Modeling and Analysis of Non-point Source Runoff and Best Management Practice Devices in Acton, MA

By

Teresa Jeannette Raine

B.S. Environmental Sciences
Massachusetts Institute of Technology, 1998

SUBMITTED TO THE DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF ENGINEERING IN CIVIL AND ENVIRONMENTAL ENGINEERING
AT THE
MASSACHUSETTS INSTITUTE OF TECHNOLOGY

JUNE 1999

Signature of Author _____

Department of Civil and Environmental Engineering
May 19, 1999

Certified by ______________________________

Prof. David Marks
Professor of Environmental Engineering
Thesis Advisor

Accepted by ______________________________

Andrew J. Whittle
Chairman Departmental Committee on Graduate Studies

Modeling and Analysis of Non-point Source Runoff and Best Management Practice Devices in Acton, MA

By

Teresa Jeannette Raine

Submitted to the Department of Civil and Environmental Engineering on May 19, 1999 in partial fulfillment of the requirements for the degree of Master of Engineering in Civil and Environmental Engineering

ABSTRACT

As part of the *Town of Acton: Nonpoint Source Phosphorus Analysis and Control* report (Master of Engineering Group Project), the effectiveness of a point-nonpoint source trading program between phosphorus in runoff from Acton and phosphorus to be emitted by a proposed Waste Water Treatment Plant (WWTP) was to be studied. For this purpose, nonpoint sources of phosphorus as well as Best Management Practices (BMPs) devices needed to be evaluated. The P8 PC DOS program was chosen to model phosphorus runoff concentrations and to assess the feasibility of specific Best Management Practices (BMPs) within the Town of Acton. Due to the lack of available land-use data specific to Acton, a model of the entire Assabet watershed area was created to estimate runoff concentrations within Acton. The average calculated runoff concentration was 0.34 ppm. This is slightly above the national average urban runoff concentration of 0.33ppm.

A buffer strip model was created to determine its feasibility within a generic 5-acre residential area. Preliminary results showed that a 50% removal efficiency could be achieved by a buffer strip with a 30-foot flow path. Further modeling showed that increased strip vegetation had little effect on the strip's removal efficiency. It was also determined that the strip flow length was exponentially related to its removal efficiency. To achieve removals on the order of 60% to 70% a significantly longer flow path would need to be established. For the purposes of establishing a buffer strip in a residential area of Acton, a flow path length of 48 feet would provide the most effective results.

The detention pond being designed as part of the wetlands area within the North Acton Recreational Area was modeled to determine its approximate phosphorus removal capability. While P8 predicted that the pond would be able to remove 50% of the phosphorus from the 50-acres of forest and residential land it will be treating, data from existing ponds suggest that actual removal efficiency will be noticeably lower than the efficiency predicted.

The BMPs suggested for the Town of Acton will not be able provide the improvement to the Assabet River that was hoped. A study of the Assabet River water quality done for the *Town of Acton: Nonpoint Source Phosphorus Analysis and Control* group report showed that a point-nonpoint source trading program would provide only a very slight improvement to the Assabet River. The bulk of the phosphorus entering the Assabet River is from WWTPs upstream, and closer to the source of the Assabet River. For any significant results, a point-point source trading between the Acton WWTP and a WWTP farther upstream would have to occur.

Thesis Supervisor: Professor David Marks
Title: Professor of Environmental Engineering

1 Introduction[1]

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 evaluates 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). The

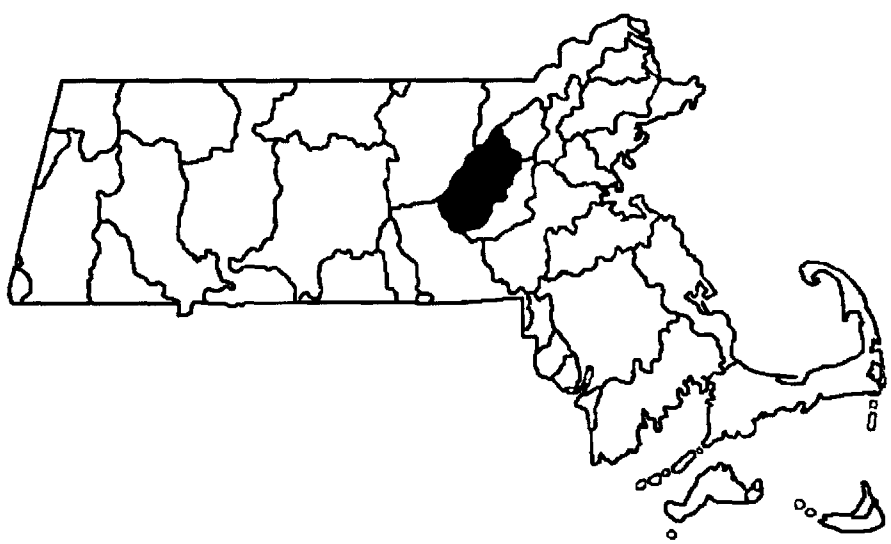


Figure 1: Location of Assabet River Basin (Source: U.S. EPA, 1999).

[1] Introduction adapted from section 1 of the *Town of Acton: Nonpoint Source Phosphorus Analysis and Control* by Steve McGinnis, Teresa Raine, Anouk Savineau, and Yukiasu Sumi (Master of Engineering Project).

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

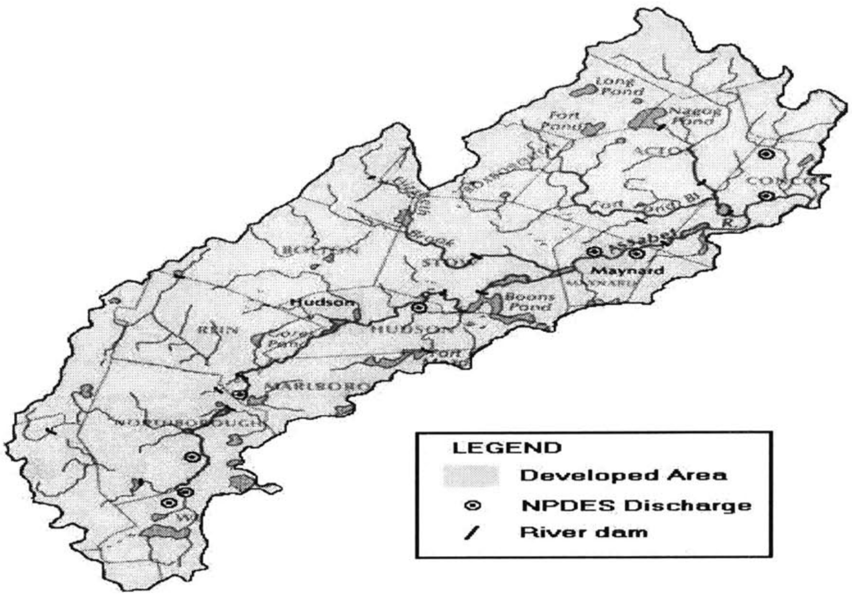


Figure 2: Map of Assabet River Basin (Source: U.S. EPA, 1999).

