City of Cambridge
Department of Public Works

PROPOSED
CONCORD – ALEWIFE AREA
STORMWATER MANAGEMENT GUIDELINES

May 2006
June 2006 (reformatted)

Note: This document is a draft and may be edited subject to the following;

City of Cambridge –	- Ordinance Committee Hearing Process
City of Cambridge -	- City Council Adoption Process
City of Cambridge -	- Citywide Stormwater Regulations & Guidelines
Department of Environmental Protection-	- Revised Stormwater Policy Document

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Cover photo courtesy of Friends of Alewife Reservation

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LIST OF ABBREVIATIONS USED

ac.	Acre(s)
ASCE	American Society of Civil Engineers
BMP	Best Management Practice
BOD	Biochemical Oxygen Demand
cfs	Cubic Feet per Second
CMR	Code of Massachusetts Regulations
COD	Chemical Oxygen Demand
col	Coliforms
CSO	Combined Sewer Overflow
DEP	Department of Environmental Protection
DPW	Department of Public Works
el.	Elevation
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
ft	Feet
GIS	Geographic Information System
HGL	Hydraulic Grade Line
I/I	Infiltration/Inflow
in.	Inch(es)
LID	Low-Impact Development
MOP	Manual of Practice
MS4	Municipal Separate Storm Sewer System
MWRA	Massachusetts Water Resources Authority
NGVD	National Geodetic Vertical Datum
NPDES	National Pollution Discharge Elimination System
NRCS	Natural Resources Conservation Service
OHM	Oily & Hazardous Materials
PAH	Polyaromatic Hydrocarbons
SOM	System Optimization Measure
SOP	System Optimization Plan
TMDL	Total Maximum Daily Load
TSS	Total Suspended Solids
WEF	Water Environmental Federation

SECTION 1 – INTRODUCTION

1.1 Concord-Alewife Area Stormwater Management Objectives

The purpose of this document is to provide guidance for development/redevelopment projects in the Concord/Alewife area so that reasonable measures are taken in these projects to address flood control and water quality. This guidance is required to protect the City of Cambridge's infrastructure and to ensure that the City can comply with the provisions of the Clean Water Act as provided for in the City's permits issued by the U.S. Environmental Protection Agency (EPA) through their National Pollution Discharge Elimination System (NPDES) programs specific to stormwater and Combined Sewer Overflows (CSO) as waters are conveyed through the City's systems to the Little River and Alewife Brook.

It is recognized that within this area there exists an overlap between the City of Cambridge Conservation Commission's jurisdiction, specifically as it relates to flood water protections within the 100-year flood zone and the City's need to protect and manage its infrastructure within the same area. Nothing in these guidelines should be construed as being in conflict with the provisions of the Wetland Protection Act or to be contrary or obstructive to the authority of the Cambridge Conservation Commission.

The Concord-Alewife area is approximately 180 acres in size and is bounded by Fresh Pond Reservation, Concord Avenue, Blanchard Road, The Alewife Reservation, and Danehy Park. Land use is predominately a mix of industrial and commercial types. According to the Cambridge Community Development Department's <u>Concord-Alewife Planning Study</u>, "… *the Concord-Alewife area is the last large commercial area of Cambridge having significant development potential and is in need of more detailed planning*." The area is adjacent to the important water resources of Fresh Pond and the Alewife Brook. Topography in the area is relatively flat and a portion of the area is within the 1982 Federal Emergency Management Agency (FEMA) defined 100-year floodplain. Those boundaries of the 100-year flood plain are currently under re-examination by FEMA and are expected to be expanded and affect more of the study area. Water quality in the Little River and Alewife Brook system is degraded due to Combined Sewer Overflows (CSOs) and stormwater runoff. The City of Cambridge is addressing CSO discharge reduction and control in partnership with the Massachusetts Water Resources Authority (MWRA). This is being done through separating combined sewer areas within the Alewife watershed, installing floatables control mechanisms at CSOs and constructing other system relief structures. Cambridge is also addressing its stormwater quality issues through the implementation of its stormwater management plan developed in compliance with the Municipal Separate Storm Sewer System (MS4) stormwater management program by the NPDES Stormwater Phase II Rule. The MS4 Phase II program was generated from the Clean Water Act (1972) and was created with the intention of improving the quality of local waterways by reducing the quantity of pollutants that stormwater picks up and carries into stormwater systems and discharges to surface waters. Water pollution from stormwater runoff is the leading cause of stormwater pollution in Massachusetts. Common pollutant definitions are provided in Appendix 1.

Redevelopment of private property in this highly impervious and urbanized area represents an opportunity to begin to address the stormwater quality and quantity concerns for private property in the area. Developing guidelines will complement the City's planning and construction efforts and will provide property owners with strategies to manage the quality and quantity of stormwater discharged from their properties. That is the primary intent of this document.

Guidelines outlining required sanitary sewer improvements for all new redevelopment projects are also provided for in this document. Future development in the area cannot exacerbate existing combined sewer overflow volumes or frequencies. Inflow and infiltration (I/I) and storage strategies are outlined so as to provide appropriate mitigation for new sanitary discharges.

With regard to stormwater discharges, the Cambridge Department of Public Works (DPW) requires development/redevelopment projects to provide on-site detention storage for the difference between the 2-year 24-hour storm event hydrograph and the 25-year 24-hour storm event hydrograph. The existing public drainage system can adequately convey a 2-year design storm. For larger events the system surcharges causing flooding, backups and ponding in various locations throughout the watershed. To this end, there are a number of traditional storage techniques (tanks and pipes) that are useful in achieving the required storage volumes; however these are less effective in terms of improving water quality. Reducing existing paved impervious areas with permeable green and vegetated open space can also create natural storage and can lessen the required on-site storage. Such controls can also improve stormwater quality, thus reducing the concentration of "pollutants of concern". [1] These controls can be broadly classified as Low Impact Development (LID) controls.

