sparks, and Micheletti, 1997).

Rahr Malting (Figure 9), touted as the largest and most advanced malting company in the world, was one of the companies that was denied a NPDES permit to increase its discharges to the Minnesota River. Realizing the rising cost of discharging to a local WWTP and seeking to expand its malting facilities, the company sought to construct a $7 million onsite treatment plant that would discharge the treated effluent to the river. However, noting that the treated malting process effluent would still contain oxygen-depleting compounds that would further deteriorate the impaired DO conditions in the Minnesota River, the MPCA refused to grant Rahr Malting a NPDES permit (Passi, 1998). Rather than spend its funds battling the MPCA in court, the

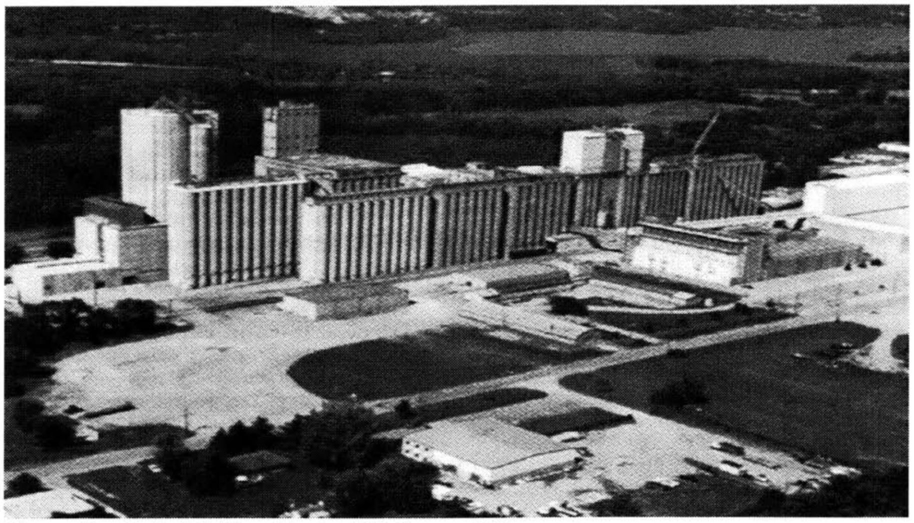

Figure 9. Rahr Malting Plant at Shakopee, Minnesota
Source: Wallace, Sparks, and Micheletti, 1997.

company decided to utilize trading measures to reduce NPS oxygen depletion of the Minnesota River through upstream agricultural BMP implementation. Due to the large alternative costs of continuing to discharge to a WWTP, Rahr Malting anticipated a future decrease in its production costs despite paying for most of the BMPs with its own funds. After three years of development, the MPCA and the EPA both approved the Rahr Malting NPDES permit in early 1997 as the first NDPES permit to incorporate pollutant trading (Hersch, 1997).

3.3.3 Passaic River Valley, New Jersey

The Passaic River Valley is a densely populated and heavily industrialized area that contains 47 municipalities and over 300 major industrial plants (Canning, 1999). Due to intense industrial development in most of the valley, the Passaic River (Figure 10) has experienced numerous

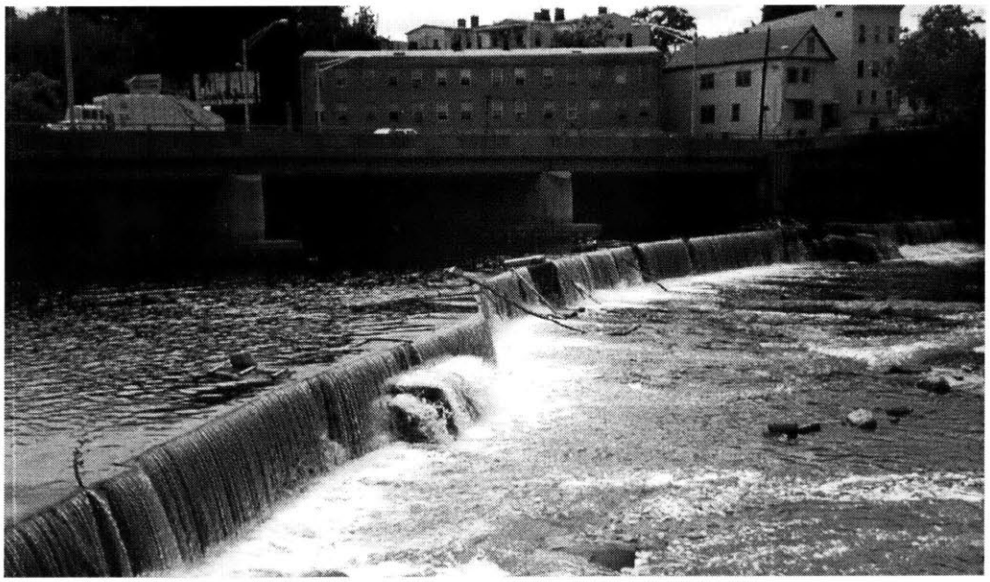

Figure 10. Lower Passaic River
Source: American Rivers, 1999

water quality problems, particularly excessive metal concentrations (Wenning, Bonnevie, and Huntley, 1994). In 1996, a joint team consisting of the New Jersey Department of Environmental Protection (NJDEP), the EPA, the Passaic Valley Sewerage Commission (PVSC), and several Passaic Valley industries conducted a pilot program to test the development of a metals trading program in the Passaic River Valley. The pilot program focused on the use of trading to meet new metals limits for pretreatment of discharges to PVSC facilities. Certain restrictions were applied to the metals trades, including the requirement that metals reductions below PVSC limits must be achieved with end-of-pipe metal reduction technologies rather than internal process changes or discontinuance of a metal waste stream (Murphy, 1997). The initial trade attempted in the program resulted in a copper trading agreement between two industrial facilities (Canning, 1999). Once approved, the trade was incorporated in the companies' NPDES permits, and twenty percent of the total amount to be traded was placed in a credit bank, where it can no longer be discharged. As a result, both companies have achieved compliance at a lower cost than possible with conventional methods, and the overall copper discharge between them has been reduced – a positive outcome from trading innovations (Murphy, 1998).

Thus far, the EPA has showcased the Passaic Valley experience as the most recent effluent trading success. In a recent report entitled, "Sharing the Load: Effluent Trading for Indirect Dischargers" the agency documented the struggles of the program (U.S. EPA Office of Policy, Planning, and Evaluation, 1998). The report was distributed by the EPA to over 600 recipients and placed on the internet for additional access and offers further insight to successful trading strategies to complement the Draft Framework for Watershed-Based Trading. The Passaic Valley pilot trades are among a small group of experiences that the EPA plans to incorporate into the final revision of the document. Particularly noteworthy of the Passaic Valley experience is the demonstration of the efficient results of cooperation between pollution sources and regulating entities. As stated by one pilot trading participant, Fabricolor Inc., "The pilot has shown people that you can work with government and can accomplish things. It doesn't have to be adversarial all of the time." Through the cooperative experience in the Passaic Valley, EPA has further defined the requirements for successful trades between WWTPs and indirect dischargers, which will also be included in the final Trading Framework revision (Canning, 1999).

3.3.4 Boulder Creek, Colorado

Although a pristine and rolling mountain waterway at its headwaters, Boulder Creek (Figure 11) quickly becomes an urban stream upon passing through Boulder, Colorado. Downstream of the Boulder WWTP, Boulder Creek suffers from elevated un-ionized ammonia concentrations that inhibit aquatic life along the 15.5 mile reach below the city (Zander, 1991a). Despite upgrades that achieved WWTP effluent ammonia concentrations within effluent limits, Boulder Creek conditions did not improve due to numerous NPS sources along the impaired creek section (Zander, 1993). Rather than implement additional plant upgrades to achieve acceptable water quality standards in Boulder Creek, the City of Boulder decided to reduce NPS pollution with the implementation of BMPs. In particular, agricultural practices around Boulder were targeted, resulting in a 120-foot wide buffer between the creek and grazing land. Additional BMPs implemented along impaired sections of Boulder Creek include streambank and riparian restoration and irrigated treatment via wetland diversions. Substantial monitoring efforts have indicated that the $1.4 million BMP investment has improved water quality in Boulder Creek to

Figure 11. Boulder Creek Upstream of Boulder WWTP Discharge
Source: Corbis, 1999.

higher levels than could have been achieved by the $3 to $7 million conventional WWTP upgrade alternative (U.S. EPA Office of Water, 1996d).

3.3.5 Wetland Trading Programs

Utilizing a broad definition of the term "navigable waters" in the CWA Section 404, the United States Army Corps of Engineers (USACE) administers dredging and filling permits (with EPA approval) for construction activities that destroy wetlands (Percival et al., 1992). Largely due to people's love for waterside living accommodations, nearly half a million acres of wetlands were lost annually to urban development until 1985, when the federal government began to enforce wetland mitigation and the rate of loss was reduced to slightly more than 100,000 acres per year. Despite this success, many officials have sought to achieve a no-net loss wetland goal (Cushman, 1997). In order to obtain a permit, most applicants must agree to create a nearby wetland of equal area to what they destroy during development (Percival et al., 1992).

In addition to the watershed-based trading programs listed above, a number of wetland trading programs (Table 1) have been developed on a statewide basis. Most of these programs allow those who have impacted wetlands to contribute funds to trust funds and nonprofit environmental organizations that undertake restoration efforts rather than attempt ad-hoc restoration efforts to achieve no-net loss of wetlands (U.S. EPA Office of Water, 1996a).

Table 1. Wetland Trading Programs
Source: Adapted from U.S.EPA, 1996a.

Program Participants	Type	Summary
Arkansas Nature Conservancy - USACE	nonpoint/ nonpoint	USACE permittees that destroy wetlands compensate the Nature Conservancy, which acquires and enhances additional wetlands.
Dade County, Florida	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Wetland Mitigation Trust Fund used for Everglades improvement projects.
Maryland Nontidal Wetland Compensation Fund	nonpoint/ nonpoint	Maryland DNR controls a wetland mitigation trust fund that is used for restoration projects.
Ohio Wetlands Foundation	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Ohio Wetlands Foundation that creates large wetlands.
Pine Flatwoods Wetlands Mitigation Trust, Louisiana	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Louisiana Nature Conservancy that administers restoration efforts.
Vicksburg District, USACE	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Ducks Unlimited, the Nature Conservancy, and other public agencies.

3.4 Trading Programs Currently Under Development

In addition to the programs that have already been implemented or expanded under the recent EPA guidelines, many other programs are currently being considered or in the initial stages of development at numerous locations across the United States. In particular, state and local officials developing trading programs to improve water quality in Long Island Sound and numerous water bodies throughout the State of Texas have invested considerable time and resources into the use of watershed-based trading for pollution control.

3.4.1 Long Island Sound

As the limiting nutrient for algae growth in saltwater systems, large water nitrogen concentrations induce the growth of dense algal blooms that are subsequently decomposed by bacteria in a process that consumes significant amounts of DO. As a result, excessive nitrogen loading from numerous Connecticut and Long Island communities has created short periods of hypoxia in Long Island Sound (LIS), a condition in which DO levels fall below the levels required to sustain aquatic life. Although short periods of hypoxia have existed since as early as the 1950s, studies since 1986 have identified an increase in the yearly duration of hypoxia that has significantly impaired the aquatic health of LIS. In recent years, hypoxia has occured in LIS from as early as late June to as late as mid-September (U.S. EPA Long Island Sound Office, 1997).

In order to reduce nitrogen loading to LIS from the surrounding communities, the Long Island Sound Study (LISS) hypoxia management program was first established in 1990, when initial status reports were prepared. Shortly after, Phase I of the LISS placed a freeze on growing nitrogen inputs to LIS, and Phase II was implemented in 1994 to begin reversing nitrogen inputs through conventional reduction methods. In 1997, the LIS watershed in both Connecticut (Figure 12) and New York was divided into eleven management zones. Proposals were developed for a comprehensive nitrogen management plan to reduce approximately 60 percent of the nitrogen loading in the watershed in order to improve DO concentrations in LIS to acceptable levels. Watershed-based effluent trading was among the water quality management proposals (Overton, 1998). Connecticut officials that control the bulk of the LIS watershed are convinced effluent trading will reduce nitrogen inputs to LIS and have estimated that the implementation of trading to achieve the reductions will save the state approximately $200 million over the use of conventional methods (Stacey, 1998). Although not opposed to an integrated watershed-trading program, New York officials have already implemented some variations of trading on a zone or "bubble" basis within the small portion of the LIS watershed under their control. Having experienced success from the bubble trading and leery of the increased administrative burden of a total watershed program involving the numerous Connecticut dischargers, New York officials have been hesitant to commit to an integrated trading program (O'Brien, 1998). However, many environmental officials see promise in the trading concept and a trading program is slowly developing (Overton, 1998).

Figure 12. West Haven, Connecticut in LISS Management Zone Three
Source: Corbis, 1999.

3.4.2 *Texas Water Resources Institute*

As part of an effort to improve water quality in the State of Texas via the WPA prescribed by the EPA, Texas water quality managers have been developing TMDLs for Texas watersheds and evaluating a number of market approaches to reducing overall pollution on a watershed basis. They are interested in utilizing the point/nonpoint aspect of trades to reduce pollution loading in predominantly agricultural watersheds. At the most recent Texas water quality conference held by the Texas Water Resources Institute (TWRI) in December 1998, pollution trading and TMDLs were of primary interest to many participants.

The TWRI recently requested the assistance of three economists to evaluate the use of market-based strategies to limit water pollution in Texas (TWRI, 1998). All three economists reported positively on the use of pollution trading in Texas watersheds, citing cost-efficient pollution control (Merrifield, 1998; Emerson, 1998; Griffin, 1998). As a result, as TMDLs are established for Texas water bodies, the TWRI plans to develop trading programs within the TMDL framework to reduce overall watershed pollution (TWRI, 1998).

3.4.3 Additional Programs in Development

In addition to the aforementioned programs, a number of trading programs (Table 2) were under consideration as the Draft Framework for Watershed-Based Trading was released, but have yet to formally develop established trading programs (U.S. EPA Office of Water, 1996a).

Table 2. Trading Programs Under Development at the State and Local Level
Source: Adapted from U.S.EPA, 1996a.

Program Participants	Type	Summary
Chatfield Basin, Colorado	point/ nonpoint	Evaluating TMDL, source targets, and potential for phosphorus trading.
Chehalis River Basin, Washington	point/ nonpoint, nonpoint/ nonpoint	TMDL has been developed and a trading study performed for Washington Department of Ecology has identified substantial benefits of trading.
Chesapeak Bay tributaries, Maryland	point/ nonpoint, nonpoint/ nonpoint	Pilot trading project has begun to evaluate the potential of trading to reduce nutrient loading to Chesapeake Bay.
Clear Creek, Colorado	point/ nonpoint	A trading program where sources adopt nonpoint sources, mostly abandoned mines, to achieve pollutant reductions.
Little Deep Fork, Oklahoma	nonpoint/ nonpoint	Agricultural and animal BMPs may be used to improve DO levels and reduce phosphorus loading in lieu of treatment plant upgrades.
Sacremento River, California	N/A	Discussions of metals trading and agrichemical input reductions have been ongoing.
San Joaquin River, California	point/ nonpoint, nonpoint/ nonpoint	The State of California, Environmental Defense Fund, and EPA have evaluated trading to reduce the cost of selenium discharge reductions.
South San Francisco Bay, California	point/ nonpoint,	Several WWTPs have evaluated trading to meet a 900 lb/year overall copper TMDL between them.
Tampa Bay, Florida	point/ nonpoint, nonpoint/ nonpoint	A trading program is being evaluated for nitrogen and total suspended solids from stormwater and the development of a stormwater fund for larger projects.
Boone Reservoir, Tennessee	point/ nonpoint	Studies have been performed to determine whether agricultural BMP implementation would be preferred over treatment plant upgrades.
Wicomico River, Maryland	point/ nonpoint	Preliminary studies have identified substantial cost savings of and water quality benefits in the Wicomico Basin.

4 ACTON, MASSACHUSETTS TRADING CASE STUDY

This case study of the Town of Acton, Massachusetts Trading Program was performed in correlation with the issues associated with watershed-based trading that have been identified in both EPA studies of its feasibility and the recent technical reviews of its limited use. As the proposed Acton Trading Program involves point/nonpoint source trades, an effort was made to ascertain the environmental issues associated with the use and quantification of the NPS component of the program. An examination of the watershed conditions conducive to successful watershed-based trading was also performed on the Assabet River Basin and the Acton Trading Program was analyzed for implementation flaws. In addition, the impact of the proposed point/nonpoint source trades on water quality in the Assabet River was examined in a QUAL2E modeling analysis that was performed by the author for a related study of the Acton Trading Program that is included as Appendix A to this thesis. Finally, as a result of the examination of the proposed Acton Trading Program, a hypothetical point/point source trade involving the Acton WWTP was outlined as an alternative trading arrangement to more efficiently reduce phosphorus loading in the Assabet River Basin.

4.1 Acton Trading Program Overview

The Town of Acton, Massachusetts, is a community located in the Assabet River Basin (See Figure 13 on the following page) that currently relies mostly upon individual sewage treatment via onsite septic systems. For the past several years, the regions of Acton known as South Acton and Kelley's Corner have been experiencing septic system failure due to shallow groundwater levels. As a result, the Town of Acton has begun designing a WWTP to serve these regions of Acton. If approved by the EPA, the Acton WWTP will discharge some of its effluent to the Assabet River at the downstream location shown in Figure 13.

Since the eutrophication of the Assabet River described in Chapter 2 has been the result of WWTP phosphorus inputs, the EPA has placed many phosphorus requirements upon the Town of Acton before approving a NPDES permit for the Acton WWTP discharge to the impaired Assabet River. The EPA has requested that the Town of Acton design a WWTP that produces effluent with a target phosphorus concentration range of 0.1 mg/L to 0.2 mg/L. In addition, the EPA has requested that the Town of Acton develop a point/nonpoint source watershed-based trading program to offset the proposed WWTP phosphorus loads to the Assabet River. Due to the impaired conditions in the Assabet River and uncertainty involved with point/nonpoint source trades, a 3:1 trading ratio has been designated by the EPA for the Acton Trading Program. Therefore, the Town of Acton must reduce three times as much phosphorus loading to the Assabet River from Acton urban runoff through the use of BMPs as the Acton WWTP discharges to the Assabet River. Referring to Figure 13, most of Acton is drained by Fort Pond Brook and Nashoba Brook; therefore, Acton urban NPS phosphorus reductions will affect the phosphorus concentrations in these brooks.

Acton NPS phosphorus sources are characteristic of a medium-density residential area with abundant vegetation, and include lawn fertilizer use and vegetative (leaf) litter. In the Town of Acton Trading Program Discussion Draft (1998), the phosphorus concentration in the Acton urban runoff was assumed to range from approximately 0.2 to 0.5 mg/L.

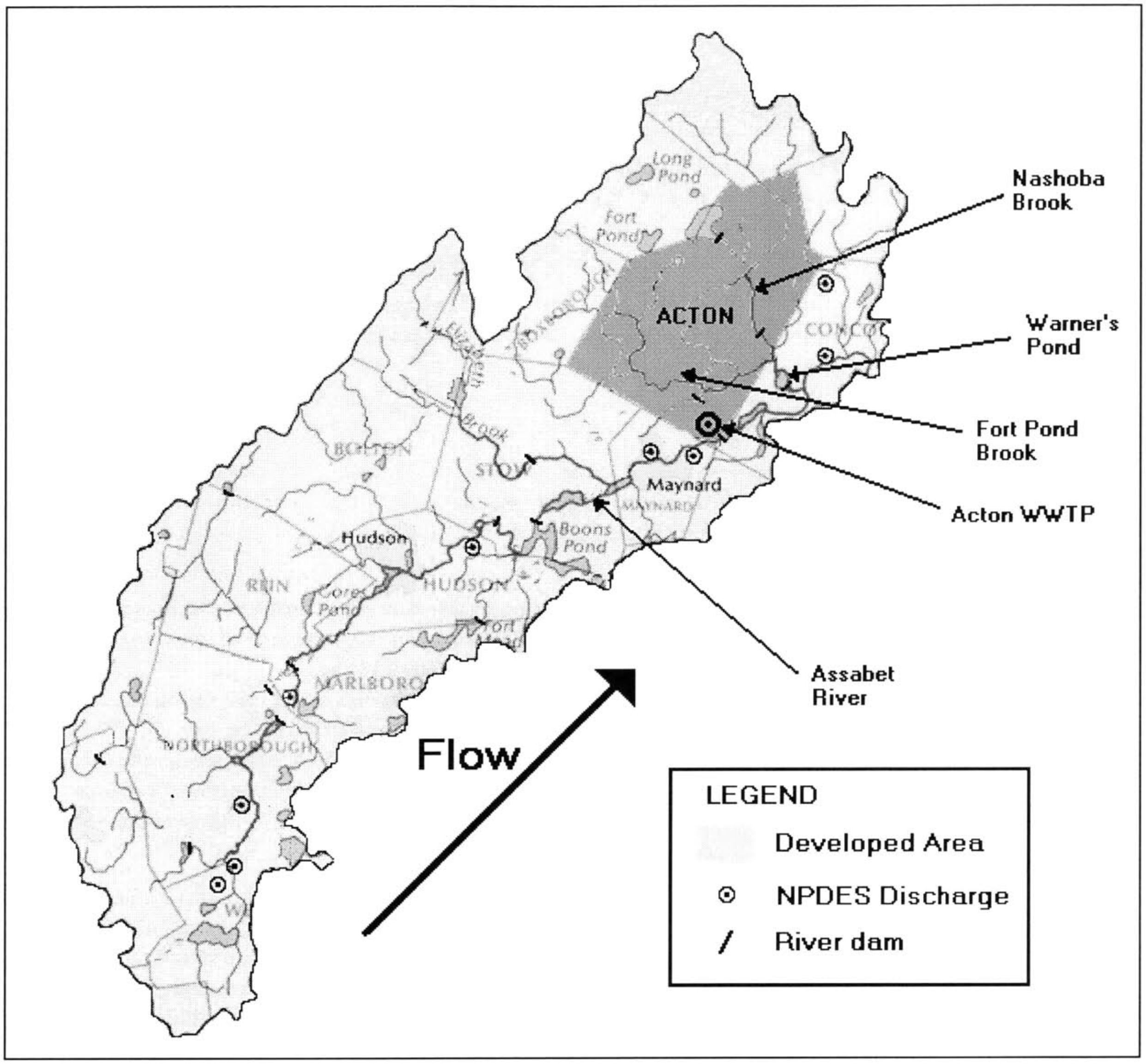

Figure 13. Acton Location in the Assabet River Basin
Source: Adapted from U.S. EPA, 1999.

This concentration range was based on results from the National Urban Runoff Project (NURP), a major urban runoff study coordinated by the EPA at 28 locations throughout the United States in the early 1980s. The NURP studies found an average urban runoff phosphorus concentration of 0.33 mg/L (U.S. EPA Office of Research and Development, 1993). At the assumed concentration range with local precipitation values, phosphorus loading in urban runoff from the 5180 hectares within Acton averages 2.9 to 7.2 kilograms of phosphorus per day (kg/day). Acton officials have determined that, with a 3:1 ratio and a phosphorus concentration target of 0.1 mg/L in the proposed WWTP effluent, a phosphorus loading reduction of approximately 0.67 kg/day must be achieved through BMP implementation. However, if technological limitations produce effluent phosphorus concentrations that are actually closer to 0.2 mg/L, the BMPs must reduce 1.91 kg/day from Acton urban runoff. As a result, BMPs must remove approximately 10 to 66 percent of phosphorus from urban surface runoff within Acton, depending upon the removal efficiency of the WWTP (Town of Acton, 1998).

4.2 General Point/Nonpoint Source Trading Environmental Issues

The environmental issues involved in the Acton Trading Program primarily concern the use of NPS pollution trades. Therefore, an effort was made to ascertain the challenges associated with the use and quantification the NPS component of the Acton Trading Program.

4.2.1 The Nonpoint Source Pollution Regulation Issue

As a point/nonpoint source trading program, the proposed Acton Trading Program will utilize NPS reductions to allow point sources to discharge pollutants to the Assabet River. With the promulgation of the CWA, a comprehensive pollution permitting system, the NPDES, was created to aid the regulation of point source discharges to the surface waters of the United States (Downing and Sessions, 1985). Although NPS pollution was included in the CWA Section 208, and despite the fact that approximately 50 percent of water pollution in the U.S. is the result of NPSs, the CWA directives for reducing NPS have been largely ignored by the EPA. Most of the nation's NPS is created by agricultural processes, and the EPA has traditionally left agricultural matters to the U.S. Department of Agriculture (USDA). However, after facing resistance to its initial attempts to direct farmers to implement BMPs to reduce NPS from agricultural fields, the USDA retreated and has ignored the NPS issue as well (Caldart, 1998).

Therefore, despite the patchwork of water quality laws on the books at the federal, state, and local level, NPS pollution has always fallen through the nation's water quality regulations. As a result, point/nonpoint source effluent trading is seen by many environmental policy experts as an innovative way of accomplishing some type of regulation of and pollution reduction from NPS pollution sources. Reducing NPS pollution was a primary emphasis of the EPA creating the Draft Framework for Watershed Based Pollutant Trading as a guide to trading programs (Pelley, 1996). However, the critics of trading programs that utilize NPS pollution reductions to allow increases in point source discharges see things differently. They contend that NPS pollution should already be regulated under the CWA; therefore, reductions in NPS pollution should not be traded to allow an increase in point source pollution (Caldart, 1998). Such backsliding issues may need to be addressed by Acton officials due the use of NPS trades in the Acton Trading Program.

4.2.2 Nonpoint Source Loading Quantification Uncertainty

Of the issues facing the Acton Trading Program, the uncertainty associated with NPS pollution loading reduction quantification is easily the most troublesome. NPS pollution depends upon random weather inputs, is difficult to measure, and is not entirely understood (Stephenson et al., 1996). Further, NPS reduction credibility from BMP implementation was a primary trading concern that arose from the EPA Administrator's Point/Nonpoint Source Trading Initiative Meeting (U.S. EPA Office of Water, 1992). Despite these obstacles, many programs attempt to incorporate NPS reductions into trading programs because they are often the largest source of pollution in a watershed and are typically the most inexpensive to reduce (Stephenson et al., 1996).

The uncertainty of assumed urban runoff phosphorus concentrations listed above results in the wide expanse, noted above as 10 to 66 percent, of required Acton urban BMP phosphorus loading reduction efficiencies (Town of Acton, 1998). However, as described in Chapter 2, the 3:1 trading ratio applied in the Acton Trading Program includes uncertainty and environmental benefit factors. Therefore, phosphorus loading reduction estimates established via the NPS modeling studies included in Appendix A should provide the accuracy required to establish tradeable NPS phosphorus reductions. In addition, studies performed by Stephenson et al. (1996) have shown that the criticism applied to the quantification of NPS loading reductions has been exaggerated due to the fact that point sources also cannot be assumed to be constant. Further, they illustrate that the assimilative capacity of the river is also not constant; therefore, the actual water quality impact of both point and NPSs varies according to climatological factors. As a result, uncertainty prevails in many aspects of both point/point and point/nonpoint source trades and must be taken into consideration by applying trades in terms of water quality impact rather than precise reduction amounts (Stephenson et al., 1996). Therefore, the Acton Trading Program has minimized the uncertainty involved in trading programs by applying the appropriate trading ratios.

4.3 Application of Successful Watershed-Based Trading in the Assabet River Basin

A nutrient trading study performed by Apogee Research (1992) found that many conditions must be present in a watershed to ensure that a trading program can successfully provide water quality improvements. Many of the conditions apply to all types of trading, with the exception of the last two requirements, which apply strictly to point/nonpoint source trading. The conditions included:

- The waterbody must be identifiable as a watershed or segment.
- Water quality goals must be able to force action.
- Accurate and sufficient data must exist to identify targets and measure reduction amounts.
- Point sources must meet CWA technology-based discharge requirements before trading.
- Point sources must be facing requirements to either upgrade or trade for reductions.
- An institutional structure is necessary to facilitate and monitor trading.
- Effective implementation mechanisms must be a component of the trading system.
- Point and controllable NPSs must both contribute significant pollutant loads.
- Nonpoint load reductions must be cheaper to obtain than point source upgrades.

An inspection of the above conditions indicates that the Assabet River Basin meets many of the criteria for the implementation of a successful trading program. As was described in Chapter 2, the watershed contains significant point and nonpoint phosphorus sources and experiences prevailing water quality problems despite the fact that the watershed's point sources have already met CWA technology-based discharge requirements. In addition, the municipal WWTPs that currently discharge to the Assabet River are facing upgrade requirements within the near future. There is also sufficient interest in obtaining water quality goals in the Assabet River, as it is a popular canoeing and recreation water body. As such the poor river water quality has given rise to the OAR, an environmental organization dedicated to improving water quality in the Assabet River. As a result, information on point and NPSs of phosphorus are available from both the MDEQE and OAR, which maintains a water quality monitoring program in the Assabet River throughout the summer months (OAR, 1999). In the Acton Trading Program, an institutional structure is being developed to facilitate trading. Therefore, the Assabet River Basin appears to characterize many of the necessary conditions for embodying a successful watershed-based trading program.

4.4 Acton Trading Program Implementation Issues

Trading programs that incorporate both point and NPSs are frought with difficulties. As noted by Stephenson et al., (1996) the use of both point and NPSs trading in a watershed-based trading program compounds the problems associated with both its design and implementation. Although the proper conditions exist for watershed-based trading in the Assabet River Basin, many of the issues associated with the Acton Trading Program are implementation and administration issues. In particular, the location of the trading program downstream of the problematic phosphorus loading to the Assabet River, the minimal phosphorus loads impacted in the program, and the impact of Warner's Pond on NPS phosphorus reductions to Assabet River are key implementation issues in the Acton Trading Program.

4.4.1 Downstream Trading Location

Referring to Figure 13, the Acton Trading Program impacts only the last six miles of the Assabet River. As can be seen in Figure 14, MDEQE sampling data show that the heavy phosphorus loading in the river occurs far upstream of Acton. The large phosphorus concentration at the

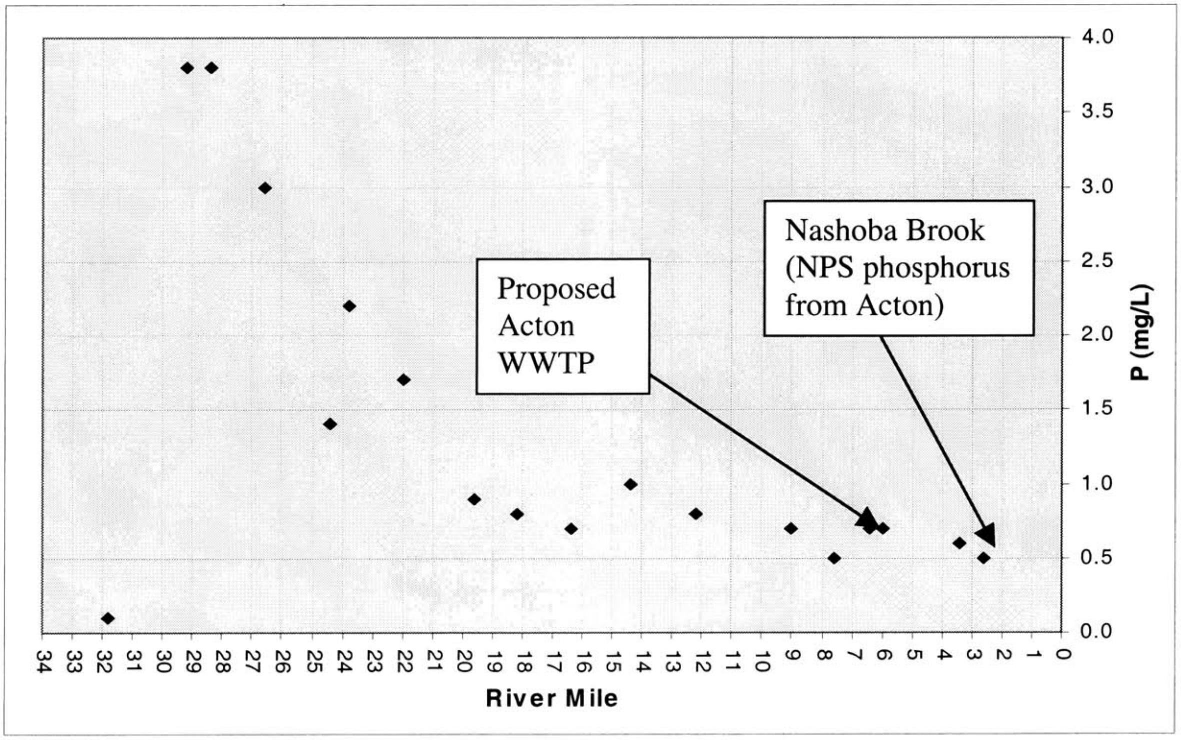

Figure 14. Measured Phosphorus Concentrations in the Assabet River
Source: Adapted from Hanley, 1989.

headwaters of the Assabet River is due to the loading from the Westborough WWTP, which in the summer months becomes the headwaters of the Assabet River (Hanley, 1989). As a result of the downstream location of the Acton Trading Program, no improvement in water quality will be seen throughout most of the Assabet River unless other upstream trading arrangements are considered.

4.4.2 Minimal Phosphorus Load Trades

Inspection of the phosphorus loading from the Assabet River Basin WWTPs and Nashoba Brook (from Acton urban runoff) as shown in Table 3 for low flows shows that the phosphorus loads targeted in the proposed Acton Trading Program (denoted by asterisks) are very small in comparison of the upstream WWTP phosphorus loads. As a result, the impacts on Assabet River water quality and the eutrophic conditions of the river are likely to be insignificant despite the substantial cost of program implementation. Phosphorus concentrations in Nashoba Brook were measured just upstream the junction with the Assabet River and represents the flows from both Fort Pond Brook and Nashoba Brook tributaries after they discharge into Warner's Pond (see Figure 13). Particularly interesting is that the phosphorus loading from Nashoba Brook (0.17 kg/d) as measured by OAR for the past several years, is lower than the Acton WWTP at even 0.1 mg/L (0.22 kg/d). Therefore, even if all phosphorus could be removed from Nashoba Brook through implementation of Acton phosphorus reduction BMPs, the actual Acton NPS phosphorus loading reduction to Acton WWTP phosphorus discharge trading ratio would not even reach 1:1, let alone 3:1. This is due to the effect of Warner's Pond on reducing Acton tributary phosphorus concentrations prior to discharging into the Assabet River.

Table 3. Assabet River Phosphorus Loading Summary for Low Summer Streamflows
Sources: Hanley, 1989; Town of Acton, 1998; OAR 1999.

Phosphorus Input	**Phosphorus Concentration (mg/L)**	**Flow rate (m^3/d)**	**Phosphorus Loading (kg/d)**
Westborough WWTP	4.00	11,786	47.0
Marlborough WWTP	7.00	5,483	38.4
Hudson WWTP	5.20	7,734	40.2
Maynard WWTP	6.30	4,161	26.2
Concord MCI WWTP	5.70	710	4.0
Acton WWTP @ 0.1 mg/L*	0.10	2,225	0.22
Acton WWTP @ 0.2 mg/L*	0.20	2,225	0.45
Fort Pond/Nashoba Brook*	0.07	3,670	0.17

* = possible Acton Trading Program phosphorus load

4.4.3 Warner's Pond Impact on Acton Phosphorus Loading Reductions to the Assabet River

In addition to the placement and small amounts of phosphorus load reductions to the Assabet River from the Acton Trading Program, the phosphorus reductions achieved in Acton may not even impact the Assabet River due to the nutrient catchment capability of Warner's Pond. As can be seen in Figure 15, most of Acton is drained by Nashoba Brook and Fort Pond Brook, both of which discharge into Warner's Pond, which then empties into the Assabet River. Studies of Warner's Pond have noted that it may act as a nutrient catchment (Town of Acton, 1998). As a

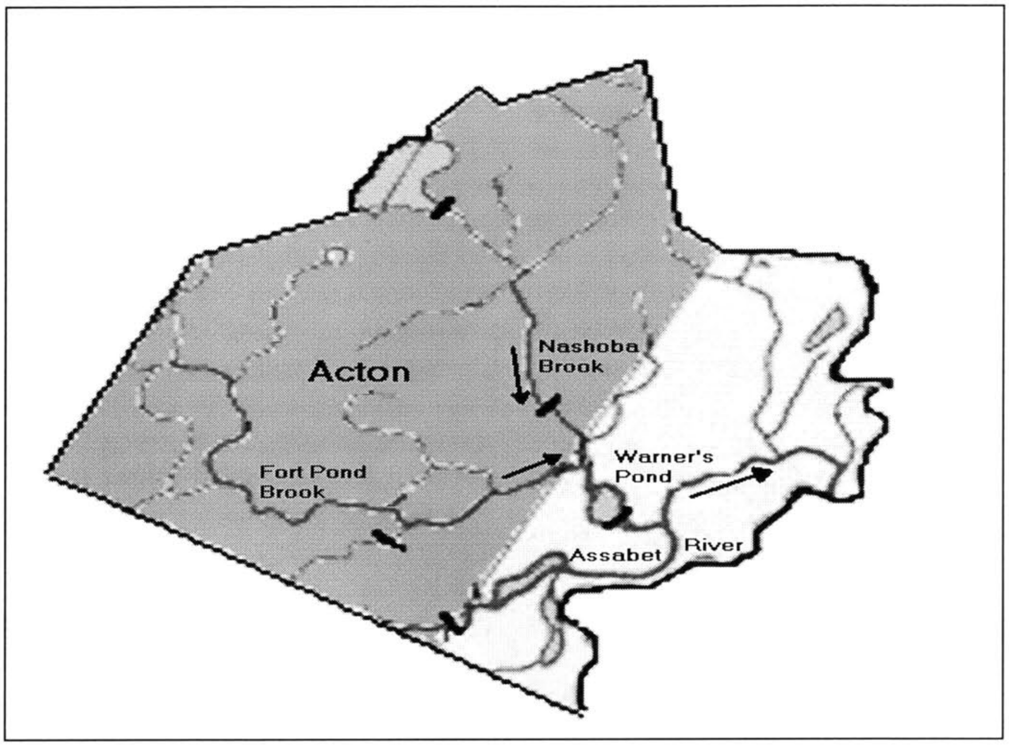

Figure 15. Acton Town Drainage to Warner's Pond
Source: Adapted from U.S. EPA, 1999.

result, the impacts of the Acton Trading Program on actual phosphorus loading reductions to the Assabet River may be negligible as only a minimal amount of phosphorus from the two Acton tributaries actually reaches the Assabet River. These conclusions are supported by the phosphorus concentrations recorded by the OAR water quality monitoring effort on Nashoba Brook downstream of Warner's Pond. The organization has consistently recorded phosphorus concentrations of approximately 0.07 mg/L at this location (OAR, 1999). Therefore, the phosphorus loads associated with urban NPS phosphorus reductions from Acton will not substantially impact water quality in the Assabet River due to the fact that much of the phosphorus from Acton is detained in Warner's Pond.

4.5 Modeling Analysis of the Proposed Acton Trading Program

A modeling analysis of the impacts of the Acton Trading Program on water quality in the Assabet River was recently completed by the author in a related study of the Acton Trading Program. This modeling analysis was performed as part of a group research project for the MIT Department of Civil and Environmental Engineering Master of Engineering Program. The final project report from this study has been included as Appendix A to this thesis. The analysis was performed with the EPA QUAL2E stream water quality model for low stream flow conditions in the Assabet River. It confirmed that the minimal NPS phosphorus loading from the Town of Acton and the impact of Warner's Pond on reducing the phosphorus concentration in Nashoba Brook downstream of Warner's Pond negates the impact of the Acton Trading Program on water quality in the Assabet River. Figure 16 shows the negligible impact of the best-case scenario

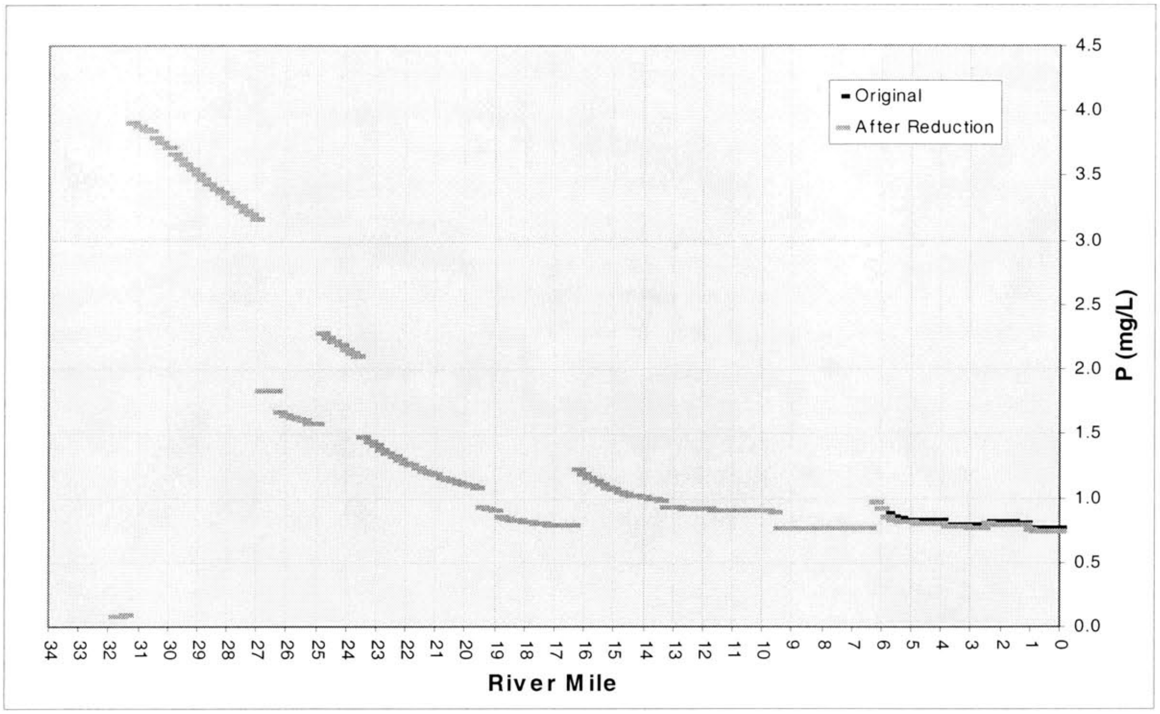

Figure 16. QUAL2E Acton Trading Program Impact on Assabet River Phosphorus Levels

for Acton urban BMP phosphorus reductions (66 percent) to Fort Pond Brook and Nashoba Brook. The small phosphorus load reductions resulting from Acton BMP implementation has little impact on Assabet River phosphorus concentrations. Similarly, the model predicts a minor reduction in the Assabet River algae concentrations shown in Figure 17.

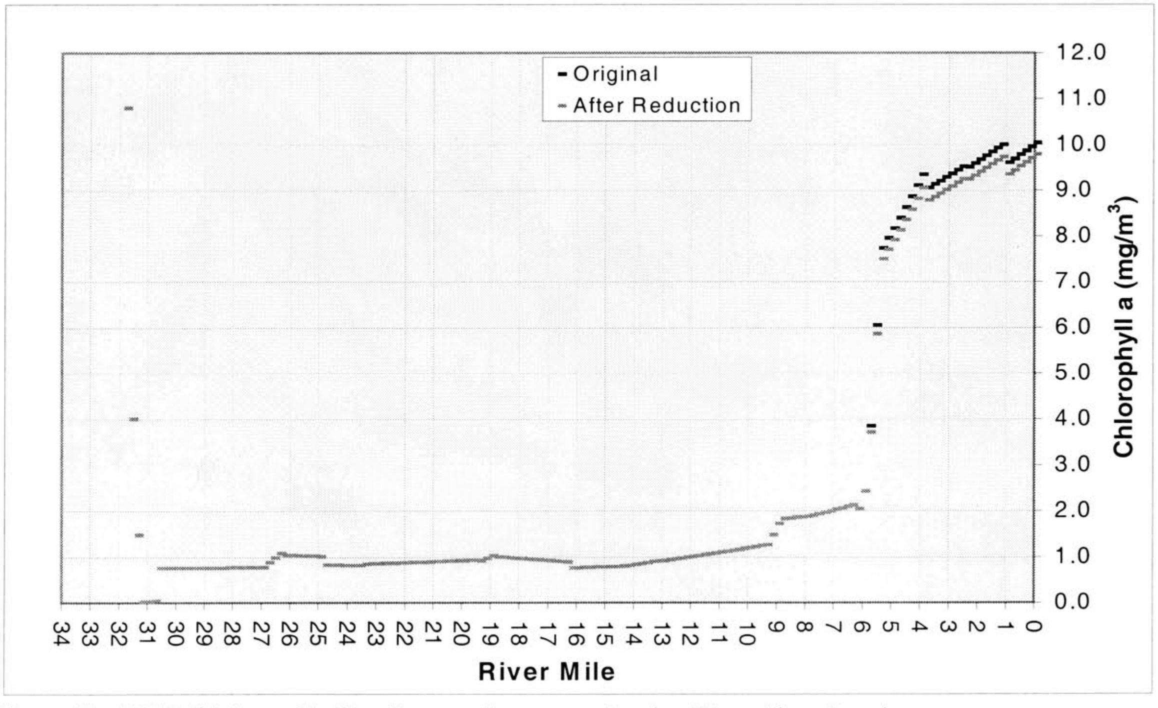

Figure 17. QUAL2E Acton Trading Program Impact on Assabet River Algae Levels

Through the analysis of a number of similar Acton BMP phosphorus reduction efficiency and Acton WWTP effluent phosphorus concentration scenarios, the QUAL2E modeling analysis illustrated that the Acton Trading Program is unlikely to have a substantial impact on water quality in the Assabet River.

4.6 Analysis of Alternative Acton/Westborough Point Source Trade

Due to the minimal predicted impact of the Acton Trading Program on water quality in the Assabet River, the QUAL2E modeling study included in Appendix A concluded that alternative trading arrangements should be considered to utilize the best water quality impact for the economic cost of improvement. The study cited significant water quality improvements in the Assabet River as the result of efficient point source trades with the municipal WWTPs located upstream of Acton. In particular, the model was used to determine the effects of a point/point source trade between the Acton WWTP and the Westborough WWTP, which is located at the headwaters of the Assabet River. A review of the model results and feasibility of the hypothetical trade are presented below.

4.6.1 Alternative Acton/Westborough Point Source Trade Modeling Analysis

The hypothetical trade scenario modeled with QUAL2E involved a 1 mg/L phosphorus trade between the Acton and Westborough WWTPs. The trade allowed the Acton WWTP to discharge a 1 mg/L maximum phosphorus concentration for a 1 mg/L reduction of the Westborough WWTP discharge phosphorus concentration (from approximately 5 mg/L to 4 mg/L). Due to the flowrate difference between the Westborough and Acton WWTP, a 5:1 phosphorus loading trading ratio is applied in this hypothetical trade scenario. Figure 18 shows

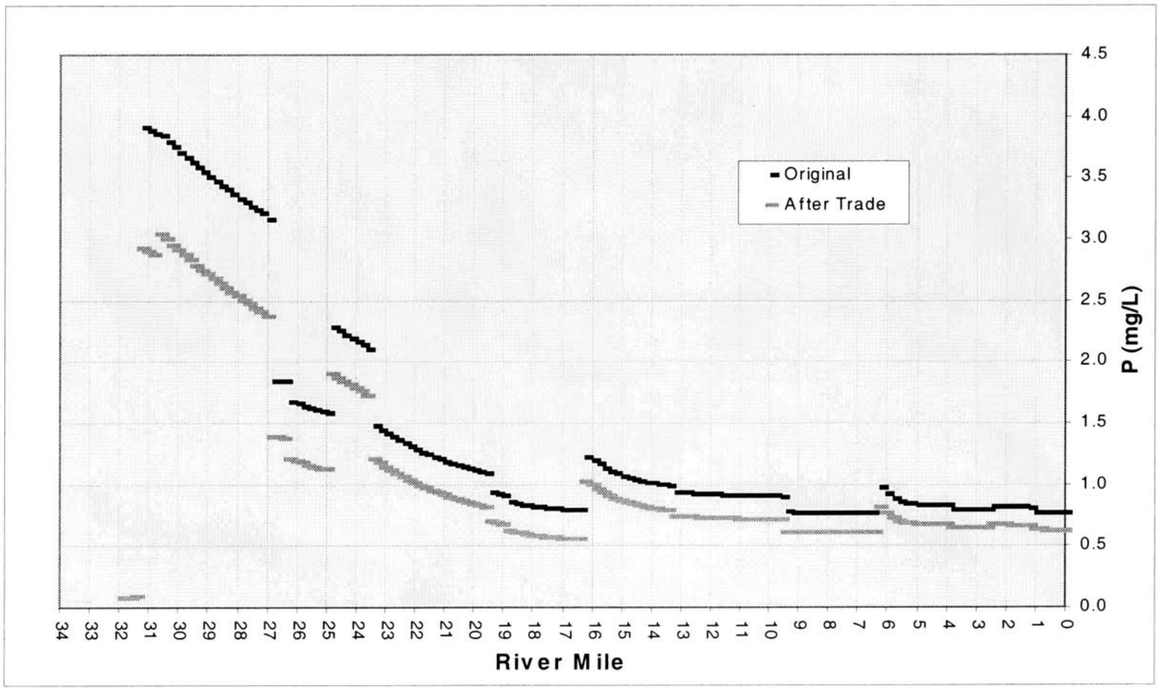

Figure 18. QUAL2E Hypothetical Acton/Westborough Trade Impact on Assabet River Phosphorus Levels

the impact of the point/point source trade on the predicted Assabet River phosphorus concentrations. The hypothetical trade achieves a much lower overall phosphorus level downstream of the Acton WWTP than the proposed Acton Trading Program. Figure 19 shows the algae concentrations that were predicted for this trading scenario. As can be seen in both figures, the hypothetical point/point source trade between the Acton and Westborough WWTPs is predicted by QUAL2E to have a much larger impact on water quality than the proposed Acton Trading Program.

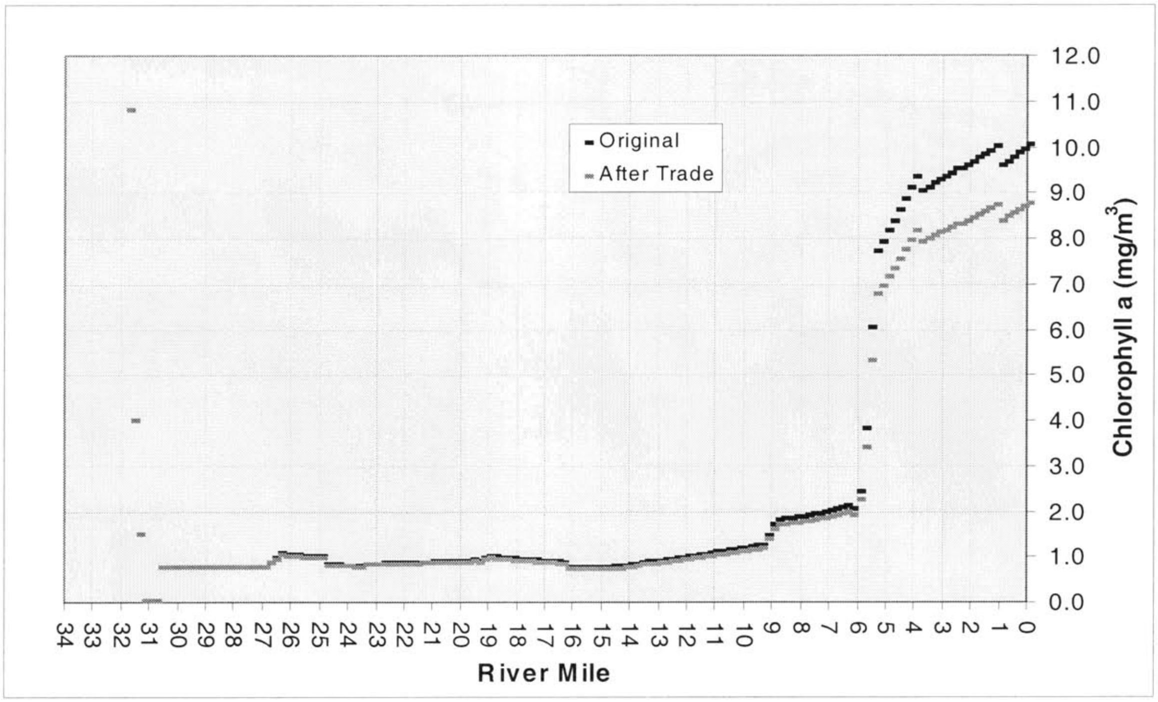

Figure 19. QUAL2E Hypothetical Acton/Westborough Trade Impact on Assabet River Alage Levels

In addition to the predicted water quality benefits of the hypothetical Acton/Westborough WWTP point source trade, inspection of the chemical addition required in both the hypothetical trade and the Acton Trading Program prove that the hypothetical trade is also much more efficient than the Acton Trading Program. In the WWTP design prepared for the report included in Appendix A, Yukiyasu Sumi concluded that the 0.1 mg/L phosphorus concentration could most easily be obtained by the excessive chemical addition of ferric chloride or aluminum salts. These chemicals remove phosphorus by forming precipitants that settle out of the wastewater and are removed as excess waste sludge. However, the phosphorus reduction achieved by chemical addition decreases as the required phosphorus removal efficiency increases. For example, an aluminum salt addition for 75 percent WWTP phosphorus removal typically requires 1.4 moles of aluminum salt for each mole of phosphorus removed. Increasing the WWTP requisite phosphorus removal efficiency to 95 percent requires the addition of 2.3 moles of aluminum salt for each mole of phosphorus removed (Pincince, 1999). Therefore, in the proposed Acton Trading Program, the excessive chemical addition is required to obtain the very high phosphorus removal efficiencies necessary to consistently produce 0.1 mg/L phosphorus effluent concentrations from the Acton WWTP are very inefficient. Since the chemicals will represent a substantial Acton WWTP operation expenditure, obtaining such high phosphorus removal efficiencies is also expensive. However, as can be seen in Figure 20, using chemical addition to remove phosphorus from an existing plant, such as the Westborough WWTP, requires no design modifications. As a result, the removal of 1 mg/L of phosphorus from the Westborough WWTP can be obtained from only the addition of chemicals to the WWTP influent. Since the required phosphorus removal in this case is very low, approximately 20 percent for the average Westborough WWTP effluent concentration of 5 mg/L, the required chemical addition rate is much lower than that required to obtain the high removal efficiencies necessary to reduce the Acton WWTP to 0.1 mg/L as proposed in the Acton Trading Program. As a result, the environmental benefits of this hypothetical trade that requires moderate phosphorus removal efficiencies at both WWTPs can be achieved much more efficiently than the Acton Trading Program, which requires excessive phosphorus removal at only the Acton WWTP to achieve a minor environmental benefit.

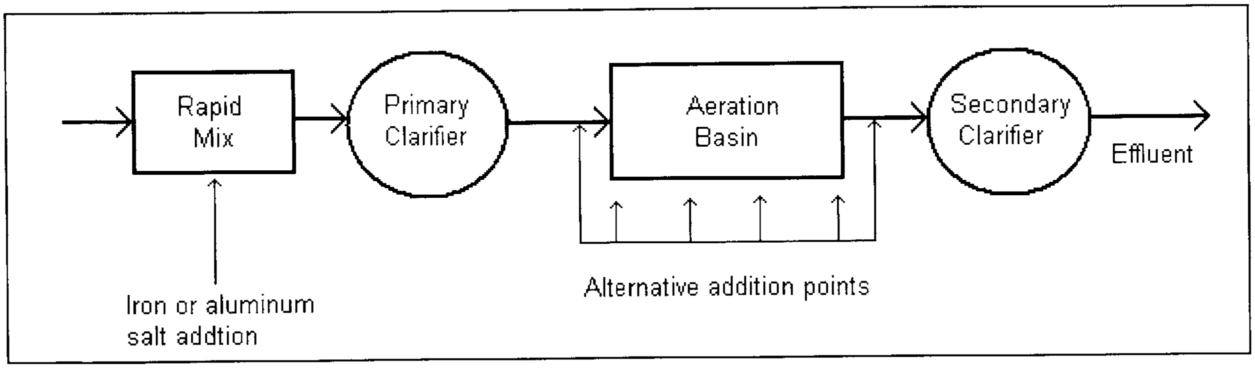

Figure 20. Chemical Addition to Existing WWTP
Source: Pincince, 1999.

5 CONCLUSIONS AND RECOMMENDATIONS

As innovative and flexible methods of maintaining water quality in unique watersheds, water pollution trading programs are extremely distinct in terms of development, implementation, and outcome. The diversity of the existing trading programs exemplifies the flexibility that exists to conform trading programs to manage nearly any site-specific watershed pollution problem. Although the use of watershed-based pollution trading is relatively unproven, observation of the existing trading programs indicates that trading has the potential to improve water quality in heavily impaired watersheds such as the Assabet River Basin.

Despite the ideal conditions for watershed-based phosphorus trading in the Assabet River Basin, the proposed Acton Trading Program is not likely to achieve its water quality objectives for the Assabet River. A comprehensive review of the Acton Trading Program and stream water quality modeling studies performed with the QUAL2E model indicate that, as proposed, the Acton Trading Program will have little impact on water quality in the Assabet River. This is due to the downstream location and minor nature of the phosphorus inputs involved in the proposed Acton Trading Program as compared to the numerous municipality WWTP phosphorus discharges upstream of Acton. Further examination of the phosphorus loads reveals that, due to the nutrient catchment capability of Warner's Pond, the tributaries that drain Acton consistently contribute less phosphorus to the Assabet River than the proposed Acton WWTP would at the target 0.1 mg/L effluent phosphorus concentration. Therefore, even if all of the phosphorus could be removed from these tributaries through implementation of Acton phosphorus reduction BMPs, the actual Acton NPS phosphorus loading reduction to Acton WWTP phosphorus discharge trading ratio would not even reach 1:1, let alone 3:1.

5.1 Recommendations for Trading in the Assabet River Basin

Since the water quality impact of the proposed Acton Trading Program is minimal and the financial costs are substantial, alternative trading arrangements should be considered by the Town of Acton. As has been shown with the hypothetical Acton/Westborough WWTP point source trade, an Acton WWTP point source trade with one of the five existing Assabet River Basin WWTPs can provide a much larger water quality impact than the proposed Acton Trading Program. Since none of the WWTPs that discharge to the Assabet River currently utilize phosphorus removal processes beyond conventional treatment, the potential exists to significantly reduce phosphorus loading from point sources into the Assabet River through chemical addition. Further, these point/point source trades could be arranged to achieve water quality improvements by utilizing efficient phosphorus removal via chemical addition to obtain moderate phosphorus discharge concentrations between the two trading WWTPs. This combined phosphorus removal method is much more efficient than the method proposed in the Acton Trading Program, which pushes chemical phosphorus removal to its limits to obtain extremely low phosphorus effluent concentrations from only the Acton WWTP.

5.2 Recommendations for Future Study

As has been seen with the proposed Acton Trading Program and the existing trading programs, trading can be inhibited by a number of factors. The Fox River and Lake Dillon trading programs are examples of two entirely opposite trading environments, one competitive and the other cooperative, that both experienced few trades due to entirely different reasons. However, recent trading programs such as the Passaic Valley experience have seen trading overcome the traditionally adversarial association between pollution sources and regulating entities to develop a cooperative relationship. Recent trading efforts have allowed Rahr Malting to expand its facilities at a lower cost than conventional methods and aided the otherwise unobtainable ecological improvements of Boulder Creek. Having been used as a case study to develop EPA guidelines from its inception, the trading program in the Tar-Pamlico watershed is also well underway. In addition, trading has provided environmental gains through cooperative wetland mitigation trading in the many programs listed in Table 1. However, a number of programs in Table 2 have been slow to develop or have stalled. Meanwhile, the cooperation required to develop a trading program in an interstate region such as the Long Island Sound watershed has slowed even the most enthusiastic water quality officials in Connecticut. Finally, although the ideal trading conditions exist in the Assabet River Basin, the proposed Acton Trading Program has been predicted to provide a minimal impact on improving water quality in the eutrophic Assabet River. Therefore, the brief history of existing trading programs remains inconclusive as an indicator of the likely success of future watershed-based pollution trading programs that are being encouraged through current EPA water quality policies.

The recent flurry of interest in watershed trading is not likely to diminish as long as EPA support is firmly behind the concept as an innovative water quality management policy. All of the basic trading concepts parallel the agency's Watershed Protection Approach and NPS reduction ideals of flexible watershed-level management that controls both point and NPSs. As a result, a more comprehensive study of watershed-based trading, particularly in the context of the similar CAAA SO_2 emissions trading, is warranted to determine whether the EPA's strong encouragement of trading is a sound environmental policy or a misplaced hope for a more efficient means of pollution control.

6 REFERENCES

Ackerman, F. And W. Moomaw. 1997. SO2 Emissions Trading: DoeS it Work? The Electricity Journal. 10(7):61-66. August/September, 1997.

American Rivers. 1999. American Rivers Home Page. http://www.amrivers.org/

Apogee Research, Inc. 1992. Incentive Analysis for Clean Water Act Reauthorization: Point/Nonpoint Source Trading For Nutrient Discharge Reductions. U.S. Department of Commerce National Technical Information Service. Springfield, Virginia.

Biswas, A.K. 1997. Water Resources: Environmental Planning, Management, and Development. McGraw-Hill. New York, New York.

Bohi, D.R., and D. Burtraw. 1997. SO_2 Allowance Trading: How Do Expectations and Experience Measure Up? The Electricity Journal. 10(7):67-88.

Brady, D.J. 1996. The Watershed Protection Approach. Water Science and Technology. 33(4-5)17-21.

Burtraw, D. 1996. The SO_2 Emissions Trading Program: Cost Savings Without Allowance Trades. Contemporary Economic Policy. 14:79-94. April.

Caldart, C. 1998. 1.811J - Environmental Law: Pollution Control Class Notes. Massachusetts Institute of Technology. November 10-12, 1998.

Canning, K. 1999. Effluent Trading: Making it Work. Pollution Engineering. February, 1999. 21-24.

Clinton, B., and A. Gore. 1995. Reinventing Environmental Regulation. March 16, 1995.

Cooperman, A.N., and W.R. Jobin. 1971. Report on Pollution of the Assabet River. Massachusetts Water Resources Commission Division of Water Pollution Control Water Quality Management Section: Boston, Massachusetts.

Corbis. 1999. Internet Home Page. http://www.corbis.com March 5, 1999.

Cushman, J. 1997. Million Wetland Acres Lost in 1985-1995. New York Times. September 18, 1997.

David, M.H., and E.L. David. 1983. Cost-Effective Regulatory Options for Water Quality Limited Streams. Water Resources Bulletin. 19(3):421-428. June 1983.