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 where WWTP plant discharges are located just above a dam in an impoundment area, as can be seen in Figure 3 (Hanley, 1989). On average, the WWTPs

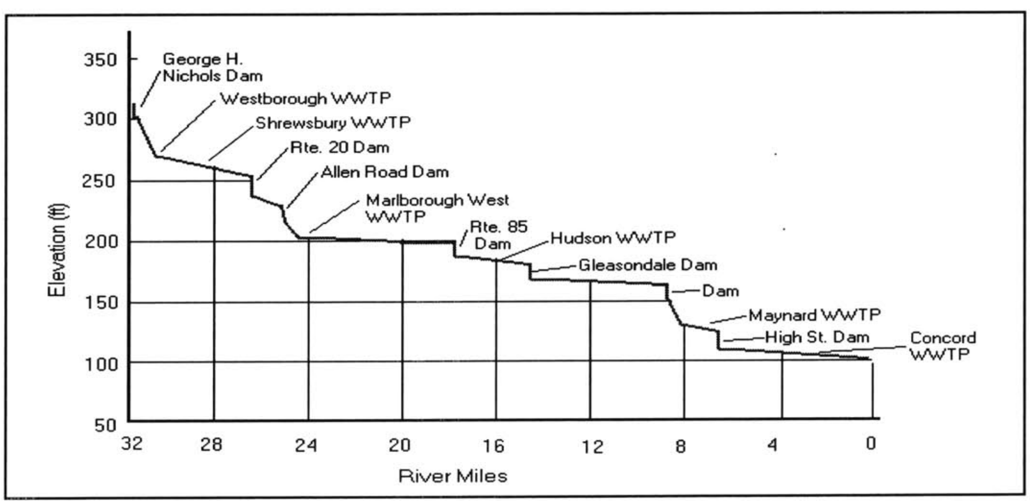


Figure 3: Dam and WWTP Locations Along the Assabet River (Source: Hanley, 1989).

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 suffered from 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 report on the 1979 sampling 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, parts of the river are covered by an algae mat (Figure 4). 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).

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

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 Nonpoint Source Modeling for Acton

To determine the effectiveness of a trading program between nonpoint sources of phosphorus and the point source created by the WWTP, nonpoint source phosphorus loading within Acton needs to be determined as well as the effectiveness of BMP devices within Acton. For any BMP plan to be considered effective, it will have to ensure that the water quality of the Assabet River will not further degrade with the additional point source. The hope is that such a program may actually help in improving the overall quality of the Assabet River in the sections that run through Acton.

2 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. The criteria for program selection included:

- Ability to estimate non-point sources of phosphorus within the watershed
- Solid reputation – Must be based on accepted principles/algorithms for modeling; produce confident results; Reliability
- Ability to assess the impact of various BMPs if implemented in Acton
- Ability to utilize the GIS data already complied for the town
- Ability to integrate the other aspects being designed for the Town of Acton project
- Ability to produce clear, confident, understandable outputs

Originally, the Town of Acton assessed four different NPS models including BASINS 2.0, GWLF, GISPLM, and P8. Of those programs two were readily accessible including the program itself, user manual, and technical support: BASINS 2.0, maintained by the EPA, and P8, created by Mr. William Walker for the Narragansett Bay Project.

2.1 BASINS 2.0

The Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) 2.0 is able to utilize GIS data such as the land-use data available through MassGIS to model entire watersheds at a time. Originally released in September, BASINS was created to (US EPA, 1999):

(1) "To facilitate examination of environmental information,
(2) To provide an integrated watershed and modeling framework, and
(3) To support analysis of point and nonpoint source management alternatives"

BASINS is comprised of several models to provide a complete picture of a watershed including a Non-point Source Model (NPSM), QUAL2E (a steady state water quality model), and TOXIROUTE (a simple dilution/decay model) as well as a post-processor. The Windows based program is powered by ArcView and can produce useful graphics of watershed land-use data and water quality reports. BASINS utilizes the Hydrologic Simulation Program – FORTRAN (HSPF) (Bicknell et al., 1996) for the NPSM.

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. There is a significant technical problem with BASINS 2.0 as well. To date, the NPSM has not run properly. Unexplained errors occurred while attempting to input data and in retrieving any output files created for NPSM runs.

2.2 P8

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 Stormwater Management Model (SWMM) (Huber and Dickinson, 1988) HSPF (Bicknell et al., 1996), and UTM-TOX (Patterson M., 1984). 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.

2.3 Model Comparison and Selection

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 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. 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.

3.1 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.

3.1.1 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.

3.1.2 Watershed Loads

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

3.1.3 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.

3.2 Particle and Water Quality Component Characteristics

The particle class characteristics are based upon the characteristics of the watersheds for impervious/pervious runoff 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).

3.3 Soil Quality

Soil quality and characteristics are important to the calculation of runoff loading. The first characteristic determined is the Hydrologic Soil Group (HSG) of the area. A listing of the four groups and their descriptions can be found in Appendix A. 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.

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

Where,
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 Meteorological Data

Runoff loading and concentration results are quite sensitive to precipitation. 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 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.

3.5 BMP Devices

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

3.5.1 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, allows 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 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 flows.

3.5.2 Device Flows

Flow within P8 is analyzed in the downstream order, one watershed or BMP device 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/hr)
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})

d_0 and d_1 are updated for each time step during the model run. With the storage volume/outflow relationship linearized, the following equation is describes the analytical solution for a device flow balance at any given time step:

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

$$\downarrow$$

$$V_{n+1} - V_n = F(V,t)$$

$$= A/K + (V_n - A/K) \exp(-Kt) - V_n$$

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 of flows over device outlets (infiltration, normal, spillway)
t = time step length (hours)
$A = Q_{in} - SUM(d_0)$
$K = SUM(d_1)$

Note that $SUM(d_o)$ and $SUM(d_1)$ denote functions. 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.

3.5.3 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 maximum 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.

3.5.4 Device Concentration

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

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

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

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

$$= M_n + Wt \qquad , \text{ if } D = 0$$

where,
D = sum of first order loss terms (hr^{-1})
C_m = average concentration during step (ppm)
V_m = 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 and Walker, 1990):

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

3.6 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 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 and Walker, 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.

4 Model of the Entire Watershed

For the Acton project, a model of the entire watershed was created 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.

4.1 Land-use data and inputs

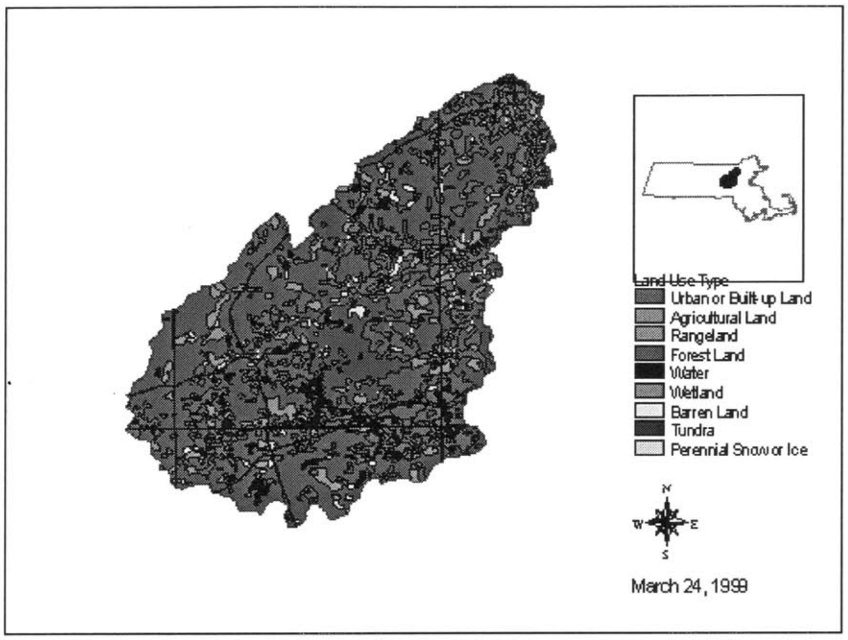


Figure 5: Land use Report for Watershed 1070005 (Assabet)

MassGIS land-use data was obtained and accessed using BASINS 2.0. Figure 5 shows the land-use report generated by BASINS. Information specific to the Acton area was assessed using GIS data provided by the Town of Acton. The 12 acres of unclassified land which 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.