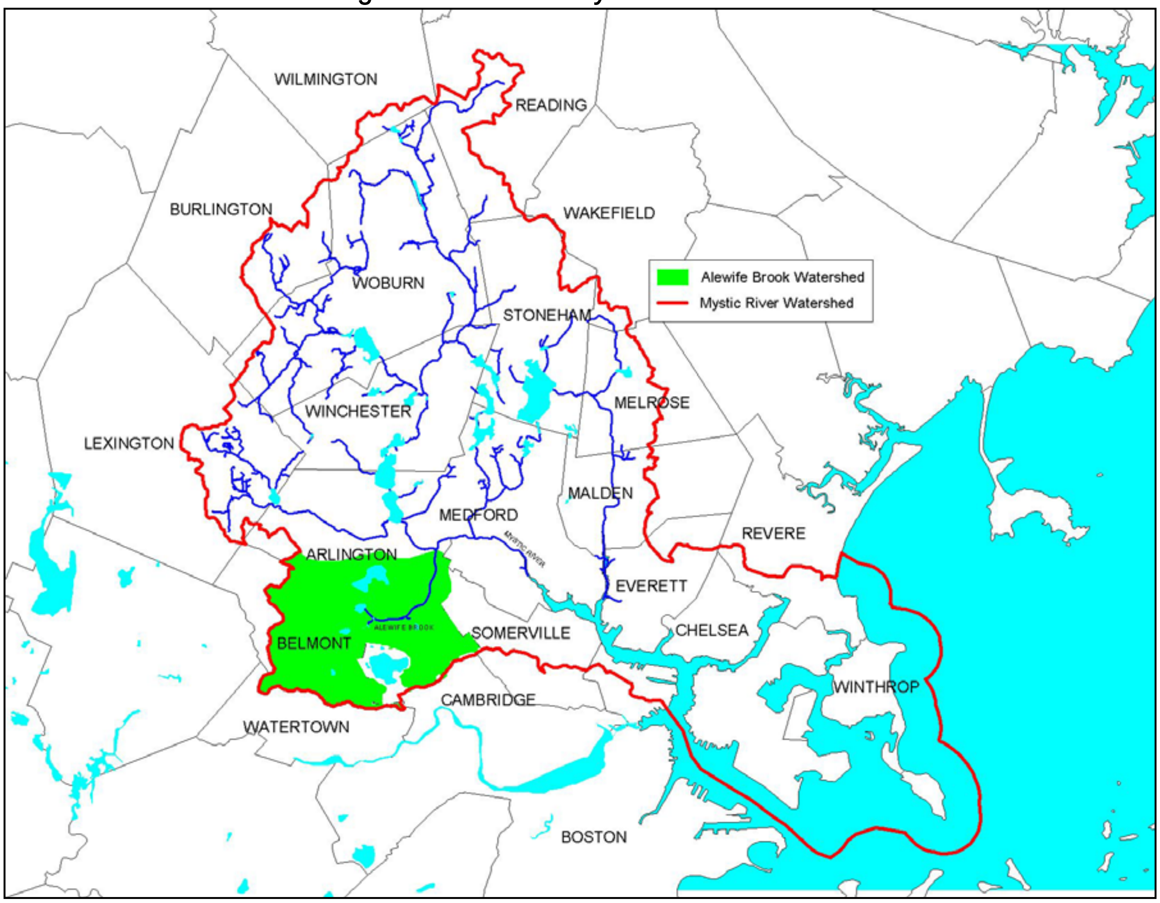
1.2 The Alewife Watershed

The Mystic River Watershed has an area of approximately 76 square miles, encompassing 21 communities north and west of Boston. The headwater of the system begins in Reading and from the Aberjona River, which flows into the Upper Mystic Lake in Winchester. The Mystic River flows from the Lower Mystic Lake through Arlington, Medford, Somerville, Everett, Charlestown, Chelsea, and East Boston before emptying into the Boston Harbor. Main tributaries to the Mystic River include Mill Brook, Alewife Brook, Malden River, and Chelsea Creek. The Mystic River watershed is home to approximately 8% of the state's population in less than 1% of its land area; the Mystic is one of the most densely populated and urban watersheds in Massachusetts.

The Alewife watershed lies in the southwest portion of the overall Mystic River watershed shown in Figure 1-1. The majority of the 8.5-square mile Alewife Brook watershed lies within three communities: Arlington (20%), Belmont (39%) and Cambridge (29%), with the balance of the area falling within portions of Somerville, Watertown and Medford. Figure 1-2 illustrates the portions of the Alewife Brook watershed in each community.