Davenport, T.E., N.J. Phillips, B.A. Kirschner, and L.T. Kirschner. 1996. The Watershed Protection Approach: A Framework for Ecosystem Protection. Water Science and Technology. 33(4-5):23-26.

Downing, D., and S. Sessions. 1985. Innovative Water Quality-Based Permitting: A Policy Perspective. Journal WPCF. 57(5):358:365.

Ellerman, A.D., P.L. Joskow, and R. Schmalensee. 1998. 1996 Update on Compliance and Emissions Trading under the U.S. Acid Rain Program. Massachusetts Institute of Technology Center for Energy and Environmental Policy Research.

Emerson, P. 1998. EDF Believes Market Forces Can Protect Texas's Environment. Environmental Defense Fund Electronic Newsletter. http://www.edf.org

Fedesco, Mark. 1998. Message from the Director. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Griffin, R. 1998. The Case for Tradable Emission Permits. Texas Water Resources. 23(3):1-3.

Hall, J.C., and C.M. Howett. 1994. Trading in the Tar-Pamlico. Water Environment & Technology. p. 58-61. July 1994.

Hanley, N.E. 1989. "Assabet River 1989 Water Quality Management Plan: Parts A, B, and C." Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control.

Hersch, P. 1997. Minnesota Issues First-of-a-Kind Discharge Permit. Pollution Online Industry News. February 28, 1997.

Industrial Economics, Inc. 1992. The Benefits and Feasibility of Effluent Trading Between Point Sources: An Analysis in Support of Clean Water Act Reauthorization. U.S. Department of Commerce National Technical Information Service. Springfield, Virginia.

Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control Technical Services Branch. 1981. The Sudbury-Assabet-Concord Basin Water Quality Management Plan.

Merrifield, J. 1998. Implementing Emission Allowance Markets. Texas Water Resources. 23(4):1-2.

Morris, D.P., and W.M. Lewis, Jr. 1992. Nutrient Limitation of Bacterioplankton Growth in Lake Dillon, Colorado. Limnology and Oceanography. 37(6):1179-1192.

Murphy, J. 1997. PVSC Kicks Off Effluent Trading Pilot Project. New Jersey Discharger. 5(1) Summer 1997.

Murphy, J. 1998. Update on Effluent Trading. New Jersey Discharger. 6(1). Summer 1998.

O'Brien, P. 1998. Nitrogen Trading – A New York Perspective. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

O'Neil, W.B. 1983. Transferable Discharge Permit Trading Under Varying Stream Conditions: A Simulation of Multiperiod Permit Market Performance on the Fox River, Wisconsin. Water Resources Research. 19(3):608-612. June 1983.

Organization for the Assabet River (OAR). 1999. Assabet River Water Quality Sampling Data.

Overton, M.S. 1998. Watershed-Based Nitrogen Trading: The Cost-Effective Solution for Long Island Sound – An Enthusiast's Perspective. . Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Passi, P. 1998. Pollution Trading Program Takes Shape. American Rivers Organization. http://www.amrivers.org/mm/pollution198.html 1998.

Pelley, J. 1996. "Effluent Trading Framework Issued by EPA Tackles Nonpoint Sources." Environmental Science and Technology. 30:386.

Percival, R., A. Miller, C. Schroeder, and J. Leape. 1992. Environmental Regulation: Law, Science, Policy. "Wetlands Protection and the Section 404 Permit Program," p. 965-984.

Pincince, A. 1999. Phosphorus Removal Via Chemical Addition. 1.85 Wastewater Treatment Course Notes. Massachusetts Institute of Technology. April 22, 1999.

Ridley, M., and B.S. Low. 1993. Can Selfishness Save the Environment? The Atlantic Monthly. September 1993. 76-86.

Roy, L.A. 1998. Organization for the Assabet River Letter to Trudy Coxe and John P. DeVillars. February 9, 1998.

Ruff, L.E. 1970. The Economic Common Sense of Pollution. The Public Interest. 19:69-85.

Salata, J. Pollutant Trading. 1998. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Shanahan, P. 1999. Personal Communication with the Author. April 7, 1999.

Smith, A.E., and A.D. Ellerman. 1998. The Costs of Reducing Utility SO2 Emissions – Not as Low as You Might Think. Massachusetts Institute of Technology Center for Energy and Environmental Policy Research. August 17. 1998.

Stacey, P. 1998. Nitrogen Trading – A Connecticut Perspective. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Stephenson, K., P. Norris, and L. Shabman. 1996. Watershed-Based Effluent Trading: The Nonpoint Source Challenge. Contemporary Economic Policy. 16:412-421.

Stewart, R. And J. Krier. 1978. Environmental Law and Policy. 2^{nd} edition. 68-75.

Texas Water Resources Institute. 1998. Using Market-Based Approaches to Protect the Environment. Texas Water Resources. 23(4):1.

Town of Acton. 1998. "Watershed Trading Program, Discussion Draft."

United States Environmental Protection Agency. 1999. Internet Home Page. http://www.epa.gov

United States Environmental Protection Agency. 1995. "Guidance Manual for Developing Best Management Practices (BMP)." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Long Island Sound Office. 1997. Long Island Sound Study. Stamford, Connecticut. EPA 840-R-97-001.

United States Environmental Protection Agency Office of Policy, Planning and Evaluation. 1998. Sharing the Load: Effluent Trading for Indirect Dischargers. Washington, D.C. EPA 231-R-98-003.

United States Environmental Protection Agency. 1993. Office of Research and Development. Handbook of Urban Runoff Pollution Prevention and Control Planning. Washington D.C. EPA/625/R-93/004.

United States Environmental Protection Agency Office of Water. 1997. Technical Guidance Manual for Developing Total Maximum Daily Loads Book 2: Streams and Rivers, Part 1: Biochemical Oxygen Demand/Dissolved Oxygen and Nutrients/Eutrophication. Washington, D.C. EPA 823-B-97-002

United States Environmental Protection Agency Office of Water. 1996(a). "Draft Framework for Watershed-Based Trading." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1996(b). "Draft Trading Update – Cherry Creek Basin, Colorado." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1996(c). "Draft Trading Update – Lake Dillon, Colorado." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1996(d). "Draft Trading Update – Boulder Creek, Colorado." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1994. President Clinton's Clean Water Initiative. Washington, D.C. EPA 800-R-94-001.

United States Environmental Protection Agency Office of Water. 1992. Administrator's Point/Nonpoint Source Trading Initiative Meeting: A Summary. Washington, D.C. EPA 841-S-92-001.

Wallace, S., C. Sparks, and B. Micheletti. 1997. Nonpoint Source Trading Creates New Discharge Opportunities. Water Environment and Technology. 9(5):18-22.

Wenning, R.J, N.L. Bonnevie, and S.L. Huntley. 1994. Accumulation of Metals, Polychlorinated Biphenyls, and Polycyclic Aromatic Hydrocarbons in Sediments from the Lower Passaic River, New Jersey. Archives of Environmental Contamination and Toxicology. 27:64-81.

West Group. 1998. Selected Environmental Statutes: 1998-1999 Educational Edition. St. Paul, Minnesota.

Zander, B. 1993. TMDL Case Study: Boulder Creek, Colorado. United States Environmental Protection Agency Region VIII Water Division. Denver, Colorado. EPA 841-F-93-006.

Zander, B. 1991(a). Innovations at Boulder Creek. EPA Journal 17(5):50.

Zander, B. 1991(b). Nutrient Trading – in the Wings. EPA Journal 17(5):47-49.

Zorpette, G. 1994. "A Slow Start for Emissions Trading." IEEE Spectrum. 31(7): 49-52.

APPENDIX A

Control of Point and Nonpoint Sources of Phosphorus in Acton, Massachusetts

By:

Anouk Savineau
Yukiyasu Sumi
Teresa Raine
Steve McGinnis

Massachusetts Institute of Technology

Department of Civil and Environmental Engineering

Master of Engineering Program

April 1999

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Executive Summary

As innovative and flexible methods of maintaining water quality in unique watersheds, watershed-based pollution trading programs are extremely distinct in terms of development, implementation, and outcome. Although the use of watershed-based pollution trading is relatively unproven in the few existing trading programs, it does seem to have the substantial potential to improve water quality in heavily impaired watersheds such as the Assabet River Basin. However, the QUAL2E modeling studies predict that the proposed Acton Trading Program will have little impact on water quality in the Assabet River. Further, the modeling studies indicate that point/point source trades will more productively achieve water quality objectives in the Assabet River. As a result, alternative point/point source trades with the four municipal WWTPs that discharge to the Assabet River upstream of Acton should be considered.

A study of the nonpoint source pollution in the Acton area showed that the average range of runoff phosphorus concentrations is approximately 0.2 to 0.5 mg/L during an average storm event. The two feasible options available for phosphorus removal include a buffer strip and a detention pond (which, as will be shown shortly, will be included in the design of the treatment wetland.). A buffer strip of lawn grass treating a 5 acre residential area achieves an average phosphorus removal of 50% producing an average treated runoff concentration of 0.17 mg/L. Modeling results show approximately 50% removal of phosphorus in a 0.18 ac-ft pond treating a 50 acre residential and forestland area. However, literature studies show that actually phosphorus removal ranges from 12% to 90%. Surface overflow from the pond averages a concentration of 0.21 mg/L. If future designs of areas are to include a detention pond, it would be beneficial to include a buffer strip at the pond overflow area.

As part of the BMP analysis, a design for a treatment wetland in a recreational park is proposed in this paper. The wetland's main function is to reduce the phosphorus concentration in storm water runoff to prevent the eutrophication of a swimming pond. A literature study shows that the wetland will be able to reduce phosphorus concentrations in runoff from approximately 0.35 mg/L to 0.12 mg/L. Final phosphorus concentrations in the swimming pond will be approximately 0.0429 mg/L, sufficiently below the threshold limit of 0.05mg/L. The wetland will consist of a detention pond, two emergent vegetation marshes, and a micro pool. Phosphorus removal will be accomplished by means of sedimentation, and plant and microbial uptake.

1 Introduction

The Town of Acton, Massachusetts, is a community located in the Assabet River Basin that currently relies mostly upon individual sewage treatment via onsite septic systems. For the past several years, the regions of Acton known as South Acton and Kelley's Corner have been experiencing septic system failure due to shallow groundwater levels. As a result, the Town of Acton has begun designing a wastewater treatment plant (WWTP) to serve these regions of Acton. If approved by the United States Environmental Protection Agency (EPA), the Acton WWTP will discharge some of its effluent to the Assabet River, which is currently in a eutrophic state due to the nutrient loading, particularly phosphorus, from existing WWTPs upstream of Acton. This project attempts to design a WWTP for the Town of Acton that produces an effluent with minimal phosphorus concentrations and analyzes the environmental impacts of the Acton WWTP phosphorus inputs to the river with both non-point source (NPS) and stream water quality modeling. In order to minimize the impact of the Acton WWTP on the impaired water quality in the Assabet River, the project also analyses the use of urban best management practices (BMPs) to reduce Acton NPS phosphorus loading to an adjacent Assabet River tributary. In an effort to publicly demonstrate the advantages of using BMPs to improve local water quality, the project will evaluate the use of a constructed wetland to reduce the nutrient loading to a swimming pond in the newly constructed North Acton Recreation Area (NARA.) Since the swimming pond eventually discharges into the Assabet River Basin, improved water quality of the swimming pond is directly related to improved water quality of the Basin.

1.1 Assabet River Overview

The Assabet River Basin is located in east-central Massachusetts (See Figure 1-1). The basin drains approximately 135 square miles and contains nineteen small towns and one city. As can be seen in Figure 1-2, the Assabet River originates in an impounded swampy area located in Westborough, Massachusetts, and stretches 31 miles through a number of highly populated

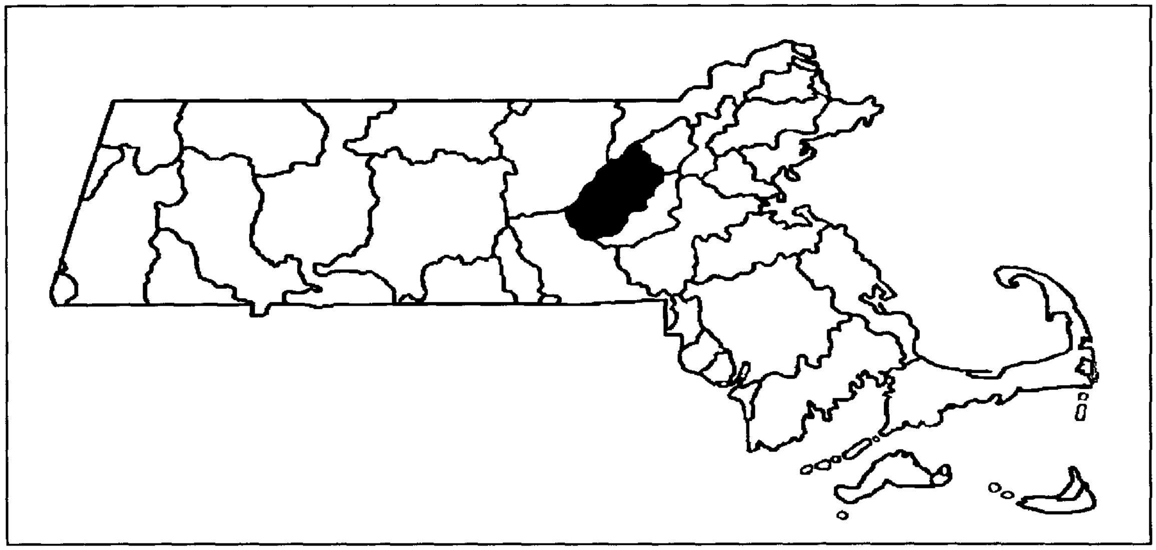


Figure 1-1. Location of Assabet River Basin (Source: U.S. EPA, 1999.)

areas. Just past the Town of Concord, the Assabet River merges with the Sudbury River to form the Concord River, which feeds the Merrimack River (Organization for the Assabet River, 1999.)

WWTPs operated by the communities of Westborough (including Shrewsbury), Marlborough, Hudson, Maynard, and Concord discharge to the Assabet River. In addition, the Massachusetts Correctional Institute (MCI) at Concord also discharges minimal flows to the river. The Assabet River follows a pattern where WWTP plant discharges are located just above a dam in an impoundment area, as can be seen in Figure 1-3 (Hanley, 1989). On average, the WWTPs

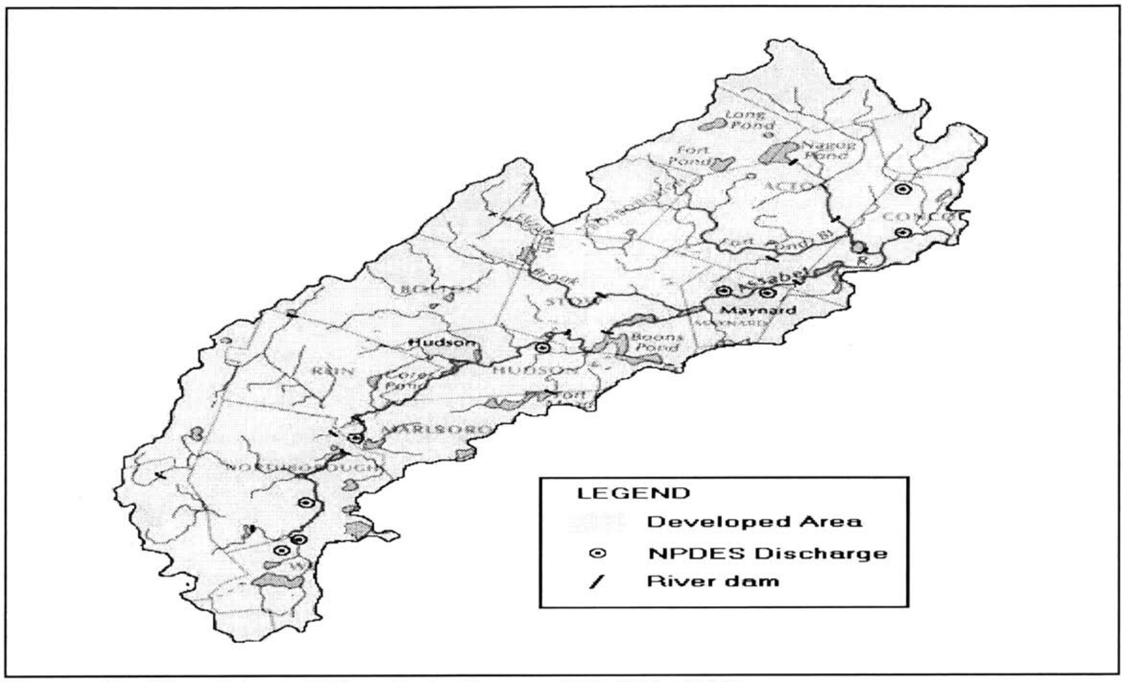


Figure 1-2. Map of Assabet River Basin (Source: U.S. EPA, 1999).

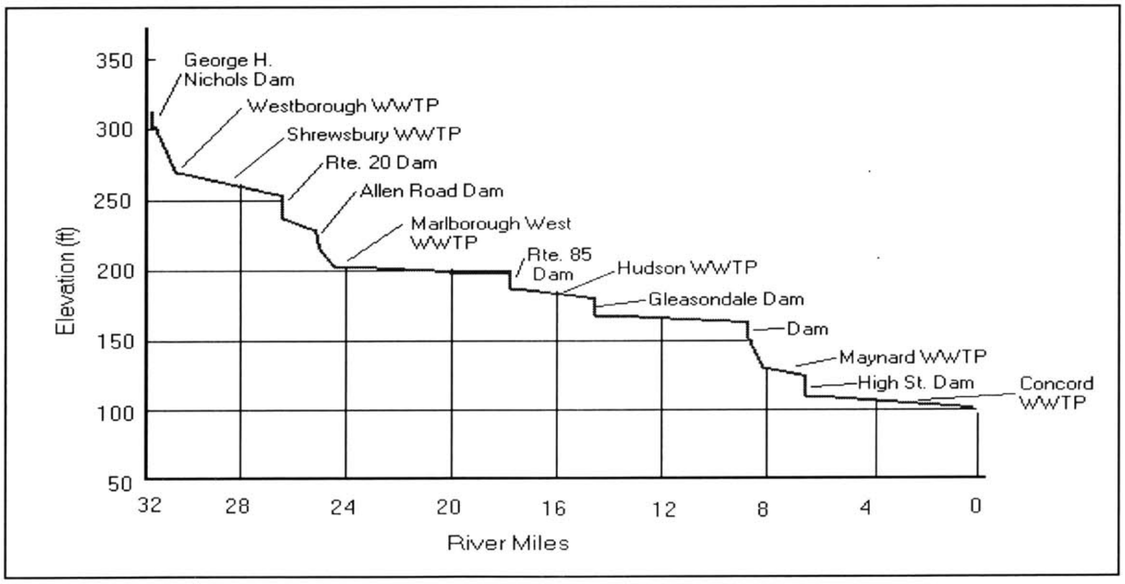


Figure 1-3. Dam and WWTP Locations Along the Assabet River (Source: Hanley, 1989).

are located approximately every six miles along the river (Roy, 1998.) The small natural gradient and numerous impoundments created by the periodic location of dams along the river produce an overall sluggish flow throughout most of the watershed, with the exception of some fast flowing sections located near Maynard and Hudson (Organization for the Assabet River, 1999.) Although the dams create impoundments that suffer from worse water quality, the water is re-aerated as the river flows over the dams, which periodically improves dissolved oxygen levels in the river (Hanley, 1989.)

1.2 Assabet River Basin Water Quality History

The Assabet River has been laden with water quality and environmental problems for many years. Poor water quality in the river first prompted the Massachusetts Department of Environmental Protection Division of Water Pollution Control to undertake extensive water quality sampling in 1965. However, the primary emphasis of the sampling was to determine dissolved oxygen and biochemical oxygen demand rather than nutrient concentrations. Subsequent sampling endeavors to assess the condition of water quality ensued in 1969, 1974, 1979, 1986, and 1987 (Hanley, 1989.) A report on the pollution of the Assabet River issued in 1971 found that phosphates from WWTP discharges were resulting in an average river phosphate concentration 60 times the allowable limit. In addition, worse conditions were observed in the numerous impoundment areas. At the time, only the Shrewsbury, Hudson, and Maynard WWTP were discharging to the Assabet River and plants were being constructed at Westborough and Marlborough. As a result of its findings, the report strongly urged communities along the Assabet River to develop phosphate removal programs (Cooperman and Jobin, 1971). The poor Assabet River water quality conditions prevailed, despite the passage of the 1970 Clean Water Act and subsequent assignment of National Pollution Discharge Elimination System (NPDES) permits to the discharging WWTPs. The 1979 sampling report also found that the Assabet River "impoundments are highly eutrophic with large amounts of aquatic growth, especially algal blooms during certain periods of the summer." Additionally, the report stated that all sections of the Assabet River were in violation of the Class B standard that had been assigned to the Assabet River in 1978. The entire river violated total phosphorus and fecal coliform standards, and only one section passed the dissolved oxygen standard for this classification (Massachusetts Department of Environmental Quality Engineering, 1981.)

The poor water quality in the Assabet River prompted the Massachusetts Department of Environmental Quality Engineering (DEQE) to develop the first water quality management plan for the Assabet River in 1981. The plan noted the problems caused by nonpoint sources, but maintained that the poor water quality in the Assabet River was largely due to excessive point source discharges from the WWTPs located along the river (Massachusetts Department of Environmental Quality Engineering, 1981.) The 1981 water quality management report was subsequently revised in 1989. The 1989 water quality management plan stressed increased nutrient studies and strict adherence to discharge limits to improve water quality in the Assabet River. Although $50 million in WWTP improvements from 1972-1989 increased overall dissolved oxygen levels, water quality studies of the Assabet River performed in 1989 indicated that WWTP nutrient loadings were still affecting the trophic state of the river (Hanley, 1989.)
In 1986, the poor water quality conditions in the Assabet River spurred the development of the Organization for the Assabet River (OAR), a non-profit organization of local residents dedicated to improving the water quality in the Assabet River. The OAR maintains a substantial water

quality monitoring program and sponsors related environmental protection programs. The group utilizes the water quality data to help enforce wastewater discharge regulations on the five WWTPs that discharge into the Assabet River (Organization of the Assabet River, 1999.)

1.3 Current Water Quality Conditions

The water quality problems suffered by the Assabet have become commonplace in many areas of Massachusetts. In addition to continued water quality difficulties resulting from municipal WWTPs, industrial discharges have also increased as several computer technology companies have located within the Assabet River Basin. Steep growth rates throughout the Assabet River Basin have forced many communities to struggle with demanding periods of rapid residential development. The trophic state of the river has continued to worsen due to excessive nutrient loading (Hanley, 1989.) During the summer of 1995, the flows in the Assabet River were recorded by the United States Geological Survey to be less than the sum of the wastewater discharges into the river (Roy, 1998). As a result, the entire stretch of the Assabet River was listed by the State of Massachusetts on its most recent "List of Impaired Waters in Massachusetts."

The Assabet River remains in a highly eutrophic state characterized by excessive algal blooms. Throughout the warm months, the river is covered by an algae mat (Figure 1-4).

Figure 1-4. Assabet River Algae Mat (Source: Steve McGinnis, 1998.)

During the summer, the layer of vegetation on the Assabet River often becomes thick enough to significantly impede canoeing through impoundment areas (Roy, 1998.) The excessive algae growth in the river remains the direct result of the presence of the excessive nutrients required to support such growth, specifically the phosphorus and nitrogen inputs (Biswas, 1997.)

1.4 Assabet River Water Quality Implications for Acton Wastewater Treatment

Since the eutrophication of the Assabet River has been the result of WWTP phosphorus inputs, the United States Environmental Protection Agency (EPA) has placed many requirements upon the Town of Acton before approving an Acton WWTP discharge to the Assabet River. The EPA has requested that the Town of Acton design a WWTP that produces effluent with a target phosphorus concentration range of 0.1 milligram per liter (mg/L) to 0.2 mg/L. In addition, the EPA has requested that the Town of Acton evaluate possible phosphorus trading and urban best management practice (BMP) programs that could be utilized by the Town of Acton to offset their proposed WWTP phosphorus discharges to the impaired Assabet River.

1.5 Project Objectives

- Analyze phosphorus concentrations in the watershed runoff and assess the efficiency of selected BMP devices. The analysis will look at inflow runoff loads and concentrations from the entire watershed. The BMP analysis will include removal efficiency and device sizing where appropriate.
- Design a constructed wetland in the North Acton Recreational Area and evaluate it for its phosphorus removal capabilities. The wetland will receive runoff from the adjacent watershed, Quarry Road, one parking lot, and from the athletic fields. The evaluation will include a complete watershed hydrology analysis, phosphorus removal strategies in the wetland, overall design of the wetland, and landscaping requirements.

1.6 References

Biswas, A.K. 1997. Water Resources: Environmental Planning, Management, and Development. McGraw-Hill. New York, New York.

Cooperman, A.N., and W.R. Jobin. 1971. Report on Pollution of the Assabet River. Massachusetts Water Resources Commission Division of Water Pollution Control Water Quality Management Section: Boston, Massachusetts.

Hanley, N.E. 1989. "Assabet River 1989 Water Quality Management Plan: Parts A, B, and C." Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control.

Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control Technical Services Branch. 1981. The Sudbury-Assabet-Concord Basin Water Quality Management Plan.

Organization for the Assabet River (OAR). 1999. OAR Home Page. http://www.ultranet.com/~oar/.

Roy, L.A. 1998. Organization for the Assabet River Letter to Trudy Coxe and John P. DeVillars. February 9, 1998.

2 Watershed Based Pollution Trading Analysis

As a fairly recent and relatively untested water quality management method, little is currently known about watershed-based pollution trading. This watershed-based pollution trading analyis is intended to provide insight to the Town of Acton on the subject in the context of current environmental control strategies. The basic definitions, guidelines, and scenarios have been outlined in order to define the broad subject of watershed-based pollution trading. In order to aid the understanding of the development of watershed-based pollution trading, background information is provided about the history of economic pollution control mechanisms. In addition, this analysis also describes the events and political forces that have driven watershed-based pollution trading to the forefront of the United States Environmental Protection Agency (EPA) water quality management strategies. Finally, the analysis also includes detailed descriptions and recent updates on the existing watershed-based trading programs to inform the Town of Acton about the successes and failures of this new water quality management strategy.

2.1 Pollution Trading Overview

Although many variations of trading programs exist, the basic concept of pollution trading involves one pollutant source compensating another nearby source to decrease the discharge of a pollutant, whether present in air emissions or water effluent, rather than decrease its discharges of the pollutant to the same environmental medium. In either case, pollution trading utilizes economic incentives to more efficiently reduce overall pollutant discharges in a given area. As the alternative source (the seller) in a pollution trade can often reduce pollutant discharges more efficiently than the original source (the buyer), the compensation for pollutant discharge reduction is profitable above the incurred reduction costs, even at higher reduction ratios. Yet, the cost of compensation borne by the buyer is still less than the original cost of compliance. As such, pollution trading can provide a cost-effective pollution reduction alternative to expensive treatment upgrades for many sources (U.S. EPA Office of Water, 1996a).

Although air pollution trading was included in the 1990 Clean Air Act Amendments (CAAA) Acid Rain Program for sulfur dioxide (SO_2) emissions, watershed-based effluent trading has received significant attention from the United States Environmental Protection Agency (EPA) as an innovative water quality management policy only within the past three years. Spurred by the proclaimed but arguable success of SO_2 trading under the CAAA (Smith and Ellerman, 1998; Bohi and Burtraw, 1997), the EPA has heavily promoted effluent trading as a flexible and efficient approach to achieving overall water quality on a watershed basis. As such, the topic has provoked a flurry of interest and speculation, despite its limited use in water quality management at the state and local level.

2.1.1 Trading Arrangements

Pollutant trades involving discharges to water bodies are limited to the watershed level and can take many forms, including trading combinations between point sources and nonpoint sources (NPS). Trades can also vary widely in terms of the number of participating sources. At the simplest level, a point source may propose a discharge permit that incorporates intraplant trading among its outfalls for a certain pollutant to increase flexibility and reduce the overall cost of compliance for that pollutant. Bilateral trades can also occur and can include both point/point source and point/nonpoint source arrangements (Downing and Sessions, 1985). In addition to bilateral trades, pollutant trading can occur within an organized watershed program or created market among many members. In such a system, the pollution credit prices may be negotiated among the watershed sources, and the credits may also be placed in a "bank" that is created by the program if not needed for immediate use (Pelley, 1996).

2.1.2 Types of Pollutants

The type of pollutants traded can include conventional pollutants (biochemical oxygen demand (BOD), oils, greases), as well as nutrients (nitrogen and phosphorus) and metals (selenium and copper). As U.S. environmental regulations intend to eliminate the discharge of toxic pollutants into the environment, toxic pollutant trading is prohibited in the United States. A vast majority of the programs currently in existence or under development have traded nutrients, particularly phosphorus. Many others focus on the improving dissolved oxygen (DO) levels in water bodies by trading BOD loading among sources. Some unique programs have also focused on metals, such as selenium and copper, as well as reducing road-deicing chemicals to help meet sodium limits. Wetland mitigation has also been incorporated into a number of point/nonpoint source trading programs that attempt to reduce nutrient loading to receiving waters (U.S. EPA Office of Water, 1996a).

2.1.3 Trading Ratios

In order to ensure overall environmental protection and compensate for the difference in environmental impacts resulting from the trade pollutant source location changes, trading ratios may be applied to the pollutant reduction transaction. As defined by the EPA, trading ratios "reflect the relative environmental benefit of reducing a unit of pollution from one source compared to another (or, conversely, the relative harm of not reducing a unit compared to another)." In addition to addressing known environmental differences, trading ratios also provide a margin of safety for trades that include uncertain environmental impacts, such as those involving NPS pollution reductions that are difficult to quantify without extensive monitoring (U.S. EPA Office of Water, 1996a).

Also known as environmental bonuses or environmental premium requirements, trading ratios are essentially the equivalent of a discharge reduction tax borne by the original source when utilizing trades rather than additional upgrades to achieve compliance (Merrifield, 1998). Depending upon the site-specific factors, trading ratios may range from as little as 1.1:1 to as much as 3:1. Higher ratios often reflect both a margin of safety and an intended environmental benefit for an impaired water body. For example, of the reduction units required for a 3:1 trade, one unit may be intended to offset the buyer source increase, another to provide a margin of safety for uncertain reduction methods, and the final unit to provide an environmental benefit by reducing the net pollutant discharge (U.S. EPA Office of Water, 1996a).

2.2 History Of Trading Program Development

During the early 1970s, a number of laws were passed by the United States Congress to protect the environment, beginning with the Clean Air Act of 1970 (CAA) designed to reduce air pollution. Soon after, the Federal Water Pollution Control Act of 1972 (FWPCA) was passed to achieve the "restoration and maintenance of the chemical, physical, and biological integrity of the Nation's waters" (West Group, 1998). In 1977, amendments to the FWPCA were passed, which have been collectively become known as the Clean Water Act (CWA). Despite the fact that many water pollutants have both anthropogenic and natural sources, national environmental command-and-control regulations (CCR) established under the CWA largely focused on improving overall water quality by reducing anthropogenic point source loads to water bodies (Zander, 1991b). The CCRs required end-of-pipe limits for point source discharges that consisted of secondary treatment for municipal point source discharges and best practicable control technology for industrial point source discharges (Shanahan, 1999). The national technology-based standards contained within CCRs, although expensive to maintain and inefficient in comparison to total-watershed pollution control approaches, were much easier for state water quality agencies to administer (Downing and Sessions, 1985). Due to the fact that the CCRs left NPS pollution virtually unregulated, increasingly stringent CWA technology-based standards still failed to achieve water quality objectives in many water bodies nationwide. In these impaired watersheds, the states were given substantial water quality management flexibility to meet ambient water quality standards (Shanahan, 1999). As a result, many water quality management officials began to search for comprehensive watershed pollution control methods to reduce the overall pollutant loading in impaired watersheds for the least economic cost (Caldart, 1998).

As compliance costs rose and problematic watersheds remained in nonattainment during the 1980s despite the full implementation of water CCR standards, the regulations became increasingly unpopular among both environmentalists and dischargers. As a result economic alternatives to environmental CCRs, especially pure market-based theories, were studied and proposed as pollution control strategies. Many of these strategies relied on previous theories, such as Hardin's oft-cited, "Tragedy of the Commons" natural resource privatization proposal to end environmental neglect as public goods (Stewart and Krier, 1978), and Ruff's (1970) optimum pollution control attained through the "invisible hand" of market mechanisms. Similarly, others merged biology and economics theories to search for methods that would naturally serve the collective interest of environmental protection by invoking selfishness to convince humans to rationally consider the environmental consequences of their actions (Ridley and Low, 1993).

However, despite the substantial attention given to the notion of utilizing economic pollution control methods in lieu of environmental CCRs, many of the economic solutions remained largely theoretical, rendering them impractical and difficult to implement. However, studies of German water quality management systems in which water quality standards were attained through a system of charges reflecting the environmental impacts of the pollutant discharge showed promise for practical economic pollution controls. As such, when the similarly practical and innovative idea of small "artificial" or "created" pollution trading markets arose in the 1980s from these studies, the concept spawned considerable interest at the state and local levels (Downing and Sessions, 1985).

2.2.1 Initial State and Local Trading Case Studies

The economic incentives associated with pollution trading or credit systems prompted some states to study how to optimize pollution reduction among a group of sources through the use of trading mechanisms. A number of state and local studies estimated that trading programs could significantly reduce pollution control costs over conventional methods (David and David, 1983). The studies culminated in a few specialized trading programs designed to control water pollution to a few impaired and recreational water bodies. As can be seen below, the programs had varying levels of success, as some programs struggled due to poor design while others developed into case studies for national guidelines.

2.2.1.1 Fox River, Wisconsin

In 1981 the State of Wisconsin created a water pollution permit program that was the first trading program of its type in the United States. The program set up initial waste load allocations (WLAs) for 14 paper plants that discharged into the Fox River (Figure 2-1), and allowed them to trade pollution rights to meet these allocations through a transferable discharge permit (TDP) system (O'Neil, 1983). Initial studies indicated that abatement costs on the Fox River could be cut by at least half if the TDP system was implemented in place of conventional control methods (David and David, 1983). However, despite this potential, the trading program suffered from virtually nonexistent trading activity - only one trade was arranged throughout the first fifteen years of the program. The lone trade occurred when a paper mill shut down its own wastewater treatment plant (WWTP) and requested that the state shift its allocation to a local WWTP.

Figure 2-1. Fort Howard Paper Plant along the Fox River, Wisconsin
Source: Wisconsin Division of Natural Resources, 1999.

The lack of trading activity in the Fox River TDP system has been attributed to competition between the program companies, which are mostly paper mills. Studies have suggested that the paper mills have been saving their allocations for future growth rather than trading them with competitors. Further evaluation of the economic incentives for trading in the paper industry have suggested that the initial savings estimates were substantially flawed, and that little economic incentive exists for paper companies to trade pollution credits.

In addition to the competition issues, trading is also limited due to the rigid program structure. Trades in which compliance cost reduction is the sole objective are prohibited through the program's demonstration of need requirements for trade approval. Existing point sources are only allowed to enter trades during expansion or if unable to meet the discharge limits with the requisite technologies. New plants are allowed to trade upon completion, but must also show a demonstration of need beyond the required treatment technologies (U.S. EPA Office of Water, 1996a).

2.2.1.2 Lake Dillon, Colorado

Phosphorus control measures undertaken in the Lake Dillon Watershed in Colorado inhibit algal growth and retain the mesotrophic state of the reservoir for both domestic water supply and recreational purposes (Morris and Lewis, 1992). Lake Dillon (Figure 2-2) is one of many Colorado reservoirs, including the Cherry Creek Reservoir noted below, in which officials strive for the highest water quality standards for these reasons. Lake Dillon is a staging reservoir for the drinking water requirements of the inhabitants of Denver, Colorado, but is also surrounded by a number of recreational communities and ski resorts. As such, the communities in the Lake Dillon Watershed have considerable interest in protecting the water quality in Lake Dillon. In 1984, the Colorado Water Quality Control Commission (CWQCC) established a maximum lake phosphorus concentration to ensure the quality of water in the reservoir. Shortly thereafter, a local group comprised of county, town, and ski resort representatives dubbed the "Phosphorus Group" began brainstorming ideas for a total watershed control approach. They came up with the "Dillon Bubble," a plan to utilize pollution trading to allow for further development in the watershed without compromising water quality in Lake Dillon (Zander, 1991b). Realizing the

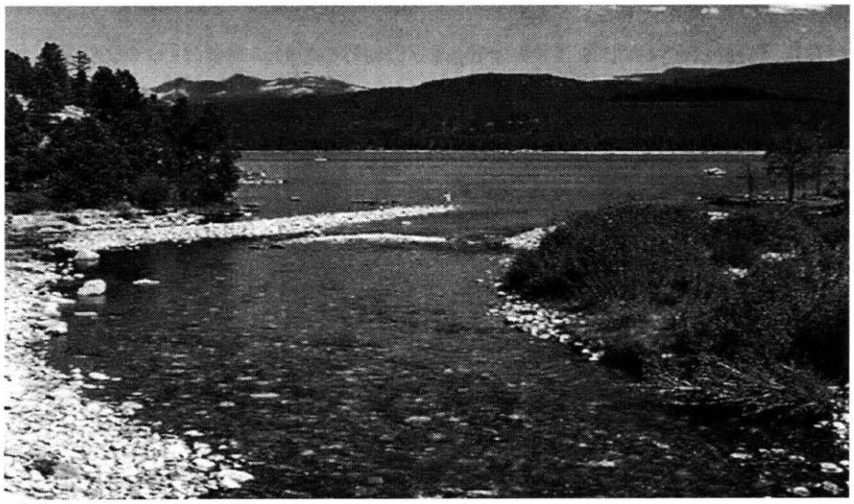


Figure 2-2. Lake Dillon
Source: USEPA, 1999.

communities' collective interest was ideal for cooperative watershed management programs, the CWQCC has been allowing point/nonpoint source phosphorus trading in the watershed since 1984.

However, similar to the Fox River trading program, few trades have occurred in the Lake Dillon trading program. Although the cooperative environment between municipalities in the Dillon Watershed is conducive to trading, point sources in the watershed have not needed trades to maintain water quality as they have already achieved some of the highest phosphorus removal capabilities in the nation (U.S. EPA Office of Water, 1996b). As a result, controlling the NPS loading to Lake Dillon from community and ski area runoff and failing community septic systems has become the primary focus of the trading program. For example, the Town of Frisco, Colorado plans to utilize stormwater controls to offset the phosphorus loading to Lake Dillon from a new golf course. Other communities and resorts are also searching for NPS load reductions to offset the increased phosphorus loading that would result from planned recreational additions and future resort developments (U.S. EPA Office of Water, 1996a).

2.2.1.3 Cherry Creek Reservoir, Colorado

Similar to Lake Dillon, Cherry Creek Reservoir is a popular recreational attraction that is visited annually by more than 1.5 million people. As a result, the Cherry Creek Basin Authority also established a total maximum daily load (TMDL) for the reservoir in 1984 to protect water quality for both drinking and aesthetic uses. However, it has been estimated that approximately 80% of the phosphorus loading in the watershed results from NPS pollution. As a result, shortly after the TMDL was established the 12 WWTPs that discharge to Cherry Creek were allowed to apply phosphorus trading credits to their discharge allowances from efforts to reduce NPS phosphorus loading in the watershed (U.S. EPA Office of Water, 1996c).

Despite the allowance of trading in the Cherry Creek Basin since 1985, formal guidelines for trading in the basin were not developed until shortly after the publication of the EPA's watershed-based trading guidelines in 1996. The guidelines allow point sources located within the Cherry Creek Basin to continue earning WLA credits by reducing NPS pollution. However, the guidelines require point sources to maintain phosphorus removal levels within designated effluent limits and demonstrate the need for an increased phosphorus discharge allowance. The Cherry Creek Basin Authority allows point sources to either contribute to authority-designated NPS control projects or devise their own NPS control project to earn discharge credits. The authority phosphorus loading reduction credits are placed in a bank to be purchased by point sources as increased discharge credits (U.S. EPA Office of Water, 1996c). In order to ensure reduction results from private NPS reduction projects, the project must be implemented and maintain NPS controls that reduce identified NPS loadings by half of the designated trade amount before any credits can be allocated to the source WLA (U.S. EPA Office of Water, 1996a).

2.2.2 Federal Watershed-Based Trading Emphasis

The watershed-based trading in the aforementioned state and local programs during the early 1980s did not go unnoticed by the EPA, which also began studying effluent trading as a method of watershed pollution control. However, the agency did not issue a formal pollution trading report for over 15 years (Pelley, 1996). A number of events in the early 1990s, including the reported success of sulfur dioxide trading under the CAAA, the Clinton administration's clean water campaign, and the EPA's move toward watershed-based water quality management, spurred widespread interest in the application of innovative trading programs to control water pollution sources. As a result, the EPA has recently made watershed-based trading one of the most encouraged approaches to watershed management.

2.2.2.1 Clean Air Act Sulfur Emission Trading

As the breakthrough that ended nearly a decade of legislative gridlock over policy solutions to the infamous acid rain problem, the Acid Rain Program under Title IV of the Clean Air Act Amendments of 1990 (CAAA) provoked considerable speculation (Ackerman and Moomaw, 1997). The unprecedented experiment in United States environmental regulation allowed electricity generation facilities to trade SO_2 emission allowances. A flurry of criticism about the expected outcome of the program ensued as economists projected poor program performance. Indeed, the program did not initially live up to its proponents' expectations, as stated by Zorpette (1994), "Almost all involved agree that the rate of trading among utilities has fallen short of aspirations." However, the industry's cost of compliance soon sunk to very low levels and SO_2 emissions dropped rapidly, leaving many proponents to proclaim the program a success.

Subsequent studies performed by Burtraw (1996) indicated that the Acid Rain Program actually required little trading to significantly reduce SO_2 emissions. Despite the proclaimed success at low trading volumes, the number of allowance transactions steadily rose over the next three years (See Figure 2-3). Although only 215 transactions involving 9.2 million allowances

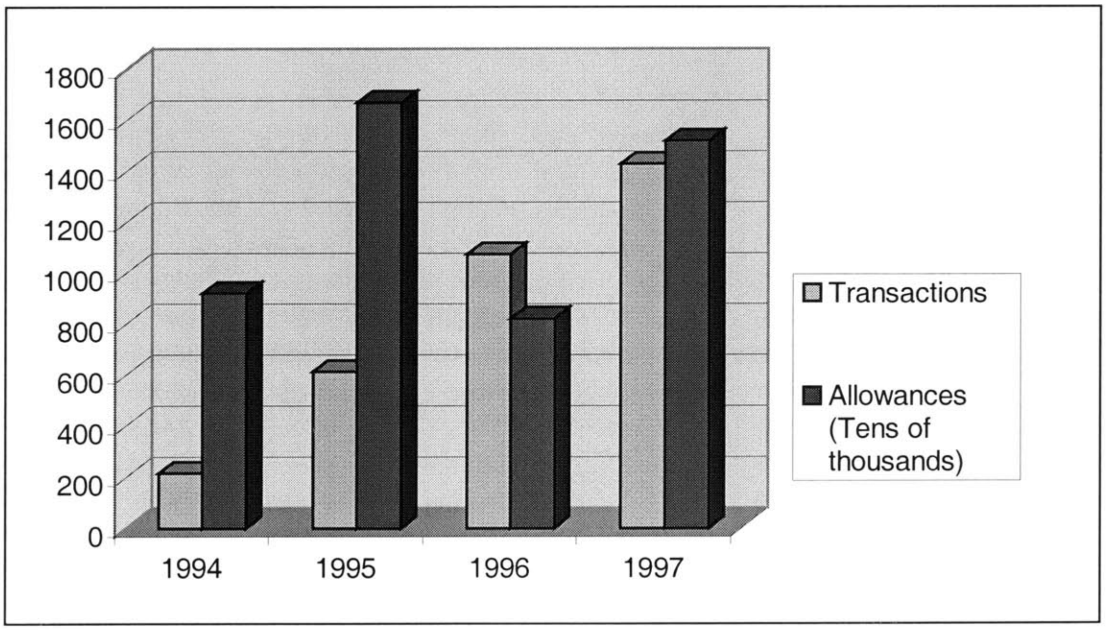


Figure 2-3. 1994-1997 SO_2 Emission Transactions and Allowances
Source: Adapted from U.S.EPA, 1999.

occurred between utilities during 1994, 1429 transactions for approximately 15.2 million transactions were recorded in 1997 (U.S. EPA, 1999). Initial results of the program indicated that in both 1995 and 1996, the flexibility and trading allowed under Title IV reduced overall SO_2 emissions by 4 million tons annually, which resulted in emissions well below the required limits set by the program (Ellerman, Joskow, and Schmalensee, 1998). As a result, the concept of watershed-based effluent trading began to gain widespread attention and became the focus of the Clinton Administration's emphasis on reinventing environmental regulation (Clinton and Gore, 1995).

2.2.2.2 *President Clinton's Clean Water Initiative*

As part of an effort to reinvent environmental regulation, the Clinton Administration issued President Clinton's Clean Water Initiative in early 1994. One of the primary concerns denoted in the initiative was the development of NPS pollution control programs by states to finally bring NPS pollution under regulatory control. The initiative first advocated the concept of a watershed-level water quality control approach that would soon become the basis for all EPA management programs in an attempt to bolster Clean Water Act Section 319 NPS pollution control programs. Citing NPS pollution as the primary contributor to water quality impairment in the United States, the initiative proposed the implementation of NPS pollution control programs at the state level. Specifically, the initiative called for the identification of impaired watersheds and the implementation of innovative and flexible local control programs comprised of voluntary and regulatory pollution controls adapted to the unique watershed conditions (U.S. EPA Office of Water, 1994). As such, President Clinton's Clean Water Initiative was one of many developments that created an incentive to utilize watershed-based pollution trading to reduce pollution loading from both point and nonpoint pollutant sources in impaired watersheds.

2.2.2.3 *Watershed Protection Approach to Water Quality Management*

As mentioned previously, despite the water quality management flexibility provided within the CWA, many states have traditionally applied a relatively uniform state-wide approach to water quality management in accord with federal statutes. However, many factors incited the EPA to promote water quality management at the watershed level in the mid-1990s. Deemed the Watershed Protection Approach (WPA), the policy was intended to provide improved water quality management by strongly encouraging unique practices that obtain overall water quality goals within each watershed (Hall and Howett, 1994). The shift to the WPA began with internal EPA comments about breaches in water quality management resulting from the failure of traditional state-level approaches to identify the impact of unique NPS pollution on impaired watersheds (Brady, 1996). Further, a primary objective of the Clinton administration in its reinvention of environmental regulation was ecosystem protection and management, which incorporates both the ecological and social characteristics of a unique geographic area into its management strategy (Davenport, et al., 1996). As a result, the water quality management lapses and the need for site-specific controls evolved into the WPA water quality management policy that set the stage for trading practices.

2.2.2.4 EPA Draft Framework for Watershed-Based Trading

Shortly after the Clinton Administration emphasized ecosystem protection and water pollutant trading as a priority of its reinvention of environmental regulation and the switch to WPA policies, the EPA issued the Draft Framework for Watershed Based Pollutant Trading in 1996. In addition to the Clinton Administration objectives, the development of the document was also spurred by the strong belief of EPA Administrator William K. Reilly in the potential of economic incentives for environmental control. In addition, a 1992 trading initiative meeting of over 120 representatives and stakeholders also strongly emphasized the need for EPA watershed-based effluent trading guidelines (U.S. EPA Office of Water, 1992). In order to fulfill this need, the Draft Trading Framework was intended to serve as a guideline for communities and companies interested in implementing watershed-based pollution trading programs.

A primary goal of the Draft Trading Framework was to provide EPA support to trading programs that many potential trading parties had avoided due to fears of noncompliance if their trading partners failed to reduce pollution per the trading agreement. The document allays these fears by encouraging and supporting the incorporation of trading arrangements into each party's National Pollution Discharge Elimination System (NPDES) permit for point/point source trades and acknowledging the need for flexibility and cooperation for point/nonpoint and nonpoint/nonpoint source trades (Pelley, 1996). As a draft document announcing the future promulgation of trading policies by the EPA, the Trading Framework was subjected to the agency regulation review process and amassed a number of public comments. The comments are currently being incorporated into the final document that is expected to be finished in late 1999 (Canning, 1999).

2.3 Recent Trading Programs

Many trading programs have been implemented as a result of the EPA's recent support of trading programs for watershed management. After the release of the Draft Trading Framework, the initial trading programs received a renewed surge of interest as case studies for trading programs in development and a second wave of effluent trading programs emerged. These include the Tar-Pamlico River in North Carolina, the Minnesota River in Southwestern Minnesota, the Passaic River in New Jersey, and Boulder Creek in Colorado (U.S. EPA Office of Water, 1996a). In addition, a number of nonpoint/nonpoint wetland trading programs have also recently been developed on a statewide basis under the Draft Trading Framework guidelines.

2.3.1 Tar-Pamlico, North Carolina

The Tar-Pamlico River Basin shown in Figure 2-4 on the following page is a large river and estuarine system in eastern North Carolina that was designated by the North Carolina Environmental Management Commission (EMC) as a nutrient-sensitive water in 1989 as the result of numerous nutrient loading studies. The studies recommended that an extensive nutrient control plan be implemented in the Tar-Pamlico River Basin that reduced both point and NPS loadings in the watershed. As a state with a federally approved coastal zone management program, North Carolina was also required to submit NPS pollution control programs to the EPA under the 1990 Coastal Zone Management Act. Having hosted the EPA Administrator's 1992

Figure 2-4. Tar Pamlico River
Source: Hall and Howett, 1994.

Point/Nonpoint Trading Initiative Meeting that led to the development of the EPA Watershed-Based Trading Framework, North Carolina environmental officials were well-informed of the issues involved with trading programs. As a result, while developing a basinwide NPS control plan for the Tar-Pamlico River Basin, the North Carolina Division of Environmental Management (NCDEM) chose an innovative watershed management trading program in cooperation with the Environmental Defense Fund and other interested parties.

The Tar-Pamlico trading program was designed to obtain overall water quality in the watershed by allowing point sources to cooperate with other point sources and NPSs to reduce nutrient loading in the watershed over a five-year period. The flexibility of the program allows point sources to choose the most cost-effective method to fulfill their individual pollution control responsibilities. They can either reduce their own discharges or the discharges of another plant, or contribute $56 per kilogram of required reduction to a fund that is used to implement best management practices (BMPs) in the basin's agricultural areas to reduce nutrient loading to streams (Hall and Howett, 1994). This fee was determined as the average cost of reduction required to achieve a 3:1 nutrient reduction ratio for cropland BMPs and a 2:1 ratio for animal BMP reductions in the Tar-Pamlico River Basin (U.S. EPA Office of Water, 1996a).

Implemented during the final years of the Trading Framework development, the Tar-Pamlico trading program was given widespread attention by the EPA as one of the largest point/nonpoint source pollution trading programs. Engineering studies of the program predicted that the initial required nutrient reductions of 425,000 kg/yr would be obtained easily through trades and operational changes. In addition, the cost savings of the program were estimated to be approximately $60 million over conventional point source control upgrades that may have forced some companies out of business (Hall and Howett, 1994).

2.3.2 Minnesota River

The Minnesota River is one of the most polluted rivers in the State of Minnesota and has historically suffered from low DO levels. However, agricultural land comprises over 90 percent of land drained by the river in the southwestern part of the state. As a result of the impaired conditions of the Minnesota River, expanding industries in the watershed were denied NPDES permits to discharge to the river in 1994. Rather than continue to impede industrial development in the Minnesota River Basin, the Minnesota Pollution Control Agency (MPCA) decided to implement a trading strategy to offset industrial expansion discharges to the river with agricultural NPS loading reductions. For every pound of BOD discharge allowed for expansion or new plant construction in a NPDES permit, the discharging company would have to agree to coordinate the implementation of upstream BMPs that would counter the new point source discharge (Wallace, Sparks, and Micheletti, 1997).

Rahr Malting, (Figure 2-5) touted as the largest and most advanced malting company in the world, was one of the companies that was denied a NPDES permit to increase its discharges to the Minnesota River. Realizing the rising cost of discharging to a local WWTP and seeking to expand its malting facilities, the company sought to construct a $7 million onsite treatment plant that would discharge the treated effluent to the river. However, noting that the treated malting process effluent would still contain oxygen-depleting compounds that would further deteriorate the impaired DO conditions in the Minnesota River, the MPCA refused to grant Rahr Malting a NPDES permit (Passi, 1998). Rather than spend its funds battling the MPCA in court, the company hired HDR Engineering, Inc., to craft a NPDES permit that incorporated innovative trading measures to reduce NPS oxygen depletion of the Minnesota River through upstream agricultural BMP implementation. Due to the large alternative costs of continuing to discharge to a WWTP, Rahr Malting anticipated a future decrease in its production costs despite paying for most of the BMPs with its own funds. After three years of development, the MPCA and the EPA both approved the Rahr Malting NPDES permit in early 1997 as the first NDPES permit to incorporate pollutant trading (Hersch, 1997).

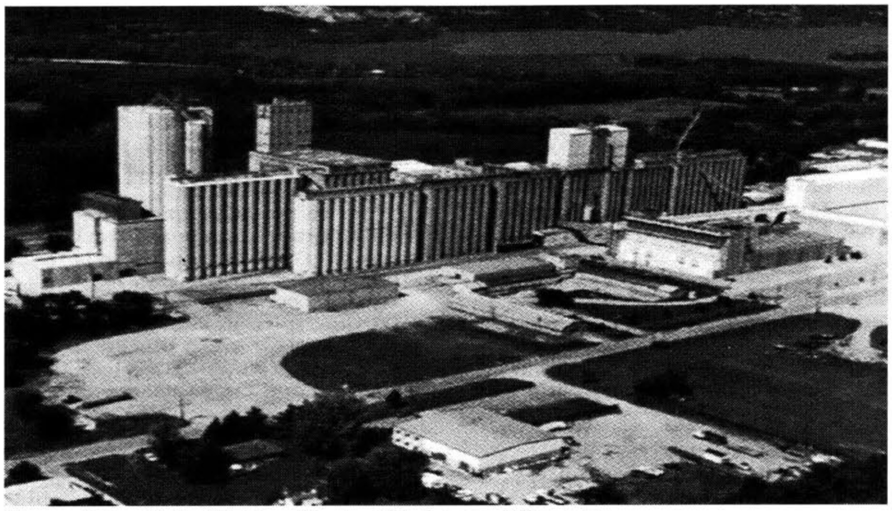


Figure 2-5. Rahr Malting Plant at Shakopee, Minnesota
Source: Wallace, Sparks, and Micheletti, 1997.

2.3.3 Passaic River Valley, New Jersey

The Passaic River Valley is a densely populated and heavily industrialized area that contains 47 municipalities and over 300 major industrial plants (Canning, 1999). Due to intense industrial development in most of the valley, the Passaic River (Figure 2-6) has experienced numerous water quality problems, particularly excessive metal concentrations (Wenning, Bonnevie, and Huntley, 1994). In 1996, a joint team consisting of the New Jersey Department of Environmental Protection (NJDEP), the EPA, the Passaic Valley Sewerage Commission (PVSC), and several Passaic Valley industries conducted a pilot program to test the development of a metals trading program in the Passaic River Valley. The pilot program focused on the use of trading to meet new metals limits for pretreatment of discharges to PVSC facilities. Certain restrictions were applied to the metals trades, including the requirement that metals reductions below PVSC limits must be achieved with end-of-pipe metal reduction technologies rather than internal process changes or discontinuance of a metal waste stream (Murphy, 1997). The initial trade attempted in the program resulted in a copper trading agreement between two industrial facilities (Canning, 1999). Once approved, the trade was incorporated in the companies' NPDES permits, and twenty percent of the total amount to be traded was placed in a credit bank, where it can no longer be discharged. As a result, both companies have achieved compliance at a lower cost than possible with conventional methods, and the overall copper discharge between them has been reduced – a positive outcome from trading innovations (Murphy, 1998).

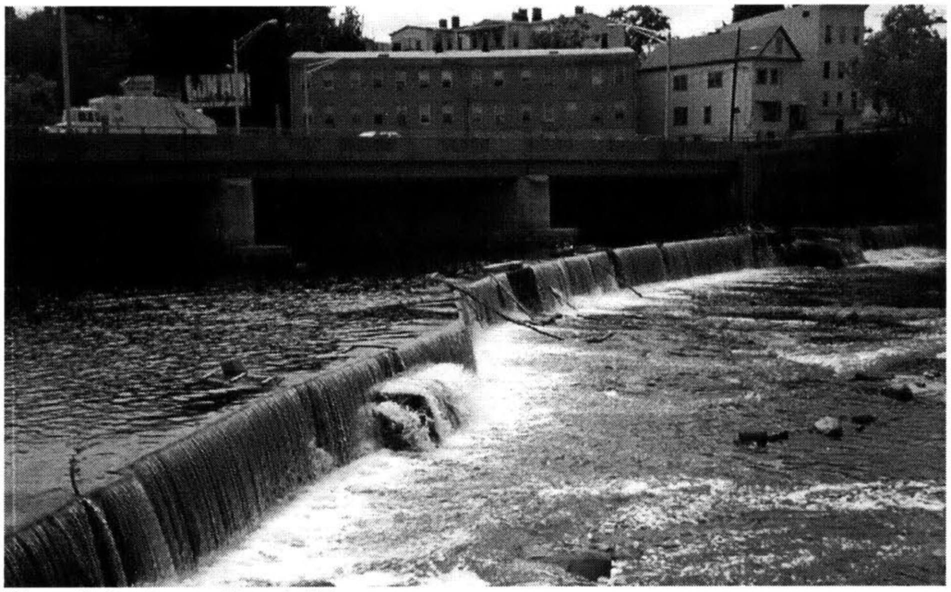


Figure 2-6. Lower Passaic River
Source: American Rivers, 1999

Thus far, the EPA has showcased the Passaic Valley experience as the most recent effluent trading success. In a recent report entitled, "Sharing the Load: Effluent Trading for Indirect Dischargers" the agency documented the struggles of the program (U.S. EPA Office of Policy, Planning, and Evaluation, 1998). The report was distributed by the EPA to over 600 recipients and placed on the internet for additional access and offers further insight to successful trading strategies to complement the Draft Framework for Watershed-Based Trading. The Passaic

Valley pilot trades are among a small group of experiences that the EPA plans to incorporate into the final revision of the document. Particularly noteworthy of the Passaic Valley experience is the demonstration of the efficient results of cooperation between pollution sources and regulating entities. As stated by one pilot trading participant, Fabricolor Inc., "The pilot has shown people that you can work with government and can accomplish things. It doesn't have to be adversarial all of the time." Through the cooperative experience in the Passaic Valley, EPA has further defined the requirements for successful trades between WWTPs and indirect dischargers, which will also be included in the final Trading Framework revision (Canning, 1999).

2.3.4 Boulder Creek, Colorado

Although a pristine and rolling mountain waterway at its headwaters, Boulder Creek (Figure 2-7) quickly becomes an urban stream upon passing through Boulder, Colorado. Downstream of the Boulder WWTP, Boulder Creek suffers from elevated un-ionized ammonia concentrations that inhibit aquatic life along the 15.5 mile reach below the city (Zander, 1991a). Despite upgrades that achieved WWTP effluent ammonia concentrations within effluent limits, Boulder Creek conditions did not improve due to numerous NPS sources along the impaired creek section (Zander, 1993). Rather than implement additional plant upgrades to achieve acceptable water quality standards in Boulder Creek, the City of Boulder decided to reduce NPS pollution with the implementation of BMPs. In particular, agricultural practices around Boulder were targeted, resulting in a 120-foot wide buffer between the creek and grazing land. Additional BMPs implemented along impaired sections of Boulder Creek include streambank and riparian restoration and irrigated treatment via wetland diversions. Substantial monitoring efforts have indicated that the $1.4 million BMP investment has improved water quality in Boulder Creek to higher levels than could have been achieved by the $3 to $7 million conventional WWTP upgrade alternative (U.S. EPA Office of Water, 1996d).

Figure 2-7. Boulder Creek Upstream of Boulder WWTP Discharge
Source: Corbis, 1999.

2.3.5 Wetland Trading Programs

Utilizing a broad definition of the term "navigable waters" in the CWA Section 404, the United States Army Corps of Engineers (USACE) administers dredging and filling permits (with EPA approval) for construction activities that destroy wetlands (Percival et al., 1992). Largely due to people's love for waterside living accommodations, nearly half a million acres of wetlands were lost annually to urban development until 1985, when the federal government began to enforce wetland mitigation and the rate of loss was reduced to slightly more than 100,000 acres per year. Despite this success, many officials have sought to achieve a no-net loss wetland goal (Cushman, 1997). In order to obtain a permit, most applicants must agree to create a nearby wetland of equal area to what they destroy during development (Percival et al., 1992).

In addition to the watershed-based trading programs listed above, a number of wetland trading programs (Table 2-1) have been developed on a statewide basis. Most of these programs allow those who have impacted wetlands to contribute funds to trust funds and nonprofit environmental organizations that undertake restoration efforts rather than attempt ad-hoc restoration efforts to achieve no-net loss of wetlands (U.S. EPA Office of Water, 1996a).

Program Participants	Type	Summary
Arkansas Nature Conservancy - USACE	nonpoint/ nonpoint	USACE permittees that destroy wetlands compensate the Nature Conservancy, which acquires and enhances additional wetlands.
Dade County, Florida	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Wetland Mitigation Trust Fund used for Everglades improvement projects.
Maryland Nontidal Wetland Compensation Fund	nonpoint/ nonpoint	Maryland DNR controls a wetland mitigation trust fund that is used for restoration projects.
Ohio Wetlands Foundation	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Ohio Wetlands Foundation that creates large wetlands.
Pine Flatwoods Wetlands Mitigation Trust, Louisiana	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Louisiana Nature Conservancy that administers restoration efforts.
Vicksburg District, USACE	nonpoint/ nonpoint	CWA Section 404 permittees that impact wetlands contribute to Ducks Unlimited, the Nature Conservancy, and other public agencies.

Table 2-1. Wetland Trading Programs
Source: Adapted from U.S.EPA, 1996a.

2.4 Trading Programs Currently Under Development

In addition to the programs that have already been implemented or expanded under the recent EPA guidelines, many other programs are currently being considered or in the initial stages of development at numerous locations across the United States. In particular, state and local officials developing trading programs to improve water quality in Long Island Sound and numerous water bodies throughout the State of Texas have invested considerable time and resources into the use of watershed-based trading for pollution control.

2.4.1 Long Island Sound

As the limiting nutrient for algae growth in saltwater systems, large water nitrogen concentrations induce the growth of dense algal blooms that are subsequently decomposed by bacteria in a process that consumes significant amounts of DO. As a result, excessive nitrogen loading from numerous Connecticut and Long Island communities has created short periods of hypoxia in Long Island Sound (LIS), a condition in which DO levels fall below the levels required to sustain aquatic life. Although short periods of hypoxia have existed since as early as the 1950s, studies since 1986 have identified an increase in the yearly duration of hypoxia that has significantly impaired the aquatic health of LIS. In recent years, hypoxia has occured in LIS from as early as late June to as late as mid-September (U.S. EPA Long Island Sound Office, 1997).