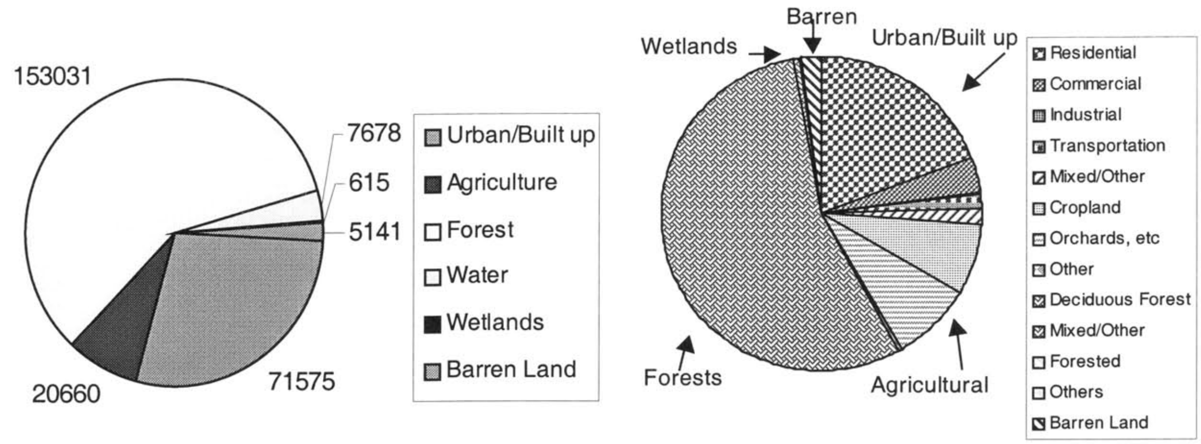


Figure 6: Watershed land-use: a) general land-use categories b) specific land-use categories

Watershed slope was 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 2: Impervious Fractions for various land uses

Assumptions were made on general soil qualities for purposes of selecting a Hydrological Soil Group (HSG). The HSG characterization describes the general texture, permeability, and drainage of The Hydrology National Engineering Handbook listed Acton 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: SCS Curve numbers for soil group B

4.2 Meteorological Data

The precipitation 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 7 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.

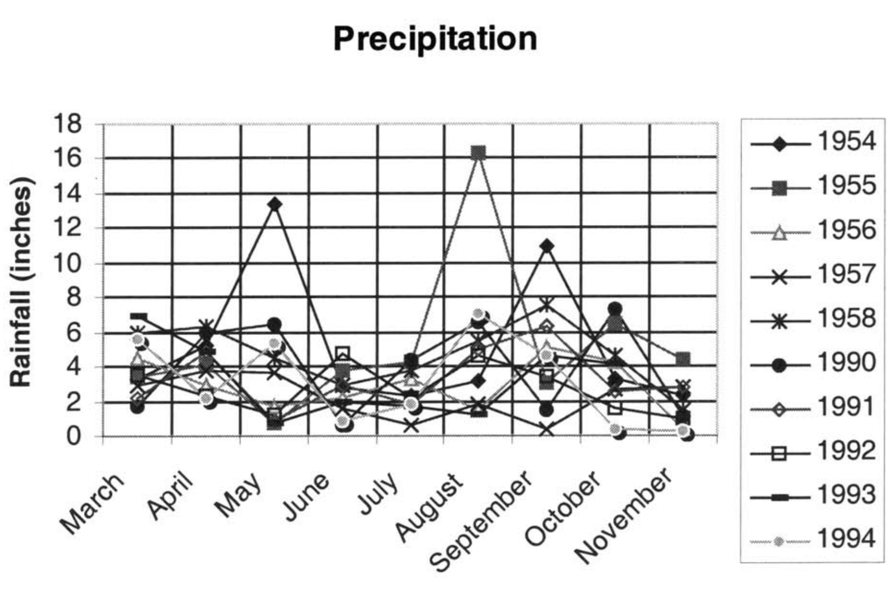


Figure 7: Monthly precipitation totals for specific years

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. P8 automatically calculates for the ET rates.

Evapotranspiration Parameters				
Calibration factor:		1		
Computed Annual ET:		21.9145 in/year		
Month	Veg. Cover Factor	Air Temp (Deg -F)	Daylight hrs/day	computed ET (in/mo)
Jan	0.5	27	9.5	0
Feb	0.5	30	10.6	0
Mar	0.5	93	11.9	0.51
Apr	0.5	48	13.4	0.87
May	0.75	59	14.6	2.36
Jun	1	66	15.2	4.16
Jul	1	72	14.9	5.01
Aug	1	70	13.9	4.09
Sep	1	63	12.5	2.55
Oct	1	54	11.1	1.53
Nov	0.75	43	9.8	0.59
Dec	0.5	34	9.1	0.25

Table 4: Evapotranspiration Parameters

4.3 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 ten days for steady state conditions to be reached.

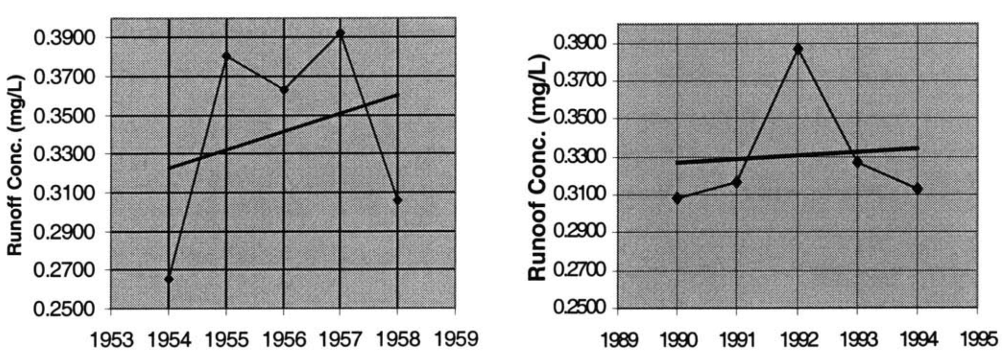


Figure 8: Total Phosphorus concentrations in watershed runoff: a) for 1954-1958 and b) for 1990-1994

Figure 8 show the results for ten runs of the complete watershed model with the various precipitation data. The average concentration for all runs is 0.336 ppm, which is approximately equal to the runoff concentration calculated 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).

The BMP devices will have to contend with runoff concentrations much higher than the average. Figure 9 shows a sample of the concentration levels as a function of storm event. The events listed are for March through November of 1991, 1992, and 1993.