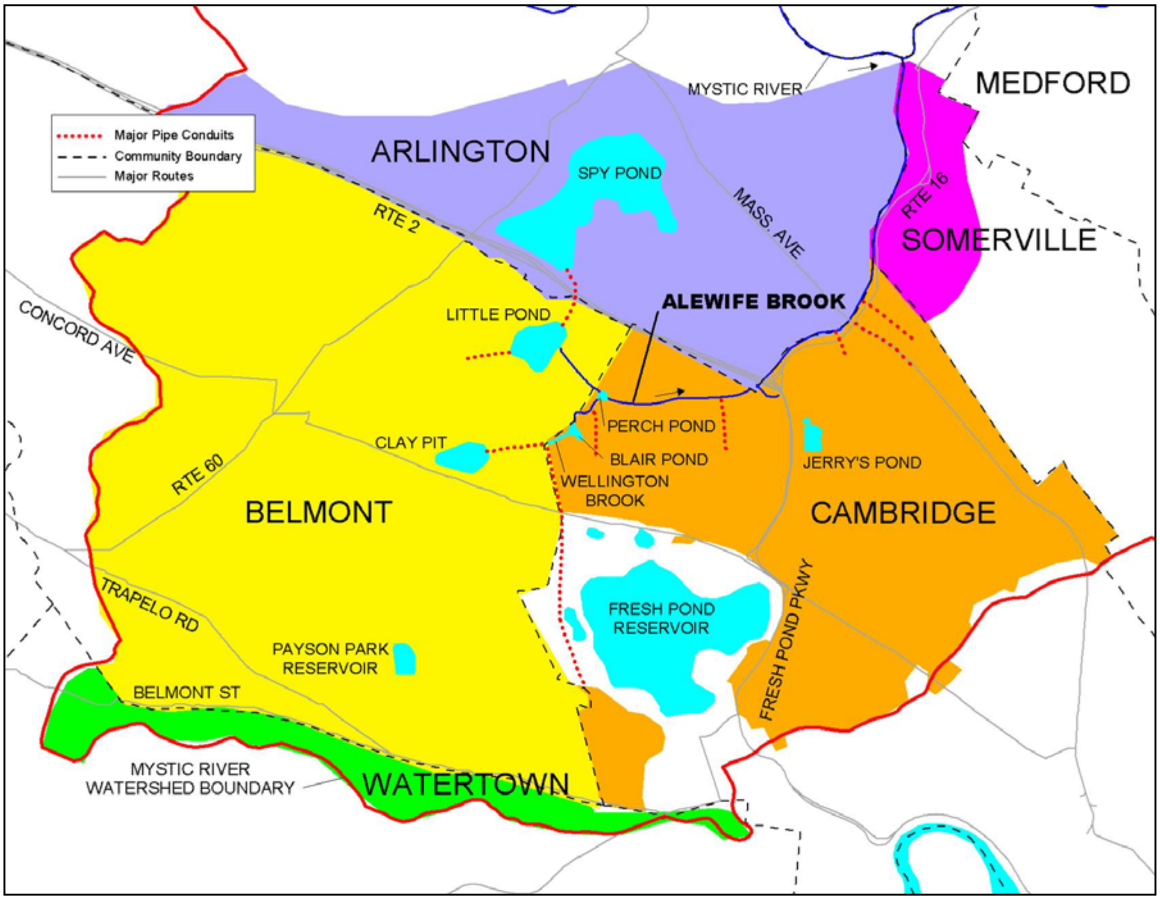
[1] Please see Section 1.6

Figure 1-1: Overall Mystic River Watershed



The Alewife Brook flows northeasterly to the confluence with the Mystic River, which discharges into Boston Harbor. The area draining to the Alewife Brook is primarily a residential urban area. Some commercial and industrial portions also lie within the basin, mainly near the northern Cambridge border. The shape of the sub-watershed is essentially that of a bowl. The steeper sloped areas of the system characterize the western, eastern and southern fringes, and the central area is predominantly flat. The system has very little topographic relief; the primary relief point being the Alewife Brook.

Figure 1-2: Alewife Brook Watershed



In addition to its topography, the other principal natural hydrologic feature of the watershed is the various ponds: Spy Pond, Little Pond, Blair Pond, and Clay Pit Pond. Spy Pond in Arlington covers an area in excess of 100 acres and flows toward Little Pond in a culvert. Little Pond in Belmont is at the upstream end of the Little River and is 18 acres in extent. Clay Pit Pond in Belmont flows toward Blair Pond in Cambridge via Wellington Brook. Blair Pond is connected to Little Pond/Little River by the continuation of Wellington Brook.

Urbanization of the sub-watershed communities, which has occurred over the past 50 years, has fundamentally changed the natural hydrologic characteristics of the area. Natural detention and storage of stormwater has been largely eliminated and replaced by impervious surfaces with constructed drainage systems. It is important to note that the Alewife Brook area has always experienced flooding, even prior to the development of the contributing municipalities.

The extent of development has increased the demand on the constructed drainage system. Movement of peak stormwater discharges through the system is limited by the conveyance rate and capacity of the trunk line pipes. As a result of flat topography and limited conveyance capacity, ponding and flooding problems occur throughout the municipal system. In summary, the natural flashiness of the system is exacerbated by the extent to which the area has been urbanized over the past century. This has resulted in fundamentally altering the natural runoff characteristics of the system.

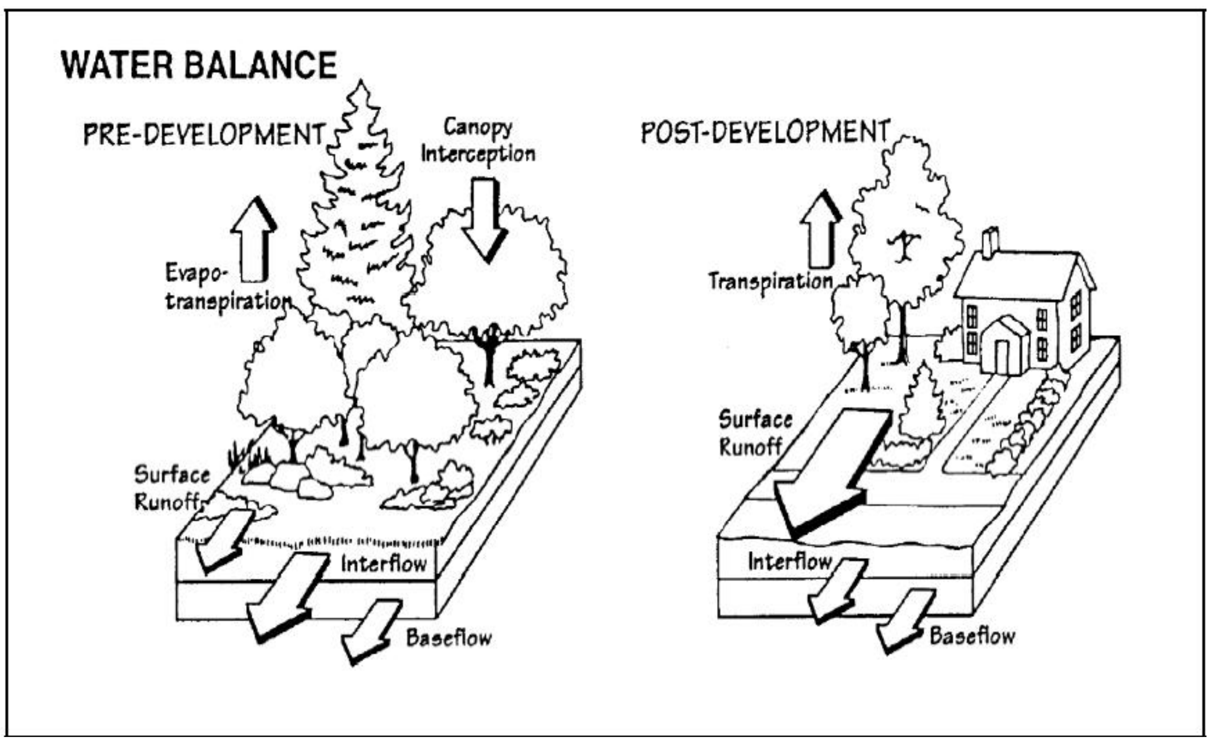
1.3 What is Stormwater Management?

Stormwater management is engineering and land management practice that incorporates into the design of development measures that mitigate or abate soil erosion, water quality degradation and flood risks. Best Management Practices (BMPs) are stormwater management techniques that store and/or treat stormwater before (source controls) or after (end of pipe controls) they enter the stormwater drainage system. BMPs can include practices such as swales, detention and retention ponds and storage tanks, deep sump hooded catch basins, etc. LID approaches generally address stormwater through small, cost effective landscape features. LID techniques include green roofs, rain gardens, cisterns, etc. Further LID control technology details that are appropriate for the Concord-Alewife area are provided in Appendix 2.