In order to reduce nitrogen loading to LIS from the surrounding communities, the Long Island Sound Study (LISS) hypoxia management program was first established in 1990, when initial status reports were prepared. Shortly after, Phase I of the LISS placed a freeze on growing nitrogen inputs to LIS, and Phase II was implemented in 1994 to begin reversing nitrogen inputs through conventional reduction methods. In 1997, the LIS watershed in both Connecticut (Figure 2-8) and New York was divided into eleven management zones. Proposals were

Figure 2-8. West Haven, Connecticut in LISS Management Zone Three
Source: Corbis, 1999.

developed for a comprehensive nitrogen management plan to reduce approximately 60 percent of the nitrogen loading in the watershed in order to improve DO concentrations in LIS to acceptable levels. Watershed-based effluent trading was among the water quality management proposals (Overton, 1998). Connecticut officials that control the bulk of the LIS watershed are convinced effluent trading will reduce nitrogen inputs to LIS and have estimated that the implementation of trading to achieve the reductions will save the state approximately $200 million over the use of conventional methods (Stacey, 1998). Although not opposed to an integrated watershed-trading program, New York officials have already implemented some variations of trading on a zone or "bubble" basis within the small portion of the LIS watershed under their control. Having experienced success from the bubble trading and leery of the increased administrative burden of a total watershed program involving the numerous Connecticut dischargers, New York officials have been hesitant to commit to an integrated trading program (O'Brien, 1998). However, many environmental officials see promise in the trading concept and a trading program is slowly developing (Overton, 1998).

2.4.2 Texas Water Resources Institute

As part of an effort to improve water quality in the State of Texas via the WPA prescribed by the EPA, Texas water quality managers have been developing TMDLs for Texas watersheds and evaluating a number of market approaches to reducing overall pollution on a watershed basis. They are interested in utilizing the point/nonpoint aspect of trades to reduce pollution loading in predominantly agricultural watersheds. At the most recent Texas water quality conference held by the Texas Water Resources Institute (TWRI) in December 1998, pollution trading and TMDLs were of primary interest to many participants.

The TWRI recently requested the assistance of three economists to evaluate the use of market-based strategies to limit water pollution in Texas (TWRI, 1998). All three economists reported positively on the use of pollution trading in Texas watersheds, citing cost-efficient pollution control (Merrifield, 1998; Emerson, 1998; Griffin, 1998). As a result, as TMDLs are established for Texas water bodies, the TWRI plans to develop trading programs within the TMDL framework to reduce overall watershed pollution (TWRI, 1998).

2.4.3 Additional Programs in Development

In addition to the aforementioned programs, a number of trading programs (Table 2-2 on the following page) were under consideration as the Draft Framework for Watershed-Based Trading was released, but have yet to formally develop established trading programs (U.S. EPA Office of Water, 1996a).

Program Participants	Type	Summary
Chatfield Basin, Colorado	point/ nonpoint	Evaluating TMDL, source targets, and potential for phosphorus trading.
Chehalis River Basin, Washington	point/ nonpoint, nonpoint/ nonpoint	TMDL has been developed and a trading study performed for Washington Department of Ecology has identified substantial benefits of trading.
Chesapeak Bay tributaries, Maryland	point/ nonpoint, nonpoint/ nonpoint	Pilot trading project has begun to evaluate the potential of trading to reduce nutrient loading to Chesapeake Bay.
Clear Creek, Colorado	point/ nonpoint	A trading program where sources adopt nonpoint sources, mostly abandoned mines, to achieve pollutant reductions.
Little Deep Fork, Oklahoma	nonpoint/ nonpoint	Agricultural and animal BMPs may be used to improve DO levels and reduce phosphorus loading in lieu of treatment plant upgrades.
Sacremento River, California	N/A	Discussions of metals trading and agrichemical input reductions have been ongoing.
San Joaquin River, California	point/ nonpoint, nonpoint/ nonpoint	The State of California, Environmental Defense Fund, and EPA have evaluated trading to reduce the cost of selenium discharge reductions.
South San Francisco Bay, California	point/ nonpoint,	Several WWTPs have evaluated trading to meet a 900 lb/year overall copper TMDL between them.
Tampa Bay, Florida	point/ nonpoint, nonpoint/ nonpoint	A trading program is being evaluated for nitrogen and total suspended solids from stormwater and the development of a stormwater fund for larger projects.
Boone Reservoir, Tennessee	point/ nonpoint	Studies have been performed to determine whether agricultural BMP implementation would be preferred over treatment plant upgrades.
Wicomico River, Maryland	point/ nonpoint	Preliminary studies have identified substantial cost savings of and water quality benefits in the Wicomico Basin.

Table 2-2. Trading Programs Under Development at the State and Local Level
Source: Adapted from U.S.EPA, 1996a.

2.5 Conclusion

The concept of pollution trading has been developing for many years, and was first utilized in a limited number of isolated trading programs in Wisconsin and Colorado watersheds. Pollution trading was first utilized on a national basis in the 1990 Clean Air Act Amendments Acid Rain Program to trade SO_2 emissions. The reported success of the Acid Rain Program, the Clinton administration's clean water campaign and reinvention of environmental regulation, and the EPA's move toward the Watershed Protection Approach to water quality management, spurred widespread interest in the application of innovative trading programs to control water pollution sources during the 1990s. As a result, the EPA has recently made watershed-based trading one the most encouraged approaches to watershed management, and has released the Draft Framework for Watershed-Based Trading to provide water pollution trading guidelines.

As innovative and flexible methods of maintaining water quality in unique watersheds, water pollution trading programs have been extremely distinct in terms of development, implementation, and outcome. The diversity of the existing trading programs exemplifies the flexibility that exists to conform trading programs to manage nearly any site-specific watershed pollution problem. Although most trading programs have been developed in impaired watersheds, they have been used to reduce pollution arising from a variety of problems, ranging from industrial metals contamination to agricultural NPS pollution. Trading programs have also been used to maintain the highest levels of water quality in valued recreational water bodies. Further, trading has been used to reduce overall pollutant loading directly to water bodies and indirect discharges to WWTPs.

As has been seen with the initial trading programs that have had significantly much more time to develop than the recent trading programs of the mid-1990s, trading can be inhibited by a number of factors. The Fox River and Lake Dillon trading programs are examples of two entirely opposite trading environments, one competitive and the other cooperative, that both experienced few trades due to entirely different reasons. However, recent trading programs such as the Passaic Valley experience have seen trading overcome the traditionally adversarial association between pollution sources and regulating entities to develop a cooperative relationship. Recent trading efforts have allowed Rahr Malting to expand its facilities at a lower cost than conventional methods and aided the otherwise unobtainable ecological improvements of Boulder Creek. Having been used as a case study to develop EPA guidelines from its inception, the trading program in the Tar-Pamlico watershed is also well underway. In addition, trading has provided environmental gains through cooperative wetland mitigation trading in the many programs listed in Table 2-1. However, a number of programs in Table 2-2 have been slow to develop or have stalled. Meanwhile, the cooperation required to develop a trading program in an interstate region such as the Long Island Sound watershed has slowed even the most enthusiastic water quality officials in Connecticut. Therefore, the brief history of existing trading programs remains inconclusive as an indicator of the likely success of the future of watershed-based pollution trading that is being encouraged through current EPA water quality policies.

The recent flurry of interest in watershed trading is not likely to diminish as long as EPA support is firmly behind the concept as an innovative water quality management policy. All of basic concepts of trading parallel the agency's Watershed Protection Approach and NPS reduction ideals of flexible watershed-level management programs that control both point and nonpoint sources. As a result, a more comprehensive study of watershed-based trading, particularly in the context of the similar CAAA SO_2 emissions trading, is warranted to determine whether the EPA's strong encouragement of trading is sound environmental policy or a misplaced hope for a more efficient means of pollution control.

2.6 Recommendations for the Town of Acton Trading Program

As outlined above, the use of watershed-based pollution trading is currently unproven in the existing trading programs. However, watershed-based pollution trading programs seem to have substantial potential to improve water quality in heavily impaired watersheds such as the Assabet River Basin. The four WWTPs that discharge to the Assabet River upstream of Acton provide ample opportunities for productive and efficient point source trades that utilize trading ratios to reduce the overall phosphorus load to the Assabet River. Further, the programs that have the most potential are those which emphasize a cooperative approach between all watershed dischargers. Therefore, if the Acton Trading Program were to expand to include other communities in the watershed, the Assabet River would experience more substantial water quality gains than can be achieved from phosphorus trades arranged solely between point and nonpoint sources in the Town of Acton. However, the limited use of watershed-based trading has shown that the success of the Acton Trading Program will ultimately hinge upon the motivation and cooperation between Assabet River Basin communities to achieve their end goal of improved water quality in the Assabet River.

2.7 References

Ackerman, F. And W. Moomaw. 1997. SO2 Emissions Trading: Doe it Work? The Electricity Journal. 10(7):61-66. August/September, 1997.

American Rivers. 1999. American Rivers Home Page. http://www.amrivers.org/

Bohi, D.R., and D. Burtraw. 1997. SO_2 Allowance Trading: How Do Expectations and Experience Measure Up? The Electricity Journal. 10(7):67-88.

Brady, D.J. 1996. The Watershed Protection Approach. Water Science and Technology. 33(4-5)17-21.

Burtraw, D. 1996. The SO_2 Emissions Trading Program: Cost Savings Without Allowance Trades. Contemporary Economic Policy. 14:79-94. April.

Caldart, C. 1998. 1.811J - Environmental Law: Pollution Control Class Notes. Massachusetts Institute of Technology. November 10-12, 1998.

Canning, K. 1999. Effluent Trading: Making it Work. Pollution Engineering. February, 1999. 21-24.

Clinton, B., and A. Gore. 1995. Reinventing Environmental Regulation. March 16, 1995.

Corbis. 1999. Internet Home Page. http://www.corbis.com March 5, 1999.

Cushman, J. 1997. Million Wetland Acres Lost in 1985-1995. New York Times. September 18, 1997.

David, M.H., and E.L. David. 1983. Cost-Effective Regulatory Options for Water Quality Limited Streams. Water Resources Bulletin. 19(3):421-428. June 1983.

Davenport, T.E., N.J. Phillips, B.A. Kirschner, and L.T. Kirschner. 1996. The Watershed Protection Approach: A Framework for Ecosystem Protection. Water Science and Technology. 33(4-5):23-26.

Downing, D., and S. Sessions. 1985. Innovative Water Quality-Based Permitting: A Policy Perspective. Journal WPCF. 57(5):358:365.

Ellerman, A.D., P.L. Joskow, and R. Schmalensee. 1998. 1996 Update on Compliance and Emissions Trading under the U.S. Acid Rain Program. Massachusetts Institute of Technology Center for Energy and Environmental Policy Research.

Emerson, P. 1998. EDF Believes Market Forces Can Protect Texas's Environment. Environmental Defense Fund Electronic Newsletter. http://www.edf.org

Fedesco, Mark. 1998. Message from the Director. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Griffin, R. 1998. The Case for Tradable Emission Permits. Texas Water Resources. 23(3):1-3.

Hall, J.C., and C.M. Howett. 1994. Trading in the Tar-Pamlico. Water Environment & Technology. p. 58-61. July 1994.

Hanley, N.E. 1989. "Assabet River 1989 Water Quality Management Plan." Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control.

Hersch, P. 1997. Minnesota Issues First-of-a-Kind Discharge Permit. Pollution Online Industry News. February 28, 1997.

Merrifield, J. 1998. Implementing Emission Allowance Markets. Texas Water Resources. 23(4):1-2.

Morris, D.P., and W.M. Lewis, Jr. 1992. Nutrient Limitation of Bacterioplankton Growth in Lake Dillon, Colorado. Limnology and Oceanography. 37(6):1179-1192.

Murphy, J. 1997. PVSC Kicks Off Effluent Trading Pilot Project. New Jersey Discharger. 5(1) Summer 1997.

Murphy, J. 1998. Update on Effluent Trading. New Jersey Discharger. 6(1). Summer 1998.

O'Brien, P. 1998. Nitrogen Trading – A New York Perspective. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

O'Neil, W.B. 1983. Transferable Discharge Permit Trading Under Varying Stream Conditions: A Simulation of Multiperiod Permit Market Performance on the Fox River, Wisconsin. Water Resources Research. 19(3):608-612. June 1983.

Overton, M.S. 1998. Watershed-Based Nitrogen Trading: The Cost-Effective Solution for Long Island Sound – An Enthusiast's Perspective. . Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Passi, P. 1998. Pollution Trading Program Takes Shape. American Rivers Organization. http://www.amrivers.org/mm/pollution198.html 1998.

Pelley, J. 1996. "Effluent Trading Framework Issued by EPA Tackles Nonpoint Sources." Environmental Science and Technology. 30:386.

Percival, R., A. Miller, C. Schroeder, and J. Leape. 1992. Environmental Regulation: Law, Science, Policy. "Wetlands Protection and the Section 404 Permit Program," p. 965-984.

Ridley, M., and B.S. Low. 1993. Can Selfishness Save the Environment? The Atlantic Monthly. September 1993. 76-86.

Ruff, L.E. 1970. The Economic Common Sense of Pollution. The Public Interest. 19:69-85.

Salata, J. Pollutant Trading. 1998. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

SEA Consultants. 1997. Fact Sheet on Water Pollutant Trading in Long Island Sound. June 23, 1997.

Shanahan, P. 1999. Personal Communication with the Author. April 7, 1999.

Smith, A.E., and A.D. Ellerman. 1998. The Costs of Reducing Utility SO2 Emissions – Not as Low as You Might Think. Massachusetts Institute of Technology Center for Energy and Environmental Policy Research. August 17. 1998.

Stacey, P. 1998. Nitrogen Trading – A Connecticut Perspective. Long Island Sound Study Update. Summer/Fall 1998. U.S. EPA Long Island Sound Office. Stamford, CT.

Stewart, R. And J. Krier. 1978. Environmental Law and Policy. 2nd edition. 68-75.

Texas Water Resources Institute. 1998. Using Market-Based Approaches to Protect the Environment. Texas Water Resources. 23(4):1.

Town of Acton. 1998. "Watershed Trading Program, Discussion Draft."

United States Environmental Protection Agency. 1999. Internet Home Page. http://www.epa.gov

United States Environmental Protection Agency. 1995. "Guidance Manual for Developing Best Management Practices (BMP)." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Long Island Sound Office. 1997. Long Island Sound Study. Stamford, Connecticut. EPA 840-R-97-001.

United States Environmental Protection Agency Office of Policy, Planning and Evaluation. 1998. Sharing the Load: Effluent Trading for Indirect Dischargers. Washington, D.C. EPA 231-R-98-003.

United States Environmental Protection Agency Office of Water. 1997. Technical Guidance Manual for Developing Total Maximum Daily Loads Book 2: Streams and Rivers, Part 1: Biochemical Oxygen Demand/Dissolved Oxygen and Nutrients/Eutrophication. Washington, D.C. EPA 823-B-97-002

United States Environmental Protection Agency Office of Water. 1996(a). "Draft Framework for Watershed-Based Trading." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1996(b). "Draft Trading Update – Cherry Creek Basin, Colorado." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1996(c). "Draft Trading Update – Lake Dillon, Colorado." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1996(d). "Draft Trading Update – Boulder Creek, Colorado." Cincinnati, OH: National Center for Environmental Publications and Information.

United States Environmental Protection Agency Office of Water. 1994. President Clinton's Clean Water Initiative. Washington, D.C. EPA 800-R-94-001.

United States Environmental Protection Agency Office of Water. 1992. Administrator's Point/Nonpoint Source Trading Initiative Meeting: A Summary. Washington, D.C. EPA 841-S-92-001.

Wallace, S., C. Sparks, and B. Micheletti. 1997. Nonpoint Source Trading Creates New Discharge Opportunities. Water Environment and Technology. 9(5):18-22.

Wenning, R.J, N.L. Bonnevie, and S.L. Huntley. 1994. Accumulation of Metals, Polychlorinated Biphenyls, and Polycyclic Aromatic Hydrocarbons in Sediments from the Lower Passaic River, New Jersey. Archives of Environmental Contamination and Toxicology. 27:64-81.

West Group. 1998. Selected Environmental Statutes: 1998-1999 Educational Edition. St. Paul, Minnesota.

Zander, B. 1993. TMDL Case Study: Boulder Creek, Colorado. United States Environmental Protection Agency Region VIII Water Division. Denver, Colorado. EPA 841-F-93-006.

Zander, B. 1991(a). Innovations at Boulder Creek. EPA Journal 17(5):50.

Zander, B. 1991(b). Nutrient Trading – in the Wings. EPA Journal 17(5):47-49.

Zorpette, G. 1994. "A Slow Start for Emissions Trading." IEEE Spectrum. 31(7): 49-52.

3 Non-point Source Pollution Analysis

For the Acton project, it was necessary to determine both the amount of phosphorus loading in the runoff and the ability of certain Best Management Practices (BMP) devices to remove that load. The calculation of runoff loads and concentrations would be used to develop the Trading program policy and determine what NPS phosphorus was available for trading with the point source of the WWTP. Once BMP devices were researched and suggested as possibilities for phosphorus removal, their feasibility within Acton needed to be estimated. It was decided that a non-point source (NPS) model would be developed to deal with all these issues.

3.1 Best Management Practice and Nonpoint Source Analysis

The study of past and current trading programs in the preceding trading policy analysis revealed that trading program characteristics are as diverse as the watersheds in which they originate. Similarly, the best management practices (BMPs) incorporated into existing nutrient trading programs also vary tremendously according to the unique nutrient sources within each watershed. Therefore, a subsequent literature analysis of BMPs suitable for phosphorus removal from urban runoff was performed to determine which BMPs would most effectively reduce phosphorus loading from the Town of Acton. The urban BMP literature analysis culminated in the recommendation of BMPs for the subsequent NPS modeling analysis.

3.2 Urban Best Management Practice Literature Analysis

Although the trading guidelines allow trading throughout the entire Assabet River Basin, calculations performed in the Town of Acton Trading Program Discussion Draft indicate that the desired 3:1 phosphorus loading reduction can be achieved solely through urban BMP implementation in Acton. As a result, the BMP analysis was restricted to urban BMPs suitable for implementation within Acton town limits.

3.2.1 Best Management Practice Definition

Best management practices include any type of method used to reduce the impact of nonpoint source (NPS) or diffuse pollution on an environmental medium. The BMPs include both structural controls that contain or remove pollution after it has been transported from the source and nonstructural controls that consist of ordinances and voluntary actions that reduce NPS pollution at the source or input to the watershed (Novotny and Olem, 1994).

3.2.2 Acton Runoff Phosphorus Removal Objectives

In the Town of Acton Trading Program Discussion Draft (1998), the phosphorus concentration in the Acton urban surface runoff was assumed to range from approximately 0.2 to 0.5 milligrams per liter (mg/L). This concentration range was based on results from the National Urban Runoff Project (NURP), a major urban runoff study coordinated by the EPA at 28 locations throughout the United States in the early 1980s. The NURP studies found an average urban runoff phosphorus concentration of 0.33 mg/L (U.S. EPA Office of Research and Development, 1993).

At the assumed concentration range with local precipitation values, phosphorus loading in urban runoff from the 5180 hectares within Acton averages approximately 2.9 to 7.2 kilograms of phosphorus per day (kg/day).

Acton officials have determined that, with a 3:1 ratio and a phosphorus concentration target of 0.1 mg/L in the proposed WWTP effluent, a phosphorus loading reduction of approximately 0.67 kg/day must be achieved through BMP implementation. However, if technological limitations produce effluent phosphorus concentrations that are actually closer to 0.2 mg/L, the BMPs must reduce 1.91 kg/day from Acton urban runoff. As a result, BMPs must remove approximately 10 to 66 percent of phosphorus from urban surface runoff within Acton, depending upon the removal efficiency of the WWTP (Town of Acton, 1998). In order to sustain adequate phosphorus removal levels from Acton runoff during seasonal runoff phosphorus concentration fluctuations, this BMP analysis was directed toward achieving overall phosphorus removal efficiencies similar to the median of the calculated range, or approximately 40 percent.

3.2.3 Financial Considerations

Due to the large cost of designing and constructing the proposed sewerage system and wastewater treatment plant in the Town of Acton, the minimization of the additional financial burden associated with BMP development and implementation was carefully considered in this analysis. Unfortunately, Novotny and Olem (1994) have emphasized that determination of urban BMP cost-effectiveness is extremely difficult due to the limited existence of accurate cost data. However, Novotny (1984) has also maintained that nonstructural BMPs are generally less expensive than structural BMPs due to the minimal design costs and lack of construction costs. In addition, the long history of self-regulation for environmental purposes in Acton (Halley, 1999) was considered when assessing the likelihood of voluntary compliance with inexpensive nonstructural BMP program objectives. As a result, considerable weight was given to nonstructural BMPs throughout this analysis, which is reflected in the final BMP modeling recommendations.

3.2.4 Urban Best Management Practice Alternatives for Phosphorus Removal

The primary focus of the EPA NURP studies was to develop a stronger understanding of urban runoff necessary for water quality management planning through the acquisition of comprehensive urban runoff data. As a result, many BMPs were researched and the surprising results spurred subsequent urban runoff studies (Bender and Rice, 1983). These studies have evaluated a number of potentially inexpensive urban phosphorus loading reduction practices, including lawn fertilizer application reduction measures, vegetative filter strips (VFS), street-sweeping practices, litter removal programs, and phosphate detergent bans.

3.2.4.1 Urban Lawn Fertilization Reductions

Research of urban pollution sources has identified a number of phosphorus sources common to urban communities of varying population densities. Of the sources identified in numerous studies, urban lawn fertilization has repeatedly been noted as one of the largest overall phosphorus sources in many residential communities (Dorney, 1986; City of Plymouth, 1999a;

Jeer et al., 1997). In order to keep lawns lush and green, homeowners either contract lawn service contractors to regularly apply fertilizer or apply it themselves, often at excessive rates to ensure a thriving lawn. Studies have estimated that fertilizer amendments are applied to approximately 50 to 80 percent of all urban lawns (Jeer et al., 1997), and that, on average, lawn phosphorus application rates exceed application rates to agricultural fields by 10:1 (Metropolitan Council, 1999). A 1997 Gallup poll indicated lawn service contractors apply approximately 22 percent of fertilizer amendments to urban lawns, suggesting that the majority of applications are performed by homeowners (PLCAA, 1999). Regardless of how fertilizer is applied to home lawns, the majority of lawn nutrient applications occur in the form of chemical fertilizers that are easily dissolved and transported in urban runoff. Research performed as part of the NURP recorded average soluble phosphorus concentrations in urban runoff of 0.12 milligrams per liter - over one third of the total runoff phosphorus concentration (U.S. EPA Office of Research and Development, 1993).

As urban lawn fertilization is a primary phosphorus source, voluntary and regulatory measures to reduce lawn fertilizer application can be relatively inexpensive strategies to significantly reduce phosphorus loading to water bodies from low-to-medium density urban areas. For example, the City of Amery, Wisconsin implemented a lawn fertilization ordinance that placed a ban on phosphates in lawn fertilizer applications in 1991 in order to protect nearby recreational lakes. Two years later, the city was forced to change the ordinance to allow application of fertilizers containing one-percent phosphate concentrations as a result of limited supplies of phosphorus-free fertilizer. However, many companies have recently begun marketing environmentally friendly fertilizers that contain little or no phosphorus. As a result of research and educational efforts at the University of Wisconsin at Madison, increased interest in phosphate-free lawn fertilizer spurred Cenex/Land O'Lakes Inc. to manufacture phosphorus-free (27-0-7) ClearLake Fertilizer (Lamker, 1998).

Similarly, the nearby City of Plymouth, Minnesota, has also implemented lawn fertilization phosphorus control measures that include both educational efforts and lawn fertilization ordinances in an attempt to reduce urban runoff phosphorus loading to nearby Parker Lake. The educational efforts attempt to inform homeowners of the environmental impacts of excessive fertilizer applications and provide instructions on how to efficiently time fertilizer applications. In addition, most of the fertilizer regulations were promulgated to promote careful application and handling of fertilizer by banning careless applications near waterways and wetlands and onto hard surfaces. However, the most restrictive lawn application regulation requires commercial fertilizer applicators to apply only phosphorus-free fertilizers on Plymouth lawns. In addition, Plymouth officials have encouraged local merchants to carry and advertise phosphorus-free fertilizers. They estimate that the combination of these practices reduces phosphorus loading from Plymouth urban runoff by 90 kilograms per year, or an average of 0.25 kilograms per day (City of Plymouth, 1999b).

3.2.4.2 Vegetative Filter Strips

Vegetative filter strips (VFS), or grass buffer strips, are flat, vegetated areas with little slope that induce slow sheet flow of storm water through the vegetation to remove pollutants (Barrett et al., 1998). The VFSs reduce pollutant concentrations in surface waters through a number of processes, including filtration, deposition, infiltration, absorption, volatilization, vegetative consumption, and decomposition. Of these pollutant removal mechanisms, infiltration is considered to be the most important, but requires large land areas to effectively remove pollutants (Keaton et al., 1998). Although settling effectively removes particulate nutrients, nutrients in urban runoff are often difficult to remove entirely via settling measures due to significant dissolved fraction of nutrients (Novotny and Olem, 1994).

Although some VFS have obtained high phosphorus removal efficiencies, many VFS studies have reported a wide variety of removal values (Woodard and Rock, 1995), some of which have ranged from 13 to 98 percent (Keaton et al., 1998). The variation in phosphorus removal efficiency can be attributed to the many site-specific factors that govern VFS phosphorus removal. Modeling studies of phosphorus behavior in VFS have indicated that the VFS size, vegetation density, and soil infiltration characteristics significantly influence phosphorus removal. Dissolved phosphorus behavior is also particularly sensitive to ground biomass buildup within the VFS. In some situations, the buildup of biomass within a VFS can actually contribute phosphorus to the runoff rather than remove it (Lee, Dillaha, and Sherrard, 1989). In addition, VFS efficiencies have been shown to decrease dramatically during overflow conditions, when the vegetation is often flattened and provides little flow impedance and pollutant removal (Novotny, 1984).

In addition to the influence of site-specific construction factors, VFS phosphorus removal efficiencies also vary due to the temporal fluctuation of the phosphorus phase present in urban runoff. A direct relationship between VFS influent phosphorus concentration and removal efficiency has been indicated in studies performed by Woodard and Rock (1995). Urban runoff studies have indicated that the amount of filterable phosphorus varies throughout the year from 13 to 97 percent (Waller and Hart, 1985). As a result, VFS phosphorus removal from urban runoff also fluctuates seasonally.

The uncertainty of site-specific VFS performance has incited the EPA to stress the limited accuracy of VFS performance predictions. Furthermore, the EPA also considers VFS to be inadequate in many situations as the sole runoff control practice (U.S. EPA Region V Water Division, 1990). Although some recent publications have recommended that regulatory agencies accept VFS as more than a pretreatment mechanism for additional structural controls, they have mainly emphasized the efficiency of VFS total suspended solids (TSS) removal rather than phosphorus removal (Barrett et al., 1998).

3.2.4.3 Street Sweeping Practices

Although street sweeping was once considered one of the most promising urban NPS control practices, studies performed on the practice as a part of the NURP produced very disappointing results. Extensive research projects undertaken at Champaign, Illinois (Bender and Rice, 1983), and Milwaukee, Wisconsin (Novotny et al., 1985) found frequent street-sweeping to be largely ineffective in improving urban runoff water quality. The results of the study were substantiated by further studies of street sweeper performance that found both the standard gutter-broom mechanical street sweeper and the vacuum-assisted gutter-broom sweeper to be only about 50 percent effective in removing particles from street gutters. In addition, in situations where very frequent (approximately every few days) street sweeping was studied, the practice actually had a negative effect on particle removal, as smaller particles are virtually unaffected by sweepers and larger particles are often merely redistributed by the sweeping process.

In addition to street sweeper inefficiency, it is estimated that more particles are blown from streets onto adjacent impervious areas such as sidewalks and lots than remain in the gutter for sweeper removal. As a result, even the most thorough street sweeping program is relatively ineffective in reducing nutrient loading from a water quality standpoint (Novotny and Olem, 1994). This is particularly true in terms of phosphorus removal, as studies have shown that only 10 percent of total urban phosphorus loading potential is removed by very frequent sweeping practices (Novotny et al., 1985). This may be due to the fact that phosphorus adsorbs much more strongly to the smaller clay particles that are not collected by street sweepers than the larger sand particles that are removed with sweeping (Pierzynski, Sims, and Vance, 1994).

3.2.4.4 Litter Removal

Although street sweeping on a regular basis may not reduce urban runoff phosphorus concentrations, seasonal vegetative litter removal may be an effective BMP to reduce overall phosphorus loading in runoff from low-to-medium density urban communities. Studies of the effects of leaves on urban runoff have found that residential areas are typically covered by 30 to 40 percent tree canopy that deposits large quantities of leaves throughout the fall (Dorney, 1986). The decomposition and transport of these leaves with urban runoff into receiving waters comprises a large nutrient source during leaf fallout (Novotny and Olem, 1994). The NURP studies performed in Milwaukee proved that street refuse or vegetative litter is one of the primary sources of urban NPS nutrient pollution, and that leaf fallout dramatically increases nutrient pollution loading in urban areas. Additional studies have confirmed that in urban areas with abundant trees, runoff phosphorus concentrations fluctuate tremendously due to influence of vegetative phosphorus sources during the summer and fall seasons (Waller and Hart, 1985). In these communities, leaf vegetative matter has been shown to contribute over half of the phosphorus loading, with urban lawn fertilization as the second closest contributor with just over one quarter of the total phosphorus loading (Dorney, 1986).

A mature tree can produce up to 25 kilograms of nutrient-enriched organic matter (Novotny et al., 1985). Leaf constituent studies have reported that approximately 1.6 to 11 milligrams of phosphorus is present in each gram of leaf organic material, depending upon the species (Waller and Hart, 1985). Urban leaf studies have also denoted that urban leaves compare similarly to leaves found in natural environments, with a range of 0.08 to 0.44 percent phosphorus found in a number of tree species (Dorney, 1986). In addition, studies have shown that small quantities of the phosphorus present in tree leaves can be easily leached in urban runoff in dissolved forms that are difficult to remove with VFS or other structural BMPs (Dorney, 1986; Waller and Hart, 1985).

As a result, the removal of leaf litter can be an effective urban BMP to reduce nutrient loading to watersheds (Novotny, 1984). In contrast to street sweeping to remove fine particles, studies have shown that litter removal and control strategies can be effective in removing approximately 50 to 75 percent of urban leaf litter and debris (Syrek, 1981). Site-studies of Hastings, Wisconsin for the NURP found that leaf litter contributed approximately 30 percent of daily phosphorus inputs in that medium-density urban area (Novotny et al., 1985). As a result, an effective litter removal program can be assumed to reduce an estimated 15 to 23 percent of the phosphorus concentration in runoff from medium-density urban areas.

3.2.4.5 Phosphate Detergent Bans

Phosphates are added to detergents to improve cleaning strength through increased dirt suspension, grease and oil emulsification, and mineral inactivation. Since the 1960s, the subject of phosphates in detergents has received considerable attention, prompting many companies to begin voluntarily reducing phosphate concentrations in numerous detergent products in the 1970s. Although no ban is currently in place in Massachusetts, statewide phosphate detergent bans have been promulgated in 11 states and phosphate limits set in three others. In addition, some counties and communities in other states have banned the use of phosphate detergents. Due to these actions, many phosphate-free detergents currently exist, including all liquid laundry detergents and most shampoos; however, no suitable replacement for phosphates has been found for automatic dishwasher detergents. Despite these reductions, phosphate detergent use is still estimated to contribute approximately 25 to 30 percent of phosphorus in household wastewater (Porter, 1991).

Although many phosphate detergent ban proponents were quick to claim water quality improvements the sole result of bans, a controversy developed as to the actual contribution of the bans to water quality as many other NPS programs were concurrently being implemented to improve water quality in many states. For many years, ban impacts incited a number of critical water quality editorial exchanges between both pro- and anti-ban researchers (Lee and Jones, 1996). However, the root of this controversy was not whether phosphorus reductions occur from bans, but whether bans produce improved water quality for the effort of reduction. Studies performed by Hoffman and Bishop (1993) determined that the phosphate detergent ban implemented by the Commonwealth of Virginia in 1988 reduced phosphorus loading to municipal WWTPs from 580,000 pounds per year (lbs/yr) to 210,000 pounds per year in the four years following the ban.

Methods developed by Lee and Jones (1996) in a collaborative study of 400 water bodies concluded that significant contributions to water quality from phosphate detergent bans only occur in water bodies in which the detergent-laden wastewater represents a significant fraction of overall phosphorus loading. Although large water bodies such as lakes are not likely to show dramatic improvements in water quality from a phosphate detergent ban, rivers that receive substantial discharges may benefit from such actions.

3.2.5 Best Management Practice Literature Analysis Conclusion

Lawn fertilization reduction measures can be relatively inexpensive strategies to significantly reduce phosphorus loading to water bodies from urban areas. Although an outright ban on the use of phosphorus fertilizers in Acton would be against Massachusetts State Laws, an educational program relying on voluntary compliance to reduce phosphorus use in Acton would likely produce significant reductions in phosphorus loading from lawn fertilizer applications. According to the nationwide lawn fertilization data previously mentioned, approximately 30 to 60 percent of homeowners personally apply fertilizer to their lawns. As such, environmentally conscious homeowners may voluntarily reduce phosphorus applications once informed of the environmental consequences of their actions. In addition, despite being an initial problem in the previous community phosphorus reduction programs, phosphorus-free fertilizer has become widely available in recent years. Many Acton lawn services have the capability to apply fertilizer with as little as 2 to 3 percent phosphorus concentrations (Grass is Always Greener, 1999; Trugreen-Chemlawn, 1999). In addition, at least one local lawn service, Lawn Doctor of Nashoba Valley (1999) confirmed that it has access to and is willing to apply environmentally friendly fertilizer applications that contain no phosphorus. Therefore, an educational effort to reduce home lawn phosphorus applications has the potential to significantly reduce phosphorus loading in Acton runoff due to traditionally high rates of voluntary cooperation with environmental programs by Acton residents.

Due to the uncertainty associated with VFS performance, the actual results of VFS installation in Acton can only be ascertained through extensive site testing. However, watershed NPS modeling for conditions within the Town of Acton should provide further insight to expected VFS performance. If the NPS modeling analysis predicts that Acton conditions are conducive to very efficient VFS removal, the Town of Acton should consider implementation of VFS throughout the town limits. Conversely, if the analysis proves that VFS phosphorus removal in the Town of Acton is minimal, the expense associated with the structural design and construction requirements to ensure proper sheet flow and velocities may outweigh the benefits of VFS installation in Acton.

Although recent improvements have increased street sweeper efficiency, sweeping remains an aesthetic practice that provides little urban runoff water quality improvement due to inefficient particle removal. In addition, the street sweeping frequency required to produce a beneficial impact on water quality in Acton would be very expensive due to labor, fuel, and sweeper maintenance expenses if performed by Acton employees or cumulative service charges if obtained from a contractor.

Despite the labor-intensive nature of an effective litter removal program, the requirements of such a program are mainly seasonal. Several rounds of litter removal during leaf fallout in autumn may be necessary to properly reduce litter contribution to urban runoff. However, leaf collection and removal from lawns is a common practice among many homeowners. A comprehensive leaf removal program in Acton could include free leaf bag removal on announced weekly dates to encourage leaf removal from homes on a regular basis during the leaf fallout period. In addition, the Town of Acton already possesses the equipment and litter storage facilities necessary for a seasonal leaf removal program and is capable of producing the funds required for an effective program.

Phosphates have been shown to be a significant source of phosphorus in household wastewater. However, if properly maintained, the use of septic systems throughout most of Acton reduces the impact of phosphate detergents on surface water quality in those regions as phosphates bind tightly to soil particles and are typically not available for surface water removal (Doenges et al., 1990). Yet, the use of phosphate detergents in the South Acton and Kelley's Corner homes served by the proposed sewerage system could potentially impact the WWTP effluent phosphorus concentration by increasing the influent phosphorus concentration. If so, the 3:1 trading ratio would subsequently require a threefold increase in BMP phosphorus reductions from Acton runoff. As phosphate detergent bans have been shown to reduce phosphorus loading to WWTPs, the implementation of measures that reduce phosphate detergent use in the sewered homes warrants consideration by Acton officials. In addition, the relationship developed by Lee and Jones strongly suggests that the reduction of phosphate detergent use would also improve water quality in rivers such as the Assabet, which in the summer months derives nearly all of its flow from WWTP discharges. Further, a primary purpose of this project is to demonstrate BMPs that will improve water quality in the Assabet River. Therefore, the successful reduction of phosphate detergent use in the sewered areas of Acton may provide the basis for future measures in the homes served by the five WWTPs that currently discharge to the Assabet River.

3.2.6 Best Management Practice Literature Recommendations for NPS Modeling

As a result of the various inefficiencies, uncertainty, and financial requirements of the structural BMPs, the most effective measures for phosphorus removal from urban runoff are nonstructural BMPs that focus on attaining source reduction through various ordinances and voluntary programs. In order to achieve the required phosphorus reduction for the Town of Acton Trading Program, a combination of these BMPs will be necessary to reduce the Town's urban phosphorus NPS to acceptable levels. Such integrated programs have long been advocated by the EPA, as reliance on a single BMP or structure to treat urban runoff can produce inadequate results (Field, 1985).

The literature analysis of urban BMPs indicates that the use of voluntary lawn fertilization and detergent use reduction measures to complement an extensive fall litter removal program comprise the most efficient and economic strategy for urban phosphorus removal in the Town of Acton. Although ordinances that ban phosphorus in lawn fertilizers and detergents will conflict with state laws and receive resistance from Acton residents, voluntary programs are likely to be received better by both state officials and Acton homeowners. As has been noted throughout this analysis, Acton residents have established a history of voluntary self-regulation under environmental programs. As such, an integrated urban runoff management program that incorporates aspects of voluntary phosphorus reductions in lawn fertilizer applications with cooperative leaf and litter removal measures is recommended to reduce urban runoff phosphorus concentrations. Further, measures should be taken to reduce the use of phosphate detergents in the sewered homes through voluntary measures or economic incentives, such as sewer hookup fee reductions.

In essence, the phosphate detergent reduction measures in the sewered homes will help lower the WWTP effluent phosphorus concentration, in effect "lowering the bar" for remaining trading program reduction requirements. The lawn fertilization measures will strive to reduce dissolved phosphorus sources throughout Acton, while the leaf litter removal program will work to reduce the amount of particulate phosphorus in Acton by removing vegetative matter and debris. As a result, the integrated program should achieve the effluent and urban runoff trading program phosphorus goals.

3.2.6.1 Recommendation Performance Calculations

Assuming the implementation of a phosphate detergent reduction program for the sewered homes in Acton will help achieve target WWTP effluent phosphorus concentrations of 0.1 mg/L, the trading program urban runoff phosphorus loading reduction requirements will be reduced significantly. As a result, the voluntary lawn phosphorus fertilizer reduction and leaf litter removal programs must achieve a combined phosphorus reduction of approximately 0.65 kg/day from Acton runoff rather than 1.91 kg/day.

By extrapolating data found in the Plymouth, Minnesota, study to Acton conditions with an assumption of 50 percent efficiency, rough estimates of expected phosphorus reductions from Acton urban runoff can be determined for the lawn phosphorus fertilization reduction program. From this data, voluntary compliance with lawn fertilizer reduction measures within Acton could reduce phosphorus from the town's urban runoff by an estimated 0.1 kg/day to 0.15 kg/day kilograms per day. Assuming the 15 to 23 percent phosphorus concentration reduction from Acton urban runoff that was calculated above, the annual phosphorus loading removal in the Town of Acton from litter removal could average 0.4 kg/day to 1.6 kg/day. Therefore, assuming minimum effectiveness of both BMPs, the integrated program would come close to removing the required 0.65 kg/day necessary to meet the minimal trading reduction requirements. If the average values for phosphorus removal apply for Acton, the integrated program would remove approximately 1.1 kg/day of phosphorus per day from Acton runoff.

3.3 NPS Model selection

A program was needed that would be able to handle both the calculation of phosphorus loading in surface runoff and the analysis of BMP devices. Originally 2 programs were being looked at for use: BASINS 2.0, maintained by the EPA, and P8, created by Mr. William Walker for the Narragansett Bay Project.

Basins 2.0 utilizes MassGIS data to model entire watersheds at a time. The Windows based program is powered by ArcView and can produce useful graphics of watershed land-use data and water quality reports. However, with BASINS, many areas need to be generalized. For example, a general value needs to be assigned for the impervious fraction of all residential areas within the watershed. BASINS also simply models the water quality. It does not allow for the design or implementation of BMP plans. BASINS 2.0 is a relatively new modeling program and has few reviews or recommendations.

P8 does not have the graphics capability that BASINS does. It is a simple DOS program that utilizes the algorithms from other tested urban runoff models including SWMM, HSPF, D3RM and TR-20. However, P8 does require a minimal amount of input to produce runoff calculations and loading estimates. P8 also, has the ability to calculate device efficiencies and size devices given a set of removal specifications. P8 requires less computer space, is significantly more user-friendly, and produces reliable results. Since its original release, several versions have been created to correct any 'bugs' and add new features.

The reliability and capabilities of P8 have made it the model of choice. P8 will be utilized to assess watershed runoff and local removal capabilities. BASINS 2.0 was used to produce land-use data and water-quality reports.

3.4 NPS Model: P8

P8 stands for Program for Predicting Polluting Particles Passage thru Pits, Puddles and Ponds. The program was designed by Mr. William Walker for IEP, Inc., USEPA/Rhode Island DEM/Narragansett Bay Project, Wisconsin Department of Natural Resources, Minnesota Pollution Control Agency, and CH2Hill, Inc. Several different existing watershed simulation models were evaluated and adapted for the creation of this program including HSPF, Stormwater Management Model (SWMM), PRS, UTM-TOX, and SWAM. P8 was developed with the intentions of being used by local planners and engineers involved in the evaluation and design of local urban BMP devices. (Walker, 1989)

For this project, P8 Version 2.3 (January 1999) was used. P8 provides continuous mass balance and water balance calculation for a system of "watersheds", removal devices, particle classes, and water quality components.

3.4.1 Basic Model Program Details

Normally, a watershed is considered to be a large area with various types of land uses, soil qualities, and pervious and impervious fractions that has all its surface water running off into streams which eventually form one river leaving the watershed. The P8 model is used to help analyze a specific device or sets of devices to be used in a specific area such as a mall parking lot area or a residential area. Within P8, the term "watershed" refers to any size area with the surface runoff draining out to one point. "Watersheds" within P8 can be defined with only one set of land use criteria and soil quality.

3.4.1.1 Watershed Runoff Volumes

Pervious areas runoff volumes are computed using the SCS curve number technique (USDA, 1964). The SCS curve number technique is used in the calculation of continuous watershed simulations for only pervious watershed fractions (Haith and Shoemaker, 1987). The model assumes that runoff from impervious areas only start after the cumulative storm precipitation exceeds the specified depression storage. There is no lag time for the runoff to reach any specified device.

3.4.1.2 Watershed Loads

Particle concentrations for pervious area runoff are calculated using a method similar to the sediment rating model from SWMM. (Huber and Dikinson, 1988):

$$C_p = C_{po}\, I^f$$

Where,
C_p = Particle Concentration in pervious runoff (ppm)
C_{po} = Concentration at a runoff intensity of 1 inch/hr (ppm)
I = runoff intensity from pervious area (in/hr)
f = exponent (~1, Huber and Dikinson, 1988)

Particle concentrations for impervious area runoff are calculated from a combination of particle accumulation and washoff and fixed runoff concentration. Particle accumulation and washoff is similar to the exponential washoff relationship utilized by the SWMM (Huber and Dikison, 1988) described as:

$$\frac{dB}{dt} = L - kb - fsB - ar^{c}B$$

Where,
B = buildup or accumulation on impervious surface (lbs/acres)
L = rate of deposition (lbs/acre-hr)
k = rate of decay due to non-runoff processes (1/hr)
s = rate of street sweeping (passes per hr)
f = efficiency of street sweeping (fraction removed per pass)
a = washoff coefficient
c = washoff exponent
r = runoff intensity from impervious surfaces (in/hr)

3.4.1.3 Modeling an entire watershed

To model an entire watershed, virtual watershed areas can be created for each land type. All the virtual watersheds are then routed to one device or simply to the outflow. To model an area of a few acres, one watershed is set up and routed to the devices, to another watershed, or to the outflow.

3.4.2 Particle and Water Quality Component Characteristics

The particle class characteristics are based upon the characteristics of the watersheds for impervious/pervious runoffs and street sweeping, and the characteristics of the devices such as settling velocities and filtration efficiency. Water quality characteristics are based upon the average weight distributions across particle classes (mg/kg).

Particle and water quality component sets are provided by P8. Calibrations are based upon the "typical urban runoff" values arrived at under the National Urban Runoff Program (NURP) (Athayede et al, 1983). The project used a distribution of particle settling velocities calculated from the NURP results (U.S. EPA, 1986) and a concentration distribution calculated using the NURP 50th percentile (or median) sites (Athayede et al, 1983).

3.4.3 Soil Quality

Soil Quality and characteristics are important to the calculation of runoff loading. The first characteristic determined is the Hydrologic Soil Group of the area. A listing of the four groups and their descriptions can be found in the appendix. For the purposes of this project, HSG B was assigned to the entire watershed based on previous studies of the area. This group is described as a "moderate" soil: moderate to well drained; moderately fine to moderately coarse texture; moderate permeability.

Next, the Soil Conservation Service (SCS) curve number can be assigned for each specific land use. The curve numbers are related to the maximum retention of water in the soil.

$$S = \frac{1000}{CN} - 10$$

S = Potential maximum retention [inches]
CN=SCS Curve Number

High curve numbers (up to 100) indicate near complete runoff with little retention, and low numbers indicate high retention and reduced runoff.

3.4.4 Meteorological Data

Runoff loading and concentration results are quite sensitive to the precipitation characteristics inputted. A storm event's duration and intensity determine the amount of phosphorus that is mobilized as well as the ability of a BMP to remove contaminants and sediment. While temperature data is not as important as the precipitation totals, it affects runoffs and efficiencies through evapotranspiration (ET) rates. Also needed for ET rate calculations are the vegetation cover fractions, which define the amount of growth in available pervious areas.

Precipitation and temperature data is available through various agencies in various intervals, from values every 15 minutes to monthly totals. P8 uses hourly precipitation data for the model. For temperature, either monthly averages or hourly temperature data can be used. For this project, hourly precipitation data and monthly temperature averages from Logan Airport (Boston) will be used. The default settings for vegetation cover fractions were left in place.

3.4.5 BMP Devices

Once the various virtual watersheds are defined, removal methods are analyzed specific to the area being treated.

3.4.5.1 Types of BMP Devices

There are six specific types of devices available through P8 and one general device that can be user defined to fit the needs of the model and removal. Each device requires the user to define the device size and basic characteristics. P8 can also size a device specific to a water quality component removal and watershed area. The table below lists all available devices. Numbers 1 to 4 are devices used for removal. Numbers 5 and 6 are not removal devices, however, they allow runoff to be redirected to or from the removal devices and watersheds. The different pipes also help retard watershed flows and response. Number 7, the aquifer, allow the user to keep account of groundwater concentrations and infiltration.

Device #	Type	Description
1	Detention Pond	Pond area with a permanent pool, normal outlet (wet pond) and an optional flood pool which empties between storm events (dry pond)
2	Infilitration Basin	Basin area that acts as a storage area while water infiltrates; usually comprised of crushed stone
3	Swale/Buffer Strip	vegetation strip that treats overland sheet flow
4	General	to be defined by user
5	Pipe / Manhole	Linear Reservior with one outlet
6	Splitter	Linear Reservior with two outlets: a normal outlet, and an alternative flood outlet
7	Aquifer	Linear Reservior that holds inflow from pervious areas of the watershed and exfiltration from other devices, and outflows through the baseflow.

Table 3-1: Listing of BMP Devices Available with P8

For the Acton project, we want to look at urban devices that can be easily implemented in the town and possibly in other towns throughout the watershed. Devices 1, 2, and 3 provide options for removal of pollutants, where as devices 5 and 6 are used for routing.

3.4.5.2 Device Flows

Flow within P8 is analyzed in the downstream order, one at a time. When the model is first executed, it sorts all the watersheds and devices into the downstream order and a table is created with elevation/volume/discharge calculations based on user inputs. The storage volume and outflow is related through the linear approximation:

$$Q = d_o + d_1 V$$

where,

Q = outflow for a given device and outlet (ac-ft)

V = current device volume (ac-ft)

d_o = intercept of outflow vs. storage volume curve (ac-ft/hr)

d_1 = slope of outflow vs. storage volume curve (hr^{-1})

d0 and d1 are updated for each time step during the model run. With the storage volume/outflow relationship linearized, the flowing equation is describes the analytical solution for a device flow balance at any given time step:

$$\frac{dV}{dt} = Q_{in} - \mathrm{SUM}\,(Q)$$

$$\Big\downarrow$$

$$\begin{aligned} V_{n+1} - V_n &= F\,(V,t) \\ &= A/K + (V_n - A/K)\exp(-Kt) - V_n \end{aligned}$$

$$A = Q_{in} - SUM\ (d_0)$$
$$K = SUM\ (d_1)$$

where,

V_n, V_{n+1} = volume at start and end of a time step (ac-ft)
Q_{in} = total inflows to device from watersheds and upstream devices (ac-ft/hr)
SUM = sum over device outlets (infiltration, normal, spillway)
t = time step length (hours)

For each time step, the estimated volume change is calculated using the following series of calculations:

$$\begin{aligned} V_m &= V_n + 0.5\,F(V_1, t) \\ V_{n+1} &= V_n + F(V_m, t) \\ V_m &= (V_n + V_{n+1}) / 2 \\ V_{n+1} &= V_n + F(V_m, t) \\ V_m &= (V_n + V_{n+1}) / 2 \end{aligned}$$

where,

V_m = average volume during the time step (ac-ft)

Volumes are constrained according to the maximum volumes inputted by the user. Any excess volume is assumed to flow to/through the spillway outlet.

3.4.5.3 Device Outlet Capacities

The following equation (from Bedient and Huber, 1988) is used for estimating overland flow velocities (for buffer strip calculations):

$$u = 1.49\, r^{2/3}\, s^{1/2} / n$$

where,

u = overland flow velocity (ft/sec)
r = hydraulic radius (ft)
s = slope (ft/ft)
n = Manning's n

Trapezoidal geometry is assumed for calculating the hydraulic radius. If the flow reaches the maxium flow depth, excess inflows are calculated at a fixed water depth and hydraulic cross-section.

Outlet from detention ponds from their normal outlet (in this case, an orifice) are calculated using a standard hydraulic equation:

$$q_o = c_o\, a_o\, (2\, g\, h)1/2$$

where,
q_o = orifice flow (cfs)
c_o = orifice coefficient (~ 0.6)
a_o = orifice area (ft^2)
g = gravity (32.3 ft/sec^2)

For wet detention ponds, the normal outlet is used to drain off any flow during "flood" conditions. Normally, the wet pond's outflow is directed to the spillway.

3.4.5.4 *Device Concentration*

All devices are assumed to have completely mixed flows. Concentrations are computed using the flowing equations:

$$\frac{dM}{dt} = W - DM$$

$$D = Q/V_m + f\,K_1 + f\,K_2\,C_m + f\,U\,A_m/V_m$$

$$\begin{aligned} M_{n+1} &= W/D + (M_n - W/D)\exp(-Dt), && \text{if } D > 0 \\ &= M_n + Wt && \text{, if } D = 0 \end{aligned}$$

where,
D = sum of first order loss terms (hr^{-1})
Cm = average concentration during step (ppm)
Vm = average device volume during time step (ac-ft)
M_n, M_{n+1} = particle mass in device at start and end of time step (ac-ft*ppm)
t = time step length (hour)
W = total inflow load to device, from watersheds and upstream devices (ac-ft*ppm/hr)
U = particle settling velocity (ft/hr)
A_m = average device surface area during time step (acres)
K_1 = first-order decay coefficient (hr^{-1})
K_2 = second-order decay coefficient ($hr\text{-}ppm^{-1}$)
f = particle removal scale factor, device specific

Concentration averaged over the time step, Cm, is defined as (Palmstrom, 1990):

$$C_m = [W + (M_n - M_{n+1})/t]\ V_m/D \quad \text{(from mass balance)}$$

3.4.6 Limitations

Most areas have little to no runoff concentration or loading data sets, making it difficult at best to calibrate the model to a specific site. Because of this, absolute concentration and load values are not going to be as reliable as the relative removal rates. Along the same lines, the particle parameters used have been calibrated for the Rhode Island region (where the model was initially used). These parameters can vary among specific locations, again affecting the reliability of absolute values. (Palmstrom, 1990) This project assumes that the variation in particle parameters does not significantly affect the absolute values.

Another limitation to note is the model's inability to account for snowfall or snowmelt data. P8 simply assumes that all precipitation is in the form of rainfall. For this reason, runs will be done excluding the winter season.

3.5 Model of the Entire Watershed

The first step is to model the entire watershed in order to calculate what is available for use in the trading program. Also, we want to make some determination of what concentrations are coming from the various areas and estimate phosphorus loading. Complete GIS land-use data for the town of Acton is not available, so it will be assumed that the results obtained for the entire watershed are also characteristic of Acton.

3.5.1 Land-use data and inputs

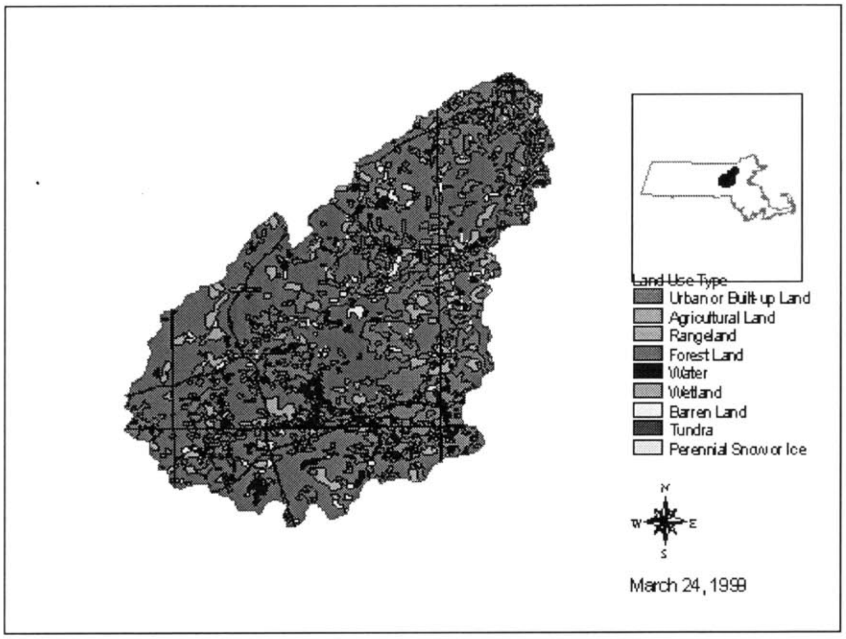


Figure 3-1: Land use Report for Watershed 1070005 (Assabet)

Land-use data was obtained through Mass GIS/BASINS 2.0 for the entire watershed. Information specific to Acton was assessed using GIS data provided by the Town of Acton. The 12 acres of unclassified land will be ignored, as it is a mere .005% of total watershed area.

The five general watershed land-uses, Urban/Built up, Agriculture, Forest, Wetlands, and Barren Land, were separated into 13 subcategories to allow for a more detailed assessment of the watershed. The 7,678 acres of surface water are not included.

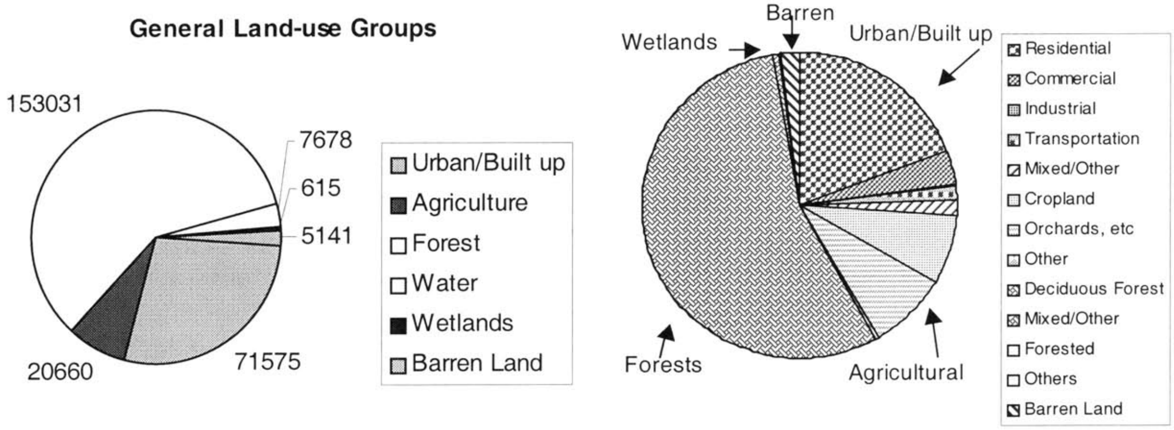


Figure 3-2: Watershed land-use: a) general land-use categories b) specific land-use categories

Watershed slope will be determined through analysis of topographic maps of the watershed and of the Acton area in particular.

Impervious fractions are assigned to each land use type based on local data from Acton. When a range of values was available for the land-type, the median value was used. For land-use densities, a medium density was assumed.

Land Use	Median	Range
Residential (medium)	0.27	.22 to .38
Commercial (medium)	0.65	.44 to .92
Industrial	0.77	.59 to 1
Transportation	0.41	.23 to .60

Table 3-2: Impervious Fractions for various land uses

First, assumptions were made on general soil qualities for purposes of selecting a Hydrological Soil Group (HSG). It was determined that the Boston area is considered in general to be in HSG B (US Dept., of Agriculture, 1997)

Based on the HSG assumptions, other watershed and device characteristics were determined. For the watershed, the necessary curve numbers are assigned according to land-use and soil quality. Below is the listing of curve numbers for HSG B.

Land-use	Curve #
Grassed (fair)	69
Meadow/Idle	58
Woods (good/fair)	55/60
Construction	89

Table 3-3: SCS Curve numbers for soil group B

3.5.2 Meteorological Data

The meteorological data used will shape the results of each individual run. The P8 model came with the hourly precipitation data for years 1954 to 1958. More current data (for years 1990 to 1994) was obtained and used for all model runs. The data from the 1950s was used in modeling the entire watershed to compare flow and concentrations as well as providing a larger data set for calculating an average concentration. The data from the 1950s is also useful to the project in that the data contains both a very wet year and a year with a hurricane. Results from these years will be used to note the effects of harsh weather conditions on watershed loading and the efficiency of the various devices.

Figure 3-3 shows the monthly precipitation totals for all available data sets. Visible peaks are the hurricane in August of 1955 and two smaller peaks in May and September of 1954. The remaining monthly totals mainly lie between 1 to 7 inches. In later sections, more specific comparisons of the storm duration and intensity will be presented to help explain modeling results.

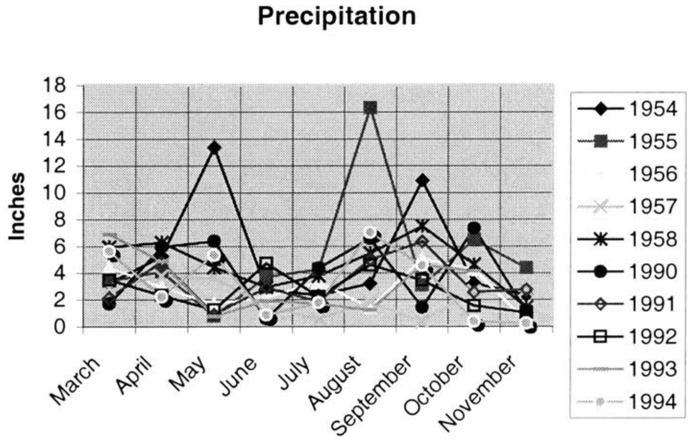


Figure 3-3: Monthly precipitation totals for specific years

Daily temperature files are optional. The user can input monthly averages along with vegetation cover factor (0-1) and daylight (hours/day). For the various models, the average monthly temperature data will be used. For fraction of vegetation cover, the default data set provided with the P8 model will be used.

3.5.3 Results

Results were to be calculated for the spring, summer, and fall months. For each year, the model ran from March 1 to November 11, keeping the information from March 11 to November 11, allowing 10 days for steady state conditions to be reached.

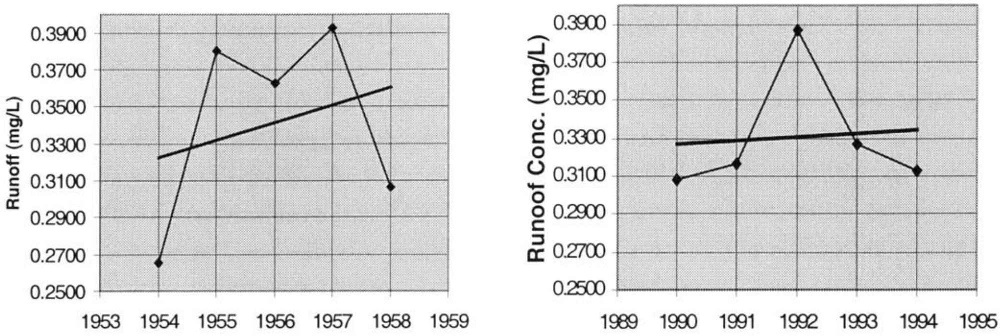


Figure 3-4: Total Phosphorus concentrations in watershed runoff: a) for 1954-1958 and b) for 1990-1994

Figures 3-4 a,b show the results for 10 runs of the complete watershed model with the various precipitation data. The average concentration for all runs is 0.336 ppm, which corresponds to the 0.33 ppm concentration arrived at by the NURP. The variety in total outflow concentrations stems from the varying intensity of storms in each specific year. Several intense storms in a year (such as in 1954) create lower concentrations due to more dilution. Alternatively, years with many light storms, such as in 1992 have caused higher concentrations in runoff since pollutants are mobilized even though runoff flow is light (less dilution).

It should be noted that the BMP devices will have to be able to deal with runoff concentrations much higher than the average. Figure 5 shows a sample of the concentration levels as a function of storm events. The events listed are for March through November of 1991, 1992, and 1993.