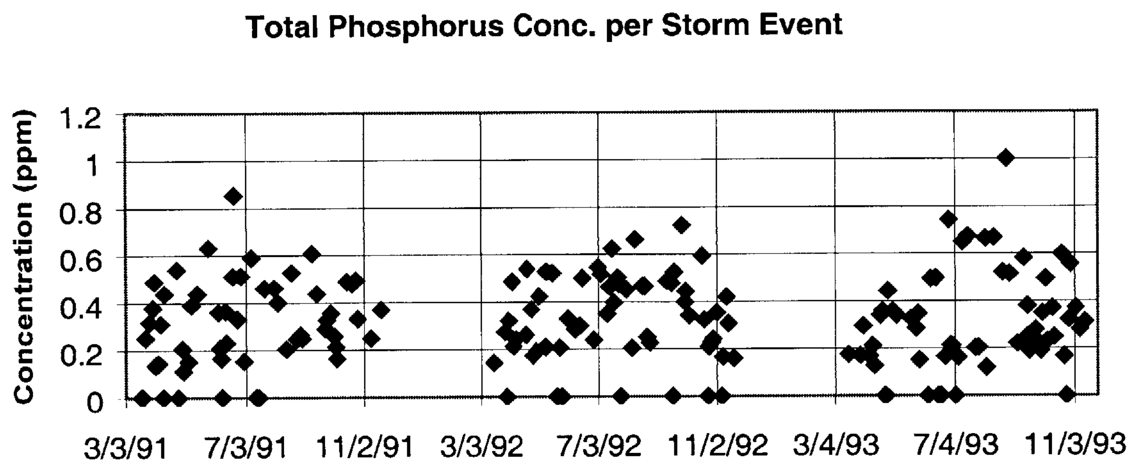


Figure 9: Concentration of Total Phosphorus in the Outflow versus Storm Event

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

5 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 needs 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 strip is a strip of vegetation that reduces the velocity of local runoff, thereby reducing the runoff's ability to carry 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 were studied. Since the size and design of the detention pond has already been determined, only efficiency were analyzed

5.1 Buffer Strip Case

A generic watershed was defined 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 5 lists the input values:

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

Table 5: General Residential Watershed Inputs

5.1.1 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 the flow depth 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 following storms. The sensitivity of particle removal rates to the Manning's coefficient increases with the defined infiltration rate. Table 6 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 6: 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 Table 7. 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) Infiltration Rate	(c)
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

Sources:	(a) Infiltration Rate	(c)
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

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

Table 7: 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, the removal rate 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 10

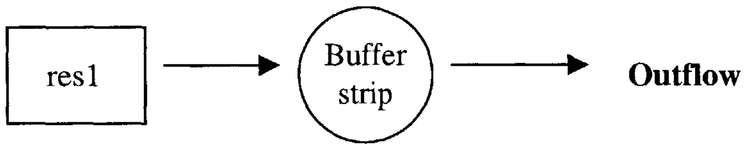


Figure 10: Schematic of General buffer strip model

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

5.1.2 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. Figure 11 shows the results from the initial runs.

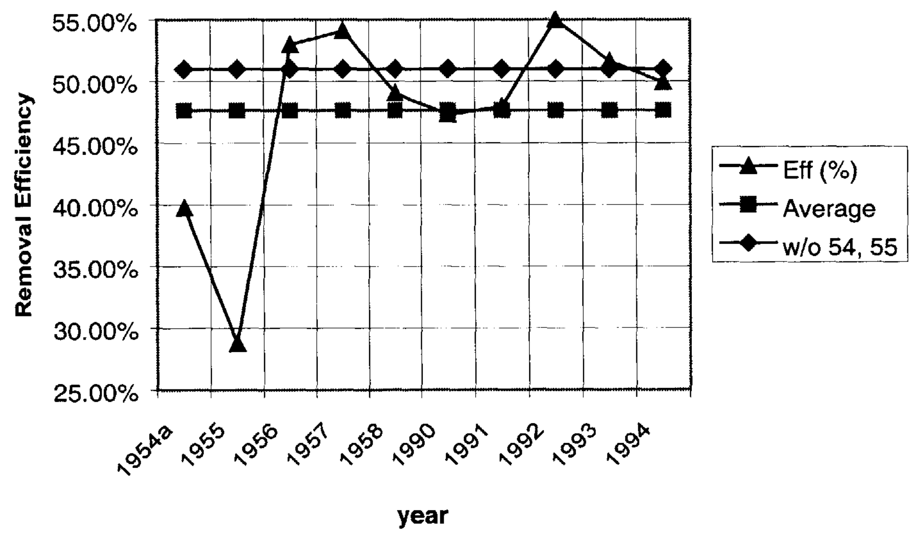


Figure 11: Initial Model Buffer Strip Efficiency

There is a notable difference between the results from 1954 and 1955 from the rest of the years. Referring back to Figure 7, which displays the precipitation data, we see significantly elevated levels of precipitation for these years. In the P8 model results for 1954 and 1955, the buffer strip "flooded" during the model run, meaning the runoff exceeded the strip's 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 occurred 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 phosphorus load of 5.1 lbs. and 1955 had a load of 7.7 lbs., the remaining years averaged 3.7 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.

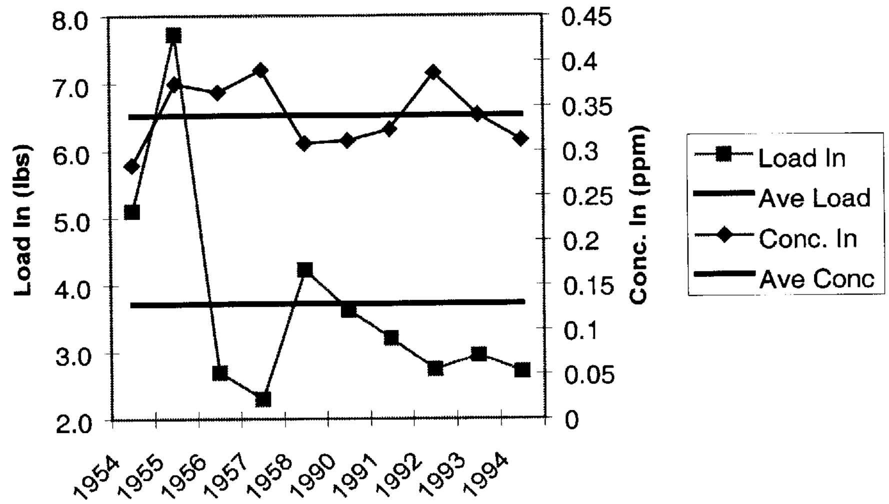


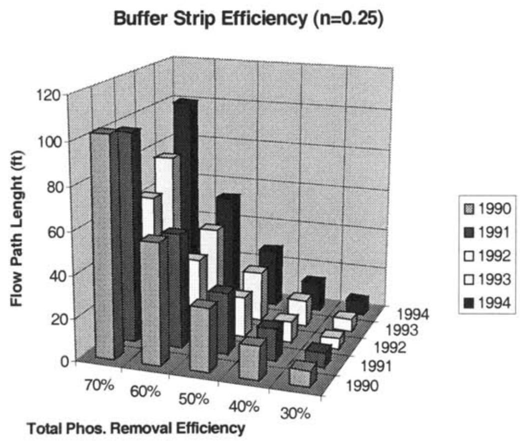
Figure 12: Inflow Loads and Concentrations from the 5 acre residential watershed

Average flow concentrations also help describe phosphorus loading to the Assabet River. For example, a specific season may have mobilized a large load of phosphorus. If there were a high inflow to match it, average phosphorus concentrations into surface waters would be lower due to dilution.