1.4 Hydrologic Impacts of New Development

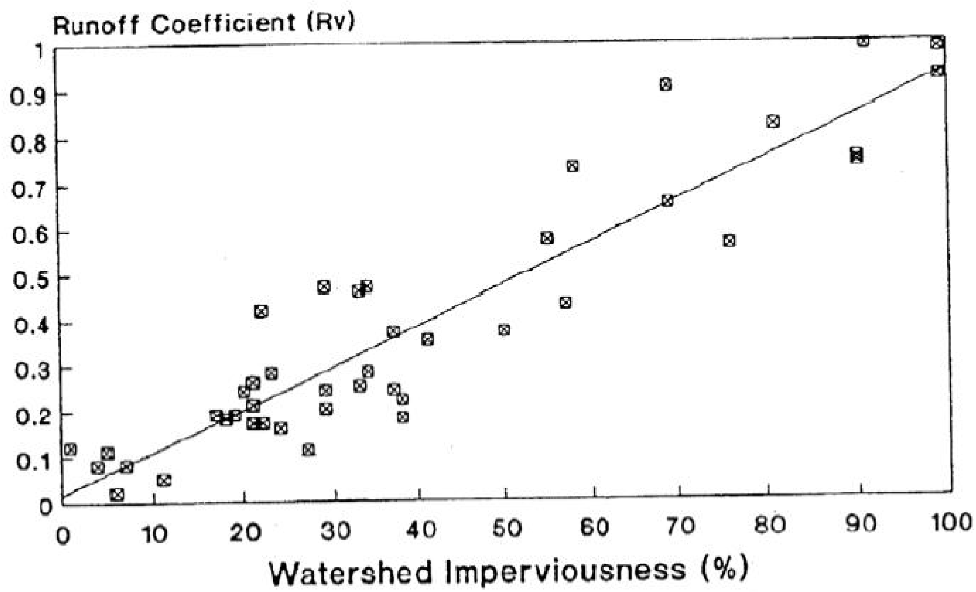
Urban development has a profound influence on the conveyance quantity and quality of receiving waters. Development can dramatically alter the local hydrologic cycle (See Figure 1-3). The hydrology of a site changes during the initial clearing and grading that occurs during construction. Trees that had intercepted and stored rainfall are removed and natural depressions (i.e. wetlands and marsh areas) that had temporarily ponded water are graded to a uniform slope. Having lost its natural storage capacity, a cleared and graded site can no longer prevent rainfall from being rapidly converted into stormwater runoff.

Figure 1-3: Water Balance at a Developed and Undeveloped Site (Schueler, 1987)



The situation can worsen after construction. Rooftops, roads, parking lots, driveways and other impervious surfaces no longer allow rainfall to soak into the ground. Consequently, most rainfall is directly converted into stormwater runoff. Figure 1-4 shows the increase in the volumetric runoff coefficient as a function of site imperviousness. The runoff coefficient expresses the fraction of rainfall volume that is converted into stormwater runoff. For example, a one-acre parking lot can produce 16 times more stormwater runoff than a one-acre meadow each year (Schueler, 1994).

Figure 1-4: Relationship Between Impervious Cover and Runoff Coefficient (Schueler, 1987)



The increase in stormwater runoff can be too much for the existing local drainage system to handle. As a result, the drainage system is often “improved” to rapidly collect runoff and quickly convey it away (using curb and gutter, enclosed storm sewers, and lined channels). The stormwater runoff is subsequently discharged to downstream receiving waters. In the Concord-Alewife area stormwater runoff is discharged to the Little River and Alewife Brook.

Development within the flood plain of a river can increase flood impacts beyond those previously mentioned. New structures constructed within the flood plain may displace floodwaters such that the elevation of the floodwaters increases. In turn, this may aggravate flood impacts elsewhere within the flooded area during significant flood events. Conservation Commissions are empowered to require development within the prescribed 100-year flood plain to provide unrestricted access to compensatory storage volume on a per-foot basis as a flood rises to the 100-year event elevation, as established by FEMA.

1.5 Water Quality Concerns from Development

Surfaces accumulate pollutants deposited from the atmosphere, leaked from vehicles, windblown from adjacent areas, or left behind by humans and/or animals. During storm events, these pollutants quickly wash off impervious surfaces, and are rapidly delivered to downstream waters. These pollutants originate from sources to include commercial, industrial and residential parking areas; roadways; automobile service stations; sewer infiltration from leaking pipes; accidents and spills; parks and residential/commercial lawns; construction sites; and active and inactive industrial sites. Some common pollutants found in urban stormwater runoff are profiled in Table 1-1. Appendix 1 provides more detail with regard to these pollutants.

Table 1-1: National Median Concentrations for Chemical Constituents in Stormwater

Constituent	Units	Concentration
Total Suspended Solids[1]	mg/l	54.5
Total Phosphorus[1]	mg/l	0.26
Soluble Phosphorus[1]	mg/l	0.10
Total Nitrogen[1]	mg/l	2.00
Total Kjeldhal Nitrogen[1]	mg/l	1.47
Nitrite and Nitrate[1]	mg/l	0.53
Copper[1]	Ug/l	11.1
Lead[1]	Ug/l	50.7
Zinc[1]	Ug/l	129
BOD[1]	mg/l	11.5
COD[1]	mg/l	44.7
Organic Carbon[2]	mg/l	11.9
PAH[3]	mg/l	3.5*
Oil and Grease[4]	mg/l	3.0*
Fecal Coliform[5]	col/ 100ml	15,000*
Fecal Strep[5]	col/ 100ml	35,400*
Chloride (snowmelt) [6]	mg/l	116

* Represents a Mean Value
Source:
1 Pooled NURP/USGS (Smullen and Cave, 1998)
2 Derived from the National Pollutant Removal Database (Winder, 2000)
3 Rabanal and Grizzard 1995
4 Crunkilton *et al.*, 1996
Schueler, 1999
Oberts, 1994

1.6 Pollutants of Concern for Alewife Brook

In 2002 the Massachusetts Department of Environmental Protection (DEP) listed the Alewife Brook from the Little River in Belmont to the confluence of the Mystic River in

Arlington/Somerville as "*Impaired or threatened for one or more uses and requiring a Total Maximum Daily Load (TMDL)*". This listing as an impaired waterway was reviewed and approved by the EPA. The "Pollutants of Concern" specific to the Alewife Brook are listed below. These were developed by DEP from visual inspection and water quality sampling survey data. These guidelines will continue to be updated to reflect evolving Federal and State water quality requirements specific to the TMDL rule.