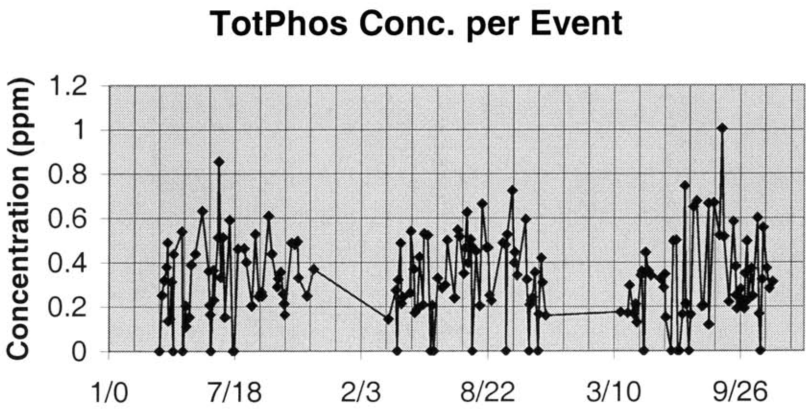


Figure 3-5: Concentration of Total Phosphorus in the Outflow versus Storm Event

As seen, concentrations range from negligible to 1 ppm, 3 times the average concentration.

3.6 Design and Analysis of BMP devices

Once the amount of phosphorus loading was determined, the BMP plan for Acton needed to be designed. The plan needed to include policies and devices that could be easily and cost-effectively implemented in Acton and possibly other areas within the watershed. Already a part of the plan is a wet detention pond located in a wetland/recreational area in Acton. From a literature study of all available options, one general BMP device, the buffer (or swale) strip, was chosen for the plan. A buffer (or swale) strip is a strip of vegetation that reduces the velocity of local runoff, thereby reducing the runoff's ability to hold sediments and nutrients. Pollutants are deposited on the strip as the runoff flows over it. The flow leaves the strip with a significantly reduced concentration of pollutants. For the buffer strip, efficiency and buffer size will be studied. Since the size and design of the detention pond has already been determined, only efficiency will be analyzed

3.6.1 Buffer Strip Case

The first step was to set up a generic watershed in which to test the effectiveness of the strip in a residential area of Acton. This watershed will be modeled as a residential area with fair grass areas and soil qualities (HSG B), and medium housing density (1 to 3.9 units per acre). A 2% slope in the land, was assumed which coincides with the slope assessment of the watershed discussed earlier. The complete description of the residential watersheds can be found in the appendix. Table 3-4 lists the input values:

Pervious Curve Number	69
Impervious Fraction	0.26
Depression Storage (in.)	0.02

Table 3-4: General "Residential Watershed" Inputs

3.6.1.1 Strip Parameters/Inputs

Several inputs are needed to define the buffer strip and its removal efficiency. For this case, we are looking for a buffer strip that will fit in a typical urban residential area and be easily created. First, dimensions need to be set. The starting point for the dimensions was set by looking through literature for a typical flow path and setting a bottom width that seemed feasible for the area. For the 5-acre residential area, the initial strip was set up with a flow path length of about 30 feet and a bottom width of 150 feet. Flow path slope is 2%, which coincides with the slope calculated for the Acton area.

Next, flow and soil infiltration characteristics need to be defined. The Manning's coefficient (*n*) mentioned above characterizes the resistance to overland flow and land surface roughness. First is the flow depth, which defines the maximum flow depth at which the specified Manning's coefficient applies for the computation of the overland sheet flow. The TR-55 puts this on the order of 0.1 feet (USDA/SCS, 1985). As the Manning's coefficient increases, so does the depth and duration of flow in buffers during and flowing storms. Sensitivity of the Manning's coefficient to particle removal rates increases with the defined infiltration rate. Table 3-5 below shows typical values for n based on coverage:

Cover	*n*	Source
Dense Growth	0.40 - 0.50	Bedient & Huber (1988)
Pasture	0.30 - 0.40	
Lawn	0.20 - 0.30	
Bluegrass Sod	0.20 - 0.50	
Short-grass prarie	0.10 - 0.20	
Sparse Vegetation	0.05 - 0.13	
Bare Clay-Loam Soil	0.01 - 0.03	

Table 3-5: Manning's coefficient for various types on vegetation coverage

For this model, I looked at both a strip with a typical lawn coverage (n =0.25), and with dense growth (n=0.45).

The choice of an infiltration rate is based on the soil type of the watershed. P8 provides the user with several options as seen in Tables 6. For this model, a value of 0.26 in/hr which correlates with SCS Soil group B, or a "silt loam" soil type was used.

Sources:	(a)	(c)
	Infiltration Rate	
SCS Soil Group	in/hr	in/hr
Sand	4.64	8.27
Loamy Sand	1.18	2.71
Sandy Loam	0.43	1.02
Silt Loam	0.26	0.27
Loam	0.13	0.52
Sandy Clay Loam	0.06	0.17

a - McCuen (1982)
b - Shaver (1986)
c - Musgrave (1955)

Sources:	(a)	(c)
	Infiltration Rate	
SCS Soil Group	in/hr	in/hr
A	0.43	0.30 - 0.45
B	0.26	0.15 - 0.30
C	0.13	0.05 - 0.15
D	0.03	0.00 - 0.05

Table 3-6: Various Infiltration rates for SCS soil groups

The particle removal scale factor adjusts the particle removal rates for each device. Removal rates include settling velocities, as well as first- and second- order decay rates. Normally, it has a value of 1.0, and it will stay at 1.0 for this model.

The initial runs of the model looked at the runoff of a simple grass buffer strip to determine efficiency of the device and feasibility for a section of Acton's residential community. The buffer strip model is set up as seen in figure 6

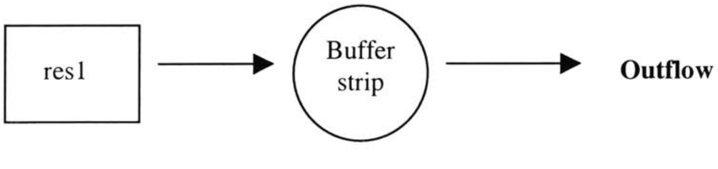


Figure 3-6: Schematic of General buffer strip model

"res1" relates to a 5 acre residential watershed. As stated before, P8 assumes that all runoff from the watershed res1 will flow through the BMP device.

3.6.1.2 Results

Several different runs were done initially to assess the efficiency of the initial buffer strip, determine the sensitivity of the results to various input values, and assess the variation of strip efficiency with precipitation. A second model series was done to determine the dimensions of the strip according to efficiency. The final set of model runs determined efficiency versus the percent of surface flow to actually run over the strip. The graph below shows the results from the initial runs

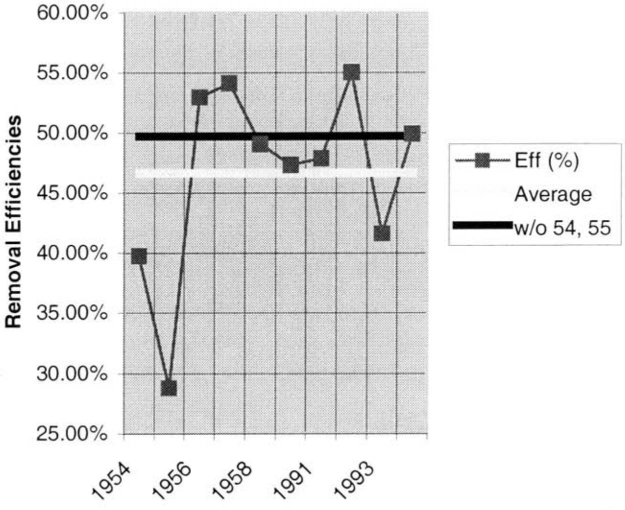


Figure 3-7: Initial Model Buffer Strip Efficiency

There is a notable difference between the results from 1954 and 1955 from the rest of the years. Referring back to figure 3-3, which displays the precipitation data, we see significantly elevated levels of precipitation for these years. P8 noted that for 1954 and 1955, the buffer strip "flooded" during the model run, meaning it exceeded its ability to even slow the runoff flowing over it. By looking at the event data for both of these years, we see that 1954 was a more consistently wet year with a few major storms, while 1955 was about normal with the exception of the hurricane that hit in late August. The intensity and duration of the storms in 1954 and the hurricane of 1955 elevated runoff velocities and flows to the point that the buffer strip had little to no effect on slowing the flow down.

The strip also had to contend with significantly elevated loads: while 1954 had a load of 5.10 lbs. and 1955 had a load of 7.73 lbs., the remaining years averaged 3.72 lbs. With drier periods preceding the hurricane in August of 1955, the large flow from the somewhat sudden storm event allowed more sediment and pollutants to mobilize.

3.6.1.3 Sizing the Strip according to Efficiencies

The initial setting showed an average of 50% phosphorus removal for a strip with a flow path of 30ft. The next step was to determine the effects of strip flow path length with removal efficiency. For the purposes of feasibility, both in terms of available land and cost of implementation, it is necessary to analyze the effect a decrease or increase in flow path length would have on the removal rate.

Figure 3-8b shows the sizes with a dense buffer strip (Manning's n = 0.45). This would be useful for area where a forested area near to the residential area could be used to act as the buffer strip. Figure 3-8a gives the results for n = 0.25, which is typical for a lawn buffer strip, a device that could easily be implemented in a residential area. With the change in the "n" variable, there is little change in the efficiency and strip size up to about a 60% removal efficiency. Flow paths vary by approximately 10 feet for the 70% removal level.

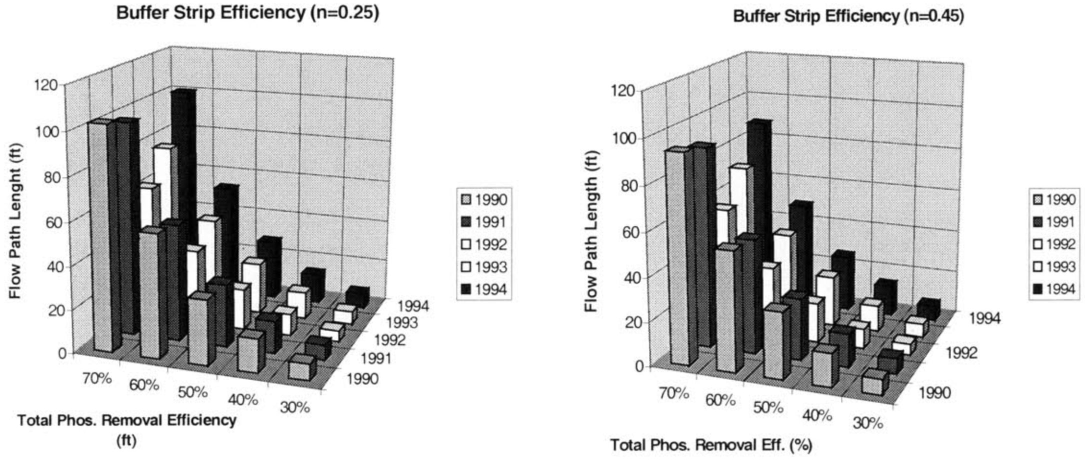


Figure 3-8: a) Shows efficiencies for a lawn buffer strip. b) Shows efficiencies for a buffers stripe with dense growth.

The average flow path lengths for each removal percentage are 7 ft for 30%, 13 ft for 40%, 26 ft for 50%, 48 ft for 60% and 88 ft for 70%. There appears to be an exponential increase in flow path length as the removal percentages increase. Therefore for locations where only a minimal amount of land is available, a strip with a short flow path will provide some noticeable phosphorus removal. Alternatively, where land availability is not an issue, it is not particularly profitable to increase the flow path past approximately 48 feet.

3.6.1.4 Comparison of Efficiency with % of runoff actually running through the Strip

Due to the positioning of the device or geography of the watershed, a BMP device may not be able to collect all of the runoff from a watershed. As stated, the results listed in figures 3-8a/b above were arrived at using the assumption that all surface water in the area will runoff into and through the buffer strip. To model phosphorus removal for situations where only a fraction of runoff is treated by the buffer strip, a second general residential area was designed following the schematic showing in figure 3-9.

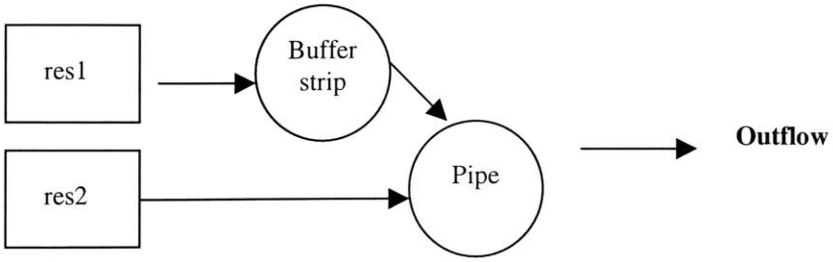


Figure 3-9: Schematic of second residential buffer strip model

This model essentially separates the single watershed into 2 separate watersheds. The first (res1) watershed flows from the buffer strip into a pipe where the runoff merges with the runoff from the untreated watershed (res2). The results are displayed in figure 3-10.

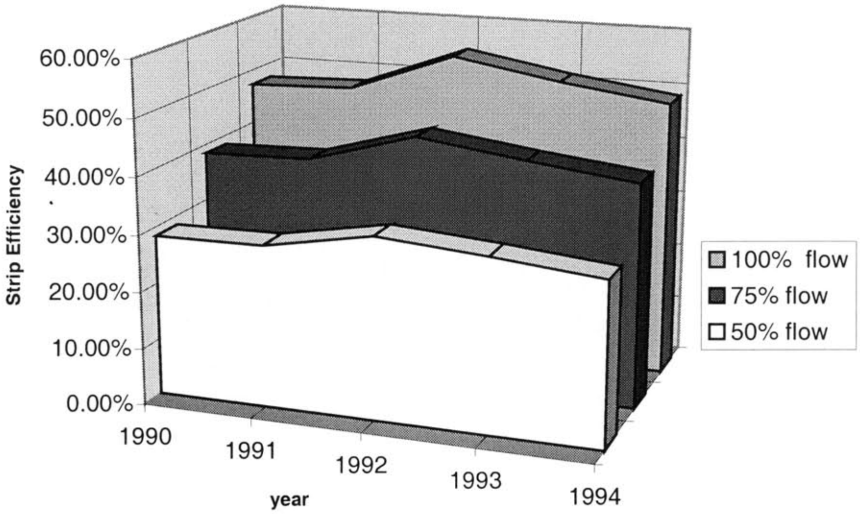


Figure 3-10: Buffer strip efficiency vs. % of runoff running over strip

The averages for each run can be found in Table 3-7 below. For each type of flow both the device efficiency and the calculated outflow concentrations is listed. The average inflow concentration was 0.3362 ppm.

% of flow treated	outflow (ppm)	dev. eff.
100%	0.1675	50.15%
75%	0.1997	40.61%
50%	0.2369	29.67%

Table 3-7: Average Results for Efficiency versus % of Runoff Treated

As expected, this device alone will not be enough to meet the target outflow concentrations. If 100% of the flow does go through then a concentration is achieved that is close to the target release concentration for the wastewater treatment facility.

3.6.2 Detention Pond

Ponds have been noted for their usefulness in removing several runoff pollutants including suspended solids, nitrogen and phosphorus. The removal is a function of particle settling and retention time within the pond. A dry detention pond has a drain of sorts close to the bottom of the pond, allowing the pond to dry out between storm events. A wet detention pond has no such removal device, water leaves the pond either through infiltration, evaporation or surface runoff if the pond floods. Both wet and dry detention ponds can be used for removal of the pollutants.

Within Acton, a wetlands recreational area will be designed and constructed. Part of the area's design includes a wet detention pond to take in runoff flow and settle out some of the pollutants. A simple model was designed to predict the removal efficiency within the pond area.

3.6.2.1 Model inputs

The area that will be treated by the detention pond is estimated at 50 acre of residential and forest land. Based on land use ratios from MassGIS data, 2 virtual watersheds are set up to model the area: first, a 16.5-acre residential area with the same characteristics as those used in the buffer strip's residential area; second, a 33.5 acre forest land with a fair amount of growth. The runoff from both these areas will completely flow into the pond.

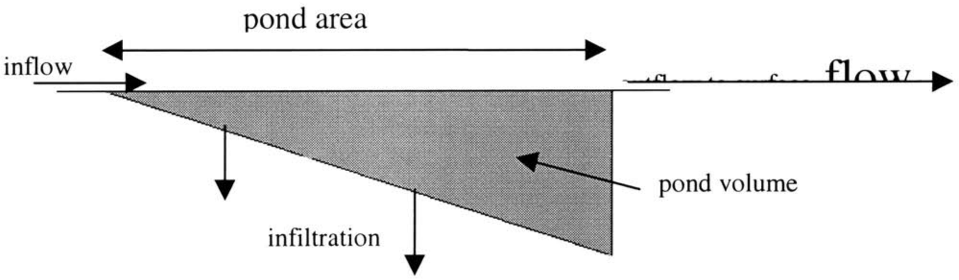


Figure 3-11: Detention Pond

The model will be a wet detention pond with a structure similar to that seen in figure 3-11. The surface area of the pond is 4000 ft^2 or approximately 0.091 acres. Pond volume is estimated at 0.18 acre-ft. The initial pond dimensions were arrived at using approximated data from the wetland/recreation park area.

3.6.2.2 Detention Pond Results

Figure 3-12 shows both the calculated efficiencies and estimated concentrations in the outflow.

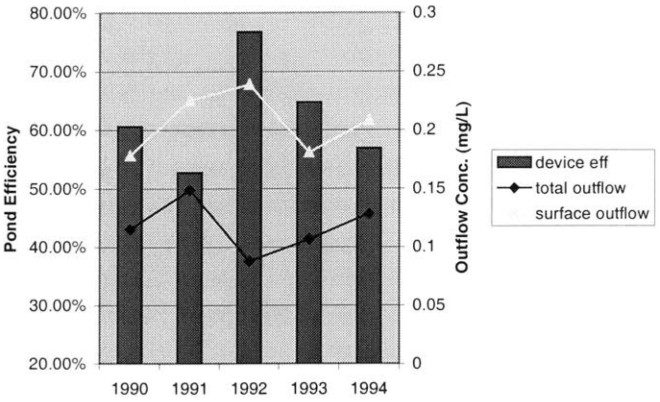


Figure 3-12: Efficiency and Concentration of outflow from the wet detention pond

The diversity in efficiency results is again due to the variety of storm event lengths and intensities. It is important to note not only the calculated total outflow but also the surface outflow. Device efficiency is calculated using the total removal loads. This total is the sum of groundwater and surface runoff loads. For detention ponds, most of the phosphorus ends up trapped in the sediment at the bottom of the pond. However it is important to look at the overflow – runoff concentrations leaving the pond. These concentration levels are just slightly lower than the concentrations entering the pond. While the pond does settle out a significant amount of phosphorus according the model results, a second device would have to be implemented downstream of the pond runoff to reduce/dilute the secondary runoff concentrations.

A study of the literature available on wet detention ponds shows that while models and calculations predict removal efficiencies on the order of 60% actual removals rates range from the low teens up to 90+%.

3.7 Recommendations for BMP / Conclusions

Devices within the Assabet area need to deal with phosphorus concentrations averaging from 0.2 to 0.6 ppm (mg/L) during any average storm event. At least one event per year had an elevated concentration of 0.8 to 1 ppm. Any BMP device implemented here will need to be able to affect at least the average range of phosphorus loads if not the yearly extremes.

With the high phosphorus loading in the watershed, buffer strips appear to be the most feasible and efficient of the BMP device options. Even ensuring the existence of a small strip of grass at the drainage location of a neighborhood will have a noticeable impact on the phosphorus concentrations running off into the Assabet or into the sewer system leading to the future WWTP.

The wet detention pond will most likely have some impact on phosphorus levels, though not to the degree that the buffer strip produce. Results are questionable at best and could possibly not be worthy enough of the cost of the pond. While the specific pond modeled will be useful in lowering concentrations entering the wetlands recreational area from the surrounding areas, implementing a similar pond on its own in other areas would not produce significant removal results.

3.8 References

Athayede, D.N. et al, "Results of the Nationwide Urban Runoff Program, Volume I, Final Report", US Environmental Protection Agency, Washington, NTIS PB84-185552, December 1983

Barrett, M.E., P.M. Walsh, J.F. Malina, and R.J. Charbeneau. 1998. Performance of Vegetative Controls for Treating Highway Runoff. Journal of Environmental Engineering. 124(11):1121-1128. November, 1998.

Bedient, P.B. and W.C. Huber, "Hydrology and Floodplain Analysis", Addison-Wesley, 1988

Bender, G.M., and W.W. Rice. 1982. Effects of Municipal Street Sweeping on Urban Storm Runoff Quality. In *Proceedings of the 1982 International Symposium on Urban Hydrology, Hydraulics, and Sediment Control.* Lexington, Kentucky. July 27-29, 1982.

City of Plymouth, Minnesota(a). 1999. High Levels of Phosphorus from Fertilizer Harm Lakes And Wetlands. http://www.ci.plymouth.mn.us/city_services/high_levels_of_phosphorus.html February 28, 1999.

City of Plymouth, Minnesota(b). 1999. Protect Water Quality: Commercial Applicators Must Use Phosphorus-Free Fertilizer on Plymouth Lawns. http://www.ci.plymouth.mn.us/city_services/fertilizer_ordinance.html February 28, 1999.

Doenges, J.M., C.P. Allan, R.J. Jontos, and C.A. Liebler. 1990. Carry Capacity of Public Water Supply Watersheds: A Literature Review of Impacts on Water Quality from Residential Development. Connecticut Department of Environmental Protection. DEP Bulletin No. 11. March 1990.

Dorney, J.R. 1986. Leachable and Total Phosphorus in Urban Street Tree Leaves. Water, Air, and Soil Pollution. 28:439-443.

Field, R. 1985. Urban Stormwater Runoff Quality Management: Low-Structurally Intensive Measures and Treatment. In *Proceedings of the NATO Advanced Research Workshop on "Urban Runoff Pollution"*. Montpelier, France. August 26-30.

Grass is Always Greener Lawn Services. 1999. Telephone Interview with the Author. March 2, 1999.

Halley, D. 1999. Acton Health Official. Telephone Interview with the Author. March 1, 1999.

Hoffman, E., "Urban Runoff Pollutant Inputs to Narragansett Bay: Comparison to Point Sources", EPA 440/5-85-001, 1985

Huber, W.C. and R.E. Dikinson, "Storm Water Management Model, Versions 4: User's Manual", Environmental Research Laboratory, US EPA, Georgia, August 1988

Jeer, S., M. Lewis, S. Meck, J. Witten, and M. Zimet. 1997. Nonpoint Source Pollution: A Handbook for Local Governments. Washington, D.C.:American Planning Association.

Keaton, M.R., P.K. Kalita, J.P. Harner, and S.K. Starrett. 1998. Evaluation of Vegetative Filter Strips for Nutrient Removal from Feedlot Runoff. Presented at the 1998 ASAE Annual International Meeting. Orlando, Florida. ASAE Paper No. 98-2014.

Lamker, J.J. Clearing the Waters. 1998. Cooperative Partners. Cenex Harvest States Cooperatives and Land O'Lakes Inc. March/April 1998.

Lawn Doctor of Nashoba Valley Lawn Services. 1999. Telephone Interview with the Author. March 2, 1999.

Lee, D., T.A. Dillaha, and J.H. Sherrard. 1989. Modeling Phosphorus Transport in Grass Buffer Strips. Journal of Environmental Engineering. 115(2): 409-427. April 1989.

Lee, G.F. and R.A. Jones. 1996. Detergent Phosphate Bans and Eutrophication. Water Research. 28:(5)749-751.

Metropolitan Council Environmental Services. 1999. P2 in the Yard...
http://www.metrocouncil.org/environmental/PollutionPrevention/yard.htm
February 28, 1999.

Novotny, V. 1984. Efficiency of Low-Cost Practices for Controlling Pollution by Urban Runoff. In *Proceedings of the Third International Conference on Urban Storm Drainage*. P. Balmer, P.A. Malmqvist, and A. Sjoberg, eds. Chalmers University of Technology, Goteborg, Sweden. 1241-1250. 1984.

Novotny, V., and H. Olem. 1994. Water Quality: Prevention Identification, and Management of Diffuse Pollution. New York:Van Nostrand Reinhold.

Novotny, V., H.M. Sung, R. Bannerman, and K. Baum. 1995. Estimating Nonpoint Pollution from Small Urban Watersheds. Journal of the Water Pollution Control Federation. 57(4) 339-348.

Palmstrom, N. and W. Walker, "P8 Urban Catchment Model: User's Manuel, Program Documentation, Evaluation of Existing Models, Design Concepts and Hunt-Potowomut Data Inventory, 1990

Pierzynski, G.M., J.T. Sims, and G.F. Vance. 1984. Soils and Environmental Quality. Lewis Publishers. Ann Arbor, Michigan.

PLCAA. 1999. Demand for Lawn/Landscape Services up 12 Percent. Professional Lawn Care Association of America Landscape-Lawn Care News. http://www.plcaa.org/news.html March 1, 1999.

Porter, E. 1991. Phosphates in Detergents. University of Idaho Cooperative Extension System Publication cis-907.id. December 1991.

Shaver, H.E., "Infiltration as a Stormwater Management Component", from B. Urbonas, L. Roesner, Urban Runoff Quality – Impact and Quality Enhancement Technology, American Society of Civil Engineers, 1986

Syrek, H. 1981. Alaska Litter. Report for the Alaskan Department on the Environment and Conservation. The Institute for Applied Research, Sacramento, California.

TruGreen-Chemlawn Lawn Services. 1999. Telephone Interview with the Author. March 2, 1999.

US Dept. of Agriculture, Soil Conservation Service, "Urban Hydrology for Small Watersheds – Technical Release Number 55", May 1985

US Dept. of Agriculture, "Part 630 Hydrology National Engineering Handbook" – Chapter 7, Natural Resources Conservation Service, September 1997.

US Environmental Protection Agency, "Methodology for Analysis of Detention Basins for Control of Urban Runoff Quality", Washington, EPA 440/5-87-001, September 1986.

United States Environmental Protection Agency. 1993. Office of Research and Development. Handbook of Urban Runoff Pollution Prevention and Control Planning. Washington D.C. EPA/625/R-93/004.

United States Environmental Protection Agency. 1990. Region V Water Division. Urban Targeting and BMP Selection. Chicago, Illinois. November 1990.

Walker, William W., "Design Calculations for Wet Detention Ponds", prepared for St. Paul Water Utility and Vadnais Lake Area Water Management Organization, October 1987.

Walker, William W., "Design Concepts for a Land-Based Water Quality Model of the Hunt-Potowonut River," prepared for Narragansett Bay Project, April 1989.

Waller, D.H., and W.C. Hart. 1985. Solids, Nutrients, and Chlorides in Urban Runoff. In *Proceedings of the NATO Advanced Research Workshop on "Urban Runoff Pollution"*. Montpelier, France. August 26-30.

Woodard, S.E., and C.A. Rock. 1995. Control of Residential Stormwater by Natural Buffer Strips. Lake and Reservoir Management. 11(1):37-45.

4 Stream Water Quality Modeling Analysis

The QUAL2E stream water quality model was used to determine the impacts of the Acton Trading Program on water quality in the Assabet River. After the model was calibrated to the Assabet River utilizing past stream water quality data, the impacts on Assabet River water quality from several phosphorus loading scenarios were predicted. In these scenarios, BMP phosphorus reduction efficiencies predicted by the NPS modeling analysis of Section 4 were represented as Acton NPS phosphorus load reductions to Nashoba Brook. The predicted phosphorus concentrations from the Acton WWTP outlined in Section 2 were also represented within the model scenario runs.

4.1 Introduction to Mathematical Modeling

Mathematical water quality models have been developing for many years as research and water quality management tools, having first been developed in the early 1920's to provide accurate dissolved oxygen (DO) concentration predictions (Beck, 1985). Computer technology became a vital part of mathematical modeling in the early 1970's, spurred from novelty status to a necessity as data requirements soared in attempts to accurately assess increasingly complex surface water systems. Once implemented, computer modeling technology soon encouraged the progression of even more complex models (Beck and van Straten, 1983). Although almost nonexistent prior to 1964, approximately half of all technical papers published in 1969 discussed the "concepts, principles, or results of the application of optimization or computer simulation techniques to water resources problems." Soon model technology was widely considered to be fully developed overall and more than just a basic research tool (Goodman, 1970).

4.1.1 Model Application and Development

A water quality model that can be directly applied to all water quality situations does not exist. As a result, the application of mathematical stream water quality models to evaluate surface water behavior is often accomplished with one of two primary approaches: the adaptation or direct application of an existing model to the problem or the development of a site-specific independent model or linked set of models to evaluate the unique problem (Beck, 1985). Development of most models follows the initial processes shown in Figure 4-1. The problem or scope of model application is identified, and a suitable theoretical construct (also called a functional representation) consisting of representative equations is developed in accordance with the general theory (also called a conceptual representation) of the model developer (Thomann and Mueller, 1987). Numerical values are then assigned to inputs and parameters, which is often labeled the computational representation (Brown and Barnwell, 1987). Together, the representative equations and assigned numerical values form the mathematical model and the initial task of model specification is complete.

Once developed, whether for a unique water quality situation or for general use, the model is then tuned during the calibration step with a set of field data extraneous to the initial model development process. Commonly, modeling exercises lack adequate field data for calibration and validation purposes, and even adequate data sets cannot provide all the insight necessary to

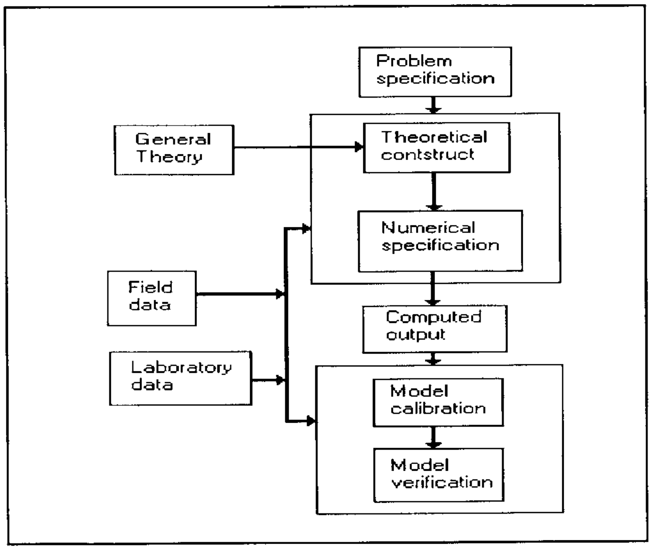


Figure 4-1. Model Development Process
Source: Adapted from Thomann and Mueller, 1987.

assign appropriate values to all model parameters. These parameters can often be determined from the ranges of reliable values present in the technical literature (Rines and Shanahan, 1993). Once a model has been calibrated with an initial set of field data, it is often subjected to further testing with additional field data to establish model validity in the model verification step. Depending on data availability, the model is often "tested" with unusual data in order to examine the its capability to predict water quality behavior under these unusual circumstances. After these processes have been completed to the extent possible with available calibration and verification data, the model is considered to be verified for use on that water body (Thomann and Mueller, 1987).

4.1.2 Model Policy and Planning Applications

Although mathematical water quality models often increase the understanding of environmental processes, a primary use of models has long been to address water quality management issues. Water quality management planning often requires an in-depth assessment of water quality that cannot be provided by field data or water quality monitoring data (Melching, 1996). Model results are frequently consulted to aid environmental policy and planning decisions to reduce the likelihood of expensive environmental control strategies that fail to improve water quality. The models provide a rational basis for water quality planning decisions by providing a credible method of analyzing the costs and benefits of a project (Beck, 1985).

The model use has great implications upon its design – models that simplify complex processes improve comprehension while very complex models are better suited to addressing management issues. A thorough analysis requires incorporation of as many of the mechanisms that result in unique water quality behavior as possible - hence the increased complexity of the water quality management-oriented model (Beck, 1985). However, the requirements of stream water quality management decisions typically require less discretization than is available from most numerical models (McCutcheon, 1989).

Due to model difficulties associated with various sources of uncertainty, the use of models as planning tools must be accompanied by a thorough knowledge of model limitations. In addition, modeling studies should be used in conjunction with monitoring and basic investigations to formulate an overall understanding of water quality behavior (McCutcheon, 1989). When used properly and with a thorough understanding of its limitations, a mathematical model can significantly increase the alternatives to a planning decision (Grimsrud, Finnemore, and Owen, 1976).

4.2 *The QUAL2E Stream Water Quality Model*

As the most recent version of a successful family of stream water quality models first developed in 1970, QUAL2E is considered by many to be the most widely accepted and used stream water quality model in the world. Its acceptance is largely due to its versatility and reliability as a water quality management and planning tool, which is the result of continuous improvements, modifications, and verification of the model. The QUAL2E version of the model has the capability to accurately predict many useful water quality variables, as well as determine the probability of error in its predictions with its QUAL2E-UNCAS companion program.

4.2.1 QUAL2E Development History

QUAL2E is the most recent of several revisions on the stream water quality model QUAL2, which was a modification of QUAL1. QUAL1 was developed by the Texas Water Development Board in 1970 as a stream water quality model that had the capability to simulate spatial and temporal water temperature, conservative mineral concentrations, biochemical oxygen demand (BOD), and DO. QUAL2 modifications to the original model were completed by the Water Resources Engineers, Inc. (now Camp, Dresser, and McKee), in 1973 for the EPA Systems Development Branch. The modifications added capabilities to simulate spatial and temporal variations in eight additional water quality components (Grimsrud, Finnemore, and Owen, 1976). QUAL2E was introduced in 1985 through a cooperative agreement between EPA and Tufts University, and contained many significant enhancements over the QUAL2 version of the model. The initial enhancements included improved algal, nitrogen, phosphorus, and DO interactions; algal growth rate provisions; temperature, DO, arbitrary non-conservative constituents; hydraulic processes; downstream boundary conditions; and input/output modifications. In 1987, further improvements were included in the QUAL2E Version 3.0, including extensive uncertainty analysis capabilities (with the companion program QUAL2E-UNCAS), reach-variable climatology input for steady-state temperature simulation, and the capability to print DO concentration plots (Brown and Barnwell, 1987).

4.2.2 QUAL2E Usage and Acceptance

As one of the first accurate stream water quality models, the QUAL models have been continually developed, maintained, and distributed for many years and have gained extensive acceptance within the modeling community. The QUAL2E model is currently one of the most widely used models to predict stream water quality behavior (Biswas, 1997). Studies of water quality models have indicated that QUAL2E is by far the most accurate one-dimensional stream water quality model under many conditions. They indicate that the model input and output files, added graphical user interface, and uncertainty analysis provide modelers with accurate results and ease of use when modeling stream water quality behavior. In addition, QUAL2E is one of few models that has been consistently and thoroughly examined for coding errors to improve model reliability. The thorough analysis performed on past versions of the model by the National Council of the Paper Industry for Air and Stream Improvement (NCASI) has had a significant impact on the acceptance of the QUAL models as reliable planning tools (McCutcheon, 1989).

QUAL2E was developed specifically to aid water quality management and planning (Brown and Barnwell, 1987). Numerous water quality planning studies worldwide have been analyzed with the use of QUAL2E, and have noted the accurate representation of water quality variables from validated versions of the model. These studies have commented on the usefulness of the model to simulate future water quality conditions and hailed the "contribution of the model to sound planning" (Rajar and Brebbia, 1997).

4.2.3 QUAL2E Model Classification Overview

QUAL2E is a one-dimensional model, which is the most common method used to model stream water quality (McCutcheon, 1989). As it is limited to one-dimensional analysis in the longitudinal direction, the QUAL2E assumes complete mixing or uniformity throughout the river width and depth (Biswas, 1981). In terms of time representation, QUAL2E utilizes both steady-state and quasi-dynamic water quality modeling processes. The quasi-dynamic time representation provides the modeler with the capability of keeping flows and loads constant while allowing environmental and meteorological parameters to fluctuate (McCutcheon, 1989). The capability to simulate both steady-state and dynamic water quality behavior make QUAL2E a very powerful and beneficial water quality management and planning tool (Brown and Barnwell, 1987). QUAL2E is also a simulation model as opposed to an optimization model. Therefore, model simulations must be run for all planning alternatives to evaluate the best option. Although the number of simulations is time intensive, the use of a simulation model such as QUAL2E provides a more accurate assessment of the economical and ecological impacts of various alternatives than a similar optimization model (Biswas, 1981).

4.2.4 QUAL2E Hydrologic Components

The QUAL2E model is applicable to dendritic streams that are laterally and vertically well-mixed. The major transport processes incorporated into the model are longitudinal advection and dispersion; lateral processes are not included in the model (Brown and Barnwell, 1987). The model can only simulate constant stream flows and waste loads (U.S. EPA Office of Water, 1995). The QUAL2E model hydraulic flow parameters shown in Figure 4-2 allow the stream to be divided into similar segments called reaches to increase modeling accuracy. Within each segment, the model assumes fairly uniform hydrogeometric characteristics and biological rate constants (Brown and Barnwell, 1987). The stream reaches are composed of computational elements of equal length that are treated as completely mixed reactors due to the one-

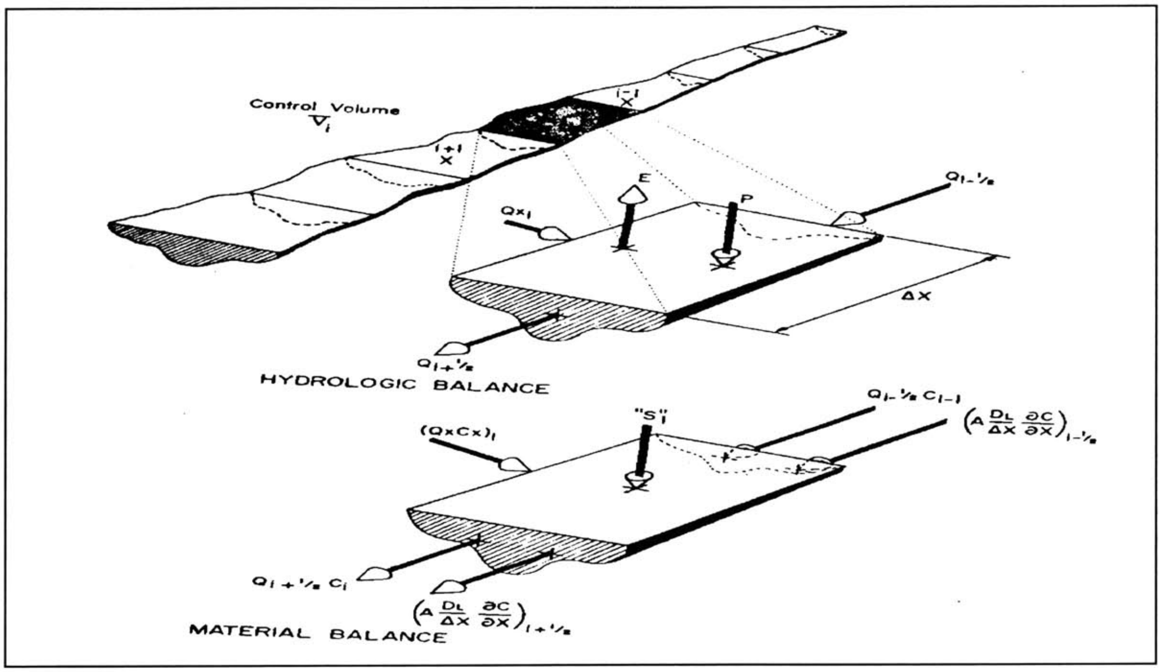


Figure 4-2. QUAL2E Stream Reach Discretization Method
Source: Adapted from NCASI, 1982.

dimensional nature of the model (See Figure 4-3). Although the requirement that computational

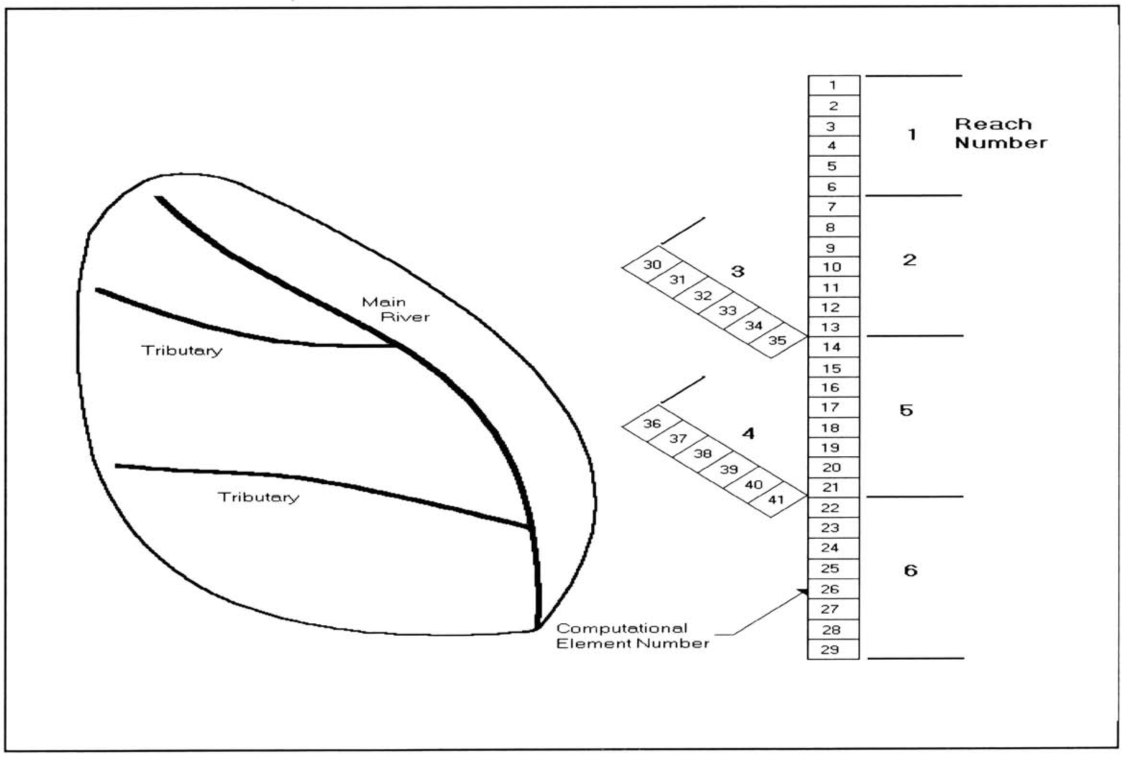


Figure 4-3. QUAL2E Stream Reach Division into Computational Elements
Source: Adapted from Brown and Barnwell, 1987.

elements be of equal length increases the difficulty of accurately modeling streams, its effects on model accuracy can be minimized by reducing element size (McCutcheon, 1989). Some studies have shown that the optimal stream reach size for modeling a fast river with QUAL2E is approximately 0.25 miles, while up to 3.2 miles per reach can be used for sluggish rivers (McCutcheon, 1985). QUAL2E is limited to a maximum of 50 reaches per simulation, and is limited a total of 500 computational elements, or 20 per reach (U.S. EPA Office of Water, 1995).

The QUAL2E model utilizes a mass balance to calculate the material flux and concentration of conservative minerals, coliform bacteria, and nonconservative materials for each computational element (U.S. EPA Office of Water, 1995). Each element is then linked to adjacent elements through the mechanisms of advective transport and dispersion (Brown and Barnwell, 1987). The model can incorporate point nutrient or waste inputs into each segment or can distribute the pollutant load evenly throughout the segment (NCASI, 1982). In addition, the model can also account for withdrawals and incremental inflows and outflows (Rajar and Brebbia, 1997). Other major processes detailed below are incorporated into the mass concentration calculations.

4.2.5 QUAL2E Water Quality Constituents

Many solid design features and intricate component relations are the primary reason for QUAL2E's water quality behavior prediction reliability. In addition to its general stream modeling components, QUAL2E can simulate up to 15 water quality constituents in branching stream systems. The constituents of concern in this study included DO, BOD, temperature, algae (as chorophyll a), the phosphorus cycle, and the nitrogen cycle.

4.2.5.1 Dissolved Oxygen and Oxygen Demand

The QUAL2E model calculates DO concentrations by incorporating sources and sinks into the mass balance equations for each computational element. The primary sources of DO in QUAL2E are algal photosynthesis and atmospheric reaeration. Special mechanisms for the effect of flow reaeration over dams are included, which can calculate reaeration values for fractional flow over and around partial dams and other obstructions (Brown and Barnwell, 1987). The primary internal DO sink is BOD, which is converted by the model to the ultimate carbonaceous biochemical oxygen demand (CBOD) of the stream. Sediment oxygen demand (SOD) is also incorporated into the model calculations (U.S. EPA Office of Water, 1995). However, QUAL2E's intricate stream DO mass balance also considers many additional processes, which are linked within the QUAL2E model with the predicted DO concentration, as is shown in Figure 4-4. Studies have indicated that the QUAL2E model very accurately predicts DO content in comparison with the classic Streeter-Phelps analysis, which has long been considered to accurately represent DO concentrations in streams for data values with limited scattering (McCutcheon, 1989).

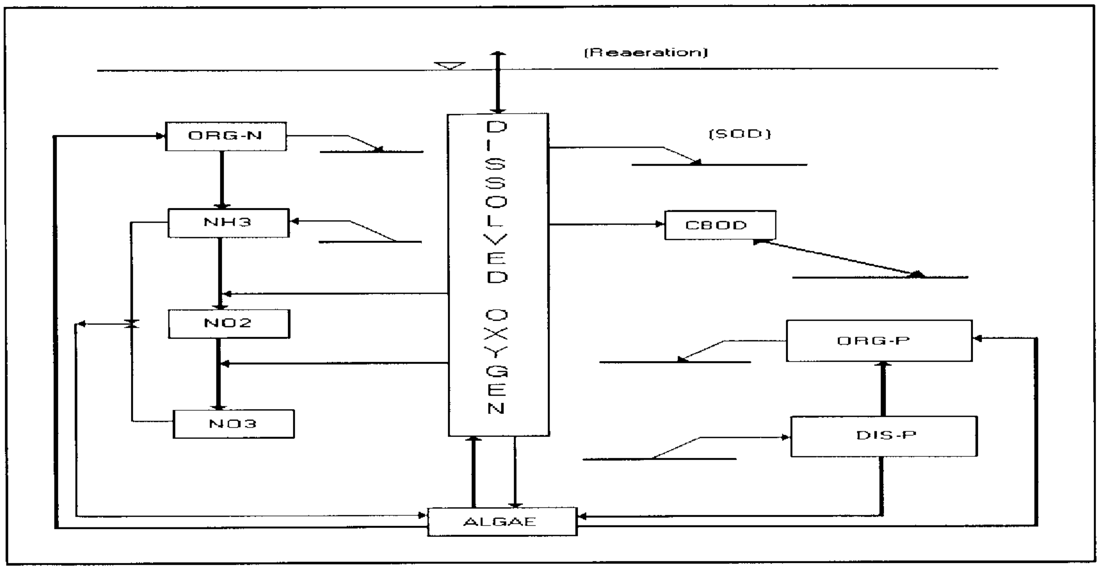


Figure 4-4. QUAL2E Constituent Interactions
Source: Adapted from U.S. EPA Office of Water, 1995.

4.2.5.2 Temperature

In attempts to model the true nature of physical, biological, and chemical processes, temperature relations within the model are significantly interrelated with all impacted processes. The temperature calculations within QUAL2E are modeled by performing a heat balance for each computational element. The balance incorporates air-water interactions including evaporation, convection, and radiation, as well as internal heat sources and sinks. Radiation as a heat source to the interface incorporates cloudiness, reflectivity, diurnal exposure, and scattering. Heat transfer across the soil-water interface at the stream bottom is neglected. The relations with other processes within the model are established via temperature corrections to standard reaction rates. Accurate temperature modeling requires extensive climatological data - input intervals of 3 hours for dynamic simulations or average local values during steady-state simulations. Dynamic simulations require that the climatological data are applied uniformly over the entire stream, whereas steady-state simulations allow data to vary by reach (Brown and Barnwell, 1987).

4.2.5.3 Algae

A very useful component of the QUAL2E model in terms of water quality planning and management applications is the prediction of algae concentrations within streams. The presence of algal blooms is the primary indicator of eutrophication within a water body and the interaction between the algal concentrations and nutrient inputs is a primary consideration in water quality management and planning (Biswas, 1981).

The QUAL2E model utilizes first order modeling equations to predict algae concentrations (NCASI, 1982). In addition, chlorophyll-a concentrations are also used as an indicator of algae biomass (U.S. EPA Office of Water, 1995). Source and sink equations are incorporated into the model to predict specific algal growth, respiration, death, and settling rates. All rate equations except those for settling rates include temperature corrections (NCASI, 1982). Algal growth rates are known to be strongly dependent upon the availability of nitrogen and phosphorus and light. Within the model, the nutrient limitation factors are calculated using Monod expressions. The QUAL2E assumes that algae utilize ammonia and nitrate as a source of inorganic nitrogen, and that dissolved inorganic phosphorus is removed by algae and returned to the system as an organic phosphorus source as a result of algae death. Additional information on QUAL2E phosphorus and nitrogen cycle are described in Sections 4.2.5.4 and 4.2.5.5.

Algal-light relationships incorporate light intensity and determine attenuation affects on algal growth through one of three available methods: a Monod half-saturation method, Smith's function, or Steele's equation (Brown and Barnwell, 1987). As a result of the strong dependence upon light inputs, algal growth predictions require daily climatological data for accurate simulations (U.S. EPA Office of Water, 1995). The effects of light averaging from the climatological data and algae self shading are also addressed within the model. The interaction among the algal growth factors is modeled by QUAL2E by three different methods: multiplicative, limiting nutrient, and harmonic mean (Brown and Barnwell, 1987).

4.2.5.4 Phosphorus Cycle

The phosphorus relationships included in QUAL2E are widely known for accurately predicting the effects of phosphorus inputs from wastewater treatment plants (WWTP). The model phosphorus cycle contains two compartments that calculate dissolved inorganic phosphorus and organic phosphorus sources and sinks (U.S. EPA Office of Water, 1995). The capability to simulate both dissolved phosphorus and organic phosphorus is a significant enhancement contained within QUAL2E over previous versions of the model. The transformation between both forms of phosphorus within the model is governed by differential equations. As previously mentioned, algae and phosphorus interactions are intricately related within the model and algal uptake of phosphorus is considered by QUAL2E to be the primary phosphorus sink mechanism within the stream (Biswas, 1981). In terms of internal phosphorus sources, the model incorporates both the release of phosphorus from respiring algae and the benthos source rate within the stream into phosphorus concentration calculations (NCASI, 1982).

4.2.5.5 Nitrogen Cycle

Although the nitrogen and phosphorus cycles utilize many similar basic mechanisms within QUAL2E, the nitrogen cycle is more complex than the phosphorus cycle and consists of four components: organic nitrogen, ammonia nitrogen, nitrite nitrogen, and nitrate nitrogen (U.S. EPA Office of Water, 1995). As with the phosphorus cycle, the conversion from one form of nitrogen to another is governed by differential equations within the model, and algal uptake is also incorporated into the nitrogen cycle as an internal sink (Biswas, 1981). The QUAL2E model considers algal uptake of nitrogen to affect mostly ammonia; therefore, an algal preference factor for this form of nitrogen has been incorporated as a primary sink mechanism within the ammonia component of the nitrogen cycle. The model also contains provisions that limit nitrification processes at locations where the model predicts low DO concentrations prevail. This mechanism increases QUAL2E's capability of accurately modeling water quality behavior in response to wastewater discharges to streams (Brown and Barnwell, 1987).

4.3 QUAL2E Calibration Process

As was stated above, modeling exercises commonly lack adequate field data for calibration and validation purposes, and even complete data sets cannot provide all the insight necessary to assign appropriate values to all model parameters. Often, parameters that lack data must be determined from ranges of reported values present in various modeling publications for similar water body characteristics and conditions (Rines and Shanahan, 1993). Due to the time constraints on the scope of this study, many QUAL2E calibration parameters were determined through extensive literature research. Multiple sources were consulted and the recommended ranges of reported values were carefully examined to determine the appropriate coefficients with insight from field studies of rivers similar to the Assabet River. The unique characteristics of the Assabet were taken into consideration as values were adjusted to provide a starting point for calibration refinements.

4.3.1 Calibration Data

In addition to literature values, QUAL2E was calibrated to the Assabet River utilizing water quality data compiled in the Assabet River Basin 1986 and 1987 Water Quality and Wastewater Discharge Data Reports (Hanley, 1987). These reports represent the documentation of an extensive water quality sampling effort that resulted in one the most comprehensive Assabet River water quality data sets recorded by the Massachusetts Department of Environmental Quality Engineering (DEQE). The extensive water quality sampling effort was performed in order to evaluate the performance and water quality response of the most recent round of WWTP upgrades along the Assabet and the integration of the Shrewsbury WWTP flows into the new Westborough WWTP. As a result, the 1987 data included all of the recent WWTP upgrades to plants that discharge to the Assabet River. The nature of the Assabet River wastewater discharges have remained virtually unchanged over the past 12 years; therefore, calibration to the data set should provide accurate model predictions for future water quality as a result of water quality management policies applied by the Town of Acton.

4.3.1.1 River Flow and Water Quality Data

The model was run for conditions existing on July 22, 1987, in coordination with the most complete water quality data collected in the 1987 report. The 1987 Assabet River water quality data were collected from 25 locations along the Assabet River, most of which were located in water quality problem areas, such as downstream from WWTPs and in dam impoundments (see Figure 4-5). River flow data from the 1987 report were also utilized in the model calibration exercise. The flow data were collected by the MDEQE from several sites with a Pygmy meter and from the United States Geological Survey (USGS) Assabet River flow gaging station located at Maynard and the Nashoba Brook station located in Acton. Numerous water quality data were collected that were useful in calibrating the model to the Assabet River. The water quality data extracted at each sampling station pertinent to the calibration exercise included temperature, pH, DO, BOD5, fecal coliform, total-Kehldahl-N, ammonia-nitrogen, nitrate-nitrogen, phosphorus, and algae concentrations. Additional water quality data were obtained from a sampling program established by the Organization for the Assabet River (OAR) that has monitored similar water quality parameters in the Assabet River for the last several years.

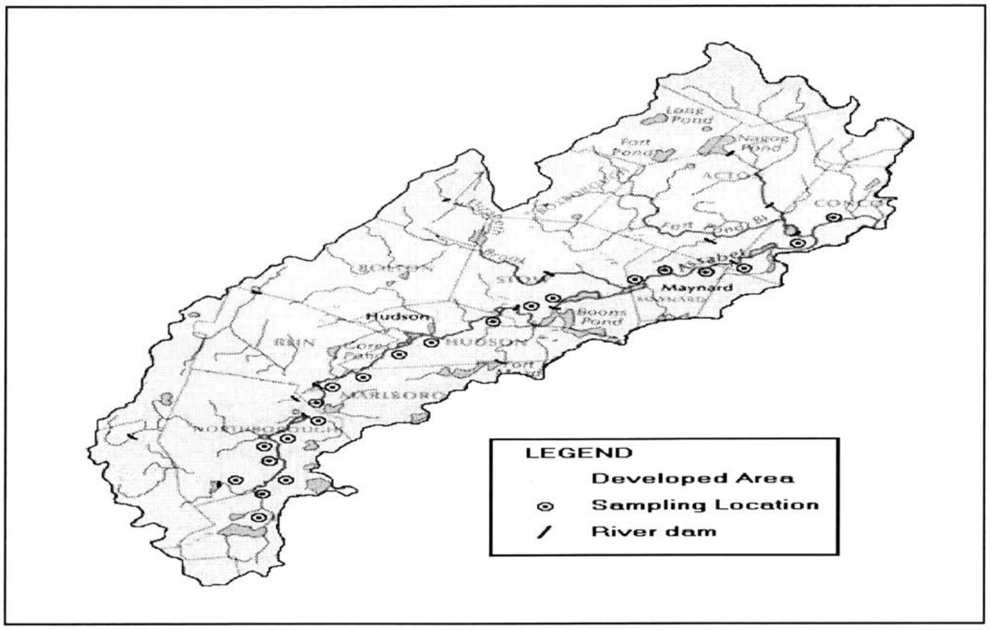


Figure 4-5. 1989 Water Quality Management Plan Water Quality Data Sampling Locations
Source: Hanley, 1989.

4.3.1.2 Climatological Data

The July 1987 climatological data for the calibration exercise were obtained from the National Climatic Data Center (NCDC) for the Logan International Airport in Boston, Massachusetts. The required data for steady-state simulation of river temperature and water quality conditions included dry bulb and wet bulb temperature, wind speed, cloud cover, and barometric atmospheric pressure. All parameter values consisted of average daily values and were assumed to apply to all reach segments. All data values were included in the NCDC data set, with the exception of cloud cover. The cloud cover value for model calibration was calculated from percent possible sunshine on July 22, 1987 with an equation developed by the Tennessee Valley Authority that was obtained from Shanahan (1984).

4.3.2 QUAL2E Hydraulic Parameters

The Assabet River and its tributaries were split into a total of 27 reaches of varying lengths that exhibited uniform water quality characteristics (see Figure 4-6). The average reach length was just under two miles per reach, and almost all reaches were less than or equal to the reach length of 3.2 miles recommended by McCutcheon (1989) for adapting QUAL2E to sluggish rivers like the Assabet. The reaches within the model were divided into computational elements of 0.2 miles, the smallest length allowed due to the model's maximum of 20 computational elements per reach. The United States Geological Survey (USGS) Marlborough, Hudson, and Maynard 7.5x15 minute quadrangle maps were utilized in the river discretization process, and emphasis was given to carefully assigning dam, point source, and junction computational elements to

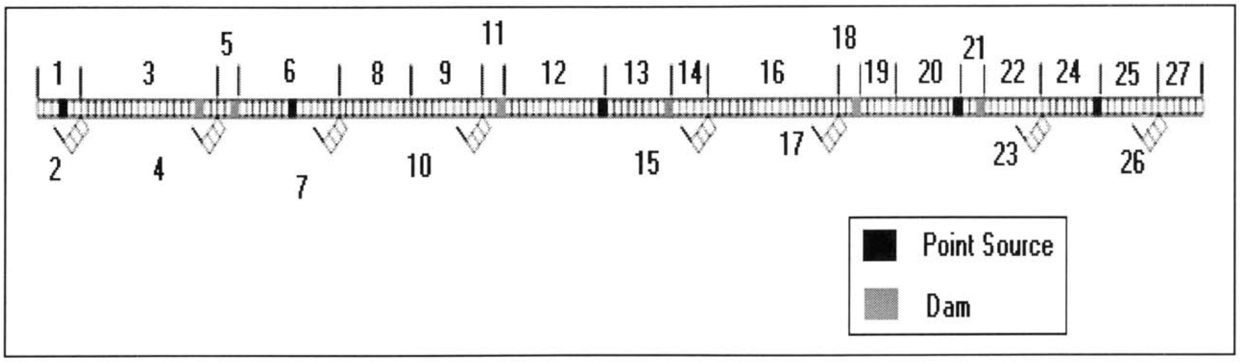


Figure 4-6. Assabet River Calibration Discretization

accurately represent the Assabet River within QUAL2E. Each reach was given a descriptive name representative of the significant water quality processes occurring within the reach that created the incentive for the resulting river discretization. A summary of the reach characteristics is provided in Table 4-1.

Reach Number	Reach Name	Tributary	Start (miles)	End (miles)	Point Source	Dam
1	Westborough WWTP		32.0	30.8	31.4	
2	Hocomonco Stream	X	0.6	0.0		
3	Route 20 Dam		30.8	27.0		27.6
4	Cold Harbor Brook	X	0.6	0.0		
5	Allen Road Dam		27.0	26.4		26.6
6	Marlborough WWTP		26.4	23.6	25.0	
7	Cooledge Brook	X	0.6	0.0		
8	After Cooledge		23.6	21.6		
9	Before Hog Brook		21.6	19.6		
10	Hog Brook	X	0.6	0.0		
11	Route 85 Dam		19.6	19.0		19.2
12	Hudson WWTP		19.0	16.2	16.4	
13	Gleasondale Dam		16.2	14.4		14.6
14	After Gleasondale		14.4	13.4		
15	Fort Meadow Brook	X	0.6	0.0		
16	Between Brooks		13.4	9.6		
17	Elizabeth Brook	X	0.6	0.0		
18	American Woolen Dam		9.6	9.0		9.2
19	Third Fast Flow		9.0	8.0		
20	Maynard WWTP		8.0	6.2	6.4	
21	Powder Mill Impoundment		6.2	5.6		5.8
22	To Second Division		5.6	4.0		
23	Second Division Brook	X	0.6	0.0		
24	Concord MCI		4.0	2.4	2.6	
25	Concord		2.4	1.2		
26	Nashoba Brook	X	0.6	0.0		
27	Final Segment		1.2	0.0		

Table 4-1. QUAL2E Reach Characteristics

Since QUAL2E is limited to nine junctions, the nine primary tributaries along the river were identified and incorporated into the model with identical reach lengths of 0.6 miles. Assabet River flow data for July 1987 were limited to four sample locations within the river. As a result, a water balance was performed between the measured flow rates in which groundwater recharge to the Assabet River was assumed negligible. Tributary inflows and point source discharges were assumed to be the sources for all flow gains between stations. Although tributary flow data for July 1987 were not available, the inflows from each tributary were estimated as a proportion of tributary drainage area in the water balance using the drainage area ratio method (Tappi Press, 1988) as shown in Table 4-2. The area ratio method was used to calculate flow inputs for all but the Nashoba Brook and Second Division Brook tributaries. The Nashoba Brook flow data were obtained from the USGS flow gaging station located at Acton, and the Second Division Brook flow data were estimated from both drainage area and adjacent tributary flow estimates. With the calculations and assumptions listed above, the flow regime in Figure 4-7 was created within

Tributary	*River Mile Location (miles)*	**Drainage Area (square miles)**	**Estimated Flow Input (cfs)**
Hocomonco Stream	31.0	1.3	1.1
Cold Harbor Brook	27.0	4.3	4.0
Cooledge Brook	23.8	5.3	5.0
Hog Brook	19.8	2.8	2.6
Fort Meadow Brook	14.6	1.2	1.1
Elizabeth Brook	11.6	4.0	3.7
Second Division Brook	6.8	1.8	1.1
Fort Pond/Nashoba Brook	5.6	3.8	1.5

Table 4-2. Flow Contribution from Primary Tributaries of the Assabet River

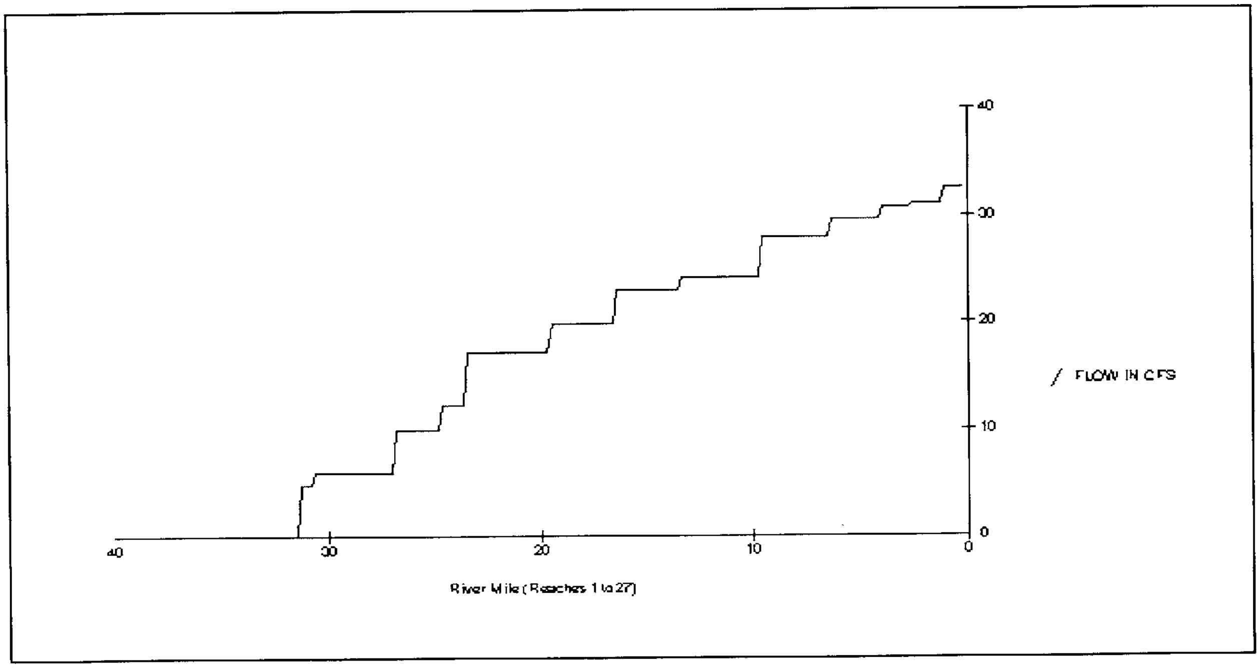


Figure 4-7. Assabet River Sreamflow Profile

QUAL2E. With the predetermined flow rates, hydraulic velocity and depth coefficients were assigned from my observation to accurately represent the Assabet River at the outset of calibration. River velocities were set to values in accordance with slow moving river velocities noted by McCutcheon (1987), and were adjusted for slow (approximately 0.5 feet per second) and fast (approximately 1.0 feet per second) reaches of the Assabet River noted by Hanley (1989). Likewise, Assabet River and tributary depth and width characteristics were adjusted to observed values. Although tributaries were largely unobserved during this time frame, they were assumed to be meandering tributaries of average depth and width resulting from vegetative obstruction, and were assigned a velocity of approximately 0.3 feet per second and a depth to width ratio of approximately 1:5. The tributaries were also assigned a Manning's roughness coefficient of 0.025, which accounts for channel vegetative obstructions.