5.1.3 Sizing the Strip according to Efficiencies

The initial model determined an average of 50% phosphorus removal by a strip with a flow path of 30 ft. The next step was to determine the effects of strip flow-path length on 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 13b shows the effect of the length of a dense buffer strip (Manning's n = 0.45) on phosphorus removal. Figure 13a 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.



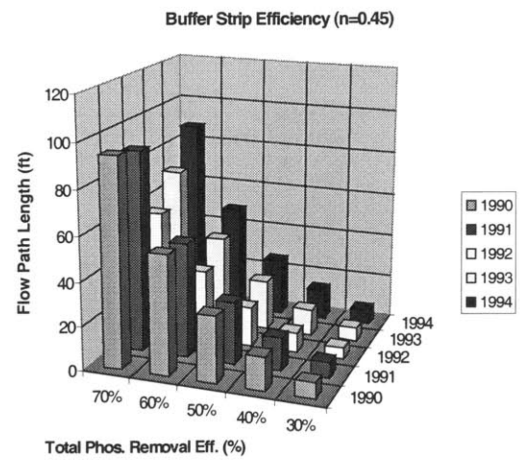


Figure 13: Phosphorus removal efficncies for: a) a lawn buffer strip. b) a buffers stripe with dense growth.

As Figure 13 shows, there is little variation between the buffer strip that uses dense growth and the strip that uses lawn grass. This shows that a well-maintained lawn area will provide efficient removal of phosphorus without requiring overgrowth or the planting of additional plant life.

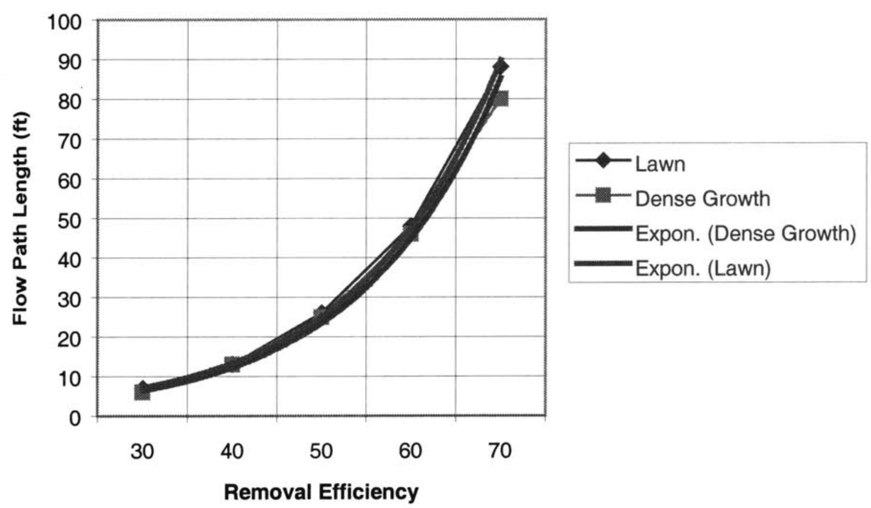


Figure 14: Average flow path length as a function of phosphourus removal efficiency and buffer strip material

There is an exponential increase in flow path length as the removal percentages increase. For a lawn buffers strip of with required efficiency (x), the flow path length (y) is defined as:

$$y = 3.72\, e^{0.65x}$$

The corresponding equation for dense buffer strips is:

$$y = 3.40\, e^{0.64x}$$

The exponential relationship between the flow path and phosphorus removal shows that for low removal rates (up to 50% removal) a small amount of land will provide a significant improvement in phosphorus removal results. On the other hand, to continue to improve phosphorus removal past 50% would require and much longer flow-path. 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.

5.1.4 Comparison of Efficiency Versus Percent of Runoff Reaching 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 Figure 13 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 15.

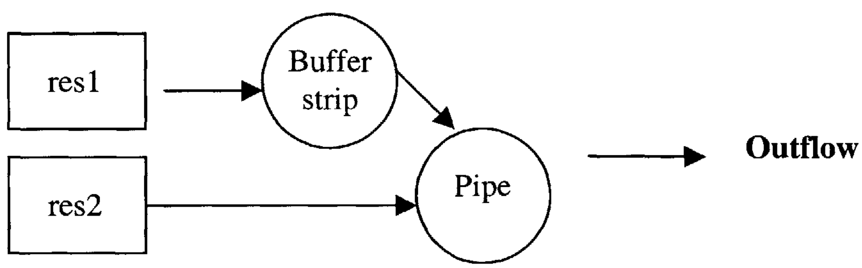


Figure 15: Schematic of second residential buffer strip model

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 16.

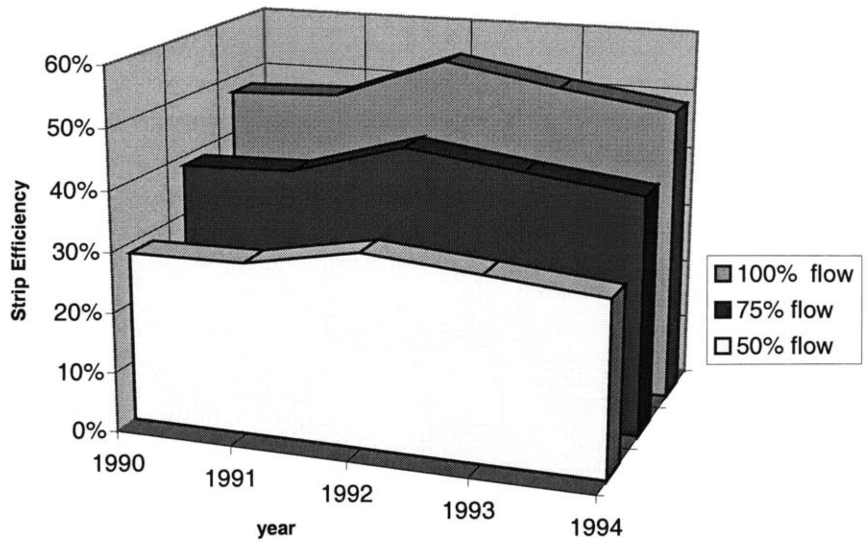


Figure 16: Buffer strip efficiency vs. percent of runoff running over strip

The averages for each run can be found in Table 8 below. For each type of flow both the device efficiency and the calculated outflow concentrations is listed. The average inflow concentration was 0.34 ppm.

% of flow treated	outflow (ppm)	dev. eff.
100%	0.17	50%
75%	0.20	41%
50%	0.24	30%

Table 8: Average Results for Efficiency Versus Percent of Runoff Treated

As expected, this device alone will not 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.

5.2 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. For the purposes of distinguishing detention ponds within P8, a dry detention pond has a drain 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, within P8. The outflow water from a wet pond leaves the pond either through infiltration, evaporation or surface overflow if the pond floods. P8 can also create a combined detention pond (normally considered to be a wet detention pond) which would include a permanent pond with an outlet/drain farther up the side of the pond as depicted in Figure 17. This allows the pond to partially drain out between storm events

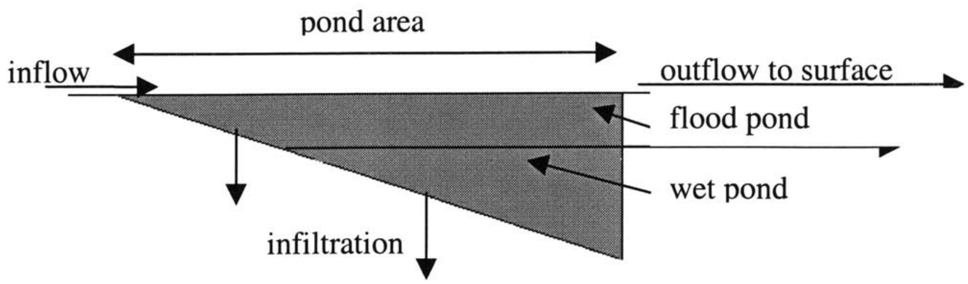


Figure 17: Combined wet/flood pond

At the North Acton Recreational Area, a treatment wetlands is being designed and will be constructed as a BMP demonstration. Part of the 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.