Alewife Brook "Pollutants of Concern"

- Metals
- Nutrients
- Organic enrichment
- Pathogens
- Oil & Grease
- Objectionable sediments (solids)

Once a water body is identified as impaired, DEP is required by the Federal Clean Water Act to develop a "pollutant budget" for each of the pollutants of concern. The pollution budget is designed to restore the health of the impaired body of water. TMDLs are designed to limit the maximum amount of the identified pollutant that can be discharged to a specific water body (e.g. Alewife Brook) to meet water quality standards, and assign pollutant load allocations to the sources. Currently, DEP has proposed a Pathogen TMDL for the Boston Harbor watershed, which encompasses the Mystic River watershed and the Alewife Brook. The TMDL established indicator bacteria limits and outlines corrective actions to meet water quality standards. Sources of indicator bacteria were found to be many and varied. Most of the bacteria sources are believed to be stormwater related (DEP 2005). TMDLs for the other pollutants of concern have not yet been proposed for this watershed.

The water quality standard designates the most sensitive uses for which the surface waters shall be enhanced, maintained and protected, prescribes minimum water quality criteria to sustain the designated uses, and includes prohibition of discharges (DEP 1996). The Alewife Brook is currently under a variance to the water quality standard until September 1, 2007 and is designated as a Class B_{CSO} waterway. Class B_{CSO} waters are designated as habitat for fish, other aquatic life and wildlife, and for primary and secondary contact recreation, but permit short-term impairment of swimming or other recreational uses due to Combined Sewer Overflow (CSO) discharges. Primary and secondary uses are supported through most of their annual period of use. A B_{CSO} classification allows limited CSO discharges until DEP has the information necessary to determine the appropriate water quality standard and level of CSO control for the receiving water. Presently there are eight (8) permitted CSOs along the Little River/Alewife Brook. The proposed level of CSO control for the outfalls is detailed in the MWRA's Notice of Project Change for the Long Term CSO Control Plan for Alewife Brook, dated April 30, 2001. DEP will make a final water quality determination at the conclusion of the variance period.

SECTION 2 – CONCORD-ALEWIFE AREA HYDROLOGY

2.1 Overview

The Concord-Alewife area is shown in Figure 2-1. The study area outlined in blue totals 182 acres with privately held property consisting of 169 acres. Approximately 149 acres of the area is impervious, 52 acres of which is roof area and 97 acres of which is pavement. The average imperviousness of the area is 88%.

Subcatchments were delineated in the Concord-Alewife area for the purpose of hydrologic/hydraulic analysis and are shown in Figure 2-2. Figure 2-3 shows the average imperviousness for each subcatchment. Figure 2-4 shows a histogram of the imperviousness of the area, based on the subcatchments. Approximately 48% of the study area has imperviousness between 95% and 100%.

The Concord-Alewife area is in the floodplain of the Little River/Alewife Brook and is flat and low-lying. The overall elevation gradient along the length of the study area is less than half a foot, and the average elevation of the study area is less than several feet above sea level. Figure 2-5 illustrates the delineation of the current FEMA 100-year and 500-year floodplains.

2.2 Soils

The Concord-Alewife area soils exhibit slow infiltration rates. Historically, the area was mined for clays and as a result of the subsequent development most of the surface soils in the area were highly disturbed by cut and fill activities and thus a significant quantity of urban fill can now be found in the area. Most of the soils now lack characteristics of naturally developed, undisturbed soils, such as defined layers and horizons, and their poor quality may serve as a constraint to restoration.

Soil boring logs extracted as part of recent construction activities, as well as onsite visual observations during previous construction projects confirm the nature of the majority of the soils in the area as being poorly draining with low permeability. Soil borings extracted for a recent DPW project in the Alewife area indicate the presence of 10-12 feet of poorly draining urban fill and peat/organic silt layers mixed together. These conditions will restrict the types of potential stormwater management controls for the area.

2.3 Groundwater

Figure 2-1 also depicts depth from ground surface to average groundwater table in and around the Concord-Alewife area. Groundwater levels were monitored throughout the Fresh Pond Reservation during the period of 1995-1997 (CDM). The average groundwater elevation observed at the monitoring station near Black's Nook adjacent to Concord Avenue was 4.4 ft NGVD. The range of measurements varied from 3.4 to 6.7 ft NGVD. The standard deviation of the observed levels at Black's Nook based on 17 readings was 1.1 feet. A ground water elevation of 4.5 ft NGVD was noted at Wheeler Street just north of Concord Avenue during recent soil boring work conducted by Cambridge DPW. Long term monitoring wells (2001-2003) placed at the south of the end of Cambridgepark Drive and near Little River indicated an average groundwater elevation of 1.0 ft NGVD.

On the basis of this information average values of groundwater elevations throughout the Concord-Alewife area were linearly interpolated starting from 4.5 ft NGVD at Concord

Avenue, 4.0 ft NGVD at Fawcett Street (south of the B&M railroad), 3.75 ft NGVD on the north side of the railroad tracks, 3.5 ft NGVD at Cambridgepark Drive, 2.0 ft NGVD at the Massachusetts Department of Conservation and Recreation's (DCR) access road, and 1.0 ft NGVD at Little River. It should be noted that this profile represents an average value interpolation with the potential to be higher during wet years. The median groundwater depth according to the data in Figure 2-6 is 3.4 feet. The shallowness of the groundwater table in the Concord-Alewife area will limit the types of effective stormwater management techniques.

2.4 Water Quality

Water quality in the Little River and Alewife Brook is poor because of stormwater and CSO discharges. The degraded condition of wetlands along the Little River and Alewife Brook also means they are less effective at their natural function of buffering and improving water quality. Recent projects by the Massachusetts Water Resources Authority, the Cities of Somerville and Cambridge, and the Towns of Belmont and Arlington have significantly improved water quality, and continuing projects will yield yet more improvements in the future.