Impoundments were noted from both the USGS maps and from visual observations of the Assabet River during a January surveillance of the river at all dam locations. All significant dam impoundments within the Assabet River were assigned an individual reach that consisted of three computational elements for a total length of 0.6 miles upstream of the dam. This characterization method was consistent with all impoundment observations with the exception of the Powder Mill Impoundment, which was noted to stretch approximately one mile from the High Street Dam. Very small velocities (approximately 0.1 feet per second) were assigned to the impoundments and an effort was made to adjust the width of the river to the observed width of the controlling dam. In the Powder Mill Impoundment reach, the velocity was set even lower at approximately 0.03 feet per second to accurately model the large volume of water impounded behind the dam. The combination of standard assigned reach velocities and slow impoundment velocities are also in agreement with total travel time studies conducted by Cooperman and Jobin (1971) for the Massachusetts Water Resources Commission (MWRC), which determined the overall average river velocity as 0.23 feet per second.

Dispersion coefficients for reach segments were determined from literature values for similar sluggish rivers as collected by Fischer et al., (1979) and are shown in Table 4-3.

Reach	**Type**	**Measured River**	**Depth (ft)**	**Width (ft)**	**Velocity (ft/s)**	**Dispersion Coefficient (ft^2/sec)**
2,3,4,7,8,9 10,12,13,14	Slow	Clinch River	1.90	118	0.69	87
1,6	Fast	South Platte	1.5	--	2.17	174
15,16,17,2022,23,24,2526,27	Slow	Clinch River	2.79	154	1.05	151
19	Fast	Copper Creek	2.79	59	1.97	226
5,11,18,21	Impoundment	Powell River	2.79	112	0.49	102

Table 4-3. QUAL2E Reach Dispersion Coefficients

Similar to the tributaries, the Manning's roughness coefficient was set to 0.025 for the first three reaches of the Assabet River due to observed weeds and vegetation in the low river flows of the first few river miles. All other reaches were set to 0.02, which accounts for rocks and small flow obstructions within the channel.

4.3.3 Biochemical Oxygen Demand and Dissolved Oxygen Rate Constants

BOD calibration to the 1987 data set showed considerable BOD loading to the Assabet River from both Hog Brook and Elizabeth Brook. The BOD decay values ranged from 0.15 day^{-1} to 3.6 day^{-1} in order to match the decay rates shown in Assabet River sampling. These values were within the range of values presented in literature decay rate tables in Bowie et al. (1985). The BOD settling rate values were estimated as negligible for the modeled flowrates with insight from a QUAL2E modeling study of the Blackstone River in Southern Massachusetts (Shanahan, 1989). The sediment oxygen demand (SOD) at the beginning of the river was assumed to be large due to the inefficiencies of both the old Westborough WWTP and Shrewsbury WWTP prior to the construction of the new Westborough WWTP in 1987. These SOD values were assumed to be at the upper range of SOD values presented in Thomann and Mueller (1987) for river reaches near and below sewage discharges, which resulted in DO levels that accurately fit the calibration data. The middle and lower Assabet River SOD values were assumed to be negligible due to a lack of evidence of high SOD values in the calibration data. The reaeration prediction method within QUAL2E utilized an equation devised by O'Connor and Dobbins to calculate the stream rearation rate as recommended by Covar (1976) for slow shallow streams.

4.3.4 Nitrogen, Phosphorus, and Algae Coefficients

Organic nitrogen hydrolysis rates were concluded from examination of several nitrogen transformation rate values reported in the literature and compiled by the U.S. EPA (1997). The approximate value observed in over half of all studies was 0.03 per day, which was entered into the model. The NH_3 and NO_2 oxidation values were determined from a set of nitrification studies in the EPA Technical Guidance Manual (1997) that utilized oxidation data collection as a method of determining nitrification rates. The NH_3 and NO_2 oxidation rates were assumed from Shanahan (1989) to be 0.45/day and 1/day for all reaches, respectively. The NH_3 and dissolved phosphorus benthos source rates per square foot of river bottom were assumed to be negligible with guidance from both the U.S. EPA Technical Guidance Manual (1997) and Shanahan (1989). In addition, the algae settling coefficient was determined to be approximately 3 ft/day. Nonalgal light extinction coefficients were taken as the low end of the range of total light extinction values for rivers referenced in the EPA Technical Guidance Manual (1997). The manual was also used as a reference for the ratio of chorophyll-a to algae, which was estimated to be approximately 16 µg/1000 mg of algae.

4.3.5 Headwater Source Data

Due to a lack of complete data from the headwater station, the headwater source data were obtained as a combination of data from the first and second sampling stations, AS01 and AS02, for the July 22, 1987 data set. Both stations are located upstream of the initial tributary and point source inputs, and as such should provide a good representation of the headwater water quality. Station AS01 was located near the headwaters of the river near the outlet of the George H. Nichols Multipurpose Dam, while station AS02 was located within one mile of the dam. The majority of the data were collected from station AS02, as the sampling effort collected very few data from station AS01.

4.3.6 Wastewater Treatment Plant Point Source Loads

Inputs to the Assabet River from WWTPs were well documented within the 1987 report. The 1987 sampling data for all WWTPs included flow rates, temperatures, BOD5, total Kheldahl nitrogen (TKN), NH_3-N, NO_3-N, total phosphorus, and fecal coliform values. DO concentrations in the WWTP effluents were not collected, and were assumed to be similar to 1987 NPDES permit limits for the treatment plants. Although July 22, 1987 effluent data were not available for the Concord MCI plant, the discharge from the plant was assumed relatively constant and the effluent data recorded from the plant on September 1, 1987 were used to include the plant's minor effluent in the modeling exercise. DO concentration data or NPDES limits were not available for this plant; therefore, the effluent DO was assumed to be 6.0 mg/L in accordance with reported wastewater concentrations.

As recommended by Bowie et al., (1985), total phosphorus concentrations were considered to be half organic phosphorus and half dissolved inorganic phosphorus. Organic nitrogen inputs for the model were determined from the difference of TKN and NH_3-N per the definition of TKN. Nitrite nitrogen levels were assumed to be negligible in discharges and throughout the rest of the river, as the conversion step from nitrite to nitrate occurs very quickly, and results in predominant levels of nitrate and very little nitrite in the natural environment. Although this intermediate step in the nitrification process is incorporated into QUAL2E, many models do not even include the process, and instead calculate the direct transformation of ammonia to nitrate (Bowie et al., 1985). Algae concentrations in wastewater effluents were assumed to be negligible due to chlorination.

4.3.7 Dam Reaeration Coefficients

Dam reaeration coefficients for the Butts and Evans dam reaeration equation utilized within QUAL2E were determined from the model value guidelines (Brown and Barnwell, 1987) for conditions assumed as moderately polluted to grossly polluted waters that flow over mostly flat, broad-crested weirs. All dams along the Assabet River were observed and photographed (Figures 4-8 to 4-13) during field trips to determine Assabet River hydrologic characteristics.

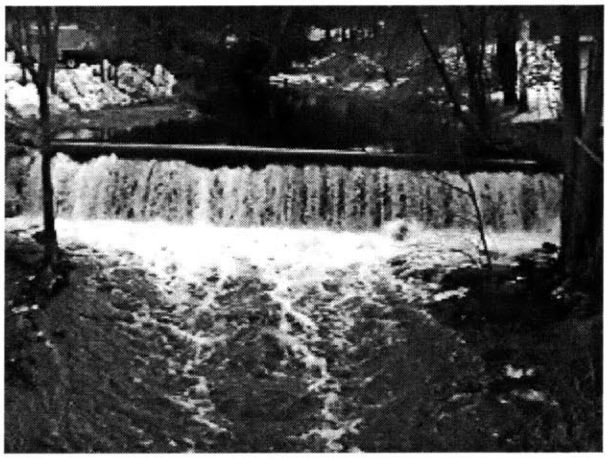


Figure 4-8. Route 20 Dam at Northborough

Figure 4-9. Allen Road at Northborough

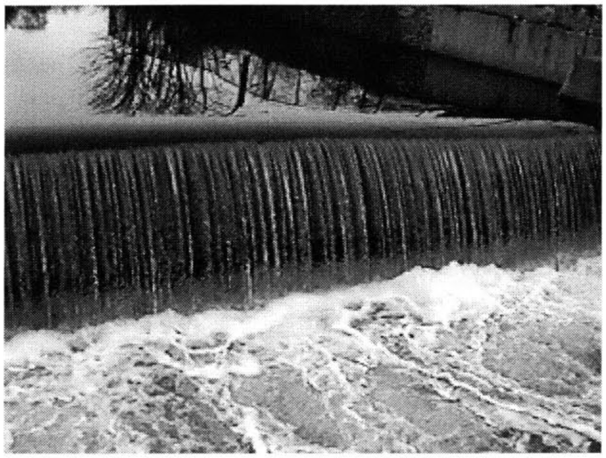


Figure 4-10. Route 85 Dam in Hudson


Figure 4-11. Gleasondale Dam

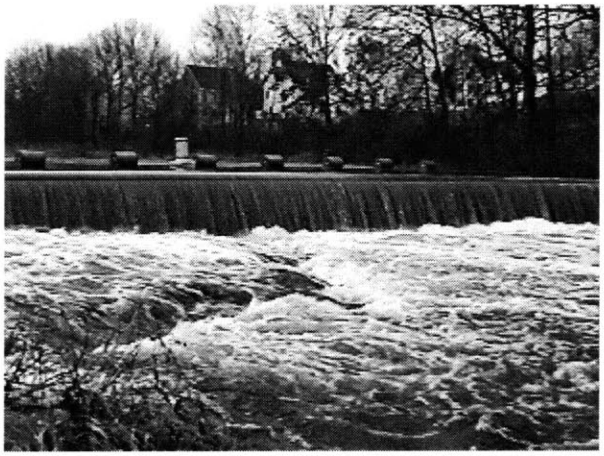


Figure 4-12. American Woolen Dam in Maynard

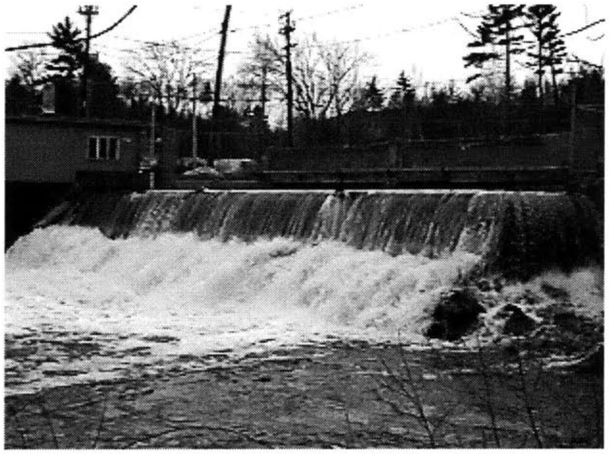


Figure 4-13. High Street Dam in Maynard

From these observations, flow drop heights and percent of flow over each dam were estimated and the resulting coefficient values are shown in Table 4-4.

Dam	a	b	Flow Over Dam (%)	Dam Height (ft)
Route 20	0.65	0.6	100	7
Allen Road	0.65	0.8	100	9
Route 85	0.65	0.6	100	8
Gleasondale	0.65	0.6	100	7
American Woolen	0.65	0.8	100	6
High Street	0.65	0.7	100	11

Table 4-4. Coefficients for the Butts and Evans Dam Reaeration Equation

4.3.8 QUAL2E Calibration Results

As was previously described, QUAL2E calibration to the Assabet River involved numerous coefficient adjustments and literature research, as well as some assumptions based upon the available water quality information. Figures 4-14 through 4-17 display the results of QUAL2E calibration to the July 22, 1987 data set collected by the MDEQE.

As can be seen in Figure 4-14, the BOD decay slope varies throughout the Assabet River. However, matching the QUAL2E BOD decay to the Assabet River was achieved without deviating from literature BOD decay coefficient guidelines. The numerous WWTP discharges and the BOD inputs from Hog Brook and Elizabeth Brook are responsible for the periodic BOD jumps. The general trend is such that BOD decay occurs rapidly enough to assimilate a WWTP discharge before accepting the next discharge.

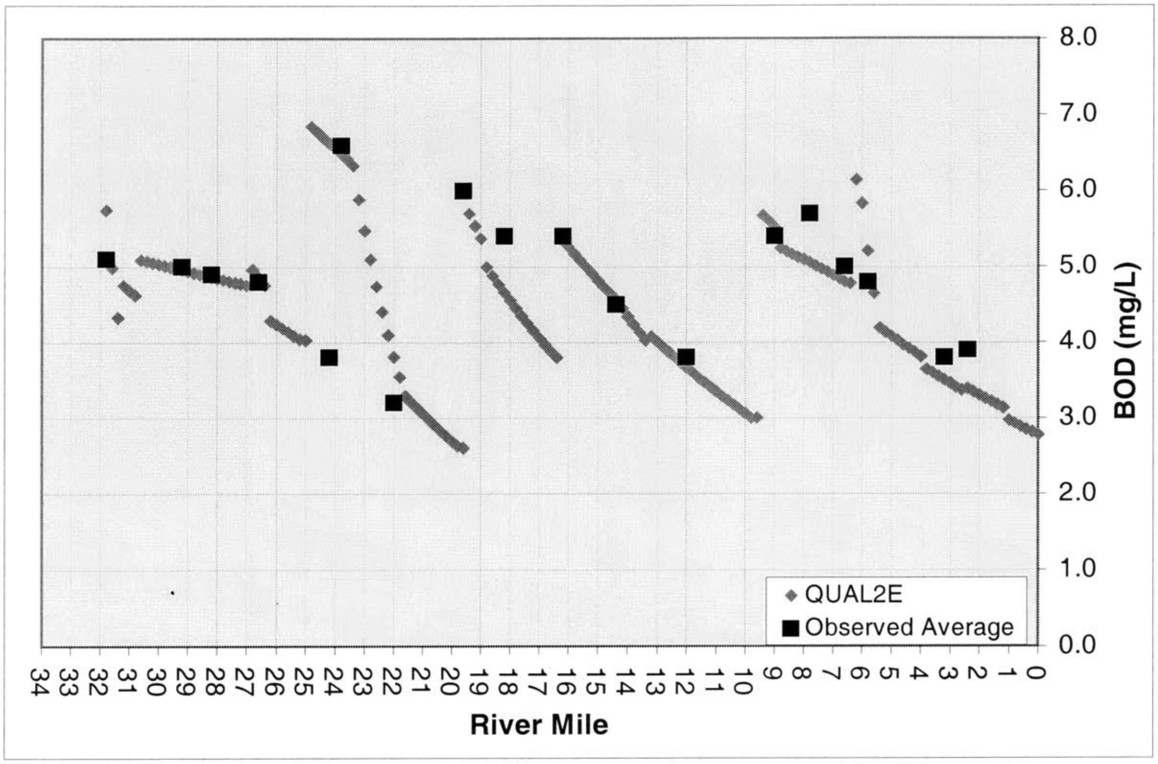


Figure 4-14. QUAL2E BOD Calibration Plot

Figure 4-15 on the following page displays the resulting DO levels from the BOD trends shown above. The general trend of predicted DO values from QUAL2E matched all DO concentration trends. The initial DO sag (mile 32 to 28) is the result of high SOD values within the water quality data near the retired Westborough and Shrewsbury WWTPs. The MDEQE reports and past records of these plants suggest that high SOD exists as a result of inefficient operation of these WWTPs prior to the Westborough WWTP upgrade and subsequent Shrewsbury tie-in. The SOD coefficients used in QUAL2E fell within the upper range of values listed by Thomann and Mueller for SOD downstream of WWTP discharges. Dam reaeration is responsible for nearly all DO jumps shown on Figure 4-15, which provided from approximately 1 mg/L up to 3 mg/L of DO per dam. These values agreed with a dam reaeration study performed by Cooperman and Jobin (1971) on the Assabet River dams, as well as with DO sampling performed by Hanley (1987) both upstream and downstream of the dams. Overall, observation of the DO levels also shows that the QUAL2E DO concentrations were mostly in agreement with the average DO levels determined from water quality samples collected from the Assabet River. However, lower Assabet River QUAL2E DO values tended to calibrate with the minimum sampled DO levels with the known DO inputs; therefore, QUAL2E predictions for these reaches should provide worst-case scenarios in terms of DO concentrations for all QUAL2E water quality predictions.

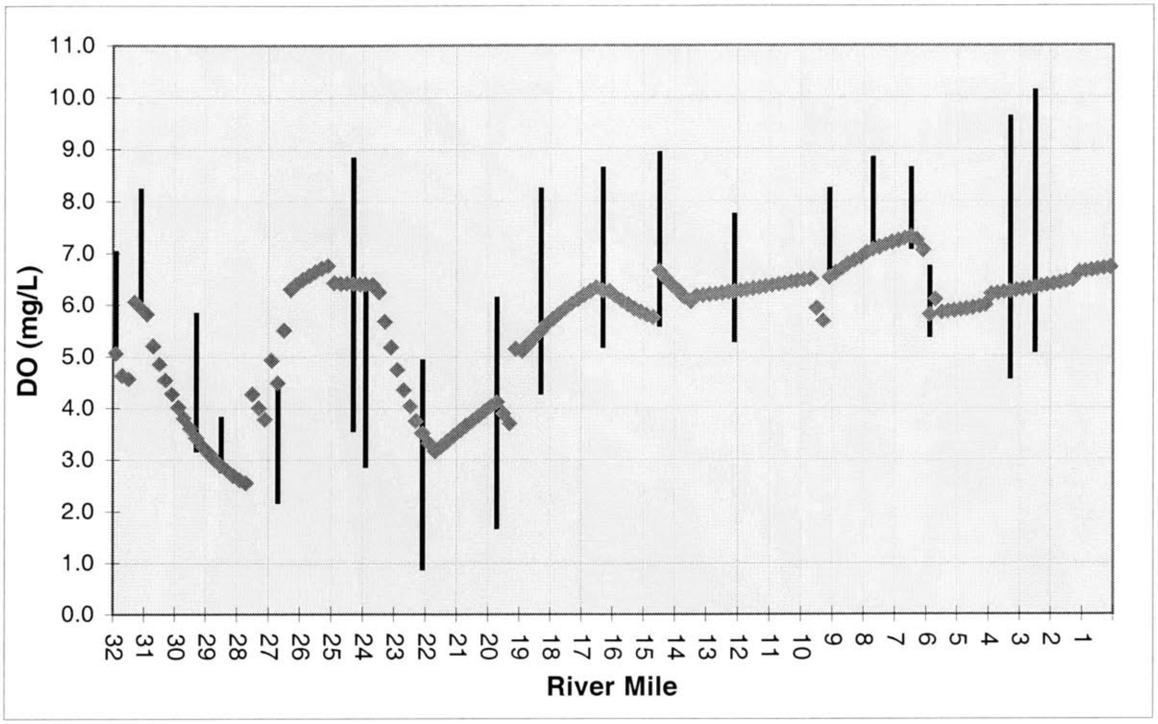


Figure 4-15. QUAL2E DO Calibration Plot

Figure 4-16 displays the results of QUAL2E calibration to total phosphorus concentrations

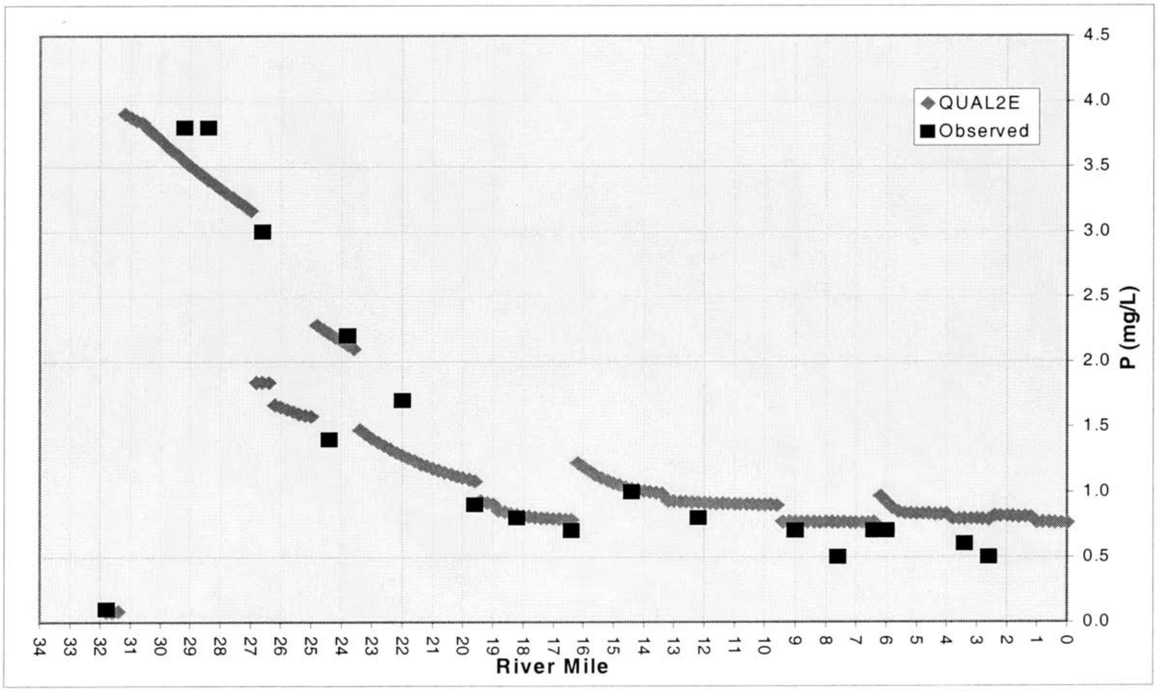


Figure 4-16. QUAL2E Total Phosphorus Calibration Plot

observed in the Assabet River from the MDEQE water quality data. Due to the varying effects of indirect discharges on WWTP phosphorus phases, the WWTP inputs were considered to be composed of both organic and dissolved inorganic phosphorus. The QUAL2E predicted phosphorus concentrations were relatively insensitive to the algal uptake at the calibrated algae concentration values. Therefore, the organic phosphorus settling rate was adjusted by reach to accommodate for increased settling in impoundments and decreased settling in turbulent sections. Once these adjustments were completed, the phosphorus calibration process was fairly straightforward as the initial phosphorus trends were very similar to the observed Assabet River phosphorus data values.

Figure 4-17 displays the related results of the QUAL2E predicted algae concentration in comparison with the MDEQE observed values. In both cases, algae concentration measurements were represented as chlorophyll-a concentrations in milligrams per cubic meter. Although few observed data points existed for QUAL2E calibration, the existing data suggest a trend of constant algae concentration after the initial high algae concentrations of the Assabet River swampy headwaters are diluted by the chlorinated discharge from the Westborough WWTP. The Assabet River algae concentrations remain uniform until reaching the Powder Mill Impoundment, where the Maynard WWTP phosphorus inputs and stagnant conditions spawn abundant algae growth.

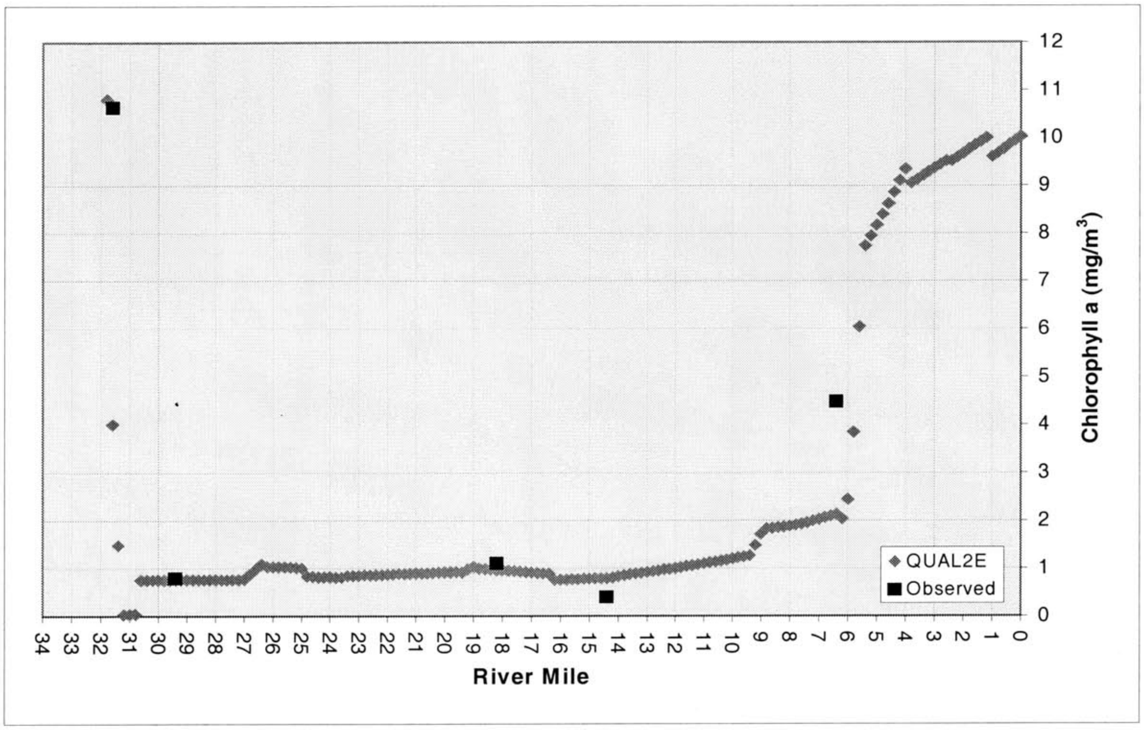


Figure 4-17 QUAL2E Algae Concentration Calibration Plot

4.4 *QUAL2E Assabet River Water Quality Predictions*

The calibrated QUAL2E stream water quality model was run for a number of hypothetical Acton WWTP phosphorus discharges and Acton BMP phosphorus loading reduction scenarios. In all cases, the NPS phosphorus reductions from Acton urban runoff obtained by BMPs were applied as phosphorus reductions to Fort Pond Brook and Nashoba Brook, both of which discharge to Warner Pond (See Figure 4-18). The phosphorus loading reductions for both brooks were applied to a single modeled tributary at the outlet of Warner's Pond, and were adjusted to account for the suspected nutrient catchment capability of Warner's Pond noted in the Town of Acton Trading Program Discussion Draft (1998). In the following modeling scenarios, Warner's Pond was assumed to be a "black box" (See Figure 4-19) that removes phosphorus from the tributary influent.

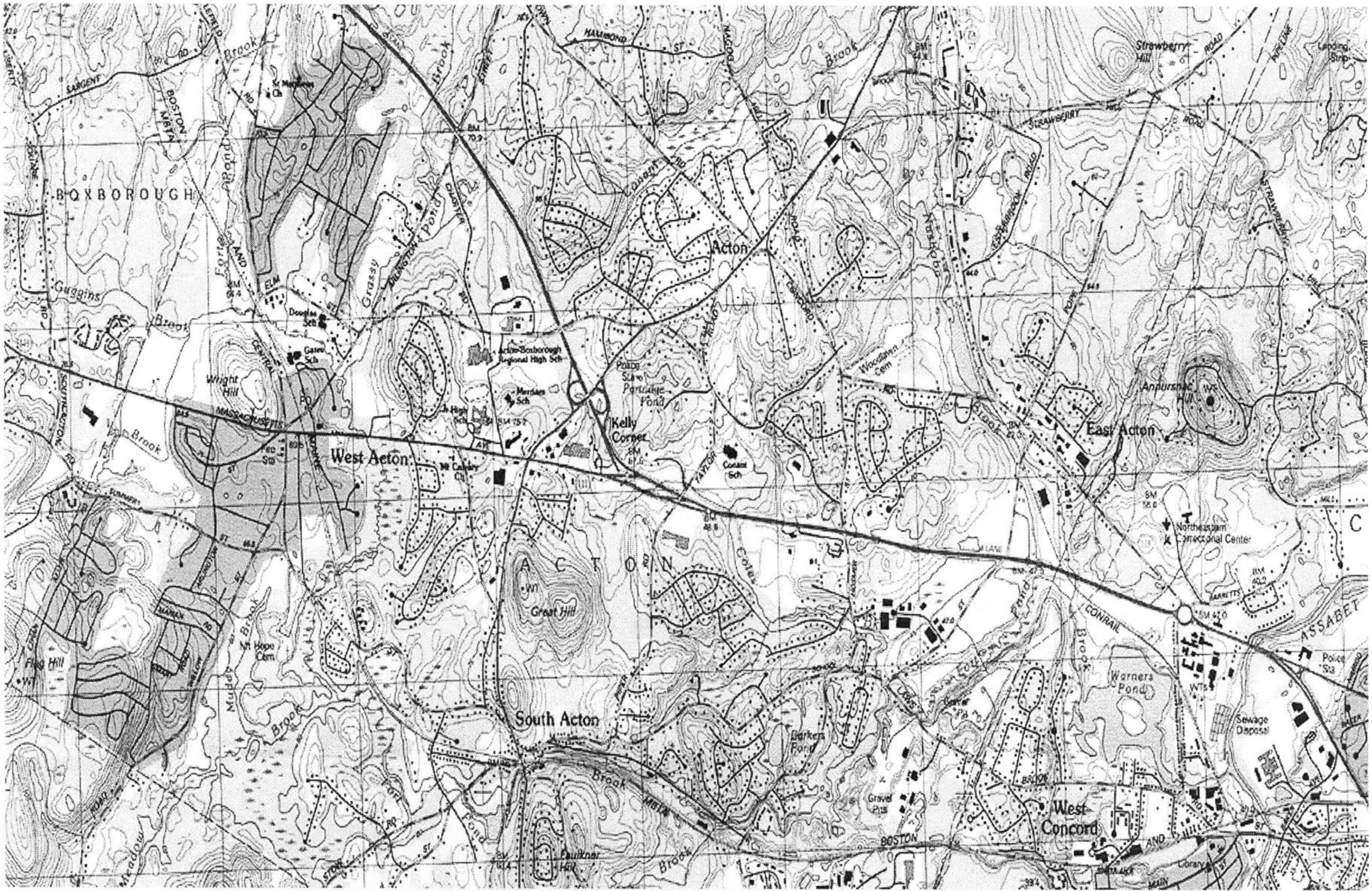


Figure 4-18. USGS Map Showing Acton Tributaries, Warner's Pond, and the Assabet River.
Source: Topo USGS Digital Map Software.

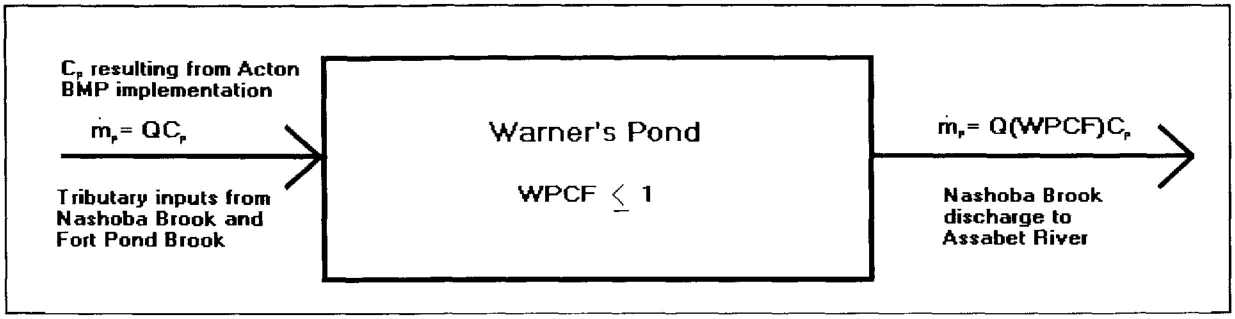


Figure 4-19. Schematic of Warner's Pond Correction Factor Application

As can be seen in Figure 4-19, the effluent phosphorus concentrations from the pond were calculated via multiplication by an assumed Warner's Pond Correction Factor (WPCF). As a result, phosphorus loading to the Assabet River from Warner's Pond was reduced by the WPCF fraction of the achieved Acton BMP phosphorus loading reductions to Fort Pond Brook and Nashoba Brook.

4.4.1 Scenario I

This model scenario represents the best-case situation for phosphorus loading reductions from the Town of Acton and the best WPCF scenario in terms of Assabet River water quality improvement. As specified in the Town of Acton Trading Program Discussion Draft (1998), this correlates to a 66 percent reduction of phosphorus from Acton urban runoff. As has been seen in the previous NPS modeling analysis, this is a possible outcome of the implementation of various BMPs throughout Acton. For this scenario, the Warners Pond correction factor (WPCF) was assumed to be unity. Therefore, Warners Pond was assumed to have no effect on Fort Pond Brook and Nashoba Brook phosphorus loading to the Assabet River. As a result, the Fort Pond Brook and Nashoba Brook phosphorus loading was represented in QUAL2E as 34 percent of the 0.07 mg/L value consistently observed in the OAR water sampling data, or 0.024 mg/L. In addition, the proposed Acton WWTP discharge of 590,000 gallons per day was incorporated into the model. The effluent phosphorus concentration was assumed to be 0.1 mg/L, which represents the lowest achievable concentration also noted by the Town of Acton Discussion Draft, and substantiated by the Acton WWTP design described in Section 5.

The QUAL2E model results for this phosphorus loading scenario are shown in shown in Figure 4-20. The impact of the maximum phosphorus reduction is negligible, as the difference in the original and after phosphorus concentrations are only slightly perceptible over the last 6 miles of the Assabet River. Likewise, the Assabet River algae concentrations as chlorophyll-a (Figure 4-21) were also barely changed by the Acton WWTP addition and BMP phosphorus reductions on the Fort Pond/Nashoba Brook model tributary. The primary impacts observed in Figure 4-21 are the dilution of Assabet River algae concentrations by the chlorinated Acton WWTP discharge and a uniform reduction in algal growth resulting from the slight reduction in river phosphorus concentrations. The addition of the Acton WWTP slightly increased BOD levels in the Assabet River (Figure 4-22), which corresponded with slightly lower DO concentrations (Figure 4-23) downstream of the Acton WWTP discharge. Overall, the downstream DO levels were reduced by approximately 0.3 mg/L by the 0.2 mg/L BOD increase downstream of the Acton WWTP.

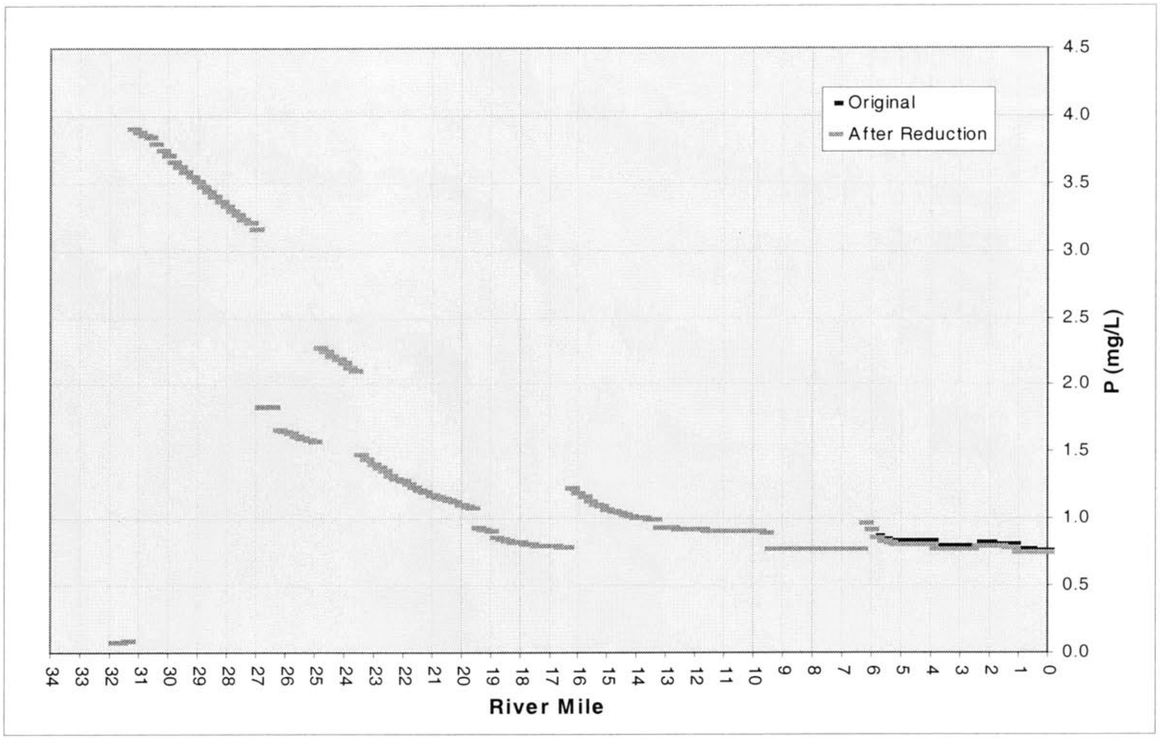


Figure 4-20. Comparison of Original and Scenario I Phosphorus Concentrations

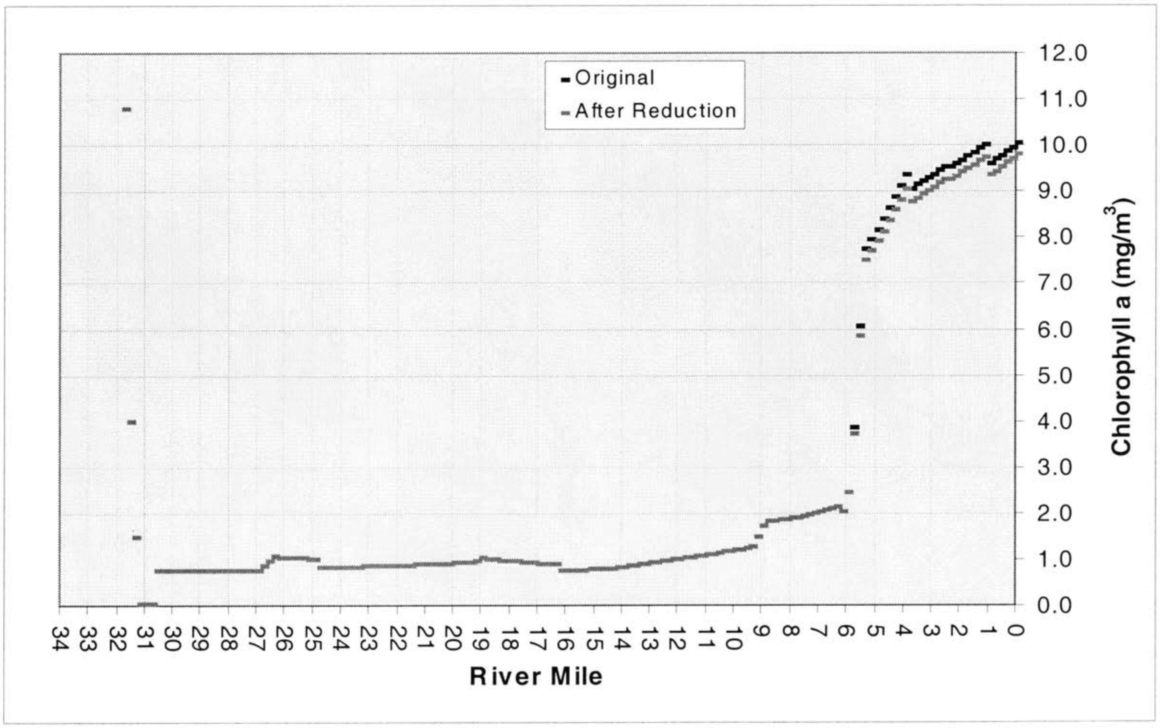


Figure 4-21. Comparison of Original and Scenario I Algae Concentrations

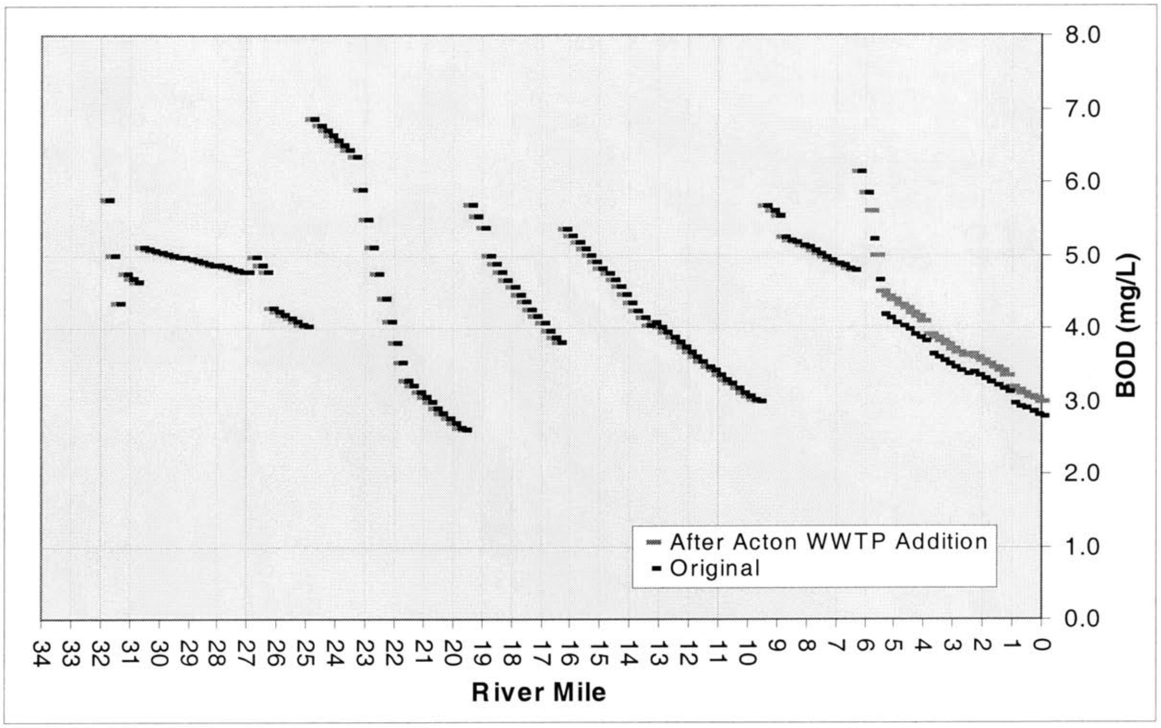


Figure 4-22. Comparison of Original and Scenario I BOD Concentrations

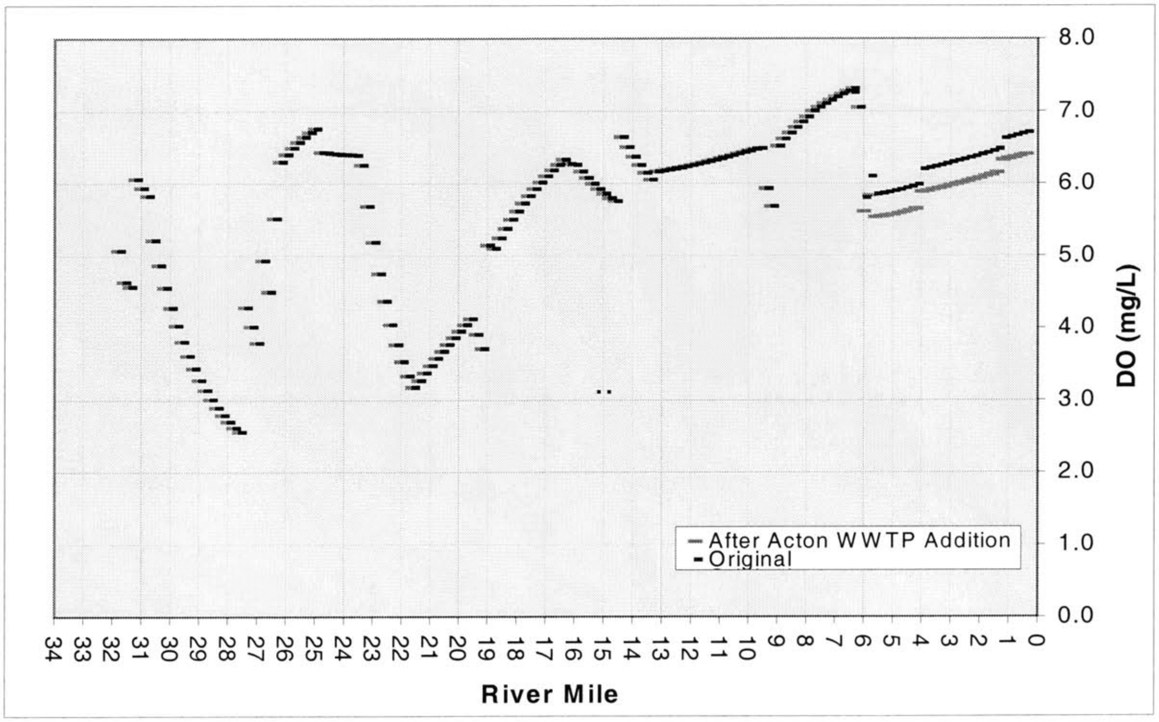


Figure 4-23. Comparison of Original and Scenario I DO Concentrations

4.4.2 Scenario II

This model scenario represents the best-case situation for phosphorus loading reductions from the Town of Acton, but an average scenario for the impact of Warners Pond on Fort Pond Brook and Nashoba Brook. The phosphorus reduction from Acton runoff was again assumed to be the optimal 66 percent. However, for this scenario, the Warner's Pond correction factor (WPCF) was assumed to be 0.5. Therefore, after being adjusted with the assumed Warner's Pond correction factor, the actual phosphorus loading to the Assabet River would be reduced by approximately 33 percent. This change was reflected in the model by adjusting the phosphorus concentration in the Fort Pond/Nashoba Brook model tributary from the 0.07 mg/L value consistently observed in the OAR water sampling data to 0.047 mg/L. The proposed Acton WWTP discharge of 590,000 gallons per day was incorporated into the model with an assumed target concentration of 0.1 mg/L.

The QUAL2E model results for this phosphorus loading scenario are shown in shown in Figure 4-24. Similar to Scenario I, the impact of the maximum phosphorus reduction is negligible, with the original and after-reduction phosphorus concentrations virtually superimposed, and the only perceptible difference in loading changes occurring downstream of the Acton WWTP.

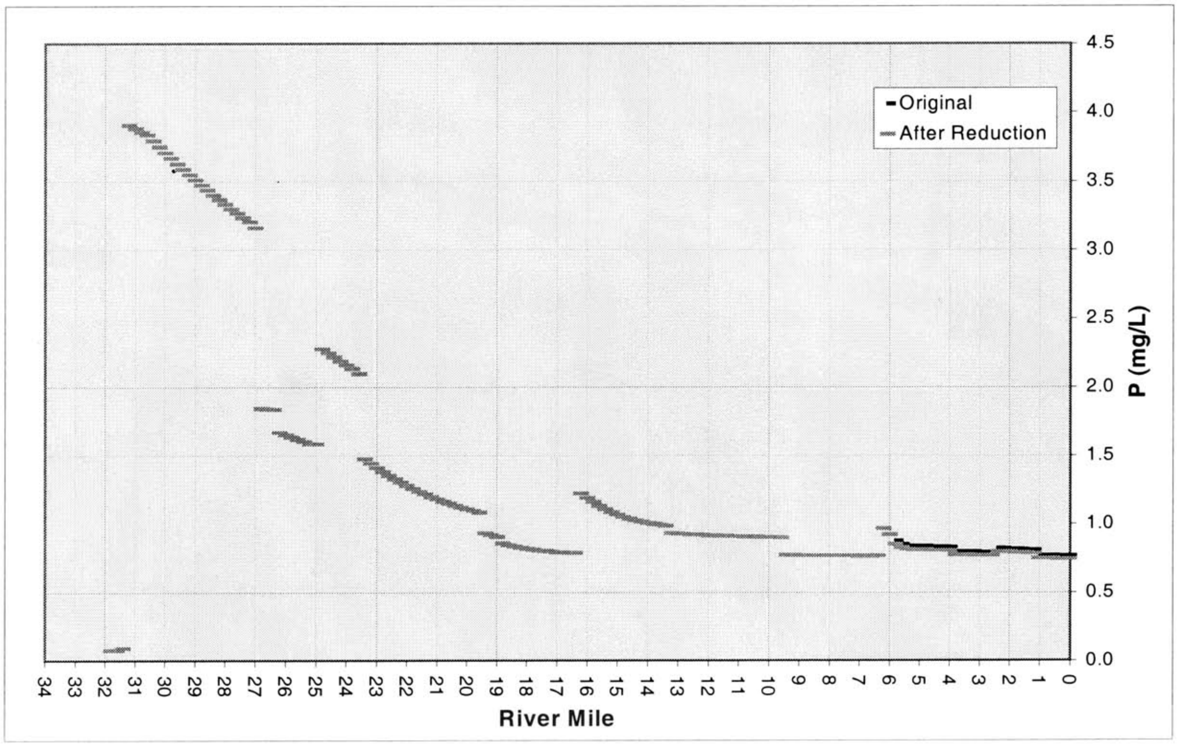


Figure 4-24. Comparison of Original and Scenario II Phosphorus Concentrations

Figure 4-25 on the following page displays the algal concentrations resulting from the phosphorus loading changes. Similar to Scenario I, the algae concentration reductions from this scenario are also minimal and due to both dilution and reduced river phosphorus concentrations.

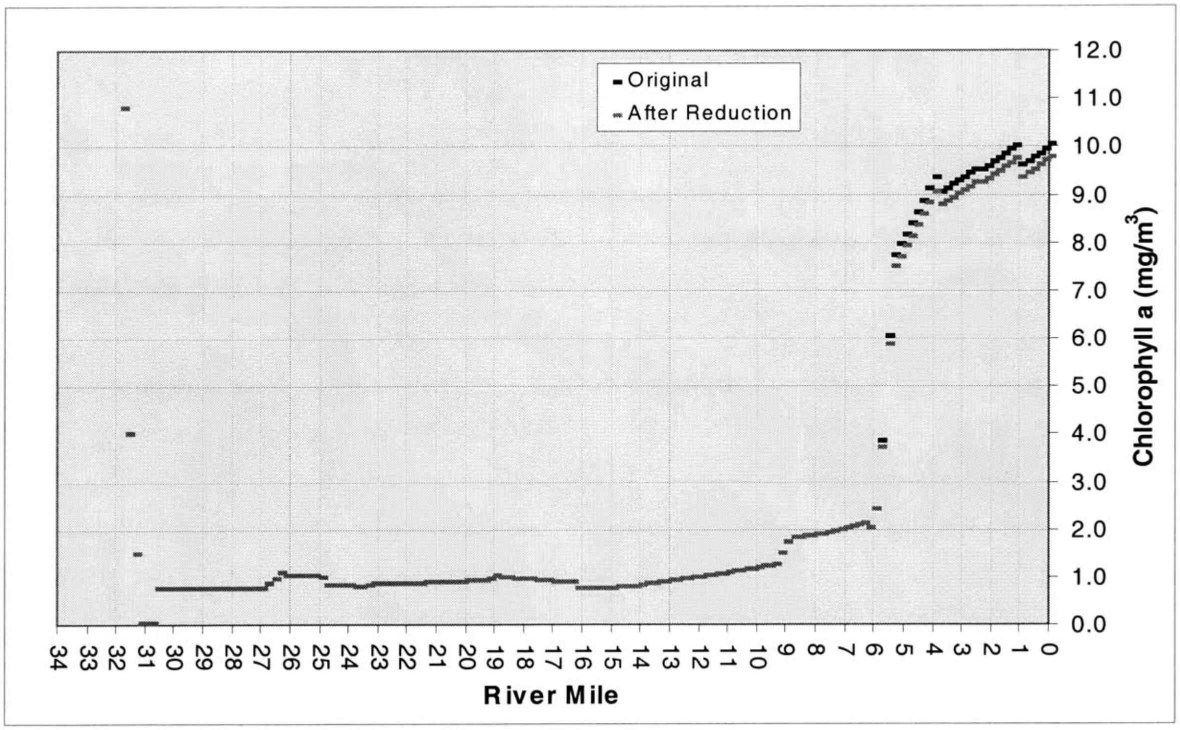


Figure 4-25. Comparison of Original and Scenario II Algae Concentrations

Predicted BOD and DO levels for this modeling scenario were identical to those for Scenario I shown in Figure 4-22 and Figure 4-23, respectively. This would be expected due to the use of the same BOD loading from the Acton WWTP in QUAL2E for Scenarios I and II, and the identical DO sources and sinks resulting from equivalent algae concentrations in both scenarios.

4.4.3 Scenario III

This model scenario represents the worst-case situation for phosphorus loading reductions from the Town of Acton, which correlates to a 10 percent reduction of phosphorus from Acton urban runoff. Therefore, after being adjusted with the assumed WPCF of 0.5, the actual phosphorus loading to the Assabet River would be reduced by only 5 percent. This change was reflected in the model by adjusting the phosphorus concentration in the Fort Pond/Nashoba Brook model tributary from the 0.07 mg/L value consistently observed in the OAR water sampling data to 0.067 mg/L. The proposed Acton effluent phosphorus concentration was assumed to be 0.2 mg/L, which represents the upper end of the concentration range goal noted by the Town of Acton Discussion Draft.

The QUAL2E model results for this phosphorus loading scenario are shown in Figure 4-26. Once again, the impacts on total phosphorus concentrations downstream of the proposed Acton WWTP are minimal, even with the 0.2 mg/L effluent concentration and virtually ineffective removal of NPS phosphorus from Acton. Figure 4-27 also shows the identical algae trend as seen before in Scenarios I and II, where the dilution and reduced phosphorus concentrations downstream of the Acton WWTP discharge slightly reduce the overall Assabet River algae concentrations.

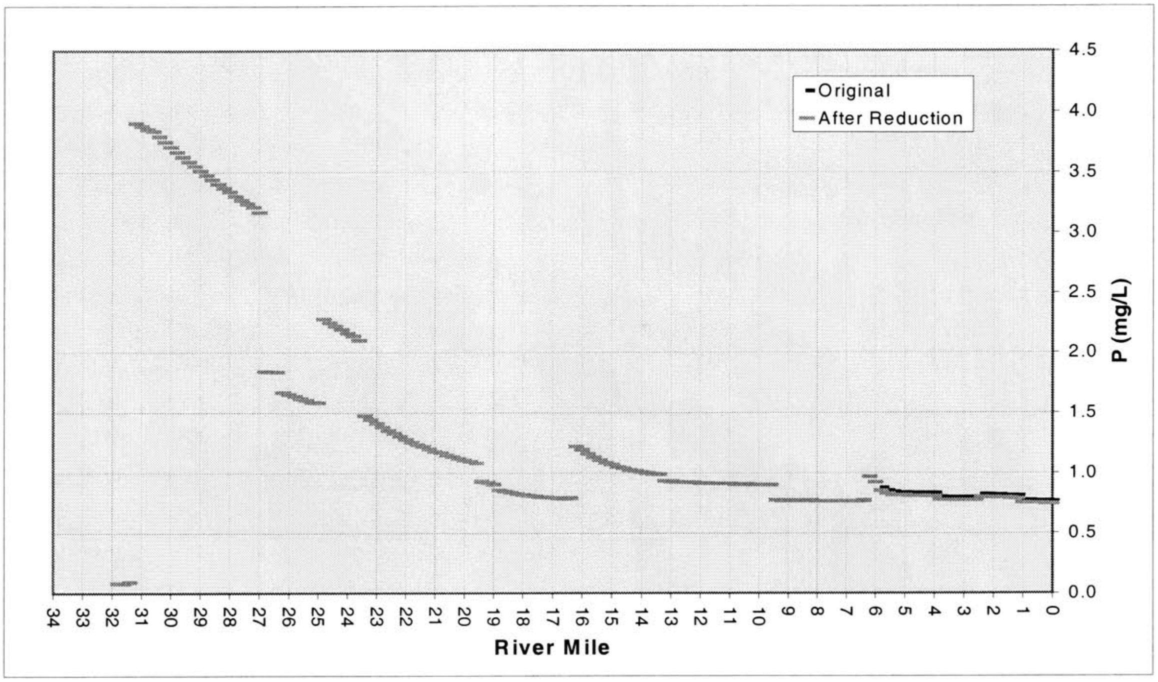


Figure 4-26. Comparison of Original and Scenario III Phosphorus Concentrations

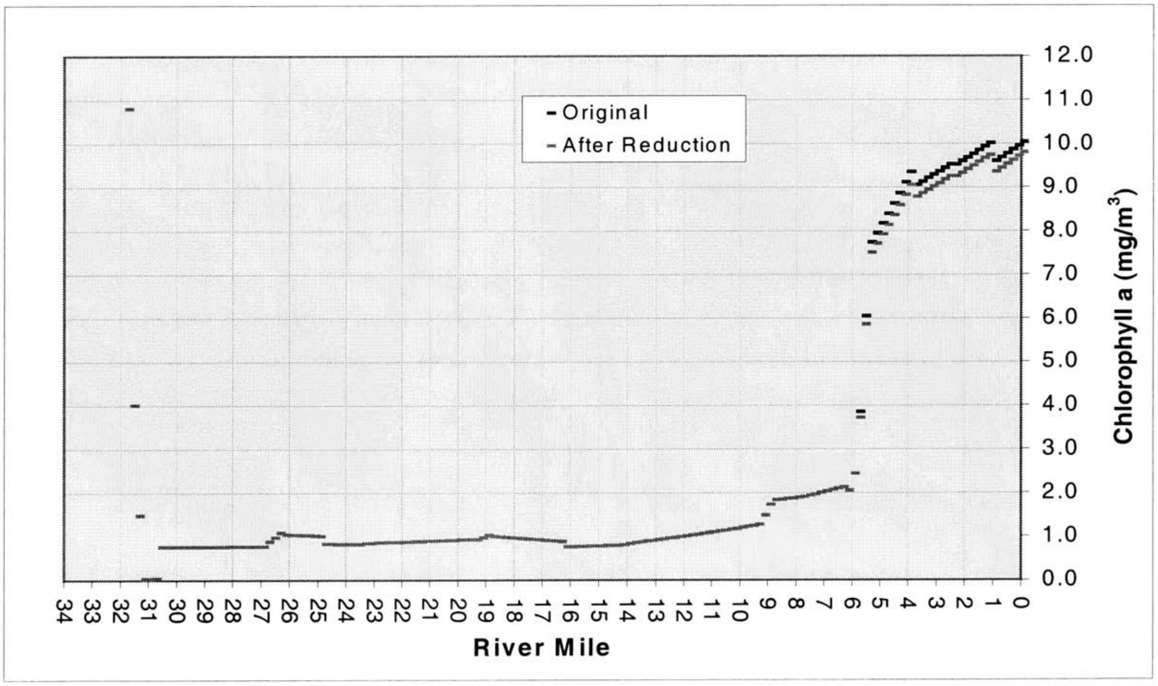


Figure 4-27. Comparison of Original and Scenario III Algae Concentrations

Predicted BOD and DO levels for this modeling scenario were also identical to those predicted for Scenarios I and II shown in Figure 4-22 and Figure 4-23, respectively.

4.4.4 Scenario IV

This scenario predicts the impacts of a hypothetical point/point source phosphorus trade with the Westborough WWTP located at the headwaters of the Assabet River rather than the Acton urban nonpoint/point source trade that has been examined thus far. This trade is well within the bounds of the current use of watershed-based pollution trading as examined in Section 2 of this report. Due to the lack of unknowns that accompany nonpoint/point source trades, the guidelines for a trade of this type would be much easier to manage. Although the financial costs required to reduce the Westborough WWTP phosphorus effluent concentrations were not examined under the scope of this study, the reductions required to achieve a 3:1 trading ratio are very small. Due to the comparatively large Westborough WWTP discharge rate, a reduction of the Westborough WWTP effluent phosphorus concentration of 1 mg/L would achieve a 5:1 phosphorus loading reduction ratio if the proposed Acton WWTP effluent were allowed to have a 1 mg/L phosphorus concentration. As a result, this scenario reduced the Westborough WWTP phosphorus effluent concentration by 1 mg/L and allowed the Acton WWTP to discharge at a phosphorus concentration of 1 mg/L.