5.2.1 Model inputs

The area that will be treated by the detention pond is estimated at 50 acres of residential and forest land. Based on land use ratios from MassGIS data, two 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 area with a fair amount of vegetation growth. The runoff from both these areas will completely flow into the pond.

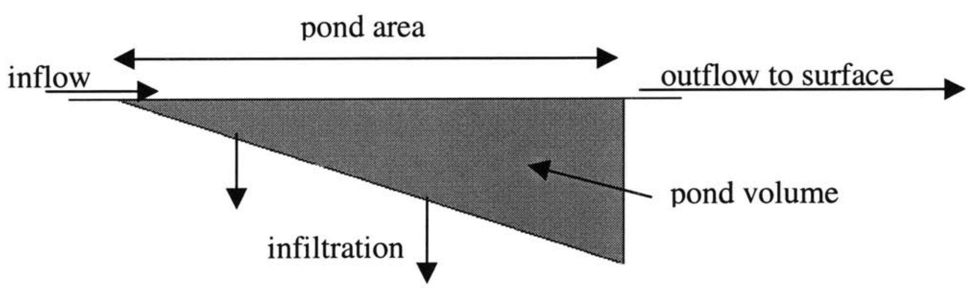


Figure 18: Detention Pond

The model will be a wet detention pond with a structure similar to that seen in Figure 18. The surface area of the pond is 4000 ft^2 or approximately 0.01 acre. Pond volume is estimated at 0.18 acre-ft. The initial pond dimensions were arrived at using data from the proposed wetland area design plans.

5.2.2 Detention Pond Results

Figure 19 shows both the calculated efficiencies and estimated concentrations in the outflow.

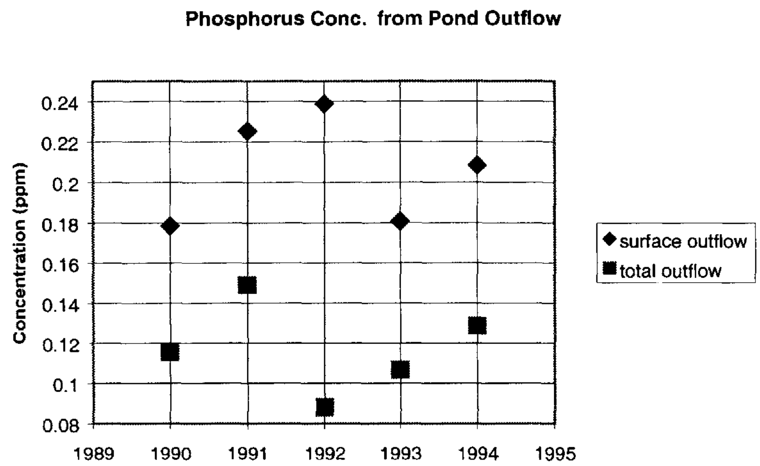


Figure 19: Concentration of outflow from the wet detention pond

The variation in efficiency results is again due to the variety of storm event lengths and intensities. 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. 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 removal rates range from the low teens up to 90+%.

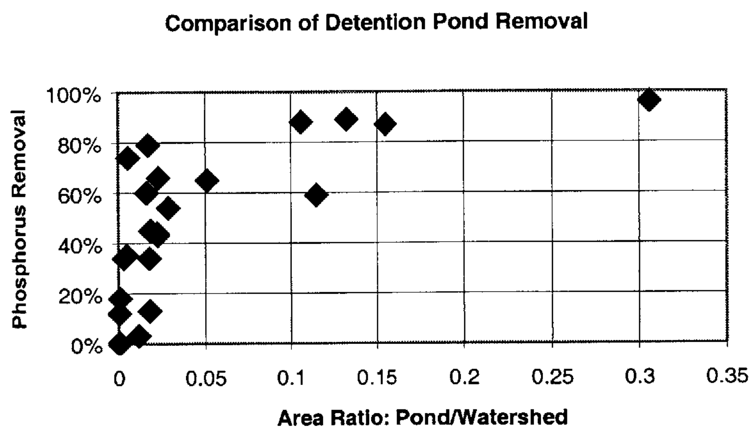


Figure 19: Results of phosphorus removal as a function of pond/watershed ratio (Walker, 1987(a)).

The phosphorus removal for ponds that are up to 5% of the area of the watershed ranges from nearly 0% up to 80%. Ponds that are 10% as large as the watersheds they are treating achieve between 60% and 98% phosphorus removal. The modeled pond is less than 1% of the area and therefore falls in that questionable phosphorus removal area. Appendix 8.13 has a list of all the data points and their sources.

6 Recommendations for BMP / Conclusions

Devices within the Assabet area need to deal with phosphorus concentrations ranging from 0.2 to 0.6 ppm (mg/L) during any average storm event. At least one event per year led to an elevated phosphorus runoff 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. Modeling results show that approximately 40% to 50% phosphorus removal can be achieved by a lawn buffer strip with 15 to 30 foot flow path. The exponential relationship between flow path length and removal efficiency means that to achieve higher phosphorus removal (60% to 70% removal efficiency), the flow path would need to be significantly increased (up to a 100-foot flow path). Phosphorus removal is not greatly improved by increasing the amount of vegetation used in the buffer strip. Buffer strips placed in residential areas where they will be able to treat a majority of the runoff in the area will noticeably decrease phosphorus concentrations in runoff flowing to the Assabet.

The wet detention pond that is part of the wetland area design will most likely have some impact on phosphorus levels, though not to the degree that the buffer strip produces. P8 predicts that the pond will be able to reduce phosphorus runoff concentrations by 50%. However, results are questionable at best. A study of detention pond efficiency as a function of the watershed/pond area ratio demonstrates that in the field, results vary from 0% to 80% removal. The benefits of building a single pond without additional BMPs to ensure phosphorus removal are not greater than the cost. While the specific pond modeled will be useful in lowering concentrations entering the treatment wetlands in the North Acton Recreational Area from the surrounding areas, implementing a similar pond on its own in other areas would not produce significant removal results. To ensure phosphorus removal, an additional BMP device, such as a wetland marsh, should be placed downstream of the detention pond.

The BMP devices modeled in this study will effectively remove NPS phosphorus in Acton. However, the goal of the Acton Watershed Trading Program, is to improve the overall water quality of the Assabet River as it runs past Acton towards the Concord River. Water quality reports on the Assabet River suggest that the amount of phosphorus entering the Assabet River from nonpoint sources in Acton is minimal when compared to the amounts entering from upstream WWTP point sources.

Water quality models done for the Acton Project (McGinnis et al., 1999) showed that the reduction in NPS phosphorus through the implementation of BMP devices provides little improvement in water quality. When modeling the results of a point-point source trade between the proposed Acton WWTP and the existing Westborough WWTP, the reduction in phosphorus and algae levels near and past Acton were significant. While it is important to control nutrient loading to surface waters, in the case of the Assabet River, water quality conditions will only see significant improvements when point sources upstream are re-evaluated and reduced.