Figure 2-1: Concord Alewife Area

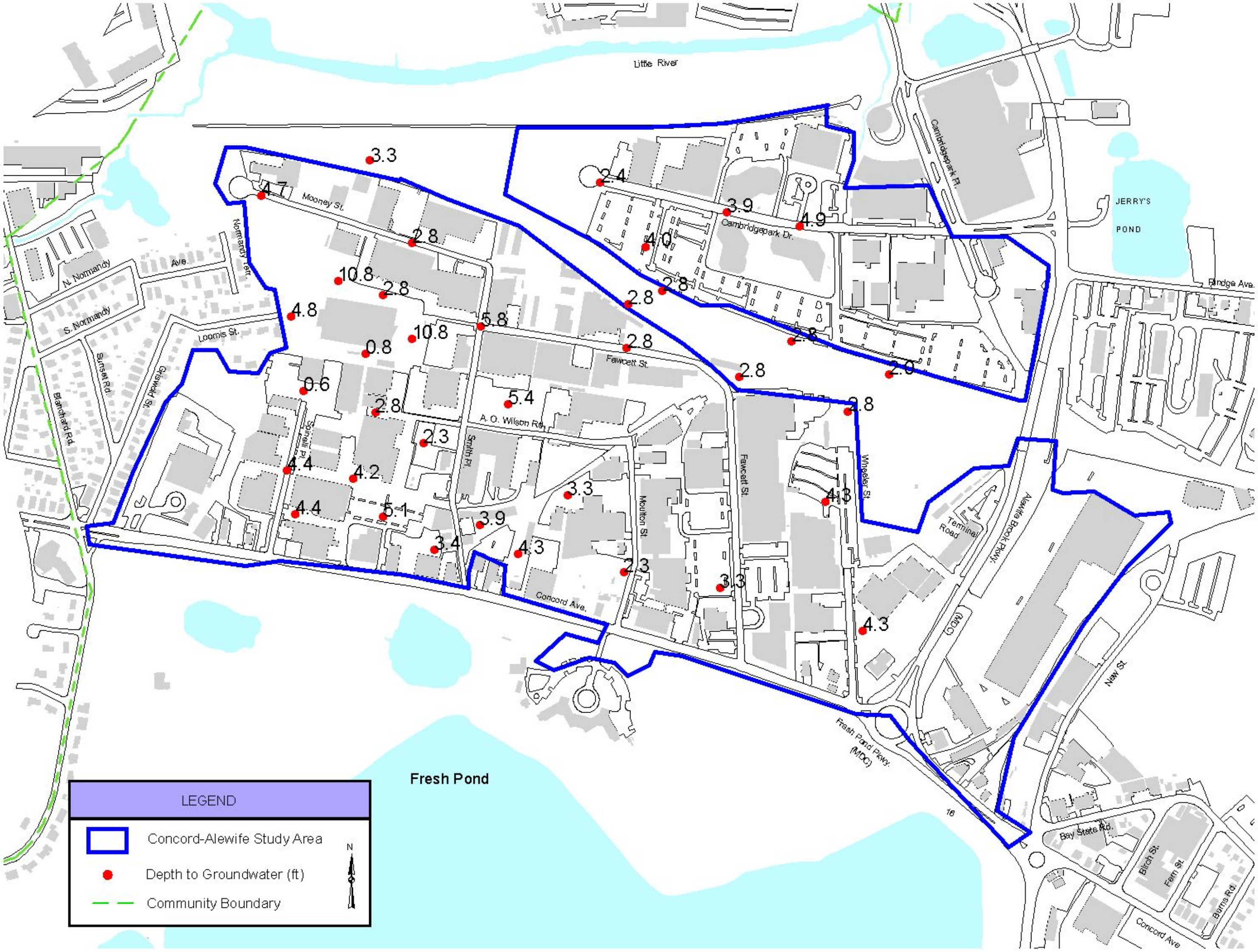


Figure 2-2: Concord Alewife Area Subcatchment Delineation

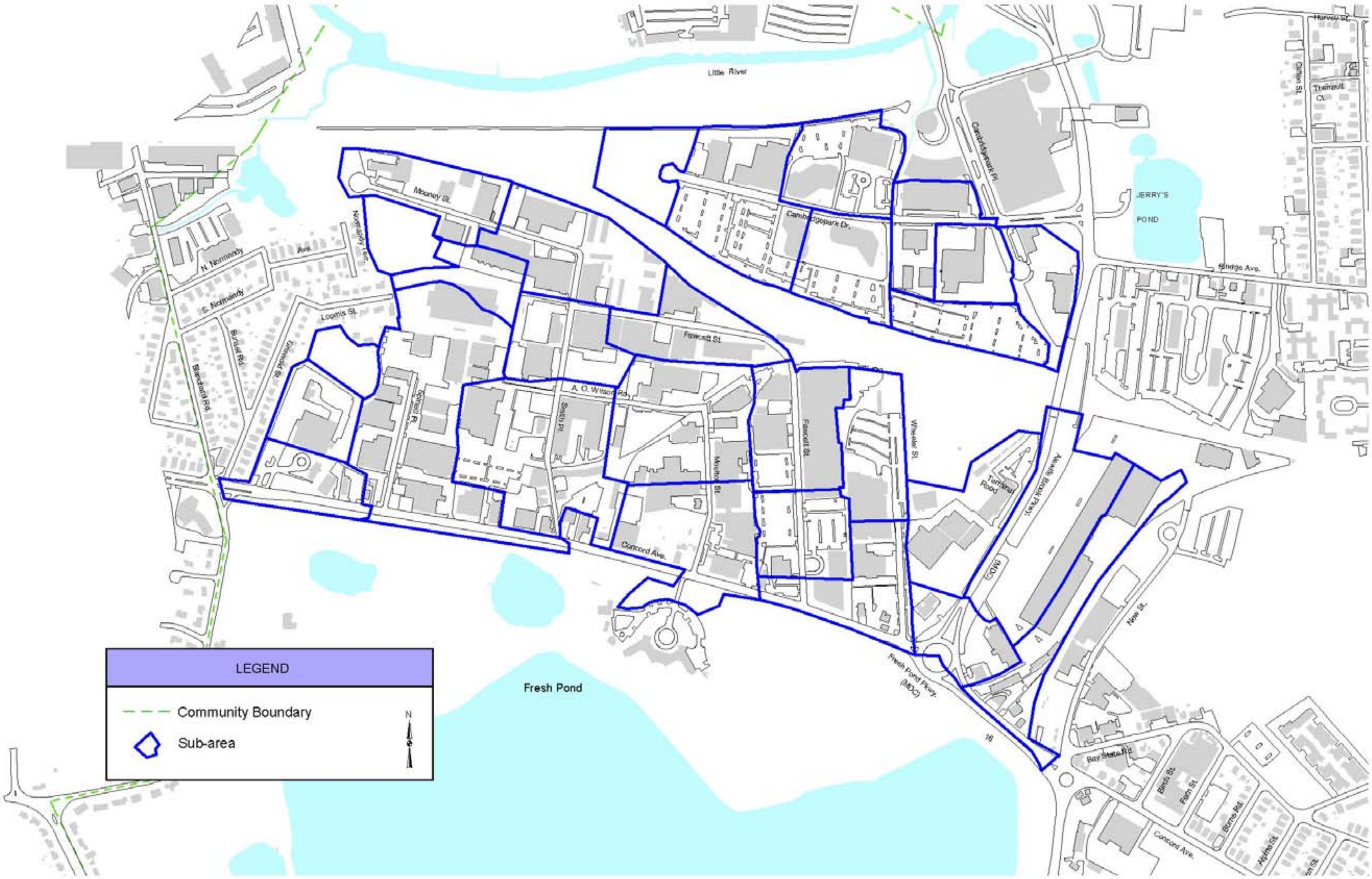


Figure 2-3: Concord Alewife Area Subcatchment Imperviousness

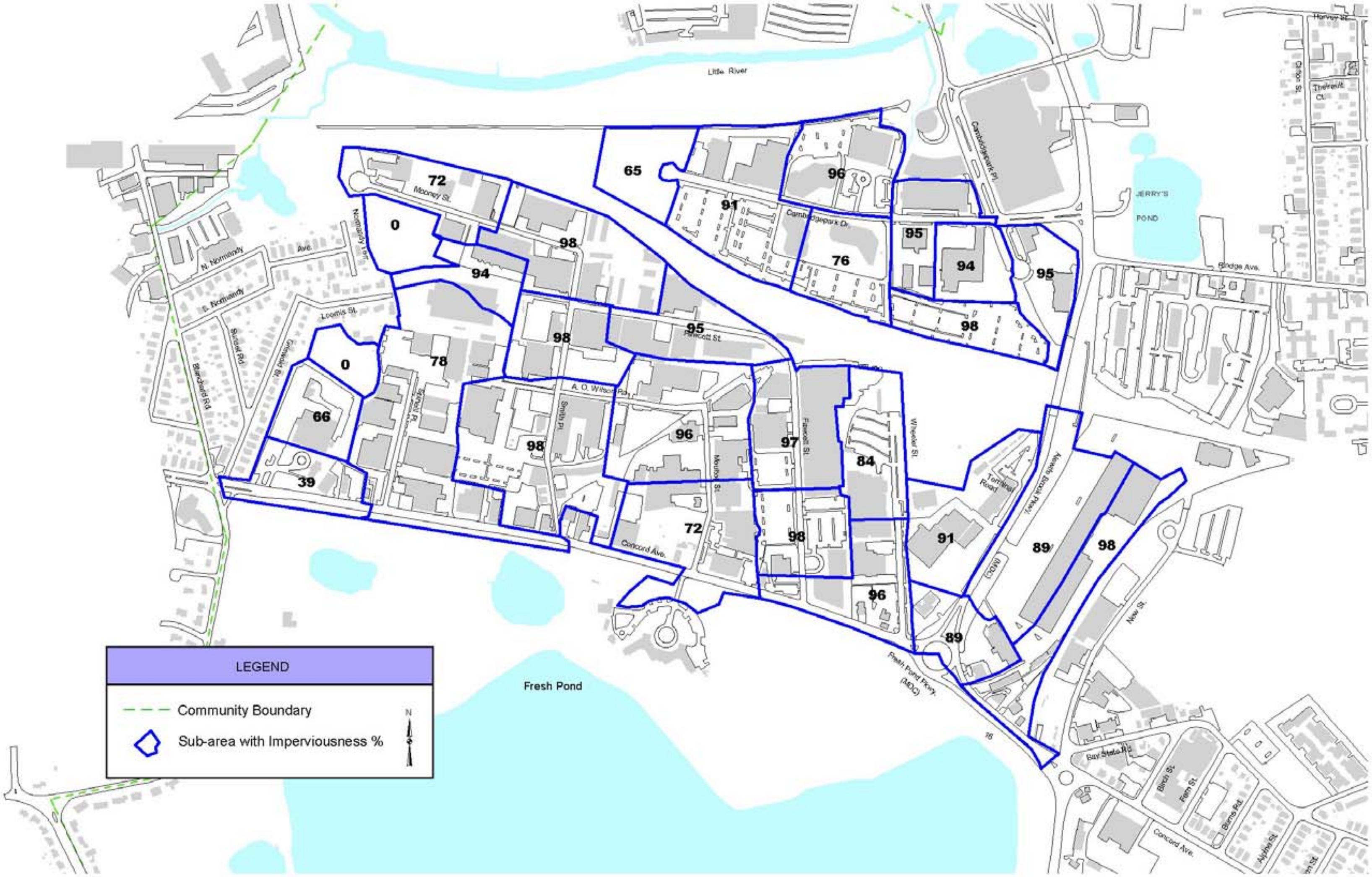


Figure 2-4: Histogram of Imperviousness for the Concord-Alewife Area

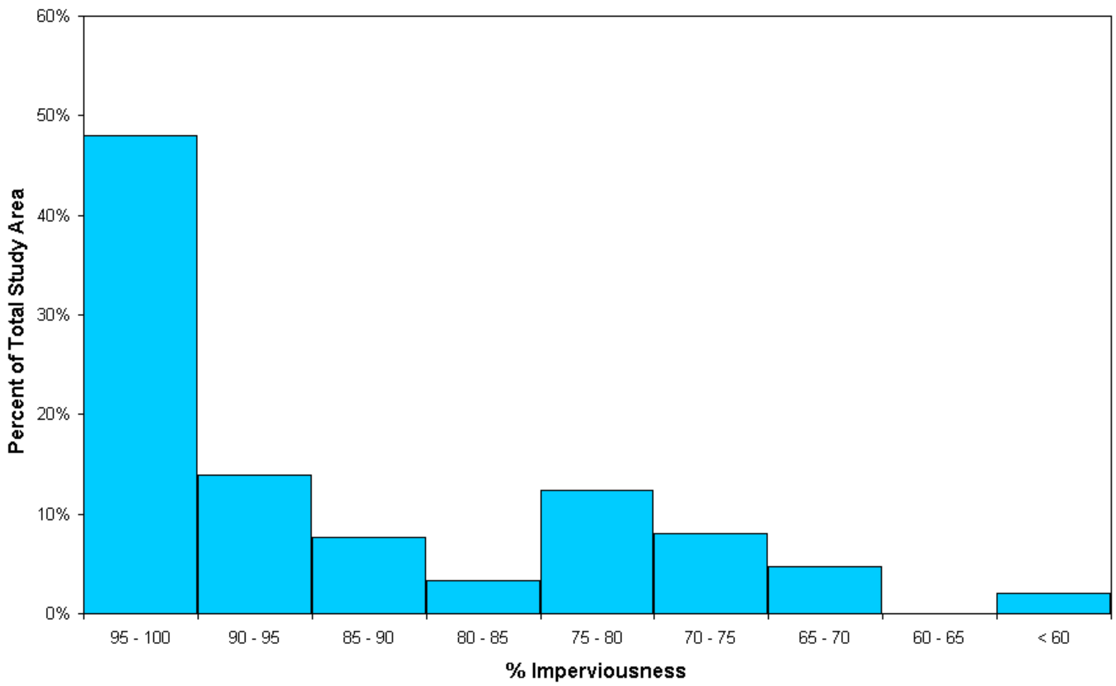
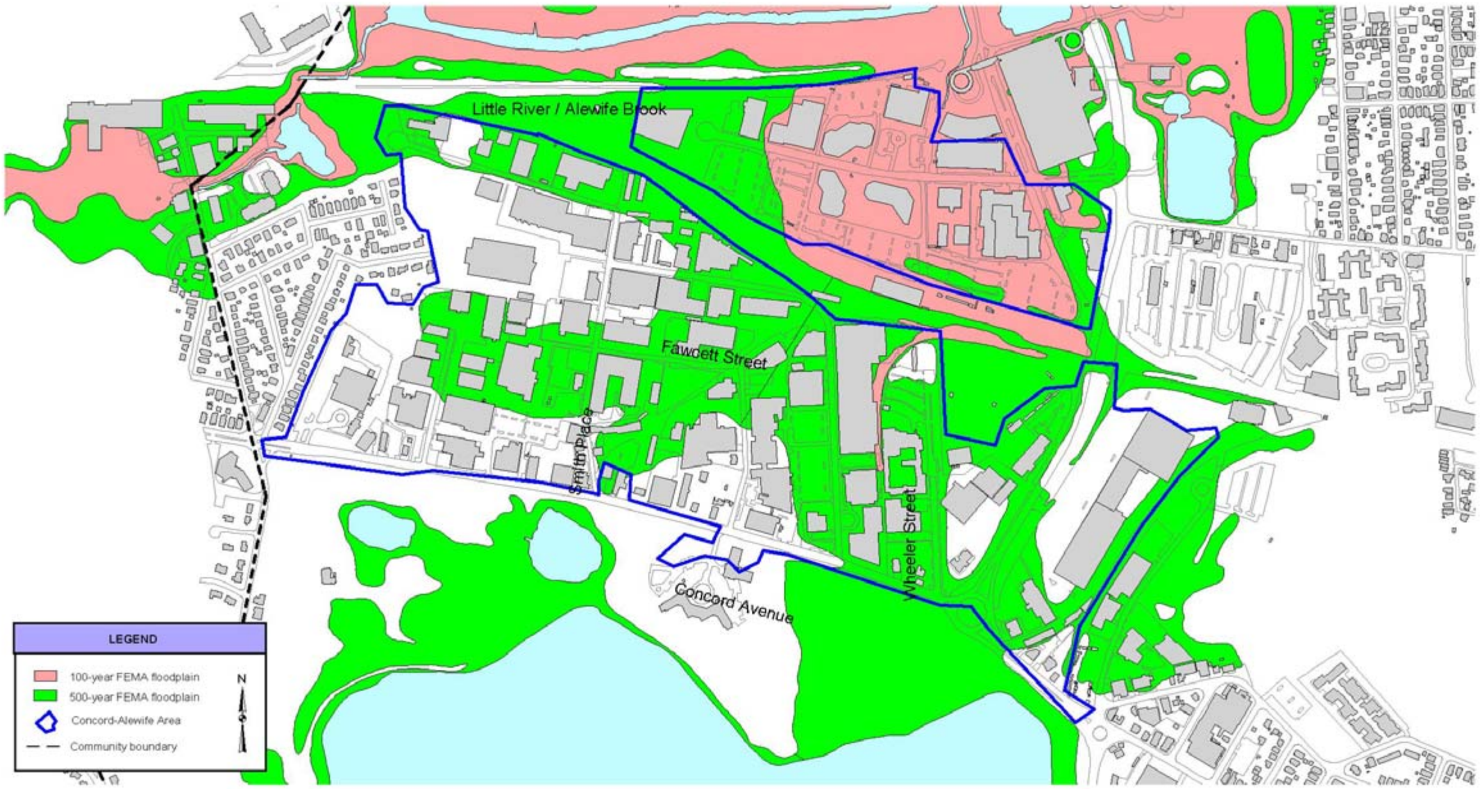


Figure 2-5: Current FEMA 100-year and 500-year Floodplain Delineation in the Concord-Alewife Area



SECTION 3 – STORMWATER CONTROL REQUIREMENTS IN THE CONCORD - ALEWIFE AREA

3.1 Introduction

Stormwater quantity and quality control is required to the maximum extent practicable on all development/redevelopment (projects) in the Concord/Alewife area. Those stormwater quantity and quality control requirements specified hereunder are applicable to the following classifications, unless deemed otherwise by the City Engineer.

(1) Where the project exceeds fifty thousand (50,000) square feet of Gross Square Area