The impacts of the point/point source trade on the predicted Assabet River phosphorus concentrations are shown in Figure 4-28. The large reduction in phosphorus loading from the Westborough WWTP at the headwaters of the Assabet River can be seen throughout the entire length of the river. As a result, this phosphorus reduction scenario achieves a lower overall phosphorus level downstream of the Acton WWTP than the previous three scenarios, while allowing a much greater effluent phosphorus concentration from the Acton WWTP.

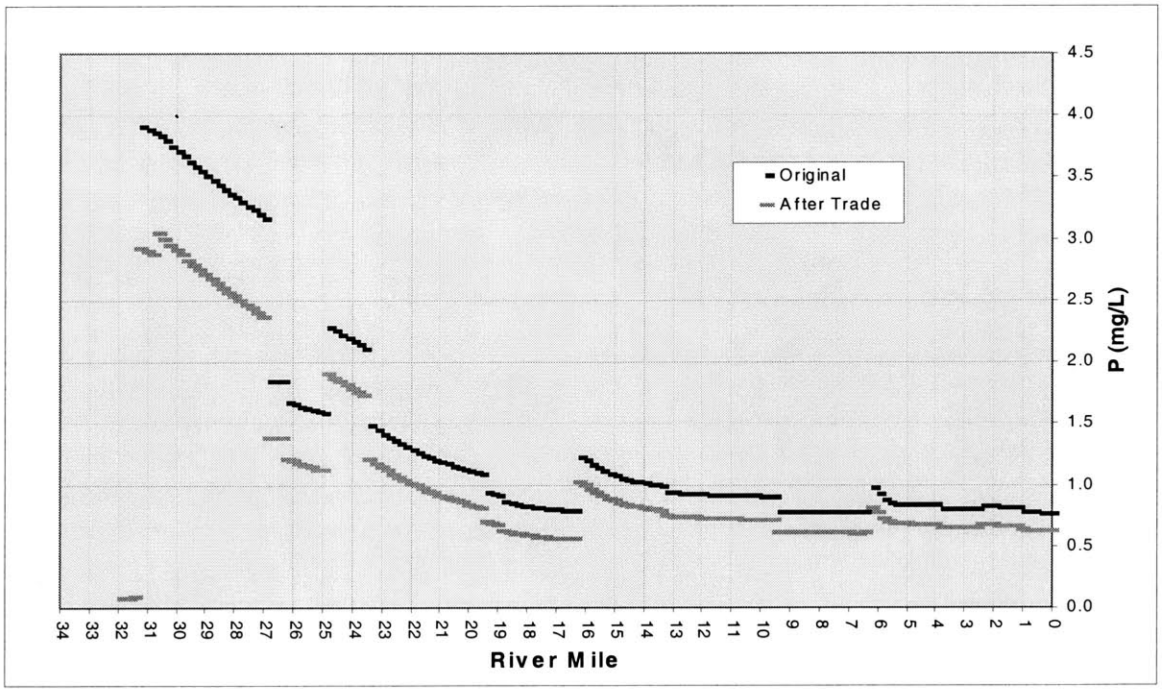


Figure 4-28. Comparison of Original and Scenario IV Phosphorus Concentrations

Figure 4-29 on the following page shows the predicted algae concentrations from this modeling

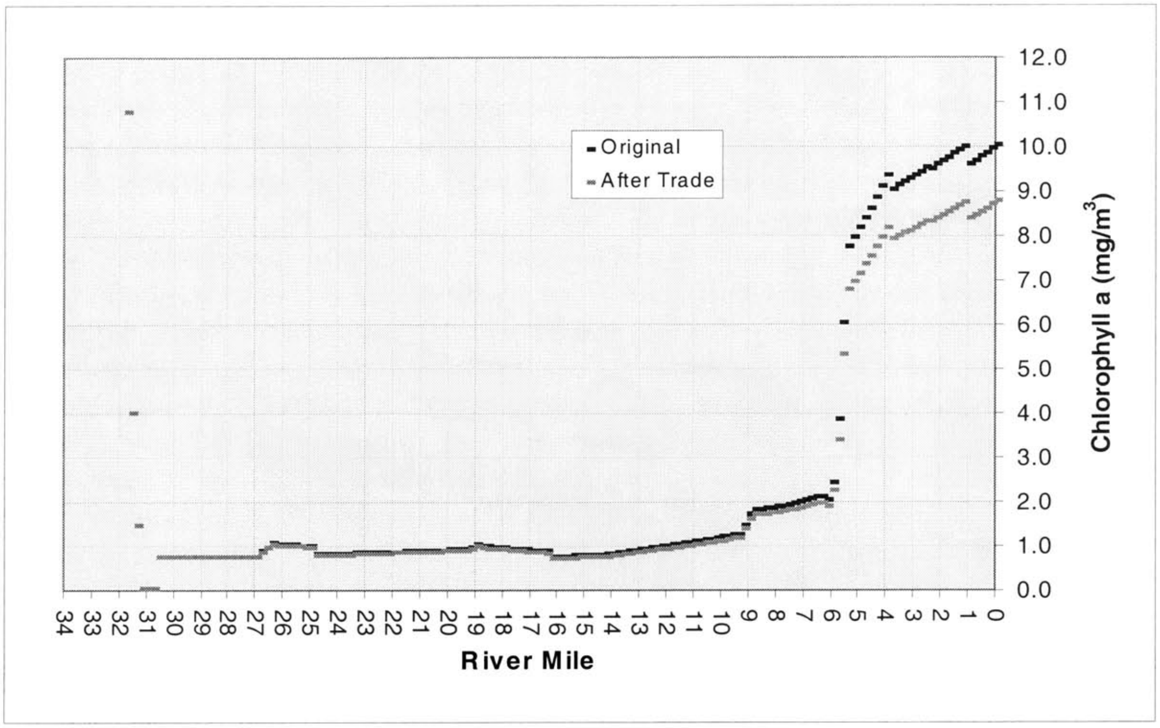


Figure 4-29. Comparison of Original and Scenario IV Algae Concentrations

scenario. Most of the river maintains sufficient phosphorus concentrations to allow the original algae concentrations to prevail. However, the reduced phosphorus concentration in this scenario does inhibit algal growth in and downstream of the Powder Mill Impoundment by approximately 1 milligram per cubic meter. Once again, the predicted BOD and DO concentrations associated with these changes were relatively unchanged from those of the previous three scenarios.

4.5 Conclusion

As can be seen in Figures 4-20 to 4-27, the predicted Assabet River Scenarios I, II, and III show little improvement in Assabet River phosphorus and algae concentrations for even extended efforts by the Town of Acton to reduce urban NPS phosphorus loading from its city limits. These model predictions are to be expected upon examination of the inconsequential phosphorus loading from both Nashoba Brook and the proposed Acton WWTP to the Assabet River (Table 4-5). As was previously mentioned, the OAR data show consistently low phosphorus

Phosphorus Input	Phosphorus Concentration (mg/L)	Flow rate (m^3/d)	Phosphorus Loading (kg/d)
Fort Pond/Nashoba Brook	0.07	3,670	0.17
Westborough WWTP	4.00	11,786	47.0
Acton WWTP @ 0.1 mg/L	0.10	2,225	0.22
Acton WWTP @ 0.2 mg/L	0.20	2,225	0.45

Table 4-5. Overall Phosphorus Loading Summary

concentrations in Nashoba Brook near the Assabet River junction that have hovered around 0.07 mg/L. As can be seen in Table 4-5, at that phosphorus concentration and with a flow rate recorded by the USGS Nashoba Brook gaging station of 3670 m^3/d, the phosphorus loading from Nashoba Brook is only 0.17 kg/d. Similarly, the proposed phosphorus loading from the Acton WWTP is also very small. The Acton WWTP, as proposed with an optimal effluent phosphorus concentration of 0.1 mg/L and a designated flowrate of 2225 m^3/d, would constitute a 0.22 kg/day additional phosphorus load to the Assabet River. When compared with the nearly 50 kg/day loading capacity of the Westborough WWTP alone, these phosphorus loads become insignificant in the overall Assabet River phosphorus loading scheme. Therefore, regardless of the Acton BMPs efficiencies that reduce phosphorus concentrations in Nashoba Brook and the ultimate phosphorus concentration of the Acton WWTP discharge, the phosphorus loading reduction efforts are unlikely to have a substantial impact on improving water quality in the Assabet River.

The minor nature of the Nashoba Brook phosphorus load to the Assabet River also provides that the assumptions made concerning the affect of the Warner's Pond Correction Factor (WPCF) on the Acton BMP phosphorus reductions are of little consequence in the predicted Assabet River concentrations as well. This conclusion is evidenced by the minimal changes observed between Scenario I (WPCF = 1) phosphorus concentrations (Figure 4-20) and algae concentrations (Figure 4-21) and its Scenario II (WPCF = 0.5) respective counterpart concentrations in Figures 4-24 and 4-25. Therefore, for the purpose of this analysis, Warner's Pond also has little impact on the predicted phosphorus and algae concentrations in the Assabet River due to urban BMP implementation in Acton.

Further examination of the phosphorus loads outlined in Table 4-5 reveals that Nashoba Brook consistently contributes less phosphorus to the Assabet River (0.17 kg/day) than the proposed Acton WWTP would at even 0.1 mg/L effluent phosphorus concentration (0.22 kg/day). Therefore, even if all phosphorus could be removed from Nashoba Brook through implementation of Acton phosphorus reduction BMPs, the actual Acton NPS phosphorus loading reduction to Acton WWTP phosphorus discharge trading ratio would not even reach 1:1, let alone 3:1. Therefore, Scenario IV was created as an attempt to ascertain the impact of alternative trading arrangements on water quality in the Assabet River. Figures 4-28 and 4-29 show that phosphorus and algae concentrations are significantly reduced by the phosphorus loading reduction from the Westborough WWTP at the headwaters of the Assabet River.

The modeling scenarios listed above predict that both the proposed Acton WWTP discharge and the use of BMPs to reduce urban NPS phosphorus loading to the Assabet River will have little impact on overall river water quality. This is due to the minor nature of the phosphorus inputs as compared to the numerous municipality WWTP phosphorus discharges upstream of Acton. One perspective of this prediction is that it assures the Town of Acton that a 0.1-0.2 mg/L phosphorus concentration discharge from its proposed WWTP should not significantly harm the Assabet River. The converse perspective is that the water quality of the Assabet River will not likely be improved the Town of Acton's extensive efforts to reduce NPS phosphorus loading to the Assabet River from Acton town limits.

4.6 Recommendation

As was also suggested by the watershed-based trading analysis in Section 2, alternative trading arrangements should be considered if a concerted effort to improve water quality in the Assabet River is to be made. Due to the numerous unknowns that plague point/nonpoint source trades, a point/point source trade with one of the other WWTPs along the river should ensure progress in achieving water quality improvements along the entire length of the Assabet River. Since none of the WWTPs that currently discharge to the Assabet River utilize phosphorus removal processes beyond conventional treatment, the potential exists to significantly reduce phosphorus loading from point sources into the Assabet River through point/point source trades. In addition, the administrative requirements for a straightforward point/point source trade would be much easier to manage than a point/nonpoint source arrangement. Particularly, determination of the phosphorus loading reductions to the Assabet River would be much easier in a point/point source trade due the availability of existing WWTP effluent monitoring. Although the financial costs required to reduce the phosphorus loading from another Assabet River WWTP were not examined under the scope of this study, the substantial flows of the WWTPs provide that the phosphorus concentration reductions required to achieve a 3:1 trading ratio with the Acton WWTP discharge would be very small. In addition, the relaxed Acton WWTP effluent phosphorus concentration limits in this trade would release funds that would have otherwise been dedicated to designing, constructing, and operating an Acton WWTP that can consistently produce an effluent with a 0.1 mg/L effluent phosphorus concentration. These funds could then be utilized to obtain the phosphorus loading reductions at the alternative WWTP necessary to meet the necessary trading ratios to improve water quality conditions in the Assabet River.

4.7 References

Beck, M.B. 1985. Water Quality Management: A Review of the Development and Application of Mathematical Models. Springer-Verlag: New York, NY.

Beck, M.B. and G. van Straten. 1983. Uncertainty and Forecasting of Water Quality. Springer-Verlag: New York, NY.

Biswas, A.K. 1997. Water Resources: Environmental Planning, Management, and Development. McGraw-Hill. New York, New York.

Biswas, A.K. 1981. Models for Water Quality Management. McGraw-Hill. New York, New York.

Bowie, G.L., W.B. Mills, D.B. Porcella, C.L. Campbell, J.R. Pagenkopf, G.L. Rupp, K.M. Johnson, P.W.H. Chan, and S.A. Gherini. 1985. Rates, Constants, and Kinetics Formulations in Surface Water Quality Modeling. 2nd ed. EPA/600/3-85/040, U.S. Environmental Protection Agency, Athens, GA.

Brown, L.C. and T.O Barnwell, Jr. 1987. The Enhanced Stream Water Quality Models QUAL2E and QUAL2E-UNCAS: Documentation and User Manual. U.S. EPA Environmental Research Laboratory. Athens, Georgia. EPA/600/3-87/007.

Cooperman, A.N., and W.R. Jobin. 1971. Report on Pollution of the Assabet River. Massachusetts Water Resources Commission Division of Water Pollution Control Water Quality Management Section: Boston, Massachusetts.

Covar, A.P. 1976. Selecting the Proper Reaeration Coefficient for Use in Water Quality Models, Presented at the U.S. EPA Conference on Environmental Simulation and Modeling, April 19-22, Cincinnati, Ohio.

Fischer, H.B., E.J. List, R.C.Y. Koh, J. Imberger, and N.H. Brooks. 1979. Mixing in Inland and Coastal Waters. Academic Press, New York, NY.

Goodman, A.S. 1970. Potential Benefits of Mathematical Models and Computers for Water Quality Management Programs. Proceedings of the National Symposium on Data and Instrumentation for Water Quality Management. Madison, WI. July 21-23, 1970.

Grimsrud, G.P., E.J. Finnemore, and H.H. Owen. 1976. Evaluation of Water Quality Models: A Management Guide for Planners. U.S. Environmental Protection Agency Office of Research and Development. Washington, D.C.

Hanley, N.E. 1987. "Assabet River Basin 1986-1987 Water Quality Data, Wastewater Discharge Data, and Water Quality Analysis." Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control.

Hanley, N.E. 1989. "Assabet River 1989 Water Quality Management Plan: Parts A, B, and C." Massachusetts Department of Environmental Quality Engineering Division of Water Pollution Control.

McCutcheon, S.C. 1989. Water Quality Modeling: Volume I Transport and Surface Exchange in Rivers. CRC Press, Inc.:Boca Raton, Florida.

McCutcheon, S.C. 1985. Water Quality and Streamflow Data for the West Fork Trinity River in Fort Worth, Texas. Water Resources Investigation Report 84-4330, U.S. Geological Survey.

Melching, C.S. 1996. Key Sources of Uncertainty in QUAL2E Model of Passaic River. Journal of Water Resources Planning and Management. 122(2):105-113. March/April 1996.

National Council of the Paper Industry for Air and Stream Improvement, Inc. (NCASI). 1982. "The Mathematical Water Quality Model QUAL-II and Guidance for Its Use – Revised Version." New York: NCASI Technical Bulletin No. 391.

Rajar, R. and C.A. Brebbia. 1997. Water Pollution IV: Modelling, Measuring, and Prediction. Computational Mechanics Publications. Boston, Massachusetts.

Rines, H., and P. Shanahan. 1993. Water Quality Modeling of the Blackstone River to Evaluate Dissolved Oxygen Impacts of the Ocean State Power Water Withdrawal. Applied Science Associates, Inc. Narragansett, Rhode Island. ASA#91-92. April 1993.

Shanahan, P. 1989. Modeling Study of the Blackstone River Calibration Notes.

Shanahan, P. 1984. Water Temperature Modeling: A Practical Guide. Proceedings of Stormwater and Water Quality Model Users Group Meeting. April 12-13, 1984:273-295. U.S. EPA Environmental Research Laboratory. EPA-600/9-85-003.

Tappi Press. 1988. 1988 Principles of Water Quality Modeling Using the QUAL2E Model Seminar. Charleston, South Carolina. April, 16, 1988.

Thomann, R.V. and J.A. Mueller. 1987. Principles of Surface Water Quality Modeling and Control. Harper and Row. New York, NY.

Town of Acton. 1998. "Watershed Trading Program, Discussion Draft."

United States Environmental Protection Agency Office of Water. 1997. Technical Guidance Manual for Developing Total Maximum Daily Loads. Book 2: Streams and Rivers. Part 1: Biochemical Oxygen Demand/Dissolved Oxygen and Nutrients/Eutrophication. Washington, D.C. EPA823-B-97-002.

United States Environmental Protection Agency Office of Water Office of Science and Technology Standards and Applied Science Division. 1995. QUAL2E Windows Interface User's Guide. U.S.EPA. Washington, D.C. EPA/823/B/95/003.

United States Geological Survey. 1987. Maynard, Massachusetts 7.5x15 minute Quadrangle Topographic Map. 42071-D3-TM-025.

United States Geological Survey. 1987. Maynard, Massachusetts 7.5x15 minute Quadrangle Topographic Map. 42071-C4-TM-025.

United States Geological Survey. 1983. Marlborough, Massachusetts 7.5x15 minute Quadrangle Topographic Map. 42071-C5-TM-025.

United States Geological Survey. 1987. Nashoba Brook Gaging Station at Acton, Massachusetts Streamflow Data.

5 Design of a Wastewater Treatment Plant for Phosphorus Removal

5.1 Background

There are five existing wastewater treatment facilities on the Assabet River upstream of the proposed Acton facility, which have caused the eutrophication of the river. Accelerated eutrophication in the river is due to excess nutrients, phosphorus in particular, being discharged from these point sources. Phosphorus and its effects on the environment are detailed in Section 5.2.

Because of the current situation of water quality in the river, the EPA and the Town of Acton have significant concerns about additional wastewater effluent discharges to the river. The EPA has proposed that an effluent total phosphorus concentration of 0.1 mg/L should be a criterion in order for a new permit application to be considered.

Phosphorus (P) is typically removed from wastewater by chemical precipitation, biological treatment, and/or physical separation. The principles of each treatment process are described in Sections 5.3, 5.4 and 5.5. Chemical precipitates are formed by adding chemicals to produce insoluble salts when combined with soluble phosphate. Phosphorus is also removed in biological treatment by utilizing an anaerobic zone before aeration. The total amount of phosphorus removed is dependent on the amount of solids produced and removed from the wastewater treatment system and on their phosphorus content. Physical separation, also known as filtration, is utilized in effluent-polishing processes to remove phosphorus-containing suspended solids.

5.2 Phosphorus and its effects on the environment

5.2.1 Overview

To address the adverse effects of nutrients on water quality, various programs have been undertaken throughout the world to control nutrients in the effluent from municipal and industrial wastewater treatment plants (WWTPs). These discharges are classified as "point sources" of nutrients (Water Environment Federation, 1998).

Although phosphorus in algal cells occurs in small amounts raging from 0.5 to 1.0% in the biomass, it has been shown to be a limiting factor in the growth of algae. A value of less than 0.005 mg/L or 5 µg/L in the ortho form is recognized as a lower growth-limiting concentration (Federal Water Pollution Control Administration, 1968). Its removal from wastewater is highly feasible because most organic and inorganic forms of phosphorus are readily removed by precipitation with the use of alum, ferric salts, or lime. It can also be removed by enhanced biological phosphorus removal processes. Where land is available, it can be removed by application to cultivated, forested, and pasture areas (Water Environment Federation, 1998).

Typically, nutrient control strategies take advantage of a microorganism's necessity for nitrogen and phosphorus. In this project, phosphorus removal is mainly targeted due to its more contributing role in terms of eutrophication in the Assabet River.

In instances where nutrient control for freshwater is deemed desirable, control of phosphorus is considered to be absolutely essential because when nitrogen becomes limiting, any excess of phosphorus can support growth of nitrogen-fixing, blue-green algae. In such cases, the nitrogen budget of a body of water will be increased, thereby materially offsetting any benefits from nitrogen removal. In marine waters, it is considered desirable to control nitrogen inputs because phosphate is in abundant supply (Water Environment Federation, 1998).

5.2.2 Effect on the environment

Eutrophication is the term used to describe the natural process by which biological productivity increases with the age of a body of water. This typically is the results of capture by phytoplankton and other aquatic growths of plant nutrients contributed by inflowing waters and of the new growth that results. These resultant organisms and plants eventually die and settle to the bottom, where they decompose to some degree. During decomposition, nutrients are released to the water above. These nutrients reach the upper waters or the euphotic zone in time, and this continual enrichment from external and recycled sources perpetually produces new growth that dies and settles to the bottom. Residue from decomposition and silt carried by inflowing waters gradually fills the lake or reservoir (Water Environment Federation, 1998).

Eutrophic is the term applied to waters with a high degree of biological productivity. Several parameters are used to determine the trophic condition of a body of water: standing crop of phytoplankton, level of chlorophyll, volume of algae, level of oxygen production, level of oxygen depletion, Secchi disk readings, or a combination of all these factors. In bodies of water that stratify during the summer, a typical method of determination measures dissolved oxygen (DO) levels in the bottom layer, or hypolimnion. If the deep waters remain aerobic during summer stagnation, the body of water is considered oligotrophic. If the dissolved oxygen becomes depleted, the waters are considered eutrophic, and the degree of eutrophy can be estimated by the time it takes for anoxic or anaerobic conditions to develop after the onset of stratification. The usefulness of this parameter requires some judgment based on temperature conditions and the relative volumes of the hypolimnion and the upper water layer, or epilimnion (Water Environment Federation, 1998).

5.2.3 Phosphorus forms, sources , and cycles

Phosphorus only exists in aquatic environments in the +5 valence-state. In essence, this limits the types of naturally occurring phosphorus compounds to salts and esters of phosphoric acid. Phosphoric acid is easily ionized to inorganic phosphates such as orthophosphates, which are readily available for biological metabolism. Bacteria and many other organisms use orthophosphates to create organic phosphates and many other organisms use orthophosphates to create organic phosphates and build solid-phase structure. Calcium phosphate is the basic component of bone. Organic phosphates are found in nucleic acids, phospholipids, adenosine triphosphate (ATP), hormones, and many other compounds that are essential to energy and reproduction (Water Environment Federation, 1998).

The most significant sources of phosphorus in wastewater are human excrement, food wastes, synthetic laundry detergents, household cleaners, and industrial and commercial discharges.

Phosphorus excreted by humans has been estimated at 0.2 to 1.0 kg/cap·a, with an annual mean of 0.6 kg phosphorus/cap·a. The next most significant source is synthetic laundry detergents. The amount of phosphorus in wastewater attributed to detergents has been estimated at 0.3 kg/cap·a. This contribution has been determined by monitoring wastewater phosphorus concentrations before and after phosphate detergent bans. Because household cleaners and commercial detergents typically are excluded in phosphate detergents ban, their contributions remain undermined. However, the soap and detergent industry has proposed 0.1 kg/cap·a as an estimate of its contribution to domestic wastewater (Jenkins and Hermonowicz, 1991). As usual, industrial and commercial contributions are highly variable, with fertilizer manufacturers, feed lots, and meat, milk, and food processors among the significant sources (Water Environment Federation, 1998).

5.2.4 Typical phosphorus concentrations

The concentration of phosphorus in domestic wastewater is affected by each of the previously cited sources. In general, phosphorus contributions from human excrement, food waste, household cleaners, and commercial detergents remain relatively consistent throughout the U.S. This is also true for industrial discharges, which are required to meet pretreatment standards. Therefore, the formulation of detergents within the wastewater collection area represents a significant variable in determining the wastewater phosphorus concentration. Phosphate detergent bans went into effect on December 1, 1988, in Virginia. The average total phosphorus concentration was 8 mg/L before the phosphate bans and 5.4 mg/L after the bans. The average orthophosphate concentration was 6 mg/L before and 3.7 mg/L after the phosphate bans (Water Environment Federation, 1998).

5.3 Overview of phosphorus removal processes

Unlike nitrogen, there is no gaseous form of phosphorus through which it can be removed from wastewater. Consequently, phosphorus must be converted to a particulate (solids) form and removed as a particulate by sedimentation, filtration, or some other solids removal process or be concentrated into a side stream using membrane treatment. Table 5-1 summarizes the various options for removing or converting phosphorus species (Water Environment Federation, 1998).

Table 5-1. Phosphorus Species and Reactions (Water Environment Federation, 1998).

Species	Comments
Organic-P	▪ Organic phosphorus can be converted to orthophosphate and polyphosphate
Orthophosphate	▪ Most abundant phosphorus species ▪ Reactive species in chemical reactions and consumed in biological growth
Polyphosphates	▪ Condensed orthophosphates ▪ Possibly reacts with metal salts ▪ Can be used for biological growth
Chemical phosphorus	▪ Precipitated phosphates formed by reacting orthophosphate with metal salts, or precipitates as phosphate hydroxides
Biological phosphorus	▪ Phosphorus incorporated into the biomass for growth ▪ Excess phosphorus may accumulate under certain conditions

To remove phosphorus from the system, three options are available (Water Environment Federation, 1998).

- Convert the phosphorus to a chemical species by adding a metal salt or lime. The efficiency of phosphorus removal is dependent on two factors: the chemical equilibrium between the phosphorus liquid and solid phases, and the efficiency of the solids removal process. Typically, the latter process controls the removal efficiency.
- Incorporate the phosphorus into the biomass. Typically, biomass contains 1.5 to 2.5% (w/w) phosphorus per volatile solids. Under certain conditions, the biomass will accumulate phosphorus levels far in excess of the nutritional requirements to 6 - 8% phosphorus ——a process referred to as enhanced biological phosphorus removal (EBPR). The phosphorus removal efficiency for biological systems depends on the phosphorus content of the sludge removed and the efficiency of the solids separation process.
- Processes that will remove essentially all pollutants from water, such as reverse osmosis, or nanofilters, can be used to remove phosphorus. Membrane treatment is expensive and not used for mainstream phosphorus removal. However, membranes used for another objective, such as total dissolved solids removal, will also remove phosphorus.

Typically, phosphorus removal processes can be grouped into three basic groups: chemical processes, biological processes and physical processes. The following subsections describe typical phosphorus removal processes. Figure 5-1 gives a basic overview of phosphorus removal processes. The selection of a specific process must be based on a case-by-case evaluation of the system economics, including both capital and operating costs.

5.3.1 Chemical phosphorus removal

The principal chemical method for phosphorus removal is chemical precipitation. The chemical precipitation of phosphorus is accomplished by the addition of the salts of multivalent metal ions that form precipitates of sparingly soluble phosphates. The multivalent metal ions used most commonly are calcium [Ca^{+2}], aluminum [Al^{+3}], and iron [Fe^{+3}]. Because the chemistry of phosphate precipitation with calcium is quite different than with aluminum and iron, the two different types of precipitation are to be considered separately (Crites *et al*, 1998).

Phosphorus removal by chemical addition is attractive for its simplicity of operation and ease of implementation, but results in increased sludge production and additional operation and maintenance costs. Chemicals are added to the wastewater at a well-mixed location, followed by flocculation and solids removal by sedimentation, filtration, or similar processes. The equilibrium of the chemical reactions favors phosphorus precipitation and will produce low residual phosphorus levels. Phosphorus levels less than 0.1 mg P/L can consistently be achieved with chemical addition and well-designed filtration facilities (Water Environment Federation, 1998).

5.3.2 Biological phosphorus removal

Conventional secondary biological treatment systems take up phosphorus from solution for biomass synthesis during BOD oxidation. Phosphorus, required in intracellular energy transfer, becomes an essential cell component. For this reason, phosphorus is taken up in an amount related to the stoichiometric requirements for biosynthesis. Typical phosphorus contents of microbial solids are 1.5 to 2% on a dry weight basis. A sequence of an anaerobic zone followed by an aerobic zone results in the selection of a population rich in organisms capable of taking up phosphorus al levels beyond the stoichiometric requirements for growth. With this environment, the biomass accumulates phosphorus to levels of 4 to 12% of the microbial solids. Wastage of these solids results in approximately 2.5 to 4 times more phosphorus removal from the system than that from conventional treatment. The organism most often associated with enhanced biological phosphorus removal belongs to the genus *Acinetobacter* (Water Environment Federation, 1992).

Rational design and operation require an understanding of the mechanism by which enhanced biological phosphorus uptake occurs. The currently accepted mechanism of enhanced biological phosphorus removal (EBPR) is summarized in Figure 6. Acetate and other short-chain fatty acids (fermentation products), produced by fermentation reactions in the anaerobic zone, are taken up and stored intracellularly, most commonly as poly-hydroxy-butyrate (PHB). In performing the anaerobic uptake of soluble organic and forming intracellular storage products, microorganisms must expend energy. They obtain this energy anaerobically through the cleavage of high-energy phosphate bonds in stored long-chain inorganic polyphosphate. This process produces orthophosphate that is released from the cell into solution. Thus, a removal of soluble BOD with concomitant release of phosphorus occurs in the anaerobic zone (Water Environment Federation, 1992).

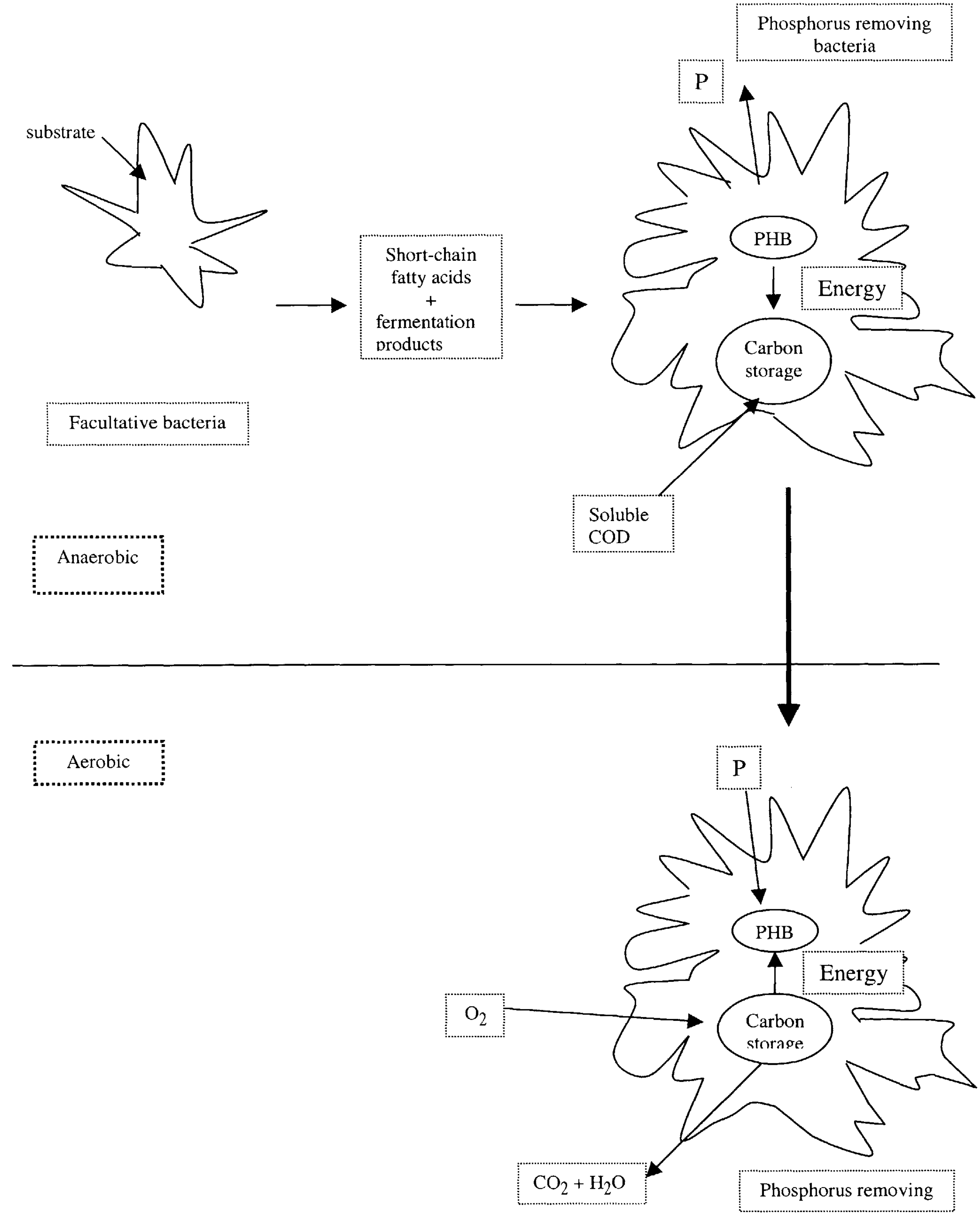


Figure 5-1. Removal Mechanisms for Excess Biological Phoshorus (PHB = poly-β-hydroxybutyrate) (Water Environmental Federation, 1992).

In the aerobic zone, a rapid uptake of soluble orthophosphate provides for the resynthesis of the intracellular polyphosphates. Accompanying this uptake, previously stored PHB is aerobically oxidized to carbon dioxide, water, and new cells. The aerobic metabolism of residual soluble BOD will also occur in this zone. Figure 1 shows typical profiles of phosphorus and BOD concentration through anaerobic and aerobic zones. Unlike the many other biological reactions of significance in wastewater treatment, the stoichiometry and kinetics of phosphorus release and uptake have not yet been fully understood. Thus, the design engineer must rely on empirical observations to obtain information for process design and modifications (Water Environment Federation, 1992).

5.3.3 Physical phosphorus removal

Due to stringent effluent requirements for phosphorus, domestic wastewater will soon demand increasing efforts to address eutrophication. Although attention has been mainly focused on the integrated removal processes of phosphorus and nitrogen, which is referred to as the tertiary treatment, it is doubtful whether the effluent qualities will be sufficient to meet the stringent requirements. The effluent-polishing (EP) process, should be introduced to WWTPs that seek for the stringent requirements. In this process, phosphorus, heavy metals, organic micro-pollutants, micro-organisms and suspended solids have to be removed. Filtration methods seem to be the most appropriate for this effluent-polishing step.

5.4 Principles of chemical phosphorus removal

The basic principle of chemical phosphorus (P) removal relies on the transformation of soluble phosphorus to a particulate form and the removal of this form (together with any phosphorus already present in a particulate form), typically by sedimentation. Removal during primary settling is limited to a fraction of the particulate form and depends on the efficiency of the primary clarifiers. In a secondary treatment process, phosphorus is incorporated to biomass and removed from wastewater through secondary sedimentation with waste biomass. As such, the quantity of phosphorus removed by a conventional secondary treatment process is a function of biomass yield and production (Water Environment Federation, 1998).

Total effluent phosphorus concentration TP can be estimated as

$$\text{Effluent TP} = \text{SP} + m\cdot(\text{effluent TSS}) \quad \cdots\cdots (1)$$

Where

SP = soluble phosphorus (mainly soluble orthophosphate), mg/L;
TSS = effluent suspended solids concentration, mg/L; and
m = phosphorus content in SS dry mass, mg P/mg SS.

For conventional activated sludge, m is 20 to 25 mg P/g volatile suspended solids (VSS) (2 to 2.5%). For chemical phosphorus removal, m varies between 40 to 100 mg P/g suspended solids (4 to 10%). The equation above emphasized the contribution and the need for effective solids removal. For example, if effluent suspended solids concentration is 20 mg/L with a phosphorus content of 5% (50 mg P/g SS), a total phosphorus concentration below 1 mg P/L cannot be achieved (Water Environment Federation, 1998).

From an engineering point of view, three parameters are of particular importance for design, operation, and analysis of chemical phosphorus removal (Water Environment Federation, 1998):

- Dose requirements,
- Minimum achievable phosphate concentration, and
- Effects of pH.

5.4.1 Ferrous iron

The main source of ferrous ion [Fe(II)] is spent pickle liquor containing mostly FeSO4 and originating from metal-processing operations. It is potentially convenient and economical source of precipitating agent, but it may contain hazardous materials (such as heavy metals) that can either pass through with the effluent or accumulate in the sludge. It should be noted, however, that commercially available technical-grade iron salts [both Fe(II) and ferric iron, or Fe(III)] may also contain a certain amount of heavy metals (Water Environment Federation, 1998).

If exposed to aerobic conditions, Fe(II) oxidizes rapidly to Fe(III). Leckie and Stumm (1970) indicated that ferric iron Fe(III) formed by oxidation of Fe(II) is more effective for phosphate precipitation than Fe(III) added directly from a stock solution. However, the practical efficiency of phosphorus removal is also affected by the settling characteristics of the precipitate. Leckie and Stumm (1970) reported that the precipitate formed by oxidized Fe(II) in clean water was inferior to those from Fe(III).

5.4.2 Ferric iron and aluminum

Chemical precipitation using aluminum or iron coagulants is effective in phosphate removal. Although coagulation reactions are complex and only partially understood, the primary action appears to be the combining of orthophosphate with the metal cation. Polyphosphates and organic phosphorus compounds are probably removed by being entrapped, or adsorbed, in the floc particles. Aluminum and ferric salts combine with phosphate ions as follows (Hammer *et al.*, 1996).

$$Al^{3+} + H_2PO_4^- \rightarrow AlPO_4(s) + 2H^+ \cdots\cdots (2)$$

$$Fe^{3+} + H_2PO_4^- \rightarrow FePO_4(s) + 2H^+ \cdots\cdots (3)$$

In addition to metal phosphate precipitants, metal complexes also form with hydroxide ions as shown below:

$$Al^{3+} + 3H_2O \rightarrow Al(OH)_3(s) + 3H^+ \cdots\cdots (4)$$

$$Fe^{3+} + 3H_2O \rightarrow Fe(OH)_3(s) + 3H^+ \cdots\cdots (5)$$

The formation of metal hydroxides adds to the complexity of predicting chemical reactions and their results. First, the formation of hydroxy-metal complexes adds a competing reaction for the added metal. The dose must therefore be adjusted to account for the metal hydroxide reactions.

Second, the reactions produce a significant amount of sludge (2.9 mg solids/mg Al for alum and 1.9 mg solids/mg Fe for ferric) that must be processed through dewatering and disposal. Third, the reactions consume a significant amount of alkalinity (5.8 mg as $CaCO_3$/mg Al and 2.7 mg as $CaCO_3$/mg Fe) (Water Environment Federation, 1998).

When an excess dose of metal salt [Fe(III) or Al(III)] is added, metal hydroxide will precipitate in addition to metal phosphate precipitating. In this case, residual phosphate concentration can be calculated from equilibrium equations if the appropriate equilibrium constants are known. Of those, only solubility products for $Al(OH)_3$ and $Fe(OH)_3$ are known with any accuracy. The values of solubility products for metal phosphates and stability constants for soluble metal-phosphate complexes were estimated from field and laboratory data on phosphate removal (Gates *et al.*, 1990). The estimated equation is as follows:

$$r\,Me^{3+} + H_2PO_4^- + (3r-1)\,OH^- \rightarrow Me_r(H_2PO_4)(OH)_{3r-1}(s) \quad \cdots\cdots (6)$$

$$r = 0.8 \text{ for Al(III), } 1.6 \text{ for Fe(III)}$$

5.4.3 Calcium

Phosphate precipitation with lime was the earliest method of phosphorus removal. Calcium forms several insoluble compounds with phosphate, among which hydroxyapatite $Ca_5(PO_4)_3(OH)$ seems to be the most important. Additionally, calcium carbonate can form depending on pH, wastewater alkalinity, and calcium dose. Significant phosphate removal can only be achieved at higher pH values. For example, removal of phosphate values below 1 mg/L requires values of pH of 10.5 to 11. For this reason lime is used either in primary treatment or following biological treatment(Water Environment Federation, 1998). The calcium or lime application doesn't seem appropriate in this project because the wastewater in the Town of Acton is assumed to be the typical municipal wastewater, in which pH is around 7, and high pH assumptions could cause biological treatment processes to break down.

5.5 Principles of biological phosphorus removal

5.5.1 Enhanced biological phosphorus removal

The following operating characteristics emerged with respect to EBPR systems (Barnard, 1976).

- The return sludge and the mixed liquor should be introduced at the inlet end of the aeration basin.
- Aeration should provide a high dissolved oxygen (DO) content at the outlet of the basin to prevent anaerobic conditions from developing in the secondary clarifier.
- The degree of nitrification should be kept to a minimum.
- Solids should be rapidly removed from the secondary clarifier and returned to the aeration basin. If the wastewater is allowed to become anaerobic, phosphorus release will lead to an increase in effluent phosphorus concentrations.
- Liquid should not be recycled to the activated-sludge process from a subsequent sludge treatment step that may release phosphates.

The importance of an anaerobic zone in the activated-sludge system to remove phosphorus and nitrogen was clearly established. Because the mechanism (microbiology and biochemistry) was still in dispute, the design of systems remained largely empirical. Several definitions of various zones within the activated-sludge system emerged:

- *Anaerobic* zone referred to a zone without aeration, devoid of DO, nitrite, or nitrate. However, the redox potential in the zone remained above that for reducing sulfate to hydrogen sulfide but below the nitrate-nitrogen gas level.
- *Anoxic* conditions referred to zones where DO was absent but nitrate was present. Anoxic zones are used for denitrification.
- *Aerobic* conditions referred to zones with DO in the liquid. Nitrification only occurs under aerobic conditions.

5.5.2 Microbiology

The microbiology of the activated sludge system is complex. Th survival of certain microorganisms in the process is not only a function of environmental conditions such as temperature, pH, and DO, but also depends on the substrate composition, such as the availability of soluble, readily biodegradable organic compounds (Water Environment Federation, 1998).

Pure culture studies to determine *Acinetobacter* growth kinetic coefficients on aerobically grown cultures, using substrates such as acetate and ethanol, demonstrated maximum growth rate (μ_{max}) in the range 4 to 30/day, dry cell yields (Y) of approximately 0.4 g/g COD, and endogenous decay rates (k_d) of 1 to 5 g/g·d have been obtained (Abbott *et al.*, 1973 and 1974; Deinema *et al.*, 1985; Ensley and Finnerty, 1980; and Hao and Chang, 1987). Cells grown under such conditions have high phosphorus contents (approximately 4 to 7%[Deinema *et al.*, 1985]). Values of μ_{max} and k_d obtained in these experiments are far higher than expected for a polyphosphate- (poly-P-) accumulating organism from the observed performance of EPR activated-sludge systems. For example, Wentzel et al. (1987 and 1988) conducted batch tests in which they found that μ_{max} = 0.75 to 0.95/day without phosphorus limitation, 0.35/day with phosphorus limitation, and k_d = 0.03 to 0.04/day. The culture used in the batch tests showed all of the attributes of EBPR activated sludge (Water Environment Federation, 1998).

For simplicity, the microorganisms that grow and participate in phosphorus uptake-release-storage typically are referred to as biological phosphorus removing microbes (bio-P). While *Acinetobacter* is the dominant bio-P organism, there are several others. These organisms flourish under the cyclic anaerobic-aerobic activated-sludge operation and incorporate a storage-uptake mechanism for soluble organic compounds (Water Environment Federation, 1998).

5.5.3 Temperature and pH

The EBPR processes are relatively insensitive to temperature changes, compared to other biological processes. The effect of temperature on EBPR systems is probably attenuated by other environmental factors, such as changes in wastewater composition and other uncontrollable factors (Water Environment Federation, 1998).

Tracy and Flammino (1985) investigated the effects of pH on EBPR and showed that pH affects the phosphorus uptake rate. The uptake rate remained at its optimum between pH 6.6 and 7.4 but decreased rapidly as the pH dropped below 6.2 (Water Environment Federation, 1998).

5.6 Screening methods

The strategy described below for selecting a phosphorus removal system is invented by us based on the screening process proposed by EPA (EPA, 1987). Initially, one has to decide what kinds of nutrients to be addressed. In this project, nitrogen is not considered as the limiting nutrient and phosphorus removal is the main goal to be addressed. Alternatives that seem to be feasible for this project are listed in the next step and these alternatives are described in Section 5.7. Technical feasibility should be the next screening process as the project goals cannot be accomplished unless the total phosphorus limitation (0.1 mg TP/ L) is not achieved (refer to Section 5.8). Non-applicable technologies are rejected with the selected alternatives being chosen as a result of each screening step. Cost-effectiveness analysis is undertaken in the fourth step (Section 5.9). Finally, the selected process as a result of the fourth step will be analyzed in terms of other environmental impacts the process might have (Section 5.2). The five steps of the selective screening process are described below.

Step 1: Categorize the facility as a new plant, and determine whether the effluent nutrient discharge limitations are for phosphorus only. In this case, eutrophication in the river is the major problem, which means phosphorus is the major determinant factor for eutrophication.

Step 2: Propose all the alternatives to be considered that are from various references and categorically classify them associated with three basic processes.

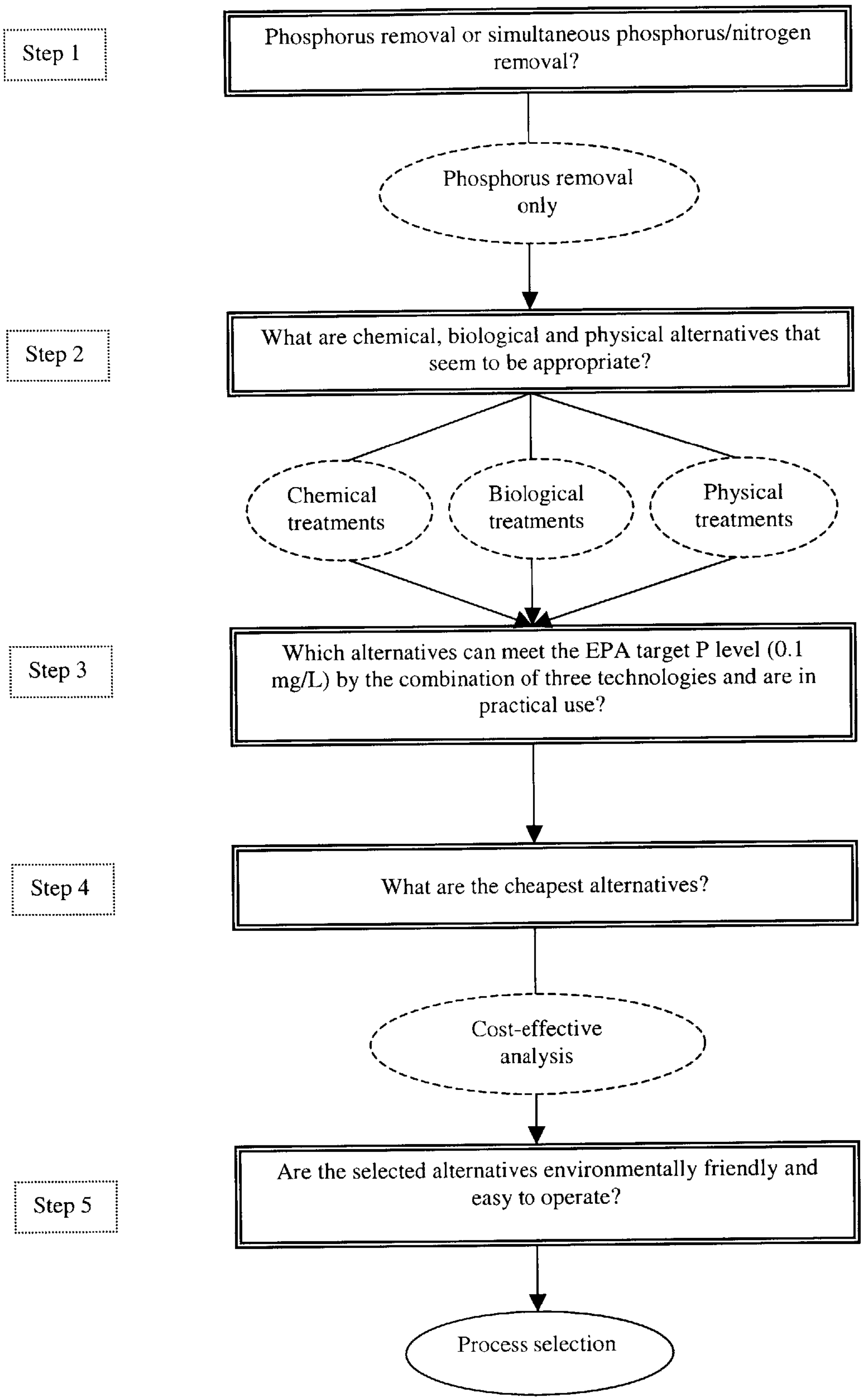
Step 3: Determine which P-removal processes can meet phosphorus limitations (0.1 mg/l in this project).

Step 4: Estimate the capital, operation and maintenance, and total present-worth costs for all applicable alternatives. This comprehensive cost-effectiveness analysis will result in the selection of a system that meets project objectives at the lowest present-worth cost.

Step 5: Non-monetary factors to be considered include:

1. Non-water-quality environmental impacts such as sludge production or waste disposal, and
2. Operator skill levels required for successful operation.

Figure 5-2. Screening Process Flow-sheet



5.7 Second step of screening (alternatives)

The following table shows the various types of technologies for phosphorus removal that seem applicable to this project. These technologies are collected through literature review or the information collected from the technical advisors.

Table 5-2. Alternatives

Technologies	Feasibility	Descriptions
Chemical treatments		
GCR20 (liquid aluminum chloride-ferric chloride)	0.18 mg/L (tertiary clarifier effluent) 0.06 mg/L (filtered final effluent)	• Used as cheaper chemical additions (about half the cost of ferric chloride) than ferric chloride or aluminum chloride, being the byproduct of a process used primarily for cleaning computer chips and circuits (Water Environment & Technology, March 1998). This technology is technically and economically feasible but not in practical use yet.
Ferric chloride	Less than 0.1 mg/L with the combination of other treatment processes	• Used in some wastewater treatment plants for high-level phosphorus removal (XCG Consultants Limited, 1996).
Alum	Less than 0.1 mg/L with the combination of other treatment processes	• More expensive than ferric chloride (XCG Consultants Limited, 1996).
Phormidium laminosum	0.05 mg/L (effluent after residence time of 12 hour)	• Using the thermophilic cyanobacterium immobilized on cellulose hollow fibres in the tubular photobioreactor at 43 °C. This technology is technically feasible but not in practical use yet.
Biological teatments		
Activated sludge treatment	0.5 to 1.0 mg/L (secondary effluent) less than 0.1 mg/L with other chemical and physical treatment processes	• Activated sludge system is appropriate for small-sized plants like this project circuits (Water Environment & Technology, Sep. 1998).

Physical treatments		
Contact clarifier and microfiltration	60-90 µg/L (without ferric dosing) 30-50 µg/L (with ferric dosing)	• Three microfiltration systems with a pore size of 0.2 µm are tested in small-scale pilot plants (Dittrich *et al*, 1996). This technology is technically feasible, yet seems expensive.
Contact clarifier and ultrafiltration	<0.1 mg/L	• This technology is presently being piloted in Hopewell, Virginia and has been installed in full-scale wastewater treatment plants in Canada and Europe.
Contact clarifier and filtration	0.05-0.1 mg/L	• Second effluent containing dissolved oxygen is dosed with ferric chloride or alum solution and then contacts with filter media (XCG Consultants Limited, 1996).

5.7.1 Chemical treatments

As mentioned in Section 5.4, chemical treatments include ferrous iron, ferric iron and aluminum, and calcium. Ferric iron and aluminum are used in the practical manner in the U.S., while ferrous iron and calcium are not practically used for high-rate phosphorus removal and calcium causes high sludge production. Therefore, two chemicals, ferrous chloride and calcium shouldn't be considered as the alternative chemical treatments. Therefore, ferric iron and aluminum are included as the alternative chemical treatments here.

The Durham Wastewater Treatment Facility, North Carolina reported that the effluent from the plant that must meet extremely stringent effluent limits in order to protect the water quality of the Tualatin River attain an effluent phosphorus concentration of 0.08 mg/L. The plant has secondary treatment processes to provide biological nutrient removal, chemical clarifiers, and mixed-media effluent filters (Unified Sewerage Agency, 1998).

One of the operators of the plant suggested that they were operating with 30 mg/L alum in both primary and tertiary clarifiers and they did a 2/3-scale bio-phosphorus removal pilot. They were able to cut the total dose to 20 mg/L in the tertiary clarifiers since, at times, secondary effluent soluble phosphorus was lower than 0.1 mg/L.

5.7.2 Biological treatments

The following table shows the matrix for the biological process selection. Though, in this project, only phosphorus removal is targeted to address eutrophication, we might have to consider the future application or the expansion of the wastewater treatment facilities. The matrix here explains which technologies are appropriate for specific target levels of nitrogen and phosphorus.

Table 5-3. Process Selection Matrix for Nutrient Removal (Water Environment Federation, 1992).

Approximate percentage removal (typical concentration, mg/L)	Nitrogen 0% (40)	Nitrogen removal	Nitrogen and phosphorus removal	Phosphorus removal	Phosphorus 0% (10)
	30%(28)	Conventional activated sludge (10 to 30%)	Conventional activated sludge	Conventional activated sludge (10 to 15%)	30%(7)
	80% (8)	- MLE - A^2/O^{TM} - PhoStrip II^{TM} - Oxidation ditch - Biodenitro^{TM} - Simpre^{TM} - UCT and VIP -	- Modified Bardenpho^{TM} - A^2/O with denite filters - Biodenitpho^{TM} - PhoStrip^{TM} - Operationally modified activated sludge - UCT - SBR - PhoStrip II^{TM}	- A/O^{TM} - PhoStrip^{TM} - Sequencing batch reactor (SBR) - OWASA	80% (2)
	(filters suggested) 95% (2)	- 4-stage Bardenpho^{TM} - Modified Wuhrman - Dual sludge - Three sludge - Post-aeration anoxic tank with methanol - Denitrification filters - Fluidized bed reactors - Phase isolation ditches	- Dual sludge with chemicals - Modified Bardenpho^{TM} with chemicals - A^2/O with denite filters and chemicals - Three sludge with chemicals	- PhoStrip^{TM}	(filters suggested) 98% (0.20)

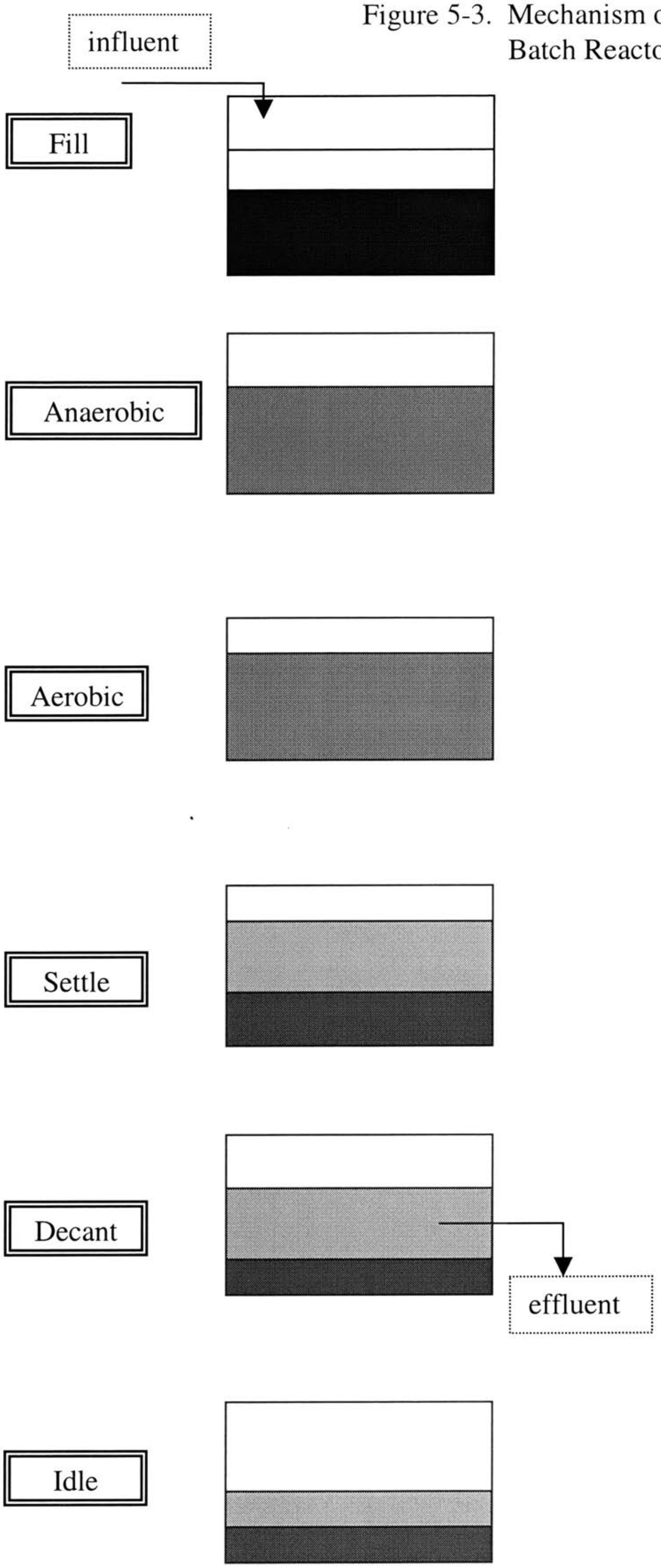
As the only alternative for biological phosphorus treatment of this project, sequencing batch reactor (SBR) is recommended because SBR is less expensive, efficient, and controllable, particularly for small-flow systems like this project. In conventional activated-sludge treatment systems, the processes are carried out simultaneously in separate tanks, whereas in SBR operation the processes are carried out sequentially in the same tank (see Figure 5-3).

Hang Sik Shin, Hang Bae Jun and Hung Suck Park reported in their research that phosphorus removal efficiencies were about 60% in the first 80 days of the experiment, 75% after 80 days, and above 95% after 120 days (1992).

Rim *et al.* reported that BOD removal was observed to be 95% on average while SS removal to be 89% on average, and that the removal rate of nitrogen was 70% in terms of total nitrogen and that of phosphorus was 77% in terms of total phosphorus (1997).

Hamamoto *et al.* reported that in the pilot plant average nitrogen and phosphorus removal rates of 86% and 82% were achieved, and that in over three years of operation, the full-scale plant realized rates of 87% and 74%, respectively (1997).

Figure 5-3. Mechanism of Sequencing Batch Reactor



Fill:
Influent enters the SBR

Anaerobic Conditions:
Complete mix of the reactor contents is achieved without use of aeration. This phase assists in control of filamentous organisms and is essential for those systems that require phosphorus removal.

Aeration:
Aeration may be intermittent to promote aerobic or anoxic conditions. Nitrification and denitrification can be achieved. The aeration source may also be operated intermittently during low flow and organic loading conditions to conserve energy

Settle:
Mixing and aeration cease. Solids/liquid separation takes place under perfectly quiescent conditions.

Decant/Sludge Waste:
The mixer and the aeration system remain off and at this time the decantable volume is removed by means of subsurface withdrawal. The reactor is immediately ready to receive the next batch of raw influent. A small amount of sludge is wasted each cycle.

Idle:
This step occurs in multiple-basin systems anytime that flow conditions are less than peak design flow. Idle time varies depending on actual flow conditions.

5.7.3 Physical treatments

(a) Solids Contact clarifier

Solids contact clarifiers provide high-rate treatment of wastewater for SS removal. The solids contact clarifier is comprised of a draft tube, (2) reaction zone, (3) sludge blanket zone, (4) a clarification zone, (5) a radial or axial turbine which recirculates the settled solids from the bottom, and (6) a concentric shaft or center column. The turbine mixes the highly concentrated settled solids with the low concentration influent and disperses the mixture into the reaction well, where the solids coagulate and settle. The flow passes through the sludge blanket and the bottom edge of the reaction well, filtering out the few particles that did remain suspended. Rotating rake arms transport the settled solids to the center of the tank for removal (WesTech, 1998).

To prevent filters form clogging, solids contact clarifiers are highly recommended before filtration. The following three alternatives utilize the filtration process with solids contact clarifiers ahead of them to reduce remaining suspended solids that are the major contributor to the concentrations of effluent total phosphorus.

(b) Contact clarifier and Ultrafiltration

Ultrafiltration (UF) is a pressure-driven unit operation in which paticulates, colloids, emulsified oils, and macromolecules are separated from a liquid feed stream upon passage through a porous semi-permeable membrane. The separation is based primarily on the size of the species in the liquid relative to the size of the membrane pores. On the separation size spectrum, UF falls between nanofiltration or NF (membrane pore sizes below approximately 0.01 μm) and microfiltration or MF (pore sizes greater than 1.0 μm) (Figure 5-4) (Monat, 1998).

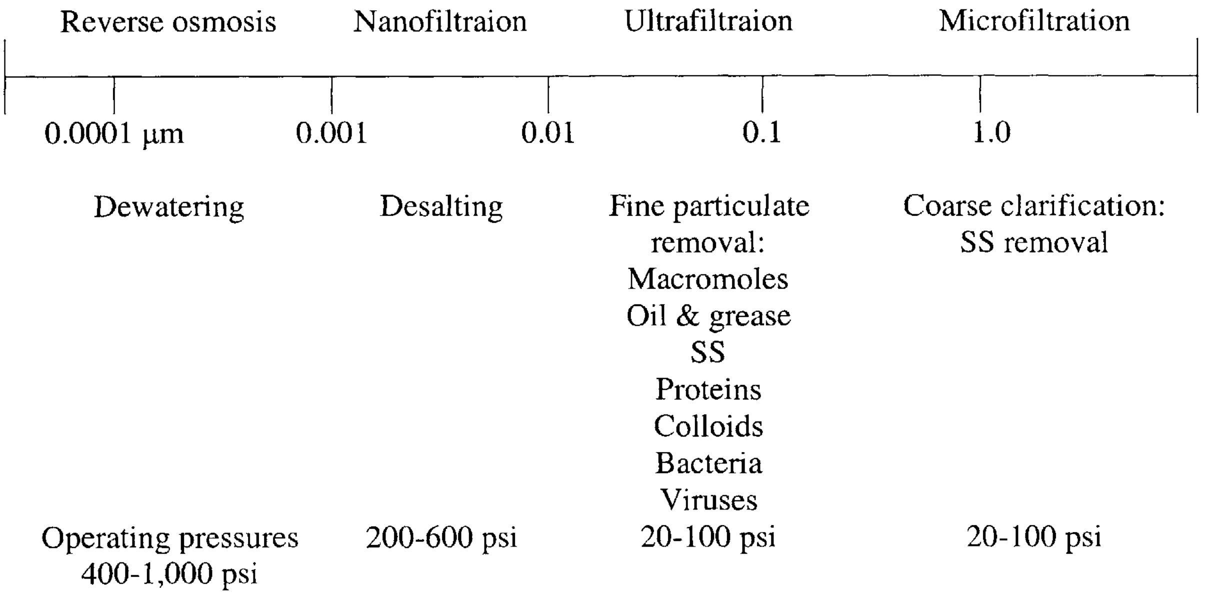


Figure 5-4. Filter Sizes and Usage (Monat, 1998)

(c) Contact clarifier and Microfiltration

Dittrich *et al.* (1996) conducted the experiment to show whether microfiltration (MF) is a technically feasible and economically competitive process for disinfection and phosphorus removal of secondary effluent and reported that the average concentrations for total phosphrous (TP) in the effluent are 60 µg/L for Memcor and the DOW units and 90 µg/L for Starcosa unit without the use of precipitants.