7 References

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

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

Bicknell, B.R., J.C. Imhoff, J.L. Kittle, A.S. Donigian Jr., and R.C. Johanson. 1993. Hydrological Simulation Program FORTRAN (HSPF): User's Manual for Release 10. EPA-600/R-93/174. U. S. Environmental Protection Agency, Athens, GA. September 1996.

Biswas, A.K. 1997. Water Resources: Environmental Planning, Management, andDevelopment. 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.

Driscoll, E.D., 1986. "Review of Papers on Dention, Retention, Wetlands, and Other Treatment" *Urban Runoff Quality, Its Impacts and Quality Enhancement Technology, Eng. Foundation Conf.* Henniker, NH, June 22-27, 1986.

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.

Hey, D.L., 1982. "Lake Elyn and urban stormwater treatment" *Stormwater Detention Facilities – Planning, Design, Operation, and Maintenance.* Conf. Proc. Henniker, NH. American Society of Civil Engineers., 1982

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

Huber, W.C. and R.E. Dickinson. 1988. Huber, W.C. and R.E. Dickinson. 1988. Storm Water Management Model User's Manual, Version 4. EPA/600/3-88/ 001a (NTIS PB88-236641/AS), U.S. Environmental Protection Agency, Athens, GA. August 1988.

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

Steve McGinnis, Teresa Raine, Anouk Savineau, and Yukiasu Sumi. 1999 "Town of Acton: Nonpoint Source Phosphorus Analysis and Control" (Master of Engineering Group Project), Massachusetts Institute of Technology, May 1999.

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

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

Patterson M., 1984. "A User's Manual for UTM-TOX, the Unified Transport Model." ORNL-6064. December 1984.

Randal, CW, 1982. "Stormwater detention ponds for water quality control, in stormwater detention facilities – planning design, operation, and maintenance." *Conference Proc., Henniker, NH. American Society of Civil Engineering*, 1982

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

Shaver, H.E.. 1986 "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

Town of Acton. 1998. Trading Program Discussion Draft. Acton, Massachusetts.

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

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

United States Environmental Protection Agency. 1982. "Results of the Nationwide Urban Runoff Program, Volume II". Water Planning Division, Washington, D.C. September 1982.

United States Environmental Protection Agency. 1986. "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. 1999. U.S. EPA Internet Home Page. http://www.epa.gov

United States Environmental Protection Agency, 1999. "About BASINS 2", http://www.epa.gov/ost/BASINS/basinsv2.htm, Updated Apr. 15, 1999

Walker, William W. 1987(a). "Phosphorus Removal by Urban Runoff Detention Basins", Lake & Reservoir Management, (obtained through: http://www2.shore.net/~wwwalker/) 1987.

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

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

8 Appendices

8.1 Calibrated Runoff Concentrations (Palmstrom and Walker, 1990)

This table contains the event-mean concentrations (ppm)

Component	NURP Median Site	
Total Suspended Solids	100	
Total Phosphorus	0.33	
Total Kjeldahl Nitrogen	1.50	
Total Copper	0.034	
Total Lead	0.020	a
Total Zinc	0.160	
Hydrocarbons	2.5	b
P8 Particle File ----------> NURP50.PAR		

a – NURP lead values reduced to account for >10-fold reduction in gasoline lead content since NURP monitoring
b – Hydrocarbons estimated from load factors reported by Hoffman (1985)

8.2 Water Quality Criteria (Palmstrom and Walker, 1990)

Component (ppm)	Level A	Level B	Level C
Total Suspended Solids	5	10	20
Total Phosphorus	0.025	0.05 d	0.10 e
Total Kjeldahl Nitrogen	2.0	1.0	0.5
Total Copper	2.0 a	0.0048 b	0.02 c
Total Lead	0.02 a	0.0140 b	0.15 c
Total Zinc	5.0 a	0.0362 b	0.38 c
Hydrocarbons	0.1	0.5	1.0

a – US EPA primary drinking water standard
b – RI standard, acute toxicity, fresh waters, hardness = 25ppm
c – NURP threshold for aquatic life, intermittent exposure, soft waters (Athayede et al, 1983)
d – US EPA (1976) guideline for eutrophication in streams
e – US EPA (1976) guideline for streams entering lakes
others are arbitrary benchmarks (no standards or criteria)

8.3 Hydrologic Soil Group Descriptions:

A -- Well-drained sand and gravel; high permeability.
B -- Moderate to well-drained; moderately fine to moderately coarse texture; moderate permeability.
C -- Poor to moderately well-drained; moderately fine to fine texture; slow permeability.

D -- Poorly drained, clay soils with high swelling potential, permanent high water table, claypan, or shallow soils over nearly impervious layer(s).

8.4 Watershed Land use details

General Subgroups				More Detailed Subgroups			
W#	Watersheds	Area	P/I	W#	Watersheds	Area	P/I
1	Urban/Built up	71575	P/I		*Urban/Built up*		
2	Agriculture	20660	P	1	Residential	52350	P/I
3	Forest	153031	P	2	Commercial	10209	P/I
4	Water	7678	P	3	Industrial	728	P/I
5	Wetlands	615	P	4	Transportation	3483	P/I
6	Barren Land	5141	P/I	5	Mixed/Other	4805	P/I
					Agricultural Land		
				6	Cropland	19670	P
				7	Orchards, etc	22686	P
				8	Other	304	P
					Forest		
				9	Deciduous Forest	151262	P
				10	Mixed/Other	1769	P
				??	*Water*	7678	P
					Wetland		
				11	Forested	360	P
				12	Others	255	P
				13	*Barren Land*	5141	P/I

8.5 Residential watershed used for Buffer Strip runs.

WATERSHED DATA			
WATERSHED NUMBER	1	NAME watershd TOTAL AREA	5 ac
OUTFLOW DEVICE NUMBER	1 for surface runoff		
AQUIFER DEVICE NUMBER	0 for percolation		
PERVIOUS CURVE NUMBER		69 (1-99)	
SCALE FACTOR FOR PERVIOUS AREA LOADS		1	
IMPERVIOUS AREA TYPE		SWEPT	NOT SWEPT
IMPERVIOUS FRACTION	-	0	0.26
DEPRESSION STORAGE	inches	0.02	0.02
IMPERVIOUS RUNOFF COEFF	-	1	1
SCALE FACTOR FOR PARTICAL LOADS		1	1
SWEEPING FREQUENCY	1/week	0	
SWEEPING START DATE	mmdd	101	
SWEEPING STOP DATE	mmdd	1231	
SCALE FACTOR FOR SWEEPING EFFICIENCY		1	

8.6 Buffer Strip Characteristics for General and Sizing runs

Swale/Buffer Strip			
Device Number	1	Label	lg strip
Bottom Elevation	feet		
Flow Path Length	feet		30
Flow Path Slope	%		2
Bottom Width	feet		150
Side Slope	ft-h/ft-v		2
Maximum Depth	feet		0.1
Manning's N			0.25
Infilitraion Rate	in/hr		0.26
Particle Removal Scale Fractor			1
Outflow Device Numbers:			
	Normal Outlet	0	
	Exfiltrate	0	