The following table shows the results of the experiment conducted by Dittrich *et al.*

Table 5-4. Average Phosphorus Concentrations in the Influent and in the Effluent of of the MF Pilot Plants With and Without Chemical Dosing (µg/L) (Dittrich *et al.*, 1996).

		Influent	Effluent		
			DOW	Starcosa	Memcor
Without ferric dosing	TP	186	62	93	60
	Dissolved phosphorus	93	-	-	-
	PO_4-P	<30	<28	52	<26
With ferric dosing (0.014 mol Fe/m^3)	TP	120	49	43	35
	Dissolved phosphorus	70	30	33	27
	PO_4-P	33	10	11	7

Oesterholt and Bult (1993) conducted a 3-month experiment to test cross flow microfiltration, and reported that iron has to be added prior to removal so that suspended solids and phosphorus can be removed in sufficient quantities. In that case, they reported 90% removal of total phosphorus when microfiltration is being applied.

The following table shows the results driven by the experiment by Oesterholt and Bult.

Table 5-5. Average Effluent Concentrations after Microfiltration Combined with Iron Flocculation (Oesterholt and Bult, 1993).

	Influent	Effluent
SS (mg/L)	20	<1
COD (mg/L)	40	27
Kj-N (mg/L)	1.7	1.4
TP (mg/L)	2.5	0.09
Escherichia coil *	26,000	2

* number of micro-organisms per 100 mL.

Oesterholt and Bult commented that from a technical point of views, microfiltration is preferred because of its high removal efficiency for all the components, but from a financial point of view, it is not feasible (1993). The economic considerations are conducted in Section 5.9 and the most cost-effective physical treatment technology is determined.

(d) Contact clarifier and Traveling bridge filter

Ross *et al.* the evaluated alternative treatment processes capable of achieving phosphorus concentrations of less than or equal to 0.10 mg/L. They reported that post-precipitation (i.e. precipitant addition to secondary effluent before filtration), demonstrated in full-scale demonstration at two Ontario WWTPs with conventional tertiary filters, was successful in achieving total phosphorus concentrations less than 0.10 mg/L in tertiary effluent. They found that effective phosphorus removal depended on the addition of adequate metal salt precipitant to reduce the soluble phosphorus concentration to about 0.03 mg/L, and good removal of suspended solids to reduce particulate phosphorus levels. For the wastewater used in their pilot studies, they also found that to achieve the target TP of less than or equal to 0.10 mg/L, the maximum suspended solids level in tertiary effluent was 5 mg/L, based on a phosphorus content of 1.5% in the solids (1996).

The test results of the experiment conducted by Ross *et al.* is shown in Table 5-6 below.

Table 5-6. Full-scale Post-precipitation Demonstration Program Results (Ross *et al.*, 1996).

Parameter	Plant 1	Plant 2
Plant type	Conventional activated sludge	Conventional activated sludge
Plant capacity	13,000 m^3/d (3.4 mgd) average 32,300 m^3/d (8.5 mgd) peak	18,200 m^3/d (4.8 mgd) average 36,400 m^3/d (9.6 mgd) peak
Filter type	Gravity-type dual media	Gravity-type, travelling bridge, sand
Hydraulic loading during study	4 m/h at average flow est. 10 m/h at peak flow	4 m/h at peak flow
Baseline filter effluent: TSS TP Soluble P	 3 mg/L 0.27 mg/L 0.21 mg/L	 <3 mg/L 0.36 mg/L 0.29 mg/L
Post-precipitation filter effluent: TSS TP Soluble P	<3 mg/L 0.09 mg/L 0.06 mg/L	<3 mg/L 0.07 mg/L 0.03 mg/L
Precipitant dosage: Baseline metal dosage Post-precip. Metal dosage	 6.2 mg Al/L (simultaneous) 7.5 mg Al/L (6.5 simult., 1.3 post)	 12.3 mg Fe/L (pre) 15.2 mg Fe/ L (12.3 simult., 3.2 post)

Notes:
Average of fifteen 24-hour composite samples collected over a hour to six week period.

They commented that by adding precipitant in the post-precipitation mode, the required dosage to achieve the desired soluble phosphorus removal efficiency was significantly lower than would have theoretically been required with single point pre- or simultaneous precipitant addition (chemical addition to primary or secondary clarifier influent). That is, high pre- or simultaneous chemical dosage leads to higher chemical costs, increased sludge production and generation of a lighter, fluffier sludge with more potential for pin-floc development (Ross *et al.*, 1996).

5.8 Third step of screening (technical feasibility)

The following table shows the alternatives technically screened out in the first step of the screening process. All the technologies that are not supposed to meet the target level of total phosphorus (0.1 mg/L) by the combination of chemical, biological and physical processes or that are not in practical use now are canceled out and are no longer the candidates for further considerations. All the technologies that are technically feasible enough to meet the EPA target level are screened out and will be considered in terms of cost effectiveness in the next step.

Alternatives screened out	Alternatives canceled out

Table 5-7. Alternatives Screened or Canceled.

Technologies	Reasons of the cancellations or selections.
Chemical treatments	
GCR20 (liquid aluminum chloride-ferric chloride)	Very attractive but not in practical use. Further more, the information is limited.
Ferric chloride	Used in some actual cases.
Alum	Used in some actual cases.
Biophysical treatment	
Phormidium laminosum	Not in practical use yet and seems too expensive.
Biological treatments	
SBR	Utilized in many cases and much information available. Cost-effective and appropriate for smaller municipalities.
Physical treatments	
Contact clarifier and microfiltration	Technically feasible. Need further analyses in terms of economy.
Contact clarifier and ultrafiltration	Technically feasible. Need further analyses in terms of economy.
Contact clarifier and filtration	Not only technically feasible, but economically sounds reasonable.

5.9 Fourth step of screening (cost-effectiveness)

The following table shows the typical sizes and the budget costs for each alternative technology screened out in the previous section. SBR that is the only biological treatment technology does not need cost-effectiveness analysis to select out. For typical wastewater treatment plants, more than 2 units of filtration systems or contact clarifiers should be prepared. The cost data are obtained from the companies that are producing each technology.

Table 5-8. Size & Costs of Alternatives.

Alternative	Number of units	Size (LxWxH) etc.	Contents	Costs
Microfiltration (Memcor)	4	• Size: 6290x2300x2800 [247 5/8”x90 1/2”x110 1/4”] • Max flow: 88560-102180 l/h [390-450 USGPM]	0.1<pore sizes<10.0 microns. Requires the following ancillary equipment to be connected to the units: feed, filtrate, waste and cleaning system manifolds, clean-in-place (CIP) chemical storage and recirculation components, compressed air supply system, and pneumatic and electrical controls. To avoid flocculation, 4units is recommended.	$1,815,000/system (including air supply system, CIP system, master PLC for full automation capability and SCADA software.) (USFilter/ Memcor)
Ultrafiltration	4		0.01<pore sizes<1.0 micron. For about 500 gpm.	$650,000/unit (including building costs) $104,000/unit (annual operating costs) (Monat, 1998)
Travelling bridge filter	4	• 9’ x 28’	For about 500 gpm.	$145,000/unit $85,000/unit (concrete units)
	3	• 9’ x 40’	For 750 gpm.	$175,000/unit $95,000/unit (concrete units)
Solids contact clarifier (DensaDeg)	2-3	• Size: 27’-9”x13’-0”x15’-0”	For 520 gpm.	$400,000/unit
	3	•	For 700 gpm.	$500,000/unit
Ferric chloride		• 30-20 mg/L		$876/ton as Fe (dry weight) (XCG Consultants Limited, 1996)
Alum		• 30-20 mg/L		$2044/ton as Al (dry weight) (XCG Consultants Limited, 1996)

5.9.1 Cost comparison of the physical treatments

For the selection of the physical treatments, the cost of travelling bridge filters, is compared with those of UF and MF. Though the contents of the cost estimation for each technology are a little different, I assume that the prices described in the above table basically have the same meaning because it generally seems that the technology of travelling bridge filters is relatively cheap, compared to the other two technologies. With limited information on the prices of these technologies, the comparison between the two technologies, UF and MF, is assumed to be less important than the comparison between UF/MF and travelling bridge filters.

I assume that each technology needs 4 units for the maximum flow rate of 1,050 gpm. (Because of the increased solids loading from a secondary sewage source or the contact clarifiers that are possibly flocculated, 4 units are highly recommended by the producers of these technologies.) The calculation is the following:

The cost estimation of MF doesn't include the O&M costs. Therefore, the comparison of the capital costs for each technology is applied here.

The capital price for 4 units of traveling bridge filters is:

$$(\$145{,}000 + \$85{,}000)/\text{unit} \times (4 \text{ units}) = \$920{,}000.$$

The capital price for 4 units of MF is:

$$\$650{,}000 \times (4 \text{ units}) = \$2{,}600{,}000.$$

Consequently, the cheapest technology is traveling bridge filters and I will select this as the proposed physical treatment technology.

5.9.2 Cost comparison of the chemical treatments

For the selection of the chemical treatments, the comparison between ferric chloride and aluminum chloride is conducted. XCG Consultants Limited reported that <u>a post-precipitation alum dosage of 16 mg/L and a post-precipitation ferric chloride dosage of 22 mg/L are needed for high-rate phosphorus removal</u> (less than 0.1 mg/L in effluent) (1996). Assuming that this estimation is correct, I calculated the costs for the chemicals as follows:

The average flow rate is 2,233 m^3/d. The annual cost for alum addition is:

$$\$2{,}044/t \times (16 \text{ g Al}/ m^3 \times 2{,}233\ m^3/d \times 365 \text{ days} \times 10^{-6} \text{ t/g}) = \$26{,}655/\text{year}.$$

The annual cost for ferric chloride addition is:

$$\$876/t \times (22\text{g Al}/ m^3 \times 2{,}233\ m^3/d \times 365 \text{ days} \times 10^{-6} \text{ t/g}) = \$15{,}708/\text{year}.$$

Although ferric chloride is cheaper than alum, we have to consider the use of two chemicals in terms of sludge production, as ferric chloride produces more sludge than alum. The calculation of sludge production is undertaken in Section 5.11.

5.10 Process diagram

As the result of the previous section, the process diagram is proposed in the next page (Figure 5-5). The process diagram shows the combination of three types of technologies (chemical, biological and physical technologies) screened out in the previous section. Two cases are proposed for further considerations.

In Case 1, first, wastewater goes into the grit chamber to remove grit, and then goes through the primary clarifiers. Primary clarifiers are used before the biological treatment process and their function is to reduce the loads on the biological treatment units. After primary sedimentation, wastewater goes into SBR, a biological treatment, which can be operated to achieve any combination of carbon oxidation, nitrogen reduction, and phosphorus removal.

The next step is effluent-polishing (EP) where the secondary effluent passes through solids contact clarifiers and then filters. Phosphorus concentrations after filtration should be less than 0.1 mg/L. The effluent from the tertiary treatment process finally passes through the disinfection process in which chlorine is used to disinfect the effluent before discharge.

As primary clarifiers are optional in small wastewater plants like this project, in Case 2, wastewater enters SBR after passing through the grit chamber. Some chemicals (alum + polymer or ferric chloride + polymer) can be added to the SBR process. After this process, wastewater is treated as in Case 1.

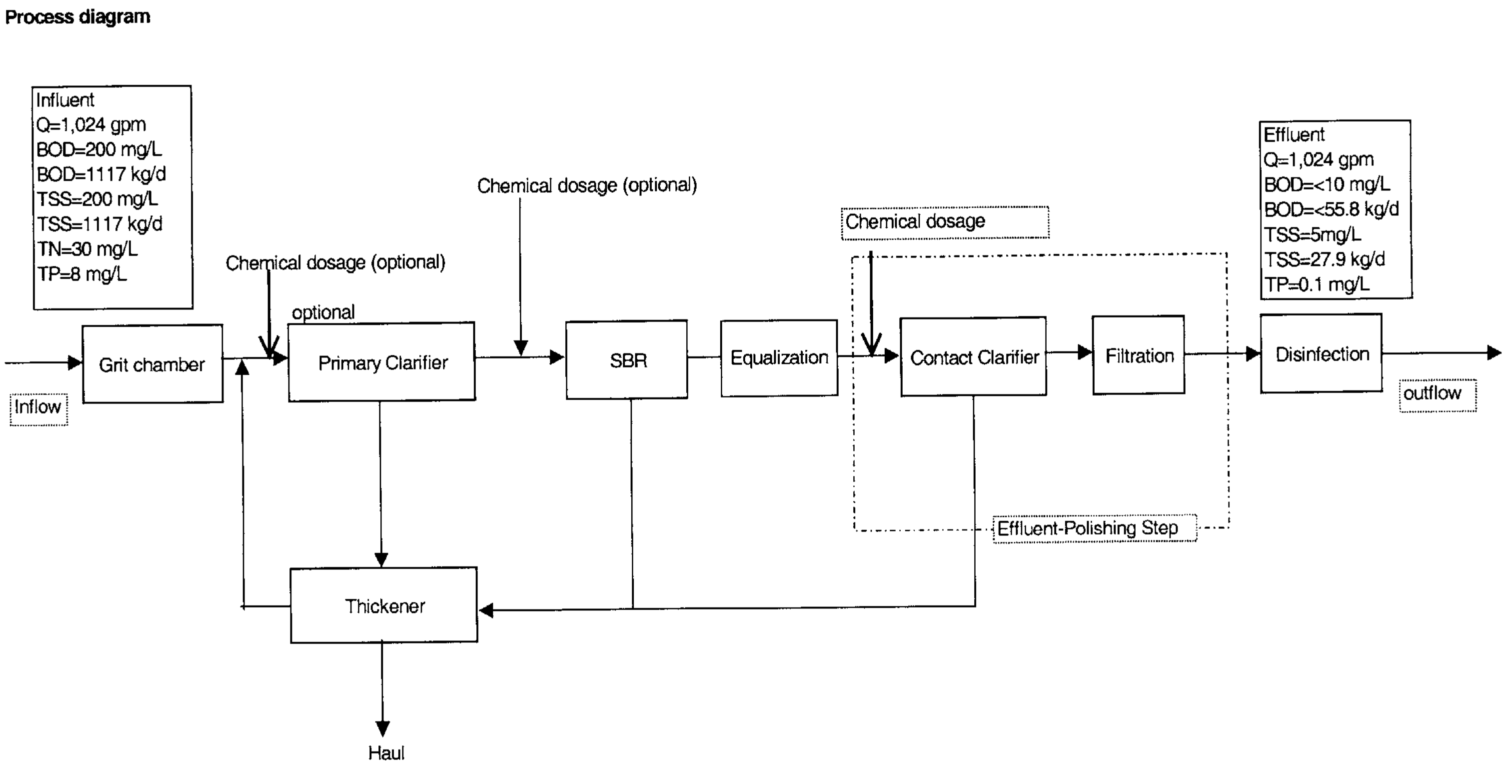


Figure 5-5. Proposed Process Diagram

5.11 BOD, TSS, TP, and sludge calculation

The concentrations of BOD, TSS and TP and the amount of sludge produced are calculated in the two cases. In Case 1, primary clarifiers are used before SBR and chemical dosing before primary clarification is applied, whereas, in Case 2, primary clarifiers are not used and SBR with chemical treatment is applied.

5.11.1 Case 1

Primary effluent and primary sludge calculation

By applying chemical enhanced primary treatment (CEPT), the removal rates for both BOD and TSS can be improved from 35% to 50% and from 50% to 75%, respectively. Therefore, assuming that the peak flow rate = 2.5 (safety factor) x 2,233 m^3/d =5,583 m^3/d and the sludge concentration = 0.05, the primary sludge production by TSS removal is calculated as follows:

Primary sludge produced by TSS removal in dry basis

$$= 200\ g/m^3 \times 5{,}583\ m^3/d \times 0.75 \times 10^{-3}\ kg/g = 837.5\ kg/d \quad \cdots\cdots (7)$$

Primary effluent concentrations of BOD and TSS are:

Primary effluent BOD = 200 mg/L x (1-0.50) = 100 mg/L, and

Primary effluent TSS = 200 mg/L x (1-0.75) = 50 mg/L.

Because the soluble orthophosphate form is removed by chemical precipitation, the dose of aluminum required to remove orthophsphate (5 mg/L of soluble phosphorus in 8 mg/L of total phosphorus) in primary treatment is as follows:

Al dosage = (Al/P)·(soluble phosphorus in primary influent – soluble phosphorus in primary effluent)

For the soluble phosphorus concentration of 0.1 mg/L in the primary effluent, the Al/P (w/w) ratio = 3 is required (Water Environment Federation, 1998). Therefore,

Al dosage = 3 (5 – 0.1) = 14.7 mg/L
Primary effluent TP = soluble P + insoluble P
= 0.1 + 3.0 = 3.1 mg/L

Using Al dosage = 15 mg/L = 15/27 = 0.555 mmole A/L and P removed = 4.9 mg/L = 4.9/39 = 0.158 mmole P/L, Al required is calculated from Equation (6):

Stoichiometric Al required = 0.8 mole A/mole P removed x 0.158 mmole/L = 0.126 mmole/L

Using excess Al added = 0.555 – 0.126 = 0.428 mmole/L, $Al_{0.8}(H_2PO_4)(OH)_{1.4}$ = 142.4 g/ mole, and $Al(OH)_3$ = 78 g/mole, the chemical sludge accrued from alum addition is calculated from Equations (2) and (4):

$$Al_{0.8}(H_2PO_4)(OH)_{1.4} \text{ sludge} = 0.158 \text{ mmole/L} \times 142.4 \text{ g/mole} = 22.5 \text{ mg/L},$$

$$Al(OH)_3 \text{ sludge} = 0.428 \text{ mmole/L} \times 78 \text{ g/mole} = 33.4 \text{ mg/L},$$

Total chemical sludge produced = 22.5 + 33.4 = 55.9 mg/L.

$$\begin{aligned}\text{Mass of chemical sludge} &= 55.9 \text{ g/m}^3 \times 5{,}583 \text{ m}^3\text{/d} \times 10^{-3} \text{ kg/g} \\ &= 312.1 \text{ kg/d} \quad \cdots\cdots (8)\end{aligned}$$

From Equations (7) and (8), the total primary sludge is:

$$312.1 + 837.5 = 1{,}149.6 \text{ kg/d} \quad \cdots\cdots (9)$$

Being calculated the same way as Al, the sludge produced by ferric chloride addition is as follows (primary effluent soluble P = 0.2 mg/L, Fe/P ratio = 5, $Al_{1.6}(H_2PO_4)(OH)_{3.8}$= 251 g/mole and $Fe(OH)_3$ = 106.8 g/mole) (Water Environment Federation, 1998):

$$\begin{aligned}\text{Chemical sludge produced} &= (38.9 + 19.4) \text{ g/m}^3 \times 5{,}583 \text{ m}^3\text{/d} \times 10^{-3} \text{ kg/g} \\ &= 325.5 \text{ kg/d} \quad \cdots\cdots (10) \\ \text{Total primary sludge} &= (7) + (10) = 325.5 + 837.5 \\ &= 1{,}163.0 \text{ kg/d} \quad \cdots\cdots (11)\end{aligned}$$

Consequently, ferric chloride produces a little more primary sludge than alum does. The primary effluent concentrations of TP are 3.1 mg/L by adding alum and 3.2 mg/L by adding ferric chloride.

Secondary effluent and activated sludge calculation

For effective operation of phosphorus removal (less than 1.0 mg TP/L), the COD:P ratio and the BOD:P ratio should be more than 40 and 20, respectively (Water Environment Federation, 1998). With the primary effluent BOD of 100 mg/L, COD of 170 mg/L and TP of 3.1 or 3.2 mg/L, the enhanced biological phosphorus removal (EBPR) treatment can be applied to the treatment of this wastewater.

The assumptions for the SBR designing are as follows (the values of μ_{max} and k_d are from Water Environment Federation, 1998, the values of Ks and *Y* are from Metcalf & Eddy, Inc., 1991):

MLVSS = 4,500 mg/L
P concentration in effluent SS = 1.5%
P concentration in activated sludge = 6.0%

μ_{max} = maximum specific growth rate = 0.8/day
Y = 0.6 g MLVSS/g BOD
k_d = 0.03/day
K_s = 60 mg BOD/L
2 units of SBR
peak flow rate (Q) = 5583 m^3/d
safety factor = 10
secondary effluent TSS = 5 mg/L.

1. Estimate the safety factor to be used in the design based on the peak loading. It appears that a safety factor of 2 should be adequate:
2. Determine the maximum growth rate for the nitrifying organisms under the stated operating conditions. The following expression can be used here:

$$\mu_{max} = 0.47\, e^{0.098(T-15)} \times DO/(K_{O2}+DO) \times [1-0.833(7.2-pH)]$$

$$\boxed{\text{Temperature correction factor}} \quad \boxed{\text{Dissolved oxygen factor}} \quad \boxed{\text{pH correction factor}}$$

where

μ_{max} = growth rate under the stated conditions of temperature, dissolved oxygen, and pH
T = temperature = 15 °C
DO = dissolved oxygen = 2.5 mg/L
K_{O2} = dissolved oxygen half velocity constant = 1.3 mg/L
pH = 7.2
k_d = 0.04/day

$$\mu_{max} = (0.47/d)\, e^{0.098(15-15)} \times 2.5/(1.3+2.5) \times [1-0.833(7.2-7.2)]$$
$$= 0.31/d$$

3. Determine μ:

$$\mu = \mu_{max} \times N/(K_N + N)$$
$$= 0.31/d \times /(0.4 +)$$

where N = effluent concentration of NH_4-N

K_N = half velocity constant = $10^{0.051T-1.158}$ = 0.4 mg/L (at 15 °C)

4. Determine the minimum design mean cell-residence time.

(a) minimum θ_c^M

$$1/\theta_c^M \approx \mu - k_d$$
$$1/\theta_c^M = 0./d - 0.04/d$$

$$= 0./\text{d}$$

$$\theta_c^M = 1/(/\text{d}) = \text{d}$$

(b) design θ_c

$$\theta_c = \text{SF}\ (\theta_c^M) = 2(\ \text{d}) = \ \text{d} \approx \ \text{days}$$

5. assuming biodegradable portion of effluent biological solids is 0.65, effluent soluble BOD (S), soluble, insoluble and total BODs are:

$$S = \text{Ks}(1 + k_d\, \theta_c)/[\ \theta_c\ (\mu_{\max} - k_d) - 1]$$
$$= 60\ \text{mg/L}\ (1 + 0.03/\text{d} \cdot 13\text{d})/\ [13\text{d}\ (0.8/\text{d} - 0.03/\text{d}) - 1]$$
$$= 9.3\ \text{mg/L}$$
$$\text{Insoluble BOD} = 0.65 \cdot 5\ \text{mg/L}$$
$$= 3.25\ \text{mg/L}$$
$$\text{Total BOD} = 3.25 + 9.3 = 12.55\ \text{mg/L}$$

6. sludge production (P_X):

$$P_X = Q[Y_S(\text{Si-S}) + Y_N(\text{Ni-N})]/\ (1 + k_d\, \theta_c)$$
$$= 5{,}583\ \text{m}^3/\text{d}\ [0.7\ \text{gVSS/g BOD}\ (100\text{-}) + 0.15\ \text{gVSS/g}\ \text{NH}_4\text{-N}\ (30\text{-})\ /(1{+}0.04/\text{d} \cdot \text{d})$$
$$= \ \text{kg VSS} \quad \cdots\cdots\ (12)$$

7. the mass of volatile suspended solids (VSS) produced in SBR:

$$\text{VSS} = Y_{\text{obs}} \cdot Q(\text{So} - \text{S})$$
$$= 0.43\ \text{g VSS/g BOD} \cdot (100 - 9.3)\ \text{g BOD/ m}^3$$
$$= 217.7\ \text{kg VSS/d}$$

8. the total mass of SS:

$$\text{SS} = \text{MLVSS}/0.8$$
$$= 217.7\ \text{kg MLVSS/d}\ /\ 0.8$$
$$= 272.2\ \text{kg MLSS/d}$$

9. the sludge to be wasted:

$$= 272.2\ \text{kg/d} - 5\ \text{g/m}^3 \times 5{,}583\ \text{m}^3/\text{d} \times 10^{-3}\ \text{kg/g}$$
$$= 244.3\ \text{kg/d} \quad \cdots\cdots\ (13)$$

10. assuming the phosphorus content = 6% in the activated sludge, the total, insoluble and soluble phosphorus concentrations in the secondary effluent are:

$$\text{P removed in WAS} = 244.3\ \text{kg/d} \times 0.06$$
$$= 14.7\ \text{kg/d}$$
$$\text{TP in secondary effluent} = (17.3 - 14.7)\ \text{kg/d}\ /5{,}583\ \text{m}^3/\text{d} \times 10^3\ \text{g/kg}$$
$$= 0.46\ \text{mg/L}$$

Insoluble P = 0.015 x 5 mg/L
= 0.075 mg/L
SP = 0.46 – 0.075 = 0.39 mg/L ······ (14)

11. P removal rate:
= (3.1 – 0.46) mg/L /3.1 mg/L
= 0.85

Tertiary effluent and sludge calculation

Assuming that alum required for tertiary treatment is 3 mg Al/L and final effluent TSS is 3 mg/L which contains 1.5% phosphorus (polymer is also added to make precipitation more effective) (Ross *et al.*, 1996),

(a) final effluent insoluble P:

Insoluble P = 0.015 x 3 mg/L
= 0.045 mg/L

(b) final effluent soluble P should be:

Soluble P = 0.1 – 0.045
= 0.055 mg/L

(c) $Al_{0.8}(H_2PO_4)(OH)_{1.4}$ and $Al(OH)_3$ sludge:

Al dose = 3 mg/L /27 mg/mmole = 0.11 mmole/L
P removed = 0.39 – 0.055 = 0.335mg/L
= 0.335mg/L /31 mg/ mmole
= 0.011 mmole/L
Stoichiometric Al required = 0.8 mole Al /mole P removed x 0.011 mmole
= 0.0086 mmole/L
Excess Al added = 0.11 – 0.0086 = 0.101 mmole/L
$Al_{0.8}(H_2PO_4)(OH)_{1.4}$ sludge = 0.011 mmole/L x 142.4 g/mole = 1.6 mg/L
$Al(OH)_3$ sludge = 0.101 mmole/L x 78 g/mole = 7.9 mg/L,

(d) the chemical and total sludge produced in the tertiary treatment:

Chemical sludge = (1.6 + 7.9) g/m^3 x 5,583 m^3/d x 10^{-3} kg/g
= 53.0 kg/d
TSS removed = 5-3 = 2 mg/L
Sludge produced by TSS removal = 2 g/m^3 x 5,583 m^3/d x 10^{-3} kg/g
= 11.2 kg/d
Sludge produced in EP step = 53.0 + 11.2 = 64.2 kg/d ······ (15)

Being calculated the same way as Al, the sludge produced in the EP step by ferric chloride is:

Chemical sludge produced = (2.7+8.3) g/m^3 x 5,583 m^3/d x 10^{-3} kg/g
= 61.4 kg/d
Sludge produced in EP step = 61.4 + 11.2 = 72.6 kg/d ······ (16)

The results are shown in Figures 5-6, 5-7 and 5-8.

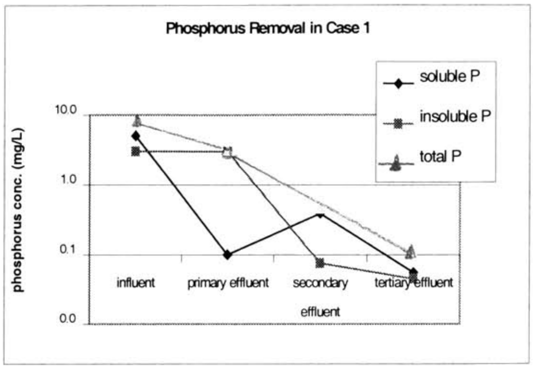


Figure 5-6.

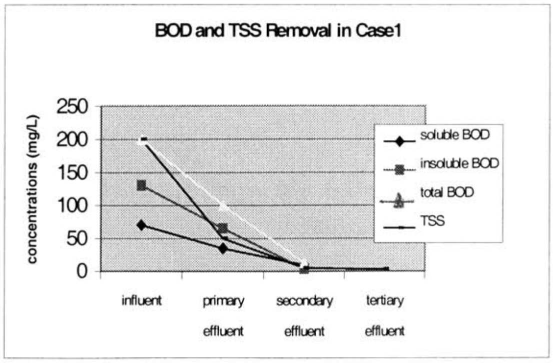


Figure 5-7.

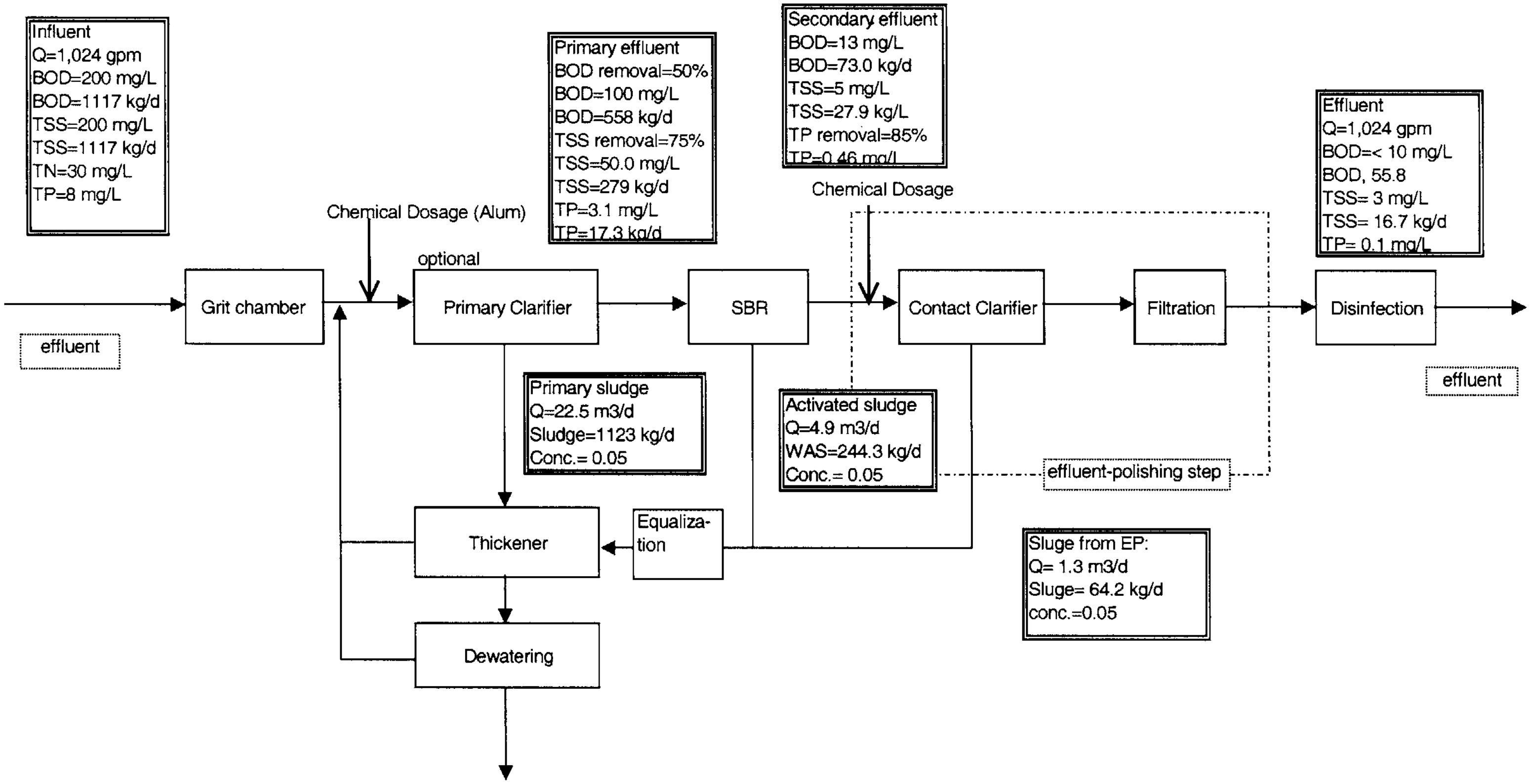


Figure 5-8. Process Diagram of Case 1

5.11.2 Case 2

Effluent after SBR and WAS calculation:

Assuming that the apparent yield = 0.43 (from Equation (12)), effluent TSS = 10 mg/L and WAS contains 8% phosphorus due to the simultaneous chemical addition (i.e. polymer) in SBR,

(a) sludge produced in SBR:

$$\begin{aligned} \text{VSS} &= Y_{\text{obs}} \cdot \text{Q(So} - \text{S)} \\ &= 0.43 \text{ g VSS/g BOD} \cdot 5{,}583 \text{ m}^3\text{/d } (00 - 9.3) \text{ g BOD/ m}^3 \\ &= 457.8 \text{ kg VSS/d} \\ \text{SS} &= \text{VSS/0.8} \\ &= 457.8 \text{ kg VSS/d / 0.8} \\ &= 572.3 \text{ kg SS/d} \\ \text{WAS} &= 572.3 \text{ kg/d} - 10 \text{ g/m}^3 \times 5{,}583 \text{ m}^3\text{/d} \times 10^{-3} \text{ kg/g} \\ &= 516.4 \text{ kg/d} \quad \cdots\cdots (17) \end{aligned}$$

(b) the concentrations of TP, insoluble P and SP in effluent from SBR:

$$\begin{aligned} \text{P removed in WAS} &= 516.4 \text{ kg/d} \times 0.08 \\ &= 41.3 \text{ kg/d} \\ \text{TP} &= (44.7 - 41.3) \text{ kg/d /5,583 m}^3\text{/d} \times 10^3 \text{ g/kg} \\ &= 0.61 \text{ mg/L} \\ \text{Insoluble P} &= 0.015 \times 10 \text{ mg/L} \\ &= 0.15 \text{ mg/L} \\ \text{SP} &= 0.61 - 0.15 = 0.46 \text{ mg} \end{aligned}$$

(c) P removal rate:

$$\begin{aligned} \text{P removal rate} &= (8.0 - 0.61) \text{ mg/L /8.0 mg/L} \\ &= 0.92 \end{aligned}$$

+Effluent from the effluent-polishing step and sludge calculation:

Assuming that alum required for this step is 3 mg Al/L and final effluent TSS is 5 mg/L which contains 1.5% phosphorus,

(a) Final effluent insoluble P:

$$\begin{aligned} \text{Insoluble P} &= 0.015 \times 5 \text{ mg/L} \\ &= 0.075 \text{ mg/L} \end{aligned}$$

(b) final effluent soluble P should be:

$$\begin{aligned} \text{Soluble P} &= 0.1 - 0.075 \\ &= 0.025 \text{ mg/L} \end{aligned}$$

(c) $Al_{0.8}(H_2PO_4)(OH)_{1.4}$ and $Al(OH)_3$ sludge:

Al dose = 3 mg/L /27 mg/mmole = 0.11 mmole/L
P removed = 0.46 – 0.025 = 0.435mg/L
= 0.435mg/L /31 mg/ mmole
= 0.014 mmole/L
Stoichiometric Al required = 0.8 mole Al /mole P removed x 0.014 mmole
= 0.011 mmole/L
Excess Al added = 0.11 – 0.011 = 0.099 mmole/L
$Al_{0.8}(H_2PO_4)(OH)_{1.4}$ sludge = 0.014 mmole/L x 142.4 g/mole = 2.0 mg/L
$Al(OH)_3$ sludge = 0.099 mmole/L x 78 g/mole = 7.7 mg/L,

(d) the sludge produced from the EP by alum:

Chemical sludge = (2.0+7.7) g/m^3 x 5,583 m^3/d x 10^{-3} kg/g
= 54.2 kg/d
TSS removed = 10-5 = 5 mg/L
Sludge produced by TSS removed = 5 g/m^3 x 5,583 m^3/d x 10^{-3} kg/g
= 27.9 kg/d
Sludge produced from the EP step = 54.2 + 27.9 = 82.1 kg/d …… (18)

Being calculated the same way as Al, the sludge produced from the EP step by ferric chloride is (3 mg Fe/L and final effluent TSS = 5 mg/L):

Chemical sludge produced = (3.5+3.4) g/m^3 x 5,583 m^3/d x 10^{-3} kg/g
= 38.6 kg/d
Sludge produced from the EP step = 38.6 + 27.9 = 66.5 kg/d …… (19)

The results are shown in Figures 5-9, 5-10 and 5-11.

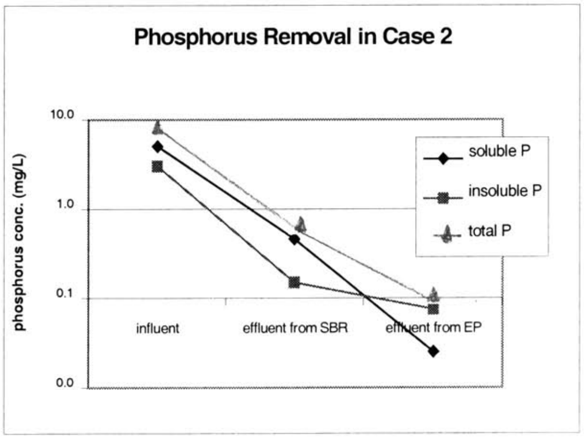


Figure 5-9.

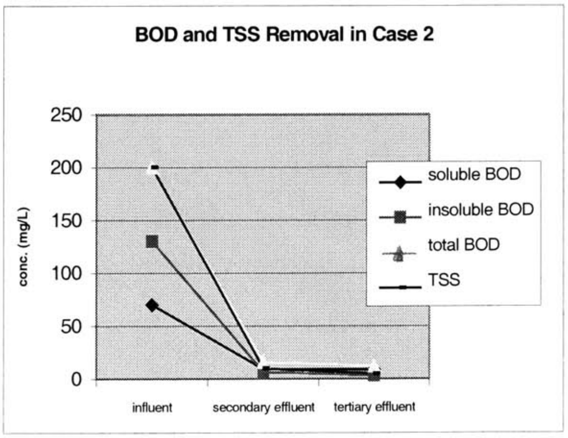


Figure 5-10.

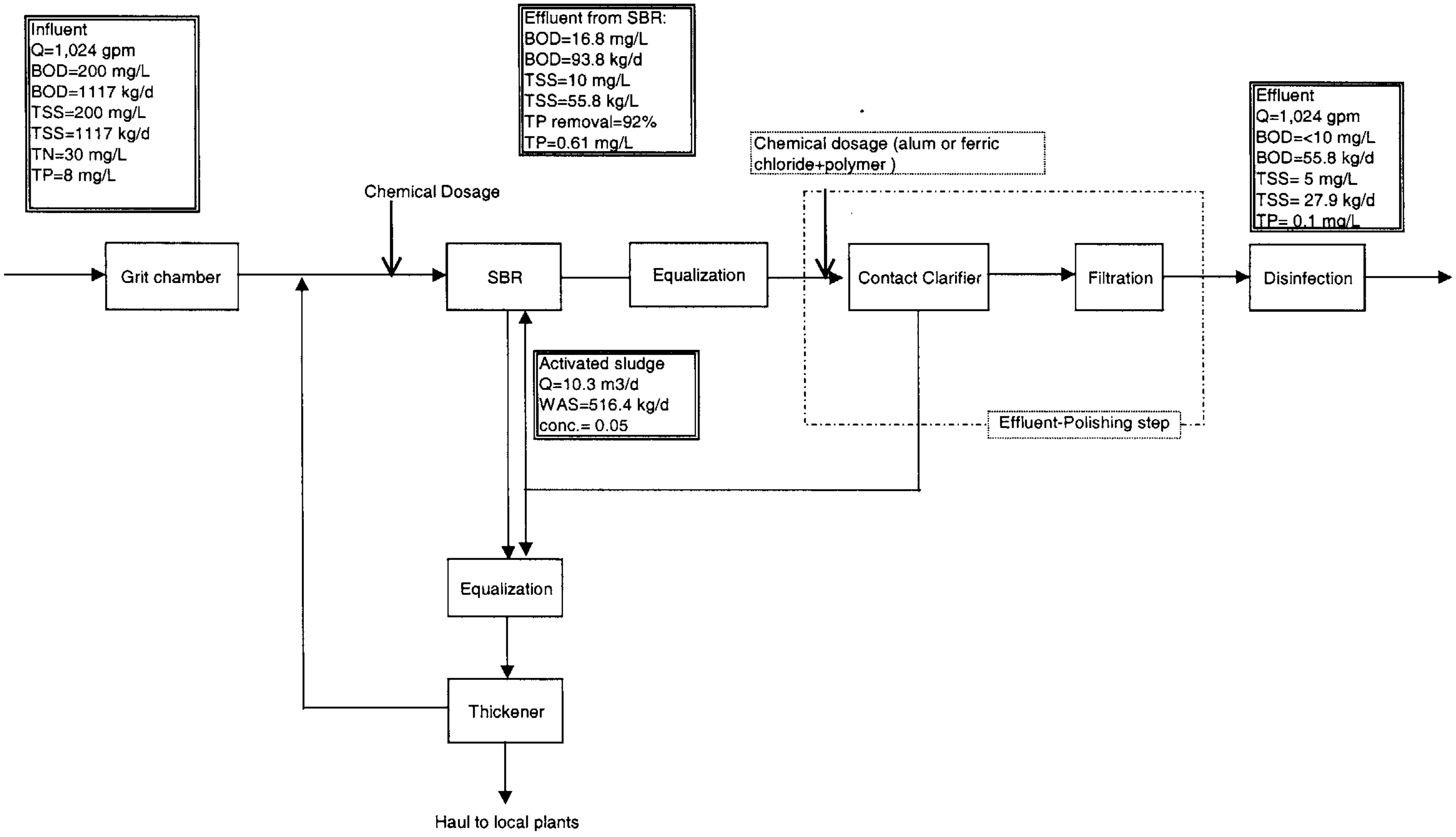


Figure 5-11. Process Diagram of Case 2

5.12 Fifth step of screening (sludge evaluation)

5.12.1 Case 1

The total sludge produced by alum addition is calculated from Equations (9), (13) and (15):

Total sludge produced = primary sludge + sludge from SBR + sludge from ER
= 1149.6 + 244.3 + 64. 2
= 1458.1 kg/d

The total sludge produced by ferric chloride addition is (from Equations (11), (13) and (16)):

Total sludge produced = primary sludge + sludge from SBR + sludge from ER
= 1163.0 + 244.3 + 72.6
= 1479.9 kg/d

In Case 1, though ferric chloride produces a little more chemical sludge than alum as shown in Figure 5-12, alum is more expensive than ferric chloride. Therefore, the application is up to the decisions by the clients who have to consider which is economically or environmentally more appropriate for the town.

The amount of 244.3 kg waste activated sludge per day is produced in SBR (from Equation (13)). This calculation is based on the assumptions applied in the previous section. The mean-cell residence time (MCRT) of 13 days is typical in terms of activated sludge treatment, and we assume that the calculation methods for the general activated sludge treatment processes are applicable to that for SBR. With these assumptions, the amount of the activated sludge produced in SBR is 244.3 kg/d. This sludge is assumed to contain 6% phosphorus due to the anaerobic step before aeration.

5.12.2 Case 2

The total sludge produced by alum addition is calculated from Equations (17) and (18):

Total sludge produced = sludge from SBR + sludge from ER
= 516.4 + 82.1
= 598.5 kg/d

The total sludge produced by ferric chloride addition is (from Equations (17) and (19)):

Total sludge produced = sludge from SBR + sludge from ER
= 516.4 + 66.5
= 582.9 kg/d

In Case 2, alum produces a little more sludge than ferric chloride. Although ferric chloride is economically and environmentally better than alum in this case, alum reduces soluble phosphorus levels more effectively than ferric chloride. Therefore, the decision-making on the chemical selection in this project is up to the clients.

As shown in Figures 5-12 and 5-13, the sludge production without the CEPT process is smaller (less than half) than with the CEPT. If Case 2 is possible in terms of the target phosphorus removal level (0.1 mg TP/L), we recommend the clients to use Case 2 without CEPT. Anyway, we have to check the effectiveness and efficiency of the proposed processes through the pilot-plant tests.

It should be noted that some assumptions applied in the previous section (i.e. WAS contains 8% phosphorus) might not be inappropriate and cannot always be relied upon, and at times the EPA target level might not be maintained. The clients and engineers should bear this point in mind.

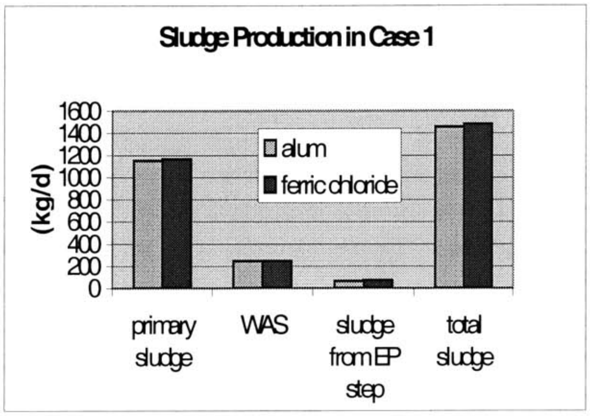


Figure 5-12.

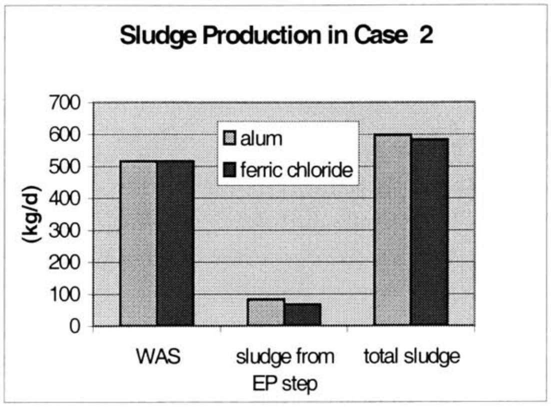


Figure 5-13.

5.13 Conclusions

The main points described in the previous sections are repeated as follows. There are 7 main points to be born in mind by the clients and engineers for the best approaches of the selection and to achieve the goals.

1. High-level phosphorus removal (less than or equal to 0.1 mg TP/L) is possible due to the advanced researches and experiments cited in the reference list.
2. There are two cases to be recommended for high-level phosphorus removal in small municipalities: (i) the combination of chemical enhanced primary treatment (CEPT), sequencing batch reactor (SBR) and physical effluent-polishing (EP) process (solids contact clarifier and filtration), and (ii) the combination of SBR and physical EP process.
3. The primary sludge produced in Case 1 (CEPT+SBR+EP) is the main contributor to sludge production and primary clarifiers are optional for small-sized phosphorus removal facilities like the proposed facilities in the Town of Acton.
4. Although ferric chloride produces more sludge than alum in Case 1, alum is about twice as expensive as ferric chloride. Further considerations are to be taken in terms of the decision-making by the clients.
5. SBR is highly recommended in this project due to its applicability to small wastewater treatment facilities. SBR can be used for nutrient removal. In case the Town of Acton comes

to need the nitrogen removal processes in the future, SBR can be easily modified to meet the demand.

6. Although Case 1 produces more sludge than Case 2, SBR functions for phosphorus removal cannot be always relied upon. Therefore, the clients should take this into consideration in terms of their final decision-making.
7. Generally speaking, the costs for the construction of the wastewater treatment facilities to reduce total phosphorus to the level of 0.1 mg P/L is quite expensive for small municipalities like the Town of Acton. To achieve more effective and efficient phosphorus reduction in the rivers and lakes in the Town of Acton, the regional wastewater treatment system is highly recommended, which enables not only the Town of Acton but also the neighboring towns and cities to strategically implement the phosphorus removal alternatives. This point is detailed in the previous chapter.

5.14 References

Bernardes, R.S.;Klapwijk, A. 1996. " Biological Nutrient Removal in a Sequencing Batch Reactor Treating Domestic Wastewater" Water Science and Technology. Vol.33, No.3:29-38.

Dittrich, J.; Gnirss, R.; Peter –Fröhlich, A.; Sarfert F. 1996 " Microfiltration of Municipal Wastewater for Disinfection and Advanced Phosphorus Removal." Water Science and Technology. Vol.34, No.9:125-131.

EPA. 1987. Design Manual: Phosphorus Removal. EPA/625/1-87/001:8-13

Gupta, Kailash; Hogge, Allen; Romero, Ivan; Swider, Clarence. 1998. "A Substitute Yields Saving." Water Environment & Technology. March 1998:37-40.

Hamamoto, Y. et al. 1997. "Development of Intermittent Cyclic Process for Simultaneous Nitrogen and Phosphorus Removal." Water Science and Technology. Vol.35, No.1:145.

Irvine, David et al. 1985. "An Organic Loading Study of Full-scale Sequencing Batch Reactors." Journal of Water Pollution Control Federation. Vol.57, No.8:847-853.

Irvine, David et al. 1998. "Sequencing Batch Reactors: A Fresh Look." Water Environment & Technology. September 1998:91-95.

Kuba, T. et al. 1993. "Biological Phosphorus Removal from Wastewater by Anaerobic-Anoxic Sequencing Batch Reactor." Water Science and Technology. Vol.27, No.5-6:241-252.

Kuba, T. et al. 1997. "Kinetics and Stoichiometry in the Biological Phosphorus Removal Process with Short Cycle Times." Water Science and Technology. Vol.31, No.4:2918-928.

Lamb, James C.; Brookhart, Morris V. 1994. "New BPR Process Achieves High Phosphorus Removal Levels." Public Works. October 1994:38-39.

Monat, Jamie P. 1998. " Synergies Between Ultrafiltration and Ion Exchange, or Why UF Should Be Used as IX Pre- and Post-Treatment." (Koch Membrane Web Site). http://www.kochomembrane.com/technical_info/separation.htm

Oesterholt, F.I.H.M.; Bult, B.A. 1993. "Improving Municipal Waste Water Quality by Effluent Polishing: A Pilot Scale Experiment at Winterswijk, the Netherlands." Water Science and Technology. Vol.27, No.5-6:277-286.

Rim, Y-T et al. 1997. "A Full-scale Test of a Biological Nutrients Removal System Using the Sequencing Batch Reactor Activated Sludge Process." Water Science and Technology. Vol.35, No.1:241.

Ross Deborah; Nutt, Stephen G.; Averill, David; Murphy, Keith. 1996. "Evaluation of Cost-effective Treatment Processes to Achieve Phosphorus Levels <0.10 mg/L in Municipal WWTP Effluent."

Shin, Hang Sik et al. 1992. "Simultaneous Removal of Phosphorus and Nitrogen in Sequencing Batch Reactor." Biodegradation Vol.3:105-111

Tanaka, Kazuhiro et al. 1991. "Study on Development of Phosphorus Removal Process by Contact Filtration." Water Science and Technology. Vol.23:739-745.

Water Environment Federation, American Society of Civil Engineering. 1992. Design of Municipal Wastewater Treatment Plant.

Water Environment Federation. 1998. Biological and Chemical Systems for Nutrient Removal.

XCG Consultants Limited et al. 1996. Research, Development and Demonstration of Cost-effective Tertiary Treatment Processes to Achieve Low Phosphorus Levels in Municipal WWTP Effluent.

6 North Acton Recreation Area Wetland Project

6.1 Background

The town of Acton recently constructed a 40-acre municipal park in North Acton, Massachusetts. The North Acton Recreational Area (NARA) was designed to include several soccer and baseball fields, an amphitheater, and a large swimming pond. While the newly founded park will benefit the town by providing recreational opportunities for its citizens, it will undoubtedly also generate or expose non-point sources of pollution that could affect the water quality of the swimming pond. An excessive inflow of phosphorus could eutrophy the pond, making it unsuitable for swimming and reducing its function as a wildlife habitat. Additionally, the pond's water eventually flows into the Nashoba Brook, which is part of the Assabet River Basin. Hence, the quality of the Assabet River Basin is directly related to the quality of the swimming pond.

Addressing the concern of eutrophication, Acton's natural resources director Tom Tidman suggested creating a treatment wetland in the park. Not only will a wetland reduce phosphorus inflow to the swimming pond, it will also indirectly reduce phosphorus inflow to the Assabet River Basin. Additionally, it will create a wildlife corridor for animals to cross the park, create a wildlife conservation area, and enhance the overall aesthetic value of the park. The wetland will be one of the many Best Management Practices (BMPs) that the town is implementing to reduce phosphorus loads to the Assabet River. Moreover, the constructed wetland will replace a smaller wetland that was flooded during the construction of the swimming pond.

Non-point sources (NPS) of phosphorus include the athletic fields (mostly through the application of fertilizer), the Town Forest, Quarry Road, the park's parking lot, and bird droppings. The wetland will intercept 3 of the 5 NPS of phosphorus: the Town Forest, Quarry Road, and the parking lot. Runoff from the athletic fields is collected and rerouted to a dry pond east of the park. Bird droppings from seagulls and geese are especially noticeable on the shoreline of the lake and are a great nuisance. Unfortunately, this non-point source is hard to control and no method is 100% effective at keeping the birds away.

Several parameters must be taken into account in the design of the treatment wetland. Listed in order of importance, these are:

1. *Provide Phosphorus Treatment*
   The wetland must be able to reduce phosphorus loading from approximately 0.2-0.5 mg/l to 0.05 mg/l, or a 75-90% reduction. Methods for removing phosphorus loading are detailed in section 6.3.
2. *Limit Costs*
   Cost is an issue in every construction project. The cost of constructing the wetland is estimated to exceed $30,000. In order to minimize costs, large construction works such as building embankments, excavating areas, and leveling terraces will be kept to a minimum. Apart from these major endeavors, most of the wetland construction will be done on a volunteer basis or by employing Concord Prison labor.

3. *Safety*
   Because the recreational area will be used by children, safety is a fundamental concern. Major efforts will be made to reduce the risk to Acton's children and adults alike by posting signs and by using plants to create a living barrier around the wetland.
4. *Aesthetics*
   Because of its high visibility in the recreational park, the wetland should be as attractive as possible. This will be achieved through extensive planting and using boardwalks and nature trails. A more detailed discussion is presented in section 6.7.
5. *Education*
   The town has placed a heavy emphasis on the educational value of the wetland. This will be achieved through educational kiosks placed on nature trails around the wetland, as explained in section 6.7.
6. *Biota diversification*
   The wetland will also be utilized to increase the number of native plant species in Acton and will serve as a wildlife corridor for Acton's numerous and diverse animals. Additionally, the wetland will attract water-friendly animals such as snapping turtles and frogs.

6.2 Watershed Hydrology

The wetland site in NARA occupies an area of approximately 0.5 acres, and collects runoff from the parking lot, from Quarry Road, and from the town forest (Figure 6-0.) If the swales (channels that collect runoff and guide it to the wetland) and the micro pool (at the bottom of the wetland) are included, the total area is approximately 1 acre.

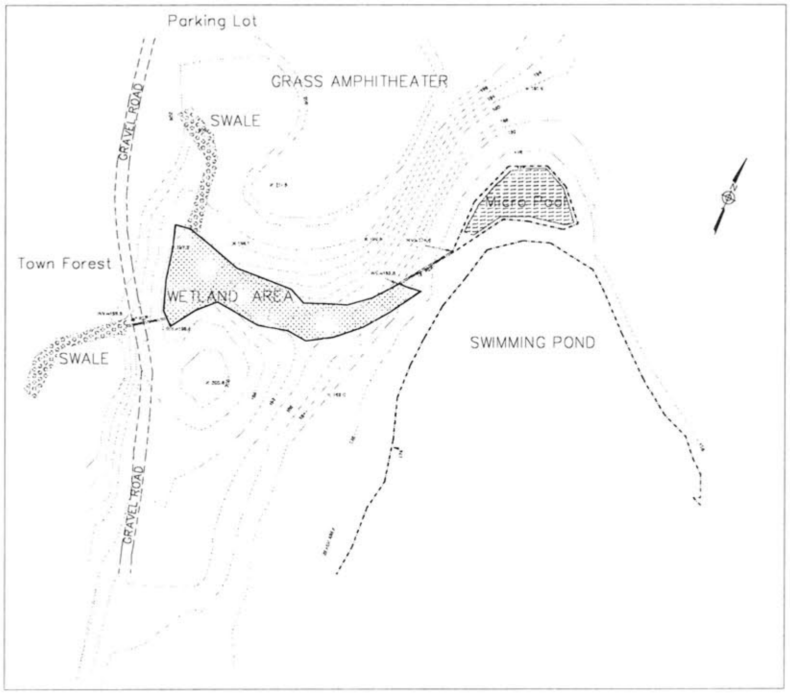


Figure 6-0: Map of the NARA wetland

Several hydrologic considerations have to be taken into account during the design of the wetland. First of all, the wetland is fairly small compared to the watershed it is in, a largely forested area of approximately 51 acres. Because of the significant size of the watershed, concern arises over the capability of the wetland to accommodate large amounts of water during storm events. Second, because the wetland is located on a fairly steep stretch of land (gradient = 4.4%), washout and erosion are legitimate problems. Lastly, the wetland must be able to withstand drought years without drying out (and hence dying.)

Hence, the first step in designing the wetland is to understand the hydrology of the watershed, to ensure that the wetland is able to manage runoff from a large storm without undergoing significant damage.

Three different runoff scenarios were analyzed: average runoff, higher-than-average runoff (flood conditions), and lower-than-average runoff (drought conditions). To determine average and drought runoff rates, the Thornthwaite water balance was used. This method calculates mean monthly runoff conditions from average monthly rain depths. For calculating flood conditions, three different methods were used: the Rational method, the Soil Conservation Service (SCS) technical release 55 (TR55), and the SCS technical release 20 (TR20) method. TR55 and TR20 are in essence that same calculation method, except that TR20 is computerized and slightly more complex. The Rational method will be used as a check.

The weather in Acton is typical of New England: wet spring and fall seasons, dry and cold winters, and dry and hot summers. Since climatologic data specific to Acton is not available, data from Boston's Logan Airport (approximately 25 miles east of Acton) was used. Because the wetland's main function is to maintain the quality of the water of the swimming pond, we are mostly concerned with the hydrology and climatology typical of the summer months, when the pond is in heavy use and when many people will be exposed to the water.

6.2.1 Thornthwaite Water Balance

The Thornthwaite water balance is one of the ways to calculate the amount of runoff that is generated during an average monthly rainstorm (Thornthwaite, 1955.) All it requires in terms of data are the mean monthly air temperatures, the mean monthly precipitation values, information on the water holding capacity of the soil and the latitude of the area of interest. In addition, the Thornthwaite method also calculates the potential evapotranspiration of the area of interest. Table 6-1 shows average monthly temperatures and precipitation depths, obtained from the NOAA (1974.)

	Jan	**Feb**	**Mar**	**Apr**	**May**	**June**	**July**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Rain Inches	3.94	3.32	4.22	3.77	3.34	3.48	2.88	3.66	3.46	3.14	3.93	3.63
Temp. Celcius	29.9	30.3	37.7	47.9	58.8	67.8	73.7	71.7	65.3	55.0	44.9	33.3

Table 6-1: Monthly average temperatures and precipitation depths for Logan Airport, Boston, Ma.

The results of the Thornthwaite water balance are shown in table 6-2 and 6-3. Both average rainfall and minimum rainfall have been examined to calculate expected runoff. The results of the Thornthwaite water balance for average rain events show that a large amount of water is expected to enter the wetland. Total volumes entering can be calculated based on the depth of runoff, the total surface area and the mean number of storms in one month (mean storm events based on Perrich, 1992.) We also know that the wetland can only retain approximately 14,000 ft^3 before the wetland overflows (section 6.4), so flooding will occur on a regular interval. In the next section (TR55) we analyze a greater-than-usual storm, and the volume of runoff discharged to the wetland. This design volume will be used to design the wetland so that no or minimal damage will be caused to the wetland.

The results of the low flow Thornthwaite water balance show that in some cases, no runoff can be expected in a month. Note that the probability of experiencing twelve months in a row without runoff is zero. Table 6-2 merely indicates that months with no runoff are possible, and should be anticipated in the wetland design.

Boston, WSFO, Massachusetts. Latitude = 42.22. watershed area of interest = 50.6 acres. 10 inches soil retention													
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Sum
Temp., F	29.9	30.3	37.7	47.9	58.8	67.8	73.7	71.7	65.3	55	44.9	33.3	
Heat index, I	0	0	0.5	2.37	5.22	8.09	10.19	9.45	7.25	4.14	1.72	0.05	49.0
Unadjusted Potential ET	0	0	0.015	0.048	0.087	0.126	0.15	0.142	0.114	0.071	0.036	0	
Correction factor	24.6	24.6	30.9	33.6	37.8	38.1	38.4	35.7	31.2	28.5	24.6	23.7	
Adjusted Potential ET, PE	0	0	0.46	1.61	3.29	4.80	5.76	5.07	3.56	2.02	0.89	0.00	27.46
Precipitation, P	3.94	3.32	4.22	3.77	3.34	3.48	2.88	3.66	3.46	3.14	3.93	3.63	
P-PE	3.94	3.32	3.76	2.16	0.05	-1.32	-2.88	-1.41	-0.10	1.12	3.04	3.63	15.31
Accumulated Water Loss	0	0	0	0	0	-1.32	-4.20	-5.61	-5.71	0	0	0	
Storage, ST	13.94	17.26	10	10	10	8.78	6.60	5.73	5.67	6.79	9.83	10.00	
Change soil moisture	0	0	0	0	0	-1.22	-2.18	-0.87	-0.06	1.12	3.04	0.17	
Actual ET	0	0	0.46	1.61	3.29	4.7	5.06	4.53	3.52	2.02	0.89	0	26.08
Moisture Deficit, D	0	0	0.00	0.00	0.00	0.10	0.70	0.54	0.04	0.00	0.00	0.00	1.38
Moisture Surplus, S	0	0	3.76	2.16	0.05	0	0	0	0	0	0	3.63	9.6
Water Runoff, RO	0.91	0.45	2.11	2.13	1.09	0.55	0.27	0.14	0.07	0.03	0.02	1.82	9.58
Snow Melt Runoff	0	0	0.73	3.27	1.63	0.82	0.41	0.20	0.10	0.05	0.03	0.01	7.25
Total Runoff, inches	**0.91**	**0.45**	**2.83**	**5.40**	**2.73**	**1.36**	**0.68**	**0.34**	**0.17**	**0.09**	**0.04**	**1.83**	**16.83**
Total Runoff, mm	22.09	11.04	71.38	137.17	69.42	34.71	17.36	8.68	4.34	2.17	0.65	44.49	423.49
Mean number of storms	5.69	5.03	5.80	5.89	5.86	5.36	5.11	5.28	4.50	4.47	5.56	5.94	
Volume Runoff, ft^3	**29315**	**16573**	**89713**	**168526**	**85406**	**46686**	**24485**	**11855**	**6952**	**3498**	**1408**	**56480**	

Table 6-2: Results from the Thornthwaite water balance using average precipitation values.