8.7 General Buffer Strip Run Results

The highlighted years (1954 and 1955) were runs where the model considered the buffer strip to have "overflowed", meaning flows were too high at some points in the year for the buffer strip to retain/effect runoff.

year	Load In	Conc. In	Surface out	Eff (%)
1954	5.10	0.2842	0.1927	39.74%
1955	7.73	0.3745	0.2925	28.77%
1956	2.69	0.3653	0.2103	52.96%
1957	2.29	0.3899	0.2236	54.10%
1958	4.22	0.3085	0.1893	49.04%
1990	3.62	0.3116	0.1898	47.29%
1991	3.2	0.3235	0.1987	47.91%
1992	2.73	0.3864	0.2173	55.01%
1993	2.95	0.3398	0.2031	51.61%
1994	2.7	0.3119	0.1944	49.91%
Average	**3.72**	**0.3396**	**0.2112**	**47.63%**
Ave (normal)	**3.05**	**0.3421**	**0.2033**	**50.98%**

8.8 Buffer Strip Sizing Run Results

n=0.25					
	30%	40%	50%	60%	70%
1990	7.5	15.5	30	57	103
1991	7	15	29	54	99
1992	5	10	19	35	63
1993	6	12.5	24	43	77
1994	7	14	28	53	100
Averages	7	13	26	48	88

n=0.45					
	30%	40%	50%	60%	70%
1990	7	15	30	54	94
1991	7	15	28	52	91
1992	5	9	18	32	57
1993	6	12	23	41	71
1994	7	14	28	49	88
Averages	6	13	25	46	80

8.9 Results for Buffer Strip Efficiency versus percent of flow to be treated by the device

	100% flow			75% flow			50% flow		
	in conc.	out conc.	dev. effic.	in conc.	out conc.	dev. effic.	in conc.	out conc.	dev. effic.
1990	0.3116	0.1646	47.29%	0.3116	0.1921	38.45%	0.3116	0.2237	28.36%
1991	0.3235	0.1688	47.91%	0.3235	0.1982	38.94%	0.3235	0.2319	28.58%
1992	0.3862	0.1742	55.01%	0.3862	0.2162	44.34%	0.3862	0.2634	32.07%
1993	0.3398	0.1654	51.61%	0.3398	0.1977	41.81%	0.3398	0.2368	30.56%
1994	0.3199	0.1645	48.91%	0.3199	0.1945	39.51%	0.3199	0.2286	28.77%
Averages	0.3362	0.1675	50.15%	0.3362	0.1997	40.61%	0.3362	0.2369	29.67%

8.10 Watersheds used

WATERSHED DATA				
WATERSHED NUMBER	1	NAME resident	TOTAL AREA	16.5 ac
OUTFLOW DEVICE NUMBER	1 for surface runoff			
AQUIFER DEVICE NUMBER	0 for percolation			
PERVIOUS CURVE NUMBER		69	(1-99)	
SCALE FACTOR FOR PERVIOUS AREA LOADS		1		
IMPERVIOUS AREA TYPE		SWEPT	NOT SWEPT	
IMPERVIOUS FRACTION	-	0	0.26	
DEPRESSION STORAGE	inches	0.02	0.02	
IMPERVIOUS RUNOFF COEFF	-	1	1	
SCALE FACTOR FOR PARTICAL LOADS		1	1	
SWEEPING FREQUENCY	1/week	0		
SWEEPING START DATE	mmdd	101		
SWEEPING STOP DATE	mmdd	1231		
SCALE FACTOR FOR SWEEPING EFFICIENCY		1		

WATERSHED DATA				
WATERSHED NUMBER	1	NAME forest	TOTAL AREA	34 ac
OUTFLOW DEVICE NUMBER	1 for surface runoff			
AQUIFER DEVICE NUMBER	0 for percolation			
PERVIOUS CURVE NUMBER		69	(1-99)	
SCALE FACTOR FOR PERVIOUS AREA LOADS		1		
IMPERVIOUS AREA TYPE		SWEPT	NOT SWEPT	
IMPERVIOUS FRACTION	-	0	0	
DEPRESSION STORAGE	inches	0	0	
IMPERVIOUS RUNOFF COEFF	-	1	1	
SCALE FACTOR FOR PARTICAL LOADS		1	1	
SWEEPING FREQUENCY	1/week	0		
SWEEPING START DATE	mmdd	101		
SWEEPING STOP DATE	mmdd	1231		
SCALE FACTOR FOR SWEEPING EFFICIENCY		1		

8.11 Wet pond inputs

Detention Pond			
Device No.	1	Label	wetpond
	Surface Area (acres)	Storage Vol (ac-ft)	Infiltration rate (in/hr)
Pond Bottom	0.08		
Permanent Pool	0.091827	0.18	0.26
Flood Pool	0	0	0.26
Particle Removal Scale Factor		1	~1.0

8.12 Wet pond results

wet pond (5 acre area)											
				year		inflow		surface outflow		total outflow	
# storms	duration	inch	flow (acft)		device eff	load	conc	load	conc	load	conc.
61	568	35.52	16.05	**1990**	60.65%	12.82	0.2938	4.84	0.1784	5.00	0.1154
55	585	30.53	14.12	**1991**	52.75%	12.11	0.3155	5.51	0.2254	5.65	0.1489
61	650	24.80	9.12	**1992**	76.79%	9.12	0.3787	1.97	0.2389	2.13	0.0879
62	637	26.37	13.02	**1993**	64.68%	10.63	0.3004	3.63	0.1808	3.79	0.1066
66	675	28.14	12.34	**1994**	56.87%	10.03	0.2991	4.24	0.2085	4.38	0.1287

8.13 Detention Pond comparison (Walker, 1987(a))

Location	Basin	ratio	mean depth	TP removal	Source
Lansing, MI	Grace No.	0.0001	0.8	0%	USEPA, 1982; Driscoll, 1983
Lansing, MI	Grace So.	0.0004	0.8	12%	
Ann Arbor, MI	Pitt	0.0009	1.5	18%	
Ann Arbor, MI	Traver	0.0031	1.3	34%	
Ann Arbor, MI	Swift Run	0.0115	0.5	3%	
Long Island, NY	Unqua	0.0184	1	45%	
Washington, DC	Westleigh	0.0285	0.6	54%	
Langsing, MI	Waverly Hills	0.0171	1.4	79%	
Glen Elyn, IL	Lake Ellyn	0.0176	1.6	34%	
Twin Cities, MN	Fish	0.0221	1.2	44%	Brown, 1985
Twin Cities, MN	Spring	0.0007	1.3	0%	
Minneapolis, MN	Harriet	0.306	8.8	96%	Endmann et al., 1983
Minneapolis, MN	Calhoun	0.1326	9.8	89%	
Minneapolis, MN	Isles	0.1554	2.4	87%	
Minneapolis, MN	Cedar	0.1062	6	88%	
Minneapolis, MN	Brownie	0.0228	4.9	66%	
Orlando, FL	Pond	0.0047	1.9	35%	Martin and Smoot, 1986
Orlando, FL	Wetland	0.0177	0.2	13%	
Orlando, FL	Pond + Wetlan	0.0224	0.6	43%	
Glen Elyn, IL	Lake Elyn	0.0161	1.6	60%	Hey, 1982
Washington, DC	Burke	0.115	2.6	59%	Randal, 1982
Columbia, MO	Callahan	0.0056	2	74%	Schrelber et al. 1980
Fiolla, MO	Frisco	0.0512	1	65%	Oliver and Grlgoropoulos, 1981