Boston, WSFO, Massachusetts. Latitude = 42.22. watershed area of interest = 50.6 acres. 10 inches soil retention													
	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Sum
Temp., F	29.9	30.3	37.7	47.9	58.8	67.8	73.7	71.7	65.3	55	44.9	33.3	
Heat index, I	0	0	0.5	2.37	5.22	8.09	10.19	9.45	7.25	4.14	1.72	0.05	49.0
Unadjusted Potential ET	0	0	0.015	0.048	0.087	0.126	0.15	0.142	0.114	0.071	0.036	0	
Correction factor	24.6	24.6	30.9	33.6	37.8	38.1	38.4	35.7	31.2	28.5	24.6	23.7	
Adjusted Potential ET, PE	0	0	0.46	1.61	3.29	4.80	5.76	5.07	3.56	2.02	0.89	0.00	27.46
Precipitation, P	0.92	1.15	1.48	1.24	0.53	0.48	0.52	1.25	0.35	0.96	1.72	1.03	
P-PE	0.92	1.15	1.02	-0.37	-2.76	-4.32	-5.24	-3.82	-3.21	-1.06	0.83	1.03	-15.83
Accumulated Water Loss			-5.61	-5.98	-8.74	-13.06	-18.03	-22.12	-25.32	-26.38			
Storage, ST	3.56	4.71	5.73	5.52	4.18	2.72	1.65	1.10	0.80	0.78	1.61	2.64	
Change soil moisture	0.92	1.15	1.02	-0.21	-1.34	-1.46	-1.07	-0.55	-0.30	-0.02	0.83	1.03	
Actual ET	0	0	0.46	1.45	1.87	1.94	1.59	1.8	0.65	0.98	0.89	0	11.63
Moisture Deficit, D	0	0	0	0.16	1.42	2.86	4.17	3.27	2.91	1.04	0.00	0.00	15.83
Moisture Surplus, S	0	0	0	0	0	0	0	0	0	0	0	0	0
Water Runoff, RO	0	0	0	0	0	0	0	0	0	0	0	0	0
Snow Melt Runoff	0	0	0	0	0	0	0	0	0	0	0	0	0
Total Runoff	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 6-3: Results from the Thornthwaite water balance using minimum precipitation values.

6.2.2 Technical Release 55 (TR55)

Technical release 55 (TR55), released by the Soil Conservation Service (SCS), provides a method for analyzing the runoff generated during a rainstorm event. TR55 is designed for small watersheds of less than 1 square mile (wetland watershed is less than 0.1 square mile.) It is assumed that runoff for the current storm event is independent of the rainfall of previous storm events, which is a reasonable assumption for small watersheds, but may prove inaccurate for larger watersheds. In addition to rainfall, other factors that affect runoff include land cover and use, soil type, watershed slope, and antecedent moisture conditions (McCuen, 1998, 1982.)

6.2.2.1 SCS 24 hour Rainfall-Runoff Depth Relation

The equation relating precipitation and runoff is:

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)}, \text{ inches}$$

Where P is the depth of precipitation for a certain design storm, and S is the potential maximum retention in inches. The depth of precipitation is chosen for a certain return period. For large construction projects such as the building of dams, often a return period of 100 years is chosen to be entirely within safety limits. As the name suggests, this is a storm that has a probability of occurring, on average, once every 100 years, and may cause serious damage. For the NARA wetland however, no human lives are at stake and no real property damage will ensue if the wetland fails. Therefore a return period of 10 years is sufficient in terms of safety limits.

The SCS method requires the 24-hour storm data input for the chosen return period. For Acton, the 10-year, 24hr storm depth value is 4.5 inches (Hershfield, 1961.) Empirical studies indicate that S, the potential maximum retention, can be estimated as follows (McCuen, 1982):

$$S = \frac{1000}{CN} - 10, \text{ inches}$$

Where CN is the runoff curve number. The runoff curve number is a function of land use, antecedent soil moisture, soil type and hydrologic conditions. Curve numbers are well tabulated and can be found in most hydrology text books (McCuen 1998, 1982.)

Once S is determined, the time-of-concentration can be found from the following equation:

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}}, \text{ hrs}$$

Where L is the time lag (i.e. the time from the center of mass of rainfall excess to the peak discharge,) Y is the slope in percent, and l is the hydraulic length in feet (McCuen, 1982.) Empirical evidence shows that the time-of-concentration, in hours, is related to the time lag by:

$$t_c = \frac{5}{3}L, \text{ hrs}$$

The time-of-concentration is a measure of the time for a particle of water to travel from the most distant point in the watershed (hydrologically speaking) to the point where the design is to be made.

6.2.2.2 *Area and Curve Number*

Curve numbers are dependent on the soil type of the area. The SCS developed a soil classification system that consists of four groups (A,B,C,D) and are described as follows:

Group A	Deep sand, deep loess, aggregated silts
Group B	Shallow loess, sandy loam
Group C	Clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay
Group D	Soils that swell significantly when wet, heavy plastic clays, and certain saline soils.

The soil type of the NARA wetland watershed was determined by the SCS and USDA, and maps of the area are available (SCS, 1991.) In addition to soil type, the curve number depends on the antecedent soil moisture. The SCS developed three antecedent soil moisture conditions (I, II, III):

Condition I	Soils are dry, but not to wilting point, satisfactory cultivation has taken place.
Condition II	Average conditions
Condition III	Heavy rainfall, or light rainfall and low temperatures have occurred within the last 5 days; saturated oil.

Condition II is the typical average condition for Acton. Group types, sub areas and curve numbers are shown in table 6-4. The total curve number for the watershed can be calculated by adding each individual curve number weighed over its area.

Area Name	Area, acres	Group type	Curve number	% of total area
Upland Forest: swamps	8.46	D	77	0.16
Upland Forest: sandy	3.21	A	25	0.06
Upland Forest: forest	35.87	B	55	0.70
Houses: roofs	1	-	98	0.02
Houses: residential lot	1.5	B	70	0.03
Road	0.66	-	98	0.01
Parking Lot	0.77	-	98	0.01
Total	51.47	-	59.21	1.0

Table 6-4: Curve number and area for the NARA watershed.

With the curve number determined, the potential maximum retention and the potential runoff depth can be calculated:

$$S = \frac{1000}{59.21} - 10 = 6.89 \text{ inches}$$

$$Q = \frac{[4.5 - 0.2(6.95)]^2}{[4.5 + 0.8(6.95)]} = 0.97 \text{ inches}$$

6.2.2.3 Slope and Hydraulic Length

The slope of the watershed area was calculated by averaging individual slopes over their hydraulic length, and is approximately 4.62%. By contrast, the total hydraulic length is not an average of the individual components, but rather the longest possible path from the watershed to the outlet, 3200ft.

Now that the slope and hydraulic length is known, the time lag and the time-of-concentration can be found:

$$L = \frac{(3200)^{0.8}(6.95+1)^{0.7}}{1900(4.62)^{0.5}} = 0.67 \text{ hours}$$

$$t_c = \frac{5}{3}(0.67 hrs) = 1.1 \text{ hours}$$

6.2.2.4 Peak Discharge

Once the area, slope, curve number, return period, and 24-hour precipitation storm depth are determined, the peak discharge can be calculated (McCuen, 1982):

1. Required Input
   A = 51.5 Acres (Drainage Area)
   T = 10 Years (Return Period)
   P = 4.5 inches (Rainfall depth for 24-hour, 10 year storm event, in Acton)
   Y = 4.62 % (average watershed slope)
   CN = 59 (runoff curve number)

2. Compute Volume of Runoff, Q
   S = 6.89 inches
   Q = 0.97 inches

3. Watershed Slope Interpolation Factor, SF
   HL = 3200 ft (Hydraulic Length)
   EA = 80 Acres (equivalent drainage area, McCuen, 1982)
   HF = 0.64 (HF = A/EA)

4. Obtain Unit Peak Discharge, QU
   QU = 36 cfs/inch (Appendix C1, Fig 2)

5. Watershed Slope Interpolation Factor, SF (McCuen, 1982)
6. SF = 1.05

7. Ponding and Swamp Storage Adjustment Factor, PF
   PPS = 16% (percent of ponds and swampy areas, based on actual drainage area, A)
   Location in watershed: Center/Spread out.
   PF = 0.58 (McCuen, 1982)

8. Peak Discharge QP, Calculations with Adjustment
   QP = QU * Q * HF * SF * PF
   QP = 13.47 cfs

9. Additional Parameters
   L = 0.67 hours (time lag)
   t_c = 1.1 hours (time of concentration)

The TR-55 method thus predicts a peak runoff off 13.47cfs for a 24-hour storm event with a 10-year return period.

6.2.3 Rational Method

As a check on the TR55 method, the rational method will similarly be used to calculate the peak runoff flow rate. The rational method is primarily used for small watershed design problems, where short duration storms are critical (McCuen, 1998.) This method relates the peak discharge to the drainage area, the rainfall intensity and the runoff coefficient as follows (McCuen, 1998):

$$q_p = CiA$$

Where q_p = peak discharge, ft^3/sec
C = runoff coefficient
A = drainage area, acres
i = rainfall intensity, inches/hr.

The rainfall intensity is obtained from an intensity-duration-frequency curve (Hershfield, 1961), using a return period and duration equal to the time of concentration. The time of concentration was found to be 1.1 hours, as explained in the TR55 method. Using a time of concentration of 1 hour, the 10-year storm predicts a rainfall intensity of 1.8 inches/hr (Hershfield, 1961.) The runoff coefficient varies with land cover, land use, soil group and watershed slope. For the watershed of our interest, the total runoff coefficient will be the sum of the runoff coefficient multiplied by the area of each subunit.

$$CA = \sum C_i A_i$$

The calculated runoff coefficients are shown in table 6-5.

Land Use	Ci	Ai	Ci*Ai
Forest	0.11	47.54	5.23
Streets	0.85	0.66	0.56
Housing: residential lot	0.23	1.5	0.35
Housing: roofs	0.85	1	0.85
Parking Lot	0.85	0.77	0.65
overall area, Ci		51.47	7.64

Table 6-5: Runoff coefficient using the Rational method.

The peak discharge can now be calculated:

$$q_p = 1.8\frac{in}{hr}(7.64acres)(\frac{1ft}{12in})(\frac{43560ft^2}{acre})(\frac{hr}{3600\sec}) = 13.87\frac{ft^3}{\sec}$$

The Rational method predicts a peak runoff discharge similar to TR55.

6.2.4 Technical Release 20 (TR20)

The SCS also developed a FORTRAN based program, TR20, to develop runoff hydrographs with a design storm as input. It's a single-event model that uses the SCS runoff equation and the SCS curvilinear unit hydrograph (McCuen, 1998.) The SCS developed four dimensionless rainfall distributions using the Weather Bureau's Rainfall Frequency Atlases (McCuen, 1982.) These distributions can be applied to different areas around the United States. The distributions are based on generalized rainfall depth-duration-frequency relationships, and they calculate incremental depths of rainfall over the storm duration (24 hours.) A type III distribution was used for Acton, and the rainfall hyetograph is shown in figure 6-1 and 6-2.

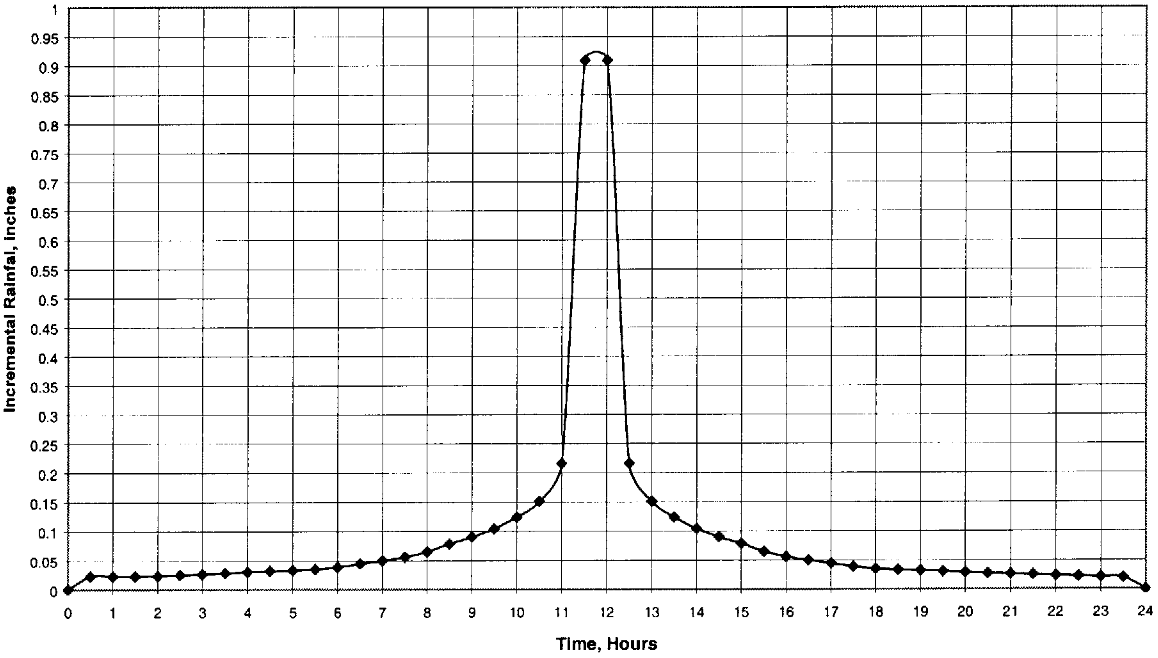


Figure 6-1: Hyetograph for Acton. Incremental Depths for 24 hour, type III storm, 10-year return period

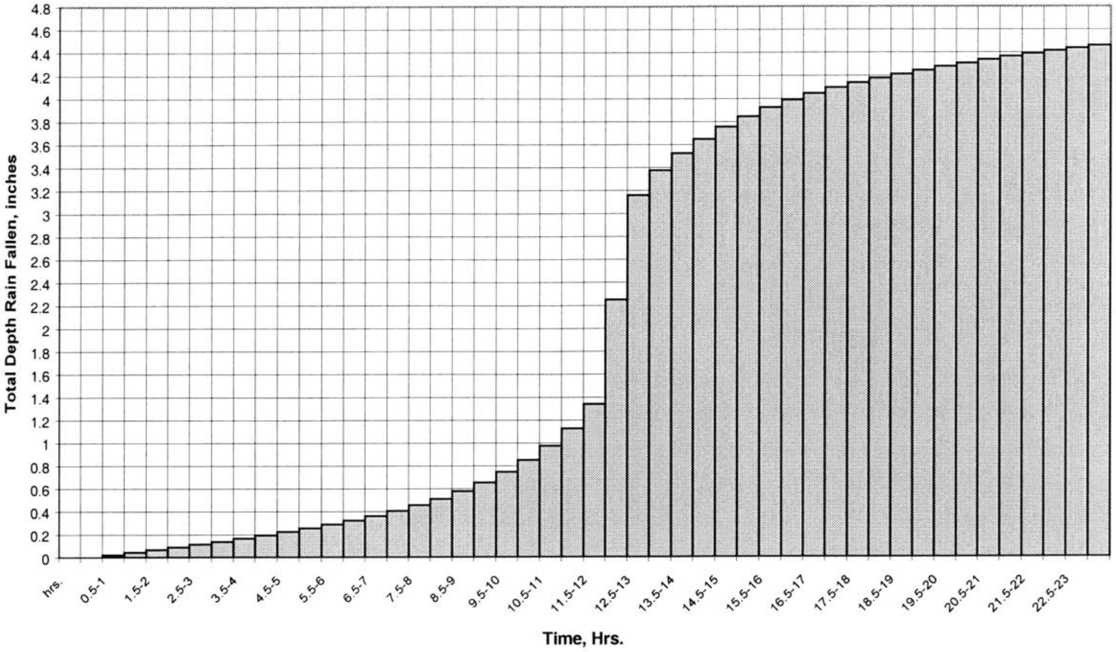


Figure 6-2: Hyetograph for Acton. Cumulative Depth for 24 hour, type III storm, 10-year return period

6.2.4.1 *Runoff Hydrograph*

A hydrograph is a graph of the runoff discharge rate, which passes a particular point, versus time (McCuen, 1982.) The hydrograph is a function of precipitation, watershed characteristics and geologic factors. A total runoff hydrograph consists of both surface runoff and baseflow. In the NARA wetland design, the baseflow will be assumed zero because we are mostly interested in the hydrology of the wetland during the summer months, when baseflow is very low.

A total runoff hydrograph shows the runoff over the whole duration of the storm and emphasizes four important concepts (McCuen, 1982):

1. Runoff occurs from precipitation excess, which equals the total precipitation minus any losses incurred such as interception, depression storage and infiltration.
2. The excess precipitation is applied at a constant, uniform rate.
3. The excess is applied with a uniform spatial distribution.
4. The intensity of the rainfall excess in constant of over a specified period of time, called the duration.

6.2.4.2 *Model Results*

TR20 requires limited input: time of concentration, design storm data, rainfall depth for a 24 hour storm (any return period), curve number, watershed area, and antacedent soil condition. The output is shown in figure 6-3. From the graph and table, the peak discharge is calculated to be 18.1cfs and the runoff depth was found to be 0.96inches, approximately the same results as TR55. Since this is the greatest discharge yet, it will be used in the design of the wetland. The total runoff volume from the watershed is approximately:

$$V = Q \times A = (0.96in)(\frac{1ft}{12in})(51.47acres)(\frac{43560ft^2}{acre}) = 179{,}363ft^3$$

Since the detention pond can only hold a maximum volume of 8000 cubic feet (see section 6.4), there will be significant flooding during such a rainstorm. This will be addressed in section 6.5.

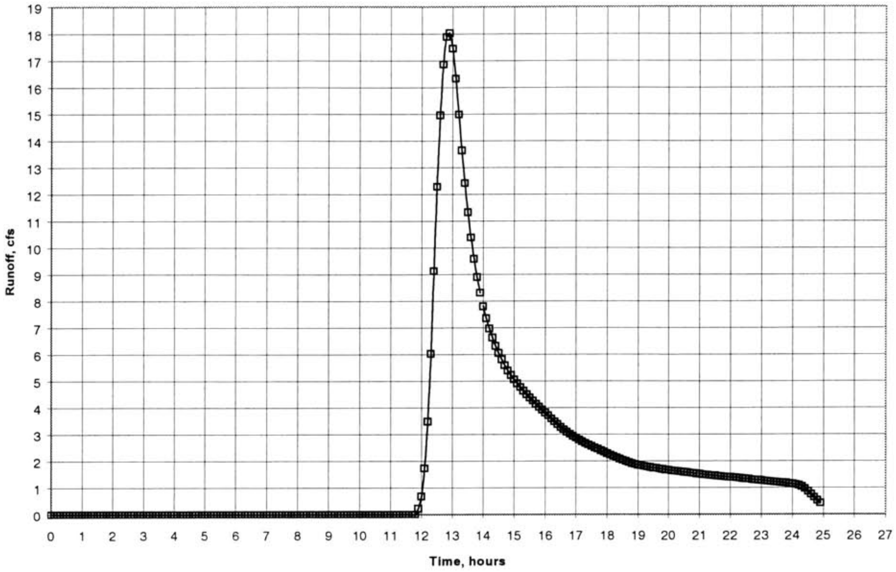


Figure 6-3: Total runoff hydrograph

6.3 Phosphorus Removal Techniques

Phosphorus is a nutrient present in storm water discharge, and is often the limiting nutrient in fresh bodies of water. Excessive amounts of this element will cause rivers, lakes and ponds to eutrophy at a faster than normal rate. Eutrophication causes algal blooms, and is of primary concern because eutrophied waters are characterized by a foul smelling odor, lack of biota life, and high turbidity.

Phosphorus in wetlands exists in several states including dissolved phosphorus, solid mineral phosphorus and solid inorganic phosphorus. Inorganic phosphorus, i.e. phosphate, dissociates as follows (Kadlec, 1996):

$$H_3PO_4 \Leftrightarrow H_2PO_4^- + H^+$$
$$H_2PO_4^- \Leftrightarrow HPO_4^{2-} + H^+$$
$$HPO_4^{2-} \Leftrightarrow PO_4^{3-} + H^+$$

Phosphorus entering a wetland can be removed through sedimentation, soil adsorption, microbial metabolism, chemical precipitation, phosphine emission and plant uptake. Specific storages of phosphorus in a peat-based wetland are shown in figure 6-4 (Kadlec, 1996.)

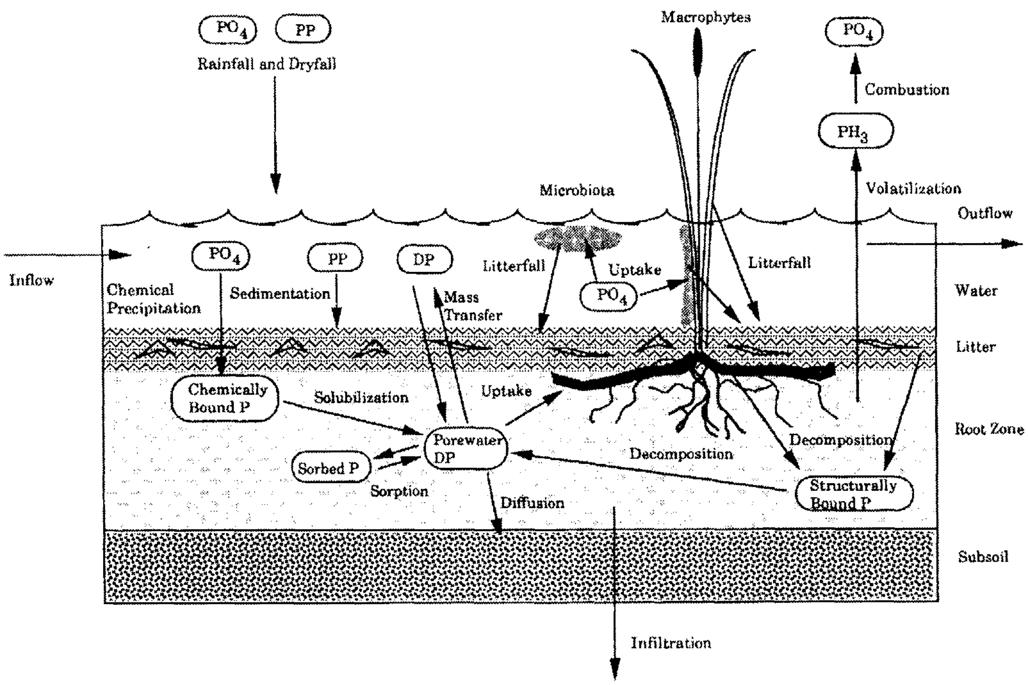


Figure 6-4: Phosphorus removal and cycling in a peat-based wetland (Kadlec, 1996)

6.3.1 Chemical Precipitation and Phosphine Emissions

Chemical precipitation involves the addition of chemicals (i.e. alum, ferric chloride, lime or ferric sulfate) to enhance precipitation of phosphorus to facilitate its removal (Tchobanoglous, 1991.) Even though it is an effective method of removing phophorus, it is also costly, requires supervision by technical personnel, increases sludge production, and generally is not appropriate for a natural system. Therefore, chemical precipitation will not further be considered or discussed.

Little is known about phosphine, a gaseous form of phosphorus. Phosphine has a high vapor pressure and may be emitted in much the same way as methane is emitted from wetlands (Kaldec, 1996), but it has not received much attention in relation to wetlands and has not been studied extensively. Since there is no widely acceptable way to quantify phosphine emissions, it will not be considered further in this analysis.

6.3.2 Sedimentation

Sedimentation is the separation of suspended particles from water by gravitational settling. Generally, if total suspended sediment (TSS) removal in a detention basin is high, removal of other pollutants that bind to particles is high as well (Stanley, 1996.) Since phosphorus binds to sediment particles quite well, it is expected that a significant amount of this nutrient will be removed. For particles with a Reynold's number less than approximately 0.5, settling can be described by Stokes' law (Tchobanoglous, 1991):

$$V_0 = \frac{1}{18}\frac{g}{\mu}(\rho_s - \rho)D^2$$

Where V_0 = settling velocity
μ = viscosity of fluid
ρ_s = density of particle
ρ = density of fluid
g = acceleration due to gravity
D = diameter of particle

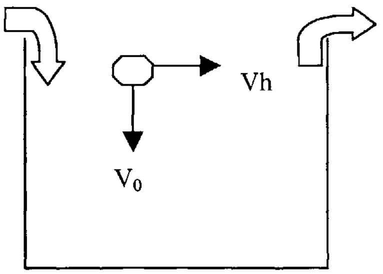


Figure 6-5: Shematic diagram of the detention pond

Figure 6-5 shows a schematic diagram of a single particle, where V_0 is the settling or vertical velocity, and V_h is the horizontal velocity (the velocity of the water.) In the design of wastewater treatment facilities, it is common to design a basin such that all particles that have a terminal velocity equal or greater to V_0 will be removed. This terminal velocity, or overflow rate, is defined as:

$$V_0 = \frac{Q}{A}$$

Where A = surface of the sedimentation basin
Q = rate of incoming water

As for all settling basins, the terminal velocity is independent of depth, up to the extent that scouring is not a factor.

For an ideal settling tank, sedimentation removal efficiency ranges from 50% to 70% (Tchobanoglous, 1991.) Field data collected by Donald Stanley show slightly different values. A stormwater detention pond in Greenville, NC was used to measure pollutant concentration removal in runoff. The mean stormwater concentrations were comparable to ranges found in Acton, approximately 0.35mg/l of total phosphorus. The removal efficiences observed were 30-58% particulate phosphorus removal, 11-46% of dissolved phosphorus, 11-46% phosphate removal (Stanley, 1996.) The specific removal efficiency of the NARA wetland will further be discussed in section 6.6.

6.3.3 Soil Adsorption

Another sink for phosphorus in wetlands is soil, where the nutrient is buried in organic form (National Small Flows Clearinghouse, vol.5.) The length of the removal period depends on the adsorption capacity of the sediment and the available wetland area; removal decreases as the adsorption sites fill up. Although the adsorption capacity of soils is finite, it can be quite large, even for sandy soils. A municipal wastewater treatment plant, using soil adsorption for the removal of phosphorus, still reports low (0.1-0.4 mg/L) concentrations after 88 years (Tchobanoglous, 1991). The degree of removal depends heavily on the contact between water and the soil matrix, so the smaller the surface area to volume ratio, the more contact there will be. Thus, shallow depths work better than deep trenches for removing phosphorus in wetlands.

Phosphorus adsorption is governed by a set of equations relating porewater concentration, soil depth, total volume, water-filled volume, particle density, water content, and mass of soil particles. The concentration between porewater and sorbed phosphorus can be defined using the Freundlich isotherm, a power-fit law (Kadlec, 1996):

$$C_s = a_p C_W^b$$

Where a_p = Freundlich phosphorus capacity factor, [mg P/kg]/[mg P/l]
b = Freundlich exponent, dimensionless
C_S = sorbed phosphorus concentration, mg/kg
C_W = porewater phosphorus concentration, mg/L

In wet soil, total phosphorus storage may be represented by the total concentration multiplied by the total volume (Kadlec, 1996):

$$V_T C_T = V_W C_W + M_S (a_p C_W^b)$$

Where V_T = total volume = $V_W + V_S$, L
V_W = water filled void, L
V_S = soil volume, L
C_T = total concentration, kg/L
M_S = mass soil particles, kg

The total concentration can further be defined as:

$$C_T = \theta C_W + \rho_b(a_p C_W^b) = [\theta + \rho_b(a_p C_W^{b-1})]C_W = fC_W$$

Where $\theta = V_W/V_T$, water content
$\rho_b = M_S/V_T$, soil bulk density, kg/L
f = phosphorus soil storage factor, dimensionless

Moreover, the phosphorus soil storage is defined as:

$$f = [\theta + \rho_b(a_p C_W^{b-1})]$$

And the phosphorus soil capacity is

$$S = f\, C_W\, \delta$$

Where S = phosphorus soil capacity, g/m^2
δ = soil depth, m

Typical values for water content (θ) in wetlands range between 0.3-0.9, bulk densities range from 0.1 g/cm^3 for peats to 1.5 g/cm^3 for mineral soils, and typical values of f range from 5 to 50. An order of estimate on the time needed to saturate the phosphorus adsorption capacity of a soil is (Kadlec, 1996):

$$t = S/J$$

where: t = time, yr.
S = phosphorus soil capacity, g/m^2
J = phosphorus removal rate, g/m^2/yr

Typical saturation times range from 1.5 months to 4.5 months. This time does not take into account any phosphorus absorbed by plants. Nor do these equations take into account temporal factors such as the age of the wetland and seasonal fluctuations. As a result, these regression equations have large standard errors. To more accurately represent phosphorus uptake, other parameters such as geographical region, age of wetland, seasonal dependence, types and density of plants, depth and duration of storm events, and temperature dependence should be accounted for. Since soil adsorption occurs over such a small time frame, it becomes somewhat negligible in our analysis of phosphorus adsorption.

Another factor that determines phosphorus uptake is the downward diffusion into soil media. Diffusion is governed by Fick's equation (Hemond, 1994):

$$J = -D\,(dC/dX)$$

Where J = flux density, $g/m^2/yr$

D = Diffusion coefficient, m^2/yr

X = downward distance, m

C = Concentration, g/m^3

The diffusion coefficient in wetlands is approximately half of the free water diffusion coefficient (Kadlec, 1996.)

$$D \cong 0.5 \times 10^{-5}\ cm^2/s = 0.016\ m^2/yr$$

Even at high gradients, the flux downward is slow and phosphorus will not penetrate to very deep depths in any significant amount of time.

6.3.4 Microbial Metabolism

Microorganisms assimilate inorganic phosphate, mineralize organic phosphorus, and are involved in the solubilization and mobilization of phosphate compounds. Microorganisms do not oxidize or reduce phosphorus, they move phosphates without altering the oxidation level. Microbes utilize phosphorus during cell synthesis and energy transport, consuming anywhere from 10 to 30% of influent phosphorus during wastewater treatment (Tchobanoglous, 1991). Only organic phosphates are available for consumption, not phosphate precipitates (Atlas, 1995). Certain microbes effectively incorporate inorganic phosphate during the production of ATP, reducing total phosphorus levels in storm water. The organism found to accomplish this most effectively (in activated sludge) belongs to the genus *Acinetobacter.* Specific bacteria that may prove useful in the uptake of phosphorus include *A. calcoaceticus*, *Pseudomonas vesicularis*, *A. lwoffii*, and *A. junii* (Jenkins, 1991.) Bacteria are naturally present in wetlands and do not need to be introduced.

ATP production is a two step process mediated by two enzymes, polyP-AMP phosphotransferase and adenylate kinase (Jenkins, 1991):

$$(\mathrm{polyP})_n + \mathrm{AMP} \rightarrow (\mathrm{polyP})_{n+1} + \mathrm{ADP}$$

$$2\ \mathrm{ADP} \rightarrow \mathrm{ATP} + \mathrm{AMP}$$

In addition to utilizing phosphorus for ATP production, for operation, and for maintenance, organisms also store phosphorus for future use. The overall reaction for aerobic respiration can be summarized as follows (Kadlec, 1996).

$$C_6H_{12}0_6 + 6H_20 + 60_2 + 38ADP + 38P = 6CO_2 + 12H_2O + 38\ ATP$$

Anoxic conditions may cause the release of phosphorus from microorganisms, thus acting as a source and not as a sink (Tchobanoglous, 1991.) Phosphorus may also be a factor in methanogenesis, which occurs under anaerobic conditions. Increasing the phosphorus content in soil can result in a shift from aerobic to anaerobic conditions in soil. Some undesirable effects of anaerobic wetlands are death of vegetation and undesirable odors, both of which can be a nuisance, especially in a recreational area. However, since the wetland in question will have fairly shallow depths, and some aeration will be provided by the spillway (creating turbulence that increases the oxygen level of the water), it is safe to assume that the area will remain aerobic.

6.3.5 Plant Uptake

Plant uptake is a slower method of phosphorus removal, as less than 1% of vegetation biomass is actually phosphorus. Increased phosphorus concentrations in a wetland, however, can spur an increase in tissue phophorus content by a factor of two to ten (Kadlec, 1996). Plants take 70% of their phosphorus from sediments and the remainder from water (Vincent, 1994). Phosphorus buried in wetland soil can thus be recycled by plant uptake. This action will also prolong the capacity of wetland soils to adsorb phosphorus.

Plant uptake is only a temporary form of storage; the nutrient is re-released when the vegetation dies and decays. Periodic harvesting or burning of the vegetation may remove this phosphorus source from the wetland, but care should be taken that the disposed vegetation is not near the swimming pond, as this could introduce phosphorus into the pond.

The plant species *Pragmatis australis* seems especially effective at removing phosphorus, but since it is an invasive species, it should not be used to plant the wetland (House, 1994.) Plants compete heavily with microorganisms for phosphorus intake; in most cases, the microorganisms use most of the available phosphate. Still, one author reports that emergent macrophytes may have an uptake capacity in the range of 50 to 150 kg $ha^{-1}year^{-1}$, and free-floating vegetation may take up phosphorus on the order of 50 to 300 kg $ha^{-1}year^{-1}$(Brix, 1994.)

6.3.6 Mechanism Summary

There are six processes by which phosphorus can be removed in stormwater runoff: chemical precipitation, phosphine emissions, sedimentation, soil uptake, microbial metabolism, and plant uptake. We've shown that chemical precipitation is not a reasonable mechanism to use in a natural setting. Too little is known about phosphine emission to be quantified at this point. Soil uptake was shown to be negligible compared to sedimentation, plant uptake, and microbial metabolism. Thus, the three most important removal processes at work in this wetland will be sedimentation, plant uptake, and microbial metabolism.

6.4 Wetland Design

The layout of the NARA wetland was almost completely predetermined by land availability and land topography. Only a small section of approximately 0.5 acres was appropriated to the treatment of stormwater. The land is located on a terraced hill; to keep the amount of construction to a minimum, the terraces will be disturbed as little as possible.

Several details apply to the entire wetland. To increase diversity, an irregular shoreline should be maintained everywhere; this will create visual isolation that will increase breeding success. Additionally, an impermeable layer of clay should be applied to the whole wetland area, to prevent infiltration of groundwater and exfiltration to the groundwater.

The wetland system will consist of two swales, a detention pond, three marshes and a micro pool, as shown in figures 6-6 and 6-7.

6.4.1 Swales

The swales leading to the detention pond are large enough to provide three key services: reduce incoming water velocities, provide preliminary treatment through planting, and increase retention through ponding. From personal observations at the site of interest, ponding occurs in all of the swales. Ponding can furthermore be encouraged by increasing the depression depth through excavation, or by placing stones behind the depression to create a damming effect. Note that such a small stone barrier is very susceptible to vandalism. Planting will cause the flow rates to decrease and will also provide preliminary phosphorus treatment.

6.4.2 Detention Pond

The upper terrace of the wetland will be transformed into a detention pond. Runoff from the parking lot, from Quarry Road, and from the Town Forest will be collected in this pond. The high bedrock elevation will limit the depth of the pond to approximately 1 to 2 feet. In addition to excavating, the pond's sides may have to be built up to achieve the desired average depth of 1 foot and the wet weather depth of 2 feet. The total surface area of the pond will be approximately $4000ft^2$ and the pond has an average width of 46 feet and average length of 95 feet. For safety reasons, the slope of the pond should not exceed a 7:1 ratio. The pond's shoreline should be irregularly shaped to increase wildlife establishment. Details are shown on figures 6-6 and 6-7.

6.4.3 Embankment I

Embankment I serves to retain runoff coming in the detention pond and serves to redistribute water slowly to marsh I. Since the bedrock is close to the surface, a bedrock foundation will be used. To prevent underseepage, cement grouting should be applied underneath the dam (US Department of the Interior, 1973.) The dam will rise approximately 2 feet above ground level, and will be constructed out of granite rocks, already available on site. The first foot above ground level will be mortared or grouted to prevent infiltration through the embankment. The top foot should not be grouted or mortared, and will act as a sieve, letting water pass through to marsh I. For safety reasons, the downstream side of the dam will be stepped and each step should not exceed 30 vertical inches. The total elevation drop from the top of the dam to marsh I is 4 feet. A schematic drawing of the dam is shown in figure 6-7.

6.4.4 Marsh I and II

Both marsh I and II consist of shallow areas and deep areas. This system of alternating water depths will: increase the retention effect of the wetland; will enhance phosphorus uptake; and will favor wildlife and vegetation establishment. Convention suggests that the shallow marsh should hold between 0 to 6 inches of water, while the deep marsh should hold between 6 to12 inches of water (Kadlec, 1996, Schueler, 1992, National Small Flows Clearinghouse, 1997.) The average dimensions of marsh I is 121 ft long by 60 feet wide and marsh II is 50 feet long by 43 feet wide (see figures 6 and 7)

6.4.5 Embankment II and III

Embankment II and III are similar to embankment I in that they will be made out of the same materials, but unlike embankment I, they will be completely impermeable (they will be cemented or grouted throughout.) Water will only be able to flow over the bank into the adjacent marsh. The total elevation drop from the top of the embankment II to marsh II is 3 feet while the drop from embankment III to marsh III is 4 feet. Like embankment I, the downstream side of the dam will be stepped, with a maximum vertical step of 30 inches, and the dams' foundations will also be similar. However, the foundation of embankment III should have a lip that extends beyond the base of the dam. The water in Marsh III flows much more rapidly than anywhere else in the wetland, and the lip will prevent erosion and scouring that could compromise the stability and safety of embankment III (figure 6-7.)

6.4.6 Marsh III

Unlike marsh I and II, which have very small slopes, marsh III will be heavily sloped. The total drop from the top of the marsh to the outlet pipe is approximately 6 feet, over an average length of 96 feet, or a 6.25% gradient. The average width of the marsh is 25 feet. The stones and small rocks that presently line this area should remain to prevent erosion. Some areas of deep water may be desired as indicated on figure 6-6. Planting this area is also recommended, both to prevent erosion and to provide phosphorus treatment. Only trees or sturdy shrubs that can resist high flow rates should be used.

6.5 Wetland Physics

Now that all hydrologic and design parameters of the wetland have been ascertained, we can determine what will happen to the wetland during dry and wet weather.

6.5.1 Velocities

Velocity is a function of slope, friction, and discharge rates. Between storms, when the discharge is small or non-existent, the Manning equation is used to relate velocity to the channel friction and slope. Even though the Manning equation has been shown not to be very accurate for wetlands it will be used due to lack of a better equation (Kadlec, 1996):

$$v = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Where v = velocity, ft/s
n = roughness coefficient
R = hydraulic radius, ft
S = slope, ft/ft

Values for the roughness coefficient are widely available in many publications. The closest approximated value for a wetland is (McCuen, 1992.)

n = 0.095 (grassed waterway)

The hydraulic radius is calculated as:

$$R = \frac{cross\,sectionarea}{wettedperimeter} = \frac{(depth)(width)}{2(depth)+(width)}$$

For optimum uptake of nutrients, suggested velocities in surface wetlands are between 0.7 and 5 cm/day or 2.66×10^{-7} and 19×10^{-7} ft/s (Kadlec, 1996.) To be within reasonable limits, the wetland will be designed for a velocity of 12×10^{-7} ft/s or 3.2 cm/day. From this velocity, we can calculate the required slope of marsh I and II. Note that the velocity in the detention pond is zero unless a discharge is applied. To achieve the desired flow rates, the slopes of marshes I and II should be very small, as shown in table 6-6.

	Velocity, ft/s	n	X-Area,ft	Wet perimeter, ft	R, ft	Slope, ft/ft
Marsh I	12×10^{-7}	0.095	60	62	0.968	6.11×10^{-15}
Marsh II	12×10^{-7}	0.095	43	45	0.956	6.22×10^{-15}

Table 6-6: Slope calculations for marshes I and II for low flow rates

The velocity in Marsh III is predetermined by its slope as shown in table 6-7.

	Slope, ft/ft	n	X-Area,ft	Wet perimeter, ft	R, ft	Velocity, ft/s
Marsh III	0.0625	0.095	25	27	0.926	3.73

Table 6-7: Velocity calculation for marsh III for low flow rate

For the purpose of calculating velocities during the 24-hour, 10-year design storm (high flow), we will assume that the velocity is uniform in each wetland cell. Average velocity is defined as the flow rate divided by the cross sectional area. Since this velocity will be used to check against scouring velocity, the most shallow depths in the wetlands have been used to calculate the greater flow rates. Note that the vegetation and the varying depth profiles are likely to produce velocities different from what is given table 6-8.

	Flow rate, cfs	Depth, ft	Width, ft	Average X-Area, ft^2	Velocity, ft/s
Detention Pond	18	1	46	46	.40
Marsh I	18	0.5	60	30	.60
Marsh II	18	0.5	43	21.5	.84

Table 6-8: Velocities in the wetland during high flow

Note that the velocity in Marsh III still follows Manning's equation during periods of high flow.

6.5.2 Scour Potential

The high flow velocities must to be compared to the maximum permissible flow velocities that do not cause scour (ASCE, 1992.) The ASCE manual on *Design and Construction of Urban Stormwater Management Systems* lists maximum permissible velocity of 4ft/sec, much greater than what is theoretically expected in the pond and marsh I & II. Marsh III comes closer to the maximum permissible velocity, but it is still below. As a safety factor, the small rocks and stones that are already in place should remain to prevent scouring and erosion.

The critical scouring velocity in detention ponds is given by Tchobanoglous (1996):

$$V_H = \left[\frac{8k(s-1)gd}{f}\right]^{1/2}$$

where V_H = minimum horizontal velocity that will just produce scour, ft/sec
k = constant which depend on type of material being scoured (0.04 for sandy material, 0.06 for more sticky, interlocking matter)
s = specific gravity of particles
g = acceleration due to gravity
d = diameter of particles
f = Darcy-Weisbach friction factor (typical values: 0.02-0.03)

If we assume an average particle size of 0.043mm (See section 6.6), the minimum scouring velocity becomes:

$$V_H = \left[\frac{8(0.05)(2.75-1)(\frac{9.8m}{s^2})(0.043\times10^{-3}m)(\frac{ft^2}{0.3048^2m^2})}{0.025}\right]^{1/2} = 0.356\frac{ft}{s}$$

The calculated maximum horizontal velocity during a high intensity storm is approximately 0.40 ft/s (from the velocity of water in the detention pond), which is slightly higher than allowable. Hence, small rocks and stones should be placed at the discharge region of the swales to slow down incoming water to prevent scouring. The floating aquatic vegetation will slow down the runoff as well.

6.5.3 Detention time

Detention times are a function of rain events and therefore fluctuate widely. In general, the longer the detention time, the better the removal of phosphorus in a wetland. During dry weather, the water will be mostly stagnant in the pond, and flowing very slowly in marsh I & II. Detention time is a function of the average velocity, as shown in tables 6-9 for average flow rates.

	Velocity, ft/s	Length, ft	Detention Time, **days**
Detention Pond	0	46	indefinite
Marsh I	$12\text{x}10^{-7}$	121	1167
Marsh II	$12\text{x}10^{-7}$	50	482
Marsh III	3.73	96	.0003
Total			~1649

Table 6-9: Detention times for low flow rates, days

Since the average time between storms is roughly between 5 and 6 days, and since almost complete flushing occurs during this time (section 6.2) the actual detention time will be much less than 1649 days. Average detention times for high flow rates is shown in table 6-10. Note that this residence time is not nearly long enough for any significant phosphorus removal to occur.

	Velocity, ft/s	Length, ft	Detention Time, **min**
Detention Pond	.40	46	1.9
Marsh I	.60	121	3.4
Marsh II	.84	50	0.99
Marsh III	3.8	96	0.0003
Total			~6.29

Table 6-10: Detention times for high flow rates, minutes

6.5.4 Pump Sizing

As demonstrated in section 6.2, there will be times when the wetland will go through periods of drought. Such an occurrence could be disastrous to the wetland vegetation and to the wetland treatment capability. To prevent this from happening, a pump should be installed in the micro pool to deliver a steady flow of water exceeding the evapotranspiration rate of the wetland. In addition to keeping the wetland functional, the recycling of water will also provide additional treatment of the swimming water, which will prove especially beneficial during months of heavy use (summer). Recall from table 6-2 that evapotranspiration peaks in July, at a rate of 5.76 inches/month. Applying this water loss over the whole wetland, the desired pump flow rate then becomes:

$$Q_{pump} \geq evaporationrate \times surfacearea$$

$$Q_{pump} \geq \frac{5.76in.}{month}(21780 ft^2)(\frac{1ft}{12in})(\frac{7.48gallons}{ft^3})(\frac{1month}{31days})(\frac{1day}{1440\min}) = 1.75GPM = 0.004\frac{ft^3}{s}$$

To be within a margin of safety, the pump will be designed to deliver a flow rate up to 4GPM. To calculate the required horsepower of the pump, the total head must be calculated (Lydersen, 1994):

$$H_m = (z_2 - z_1) + (P_2 - P_1) + \frac{v^2}{2g} + f\frac{L}{d}\frac{v^2}{2g}$$

Where: z_2 is the discharge elevation, 196ft;
z_1 is the suction elevation, 174ft;
P_2 is the pressure at discharge, 33.9 ft (atmospheric pressure);
P_1 is the pressure at section, 33.9 ft (atmospheric pressure);
v is the velocity in the pipe at maximum reading, ft/s;
g is the acceleration of gravity, 32.2 ft/s^2
f is the friction factor, dimensionless;
L is the length of run, ft;
D is the diameter of the pipe, ft.

Since the flow rate is very small, we can choose a standard 1.5" schedule 40 PVC pipe. The velocity head, $v^2/2g$, is related to pipe size and flow rate. Tables exist to facilitate calculations (Lydersen, 1994). Using a 1.5" pipe and a flow rate of 4GPM, the velocity is 0.63ft/s. Hence, the velocity head is 0.01ft.

Similarly, the friction head, $f(L/d)(v^2/2g)$ can be found from using tables (Lydersen, 1994). Assuming that two elbows will be used in laying the pipe, the equivalent length is 4.5ft per elbow = 9ft. The approximate distance between the detention pond and the micro pool is 680ft so the total combined length is 689ft. The friction loss of a 1.5" PVC pipe equals 0.12ft/ft*689ft = 82.68ft of friction loss.

Total mechanical head is equal to:

$H_m = (196-174)\text{ft} + 0.01\text{ft} + 82.68\text{ft} = 104.69\text{ft}$

A pump that can develop 4GPM flow rate against 105 feet of total head should be selected.

Horsepower is related to the mechanical head as follows:

$$HP = \frac{(Q)(H_m)(SpecificGravity)}{efficiency} = (\frac{2gal}{\text{min}})(105ft)(\frac{\text{min}-hp}{33000lb-ft})(\frac{8.34lb}{gal})(\frac{1}{0.55}) = 0.096hp$$

Because of the low flow requirements, a positive displacement pump such as a peristaltic or diaphragm pump is best suited to recycle water to the wetland. These two pumps are also able to handle abrasive liquids, such as sand or silt particles in water.

6.6 Phosphorus Removal Analysis

Now that all the wetland parameters (hydrology, layout, flow rates, detention times, etc...) are known, the phosphorus removal capability of the wetland should be re-explained. Recall that runoff is collected by the swales, enters the detention pond, flows through the marsh and empties in the detention pond. Assuming that the swales do not remove any significant amounts of phosphorus, the first site of treatment is the detention pond, removing this nutrient through sedimentation.

From section 6.3, the overflow rate is:

$$V_0 = \frac{Q}{A}$$

and as explained in section 6.4, the dimensions of the detention pond in the wetland are already predetermined, so the overflow rate is:

$A = 4000\ \text{ft}^2$

$Q = 18\ \text{ft}^3/\text{sec} \rightarrow$ design flow

$\therefore V_0 = 0.0045 ft/\sec = 16.2 ft/hr$

Recall that if the vertical velocity is greater than this, good settling will be achieved. To determine what fraction of particles will be removed, solve for the minimum diameter needed for a particle to settle. Recall Stokes' equation from section 6.3:

$$v = \frac{1}{18}\frac{g}{\mu}(\rho_s - \rho)D^2,$$

Where $v = 0.0045$ ft/sec

$g = 9.8\ \text{m/sec}^2$

$\mu = 1.3\text{cp}$, at 10°C (CRC handbook of chemistry and physics, 1989)

$\rho_s = 2.6\text{-}2.9\ \text{g/cm}^3$ for most minerals, avg. value = $2.75\ \text{g/cm}^3$ (Das, 1990)

$\rho = 1$, for water

$$D = \left[\frac{18 v \mu}{(\rho_s - \rho) g}\right]^{1/2}$$

$$D = \left[18(0.0045\frac{ft}{\sec})(1.3cp)(\frac{10^{-2} g}{cp - cm - \sec})(\frac{cm^3}{(2.75-1)g})(\frac{\sec^2}{980.7cm})(\frac{30.48cm}{ft})\right]^{1/2} = 0.043mm$$

Only particles with a diameter greater than 0.043mm will settle in the detention pond. Using the US Army Corps of Engineers' and the US Bureau of Reclamation's average diameters values for soil particles (table 6-11), we can see that gravel and sand will settle quite easily, but only the largest of the silt and clay particles will be retained in the detention pond.

Type of Soil	Gravel	Sand	Silt & Clay
Diameter, mm	76.2-4.75	4.75-0.075	<0.075

Table 6-11: Average soil particles diameters (Das, 1990)

Phosphorus mostly associates with clay particles and organic matter on soil constituents. A more thorough investigation of the runoff constituents needs to be undertaken to determine with accuracy the amount removed in the detention pond. The best removal estimates that can be given at this point are given by the study conducted by Stanley: 30-58% particulate phosphorus removal, 11-46% of dissolved phosphorus, 11-46% phosphate removal (Stanley, 1996.) A conservative value for total phosphorus removal through sedimentation is approximately 40%.

As seen in section 6.3, microbes have the capacity to remove anywhere from 10-30% of incoming phosphorus. Similarly, we saw that plant uptake can be in the order of 50 to 150 kg $ha^{-1}year^{-1}$ for emergent macrophytes, and 50 to 300 kg $ha^{-1}year^{-1}$ for free-floating vegetation (Brix, 1994.) This corresponds to an annual removal capability of 4.37-13.1 kg/year for the emergent vegetation in the NARA wetland, and 4.37-26.22 kg/yr for free-floating vegetation. We can also calculate the annual influx of phosphorus into the wetland from the yearly runoff volume: C_{avg} = 48 kg/yr. Thus, average removal capabilities of the vegetation runs between 9.1% and 54.6% removal capacity.

It is hard to estimate the total removal of phosphorus from the individual removal mechanisms; it is impossible to estimate to what extent one mechanism dominates over another. However, Kadlec suggests using a simple mass balance to determine the output concentration of phosphorus in runoff (Kadlec, 1996):

$$q\, dC/dy = -k(C-C^*) = -kC$$

Where q = hydraulic loading rate, m/day
C* = phosphorus background levels, usually zero
C = concentration of phosphorus
y = x/L = fraction distance from inlet to outlet
x = distance from inlet, m
L = total distance from inlet to outlet, m
k = uptake rate constant, m/day

Integrating this equation gives the concentration profile:

$$\ln (C/C_i) = (k/q)\cdot y$$

$$C = C_i \exp(-ky/q)$$

Where k/q = Da = Damkohler number

At the outlet of the wetland, the concentration then becomes:

$$C_o = C_i \exp(-k/q)$$

Where C_o = concentration of phosphorus at the outlet

C_i = concentration of phosphorus at the inlet

Since the NARA wetland will be a surface wetland, it can be modeled as an emergent marsh system, for which the uptake rate constant, k, equals 12.1 ± 6.1 m/yr. If no parameters but the inlet concentration is known, a simplified equation can be used to determine the outlet concentration (Kadlec, 1996).

$$C_o = 0.34\,(C_i)^{0.96}$$

Where: $0.02 < C_i < 20$ mg/l

$0.009 < C_o < 20$ mg/l

It is known that the inlet concentration in the NARA wetland is on average 0.35 mg/L. Thus, the outlet concentration becomes:

$$C_o = 0.34\,(0.35)^{0.96} = 0.12 \text{ mg/L}$$

This concentration is still approximately a factor of two greater than the outflow desired. However, the last step in the phosphorus treatment is a dilution step in the micro pool area. An investigative report done for the North Acton Recreational Park indicates that natural phosphorus levels in the pond are approximately 0.04mg/L (Pine and Swallow, 1989.) The total volume of the pond is approximately 15 million gallons at low level. Additionally, the micro pool occupies 3.9% of the total volume of the pond, so the approximate volume of water in the micro pool is 585,000 gallons. The new concentration of phosphorus in the micro pool is now:

<u>Existing phosphorus in micro pool:</u>

$$P = 585{,}000gal\left(\frac{0.04mg}{L}\right)\left(\frac{3.785L}{gal}\right)\left(\frac{kg}{10^{-6}mg}\right) = 0.086kgP$$

<u>Phosphorus concentration in wetland outflow:</u>

$$P = 71676ft^3\left(\frac{0.12mg}{L}\right)\left(\frac{28.32L}{ft^3}\right)\left(\frac{kg}{10^{-6}mg}\right) = 0.244kgP$$

<u>Phosphorus concentration in micro pool after mixing:</u>

$$C = \frac{(0.244+0.086)kg}{(585000+536316)gal}\left(\frac{10^6 mg}{kg}\right)\left(\frac{gal}{3.785L}\right) = 0.078\frac{mg}{L}$$

Note that the concentration is still slightly higher than the acceptable standard of 0.05mg/L, but after fully mixing in the swimming pool, the total phosphorus concentration in the pond will be approximately 0.0429mg/L.

As a comparison, if the runoff were allowed to flow into the swimming pond without treatment (i.e. without going through the wetland), the average phosphorus concentration in the pond would be approximately 0.051mg/L, slightly higher than what is acceptable, but not much different than the concentration after wetland treatment. Hence, when there is flooding in the wetland, the total possible phosphorus concentration in the swimming pond is still fairly low.

6.7 Landscaping

6.7.1 Vegetation

To ensure an aesthetically pleasing wetland, care should be taken to plant the wetland with desired species before invasive species appear. The establishment of native vegetation will limit the number of invasive species. Appendix B divides species that are native to this region in three categories: herbaceous emergent vegetation, submerged and floating vegetation, shrubs, and trees. Note that this database only contains a sample of wetland species; their availability should be checked with a wetland species retailer, who might also be able to suggest other species. Figure 6-8 shows the relative areas that should be planted with each respective category.

To obtain the wetland vegetation, local nurseries should be contacted that specialize in wetland vegetation (such as Environmental Research Corps (ERC) and its sister company BioMass Farms (www.wetlandsandwildlife.com)).

6.7.2 Wildlife

One of the original requirements of the wetland is that it needs to serve as a wildlife corridor, so that animals may safely cross the recreational park. The wetland will also serve as a small wildlife preservation area. One important step in achieving this goal is to diversify the vegetation in the wetland, and create irregular shorelines that promote small and numerous niches to form. Choosing plants that have a high wildlife value can also increase the number of animal species present. If desired, certain animals could be introduced artificially, such as snapping turtles and frogs, but is not a necessary step in attracting wildlife. Birds can be attracted by planting shrubs and trees where they can nest, and by installing bird houses around the wetland. Most important, the wildlife diversification of the wetland will greatly depend on the amount of human intrusion into the wetland. Providing a living barrier of shrubs and other plants around the wetland will limit human intrusion into the wetland.

6.7.3 Additional Landscaping Plans

Figure 6-9 shows additional features that may be desirable in and around the wetland. Trails around the wetland will both ensure that people can enjoy the wetland, while also limiting off-trail hiking in the wetland. Educational kiosks can be placed around the wetland, explaining the function of the wetland, and indicating the different plant species as well as animal species present. A small wooden bridge can be placed near embankment I that overlooks both the detention pond and the marshes. Both the trails and the pedestrian bridge should be wheel chair accessible.

6.8 Conclusion

The proposed treatment wetland in the North Acton Recreational Area is a prime example of a Best Management Practice that small communities throughout the United States can implement. The wetland's main function will be to remove phosphorus in storm water runoff to prevent the eutrophication of a swimming pond. On average, it will reduce the phosphorus concentration in storm water runoff from 0.35mg/L to 0.12mg/L. After mixing the runoff water with the swimming pond water, the final phosphorus concentration in the swimming pond is expected to be around 0.0429 mg/L. The main phosphorus removal methods in the wetland are sedimentation, microbial uptake, and plant uptake. The wetland will also provide many auxiliary benefits such as providing a wildlife haven, serving as an educational tool, and enhancing the overall aesthetic value of the park.

6.9 References

ASCE Manuals and Reports of Engineering Practice No.77. 1992. “Design and Construction of Urban Stormwater Management Systems.” American Society of Civil Engineers, New York, NY.

Atlas, R.M. 1995. “Principles of Microbiology”. 1st Edition, Mosby-Year Book, Inc., St. Louis, Missouri.

Brix, H. 1994. “Functions of Macrophytes in Constructed Wetlands.”, Water Science and Technology, 29(4):71-78.

CRC Handbook of Chemistry and Physics, 1989. CRC Press, Inc. Boca Raton, FL.

Das, B.M. 1990. “Principles of Geotechnical Engineering”. 2nd Edition, PWS-Kent Publishing Company. Boston, MA.

Harris, C.W. and Dines, N.T. 1988. “Time-Saver Standards for Landscape Architecture, Design and Construction Data.”, McGraw-Hill book Company. New York, NY.

Hemond, H.F. and Fechner, E.J. 1994. “Chemical Fate and Transport in the Environment.”, Academic Press, Inc. San Diego, Ca.

Hershield, David M. May 1961. “Rainfall Frequency Atlas of the United Sates for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years.” Technical Paper 40. Weather Bureau.

House, C.H., Broome, S.W., Hoover, M.T. 1994. “Treatment of Nitrogen and Phosphorus by a Constructed Upland-Wetland Wastewater Treatment System.” Water Science and Technology, 29(4):177-184.

Kadlec, R.H., and Knight, R.L. 1996. “Treatment Wetlands.” CRC Press, Inc. Boca Raton, FL.

Lydersen, B.K., D’Elia, N.A., Nelson, K.L. 1994. “Bioprocessing Engineering: Systems, Equipment and Facilities.” John Wiley & Sons. New York, NY.

McCuen, Richard H. 1982. “A Guide to Hydrologic Analysis Using SCS methods.”, Prentice-Hall, Inc., New Jersey.

McCuen, Richard H. 1998. “Hydrologic Analysis and Design, 2nd Edition.”, Prentice-Hall, Inc., New Jersey.

National Small Flows Clearinghouse. 1997. “Handbook of constructed Wetlands, volume 1 and 5.”, USDA-NRCS & USEPA Region III.

Officials of the National Oceanic and Atmospheric Administration (NOAA), US Department of Commerce. 1974. "Climates of the States, Volume I.", Water Information Center, Inc., New York.

Perrich, J.R. 1992. "The ESE National Precipitation Databook." Cahners Publishing Company.

Pine and Swallow Associates, Inc. 1998. "Environmental Investigation for the Proposed North Acton Recreation Area and Pond." Groton, MA.

Schueler, T.R. 1992. "Design of Stormwater Wetlands Systems: guidelines for creating diverse and effective stormwater wetlands in the mid-Atlantic Region.". Metropolitan Washington Council of governments, Washington, DC.

Soil Conservation Service (SCS). 1991. Interim Soil Survey for Acton, Massachusetts.
Stanley, D.W. 1996. "Pollutant Removal by a Stormwater Dry Detention Pond.", Water Environment Research, 68(6):1076-1083.

Tchobanoglous G. and Burton, F.L. 1991. "Wastewater Engineering: Treatment, Disposal and Reuse, 3rd edition.", McGraw-Hill, Inc., Boston, MA.

Thornthwaite, C.W. and Mather, J.R. 1955. "The Water Balance." Publications in Climatology, 8(1), Centerton, New Jersey.

U.S. Department of the Interior. 1975. "Design of Small Dams." A water resources technical publication. U.S. government printing office, Denver, CO.

Vincent, G. 1994. "Use of Artificial Wetlands for the Treatment of Recreational Wastewater." Water Science and Technology, 29(4):67-70.

7 Recommendations

A comprehensive review of the Acton Trading Program and stream water quality modeling studies performed with the EPA stream water quality model, QUAL2E, indicate that, as proposed, the Acton Trading Program will have little impact on water quality in the Assabet River. Therefore, alternative point/point source trading arrangements should be considered if a concerted effort to improve water quality in the Assabet River is to be made. The four WWTPs that discharge to the Assabet River upstream of Acton currently utilize no phosphorus removal beyond conventional removal processes. As a result, they provide ample opportunities for productive and efficient point source trades that utilize trading ratios to reduce the overall phosphorus load to the Assabet River.

Even if an alternative point/point source trading arrangement can be implemented, a BMP program to reduce or mitigate NPS of phosphorus in Acton should not be nullified. Of the many BMPs available to Acton, two specific ones were analyzed in this report: a buffer strip and a detention pond. A buffer strip of lawn grass treating a 5 acre residential area achieves an average phosphorus removal of 50%, producing an average treated runoff concentration of 0.17 mg/L. These results showed that the buffer strip, on its own, would be an effective and feasible BMP in Acton's recreational areas. The results from the detention pond as a stand-alone BMP device are not as promising. While modeling results show approximately 50% removal of phosphorus in a 0.18 ac-ft pond treating a 50 acre residential and forestland area, literature studies show that actually phosphorus removal ranges from 12% to 90%. Moreover, surface overflow from the pond averaged a concentration of 0.21 mg/L, over 2 times the target phosphorus concentrations. If future designs of areas are to include a detention pond, it would be beneficial to include a buffer strip at the pond overflow area.

As part of the BMP analysis, a literature study on treatment wetlands showed that wetlands can reduce phosphorus concentrations in runoff from approximately 0.35 mg/L to 0.12 mg/L. Moreover, a treatment wetland can provide auxiliary benefits such as providing a habitat for wildlife, increasing vegetation diversity, and enhancing the overall aesthetic value of the surrounding area. Therefore, it is recommended that a comprehensive BMP program for Acton include small treatment wetlands, such as the one designed for the North Acton Recreational Area.

Of course, if no alternatives to the proposed Acton Trading Program are feasible, high-level phosphorus removal is possible, albeit at a price. Chapter 5 shows that phosphorus concentrations of less than or equal to 0.1 mg TP/L are attainable in WWTP discharges. There are two options to be recommended for high-level phosphorus removal in small municipalities. The first option is a combination of chemical enhanced primary treatment (CEPT), sequencing batch reactor (SBR), and physical effluent-polishing (EP) (solids contact clarifier and filtration). The second option is a combination of SBR and physical EP processes. In terms of sludge production, option 1 is more favorable than option 2 as the primary sludge produced in former (CEPT+SBR+EP) is the main contributor to sludge production. In terms of phosphorus removal reliability, option 2 is more preferable due to its primary settling process.