(2) Where the project parcel or parcels equals or exceeds one acre in size.

(3) Where the project includes outdoor parking for 10 cars or more.

Stormwater quantity and stormwater quality controls are not mutually exclusive and in many instances the control solutions specified for water quantity control provide significant water quality benefits as well. It can be generally stated that stormwater quality control can be achieved by a variety of stormwater storage and sedimentation techniques. The water quality runoff volume can be applied toward the total runoff quantity control volume to be retained onsite, provided the post-development peak discharge rate requirements are met. Many of the controls specified hereunder have been adopted from the Massachusetts Department of Environment Protections (DEP) Stormwater Management Policy most particularly as they relate to quality controls.

Practices that are acceptable from a water quantity perspective must be capable of meeting the following criteria

1. Storing the difference between the 2 year 24 hour pre-construction runoff hydrograph and the post construction 25 year 24 hour runoff hydrograph.
2. Ensuring that the post project peak discharge from the project area does not exceed the pre project peak discharge from the project area.
3. Ensuring that the stormwater runoff as a result of the project does not have an negative impact on abutting property
4. Ensuring that there shall be no reduction in groundwater discharge as a result of the project.

The runoff volume to be treated for water quality is based on the DEP Stormwater Management Policy. Practices that are acceptable for water quality treatment must be capable of meeting the following criteria:

1. Treating the full water quality volume, and

2.Removing 80% TSS and 98% trash and floatables for new developments in accordance with DEP Stormwater Management Guidelines

3) Are capable of TSS and trash and floatables removal to the maximum extent practicable for redevelopment projects

With the exception of “stormwater hot spots” noted below, all treatment systems capable of satisfying the TSS and trash removal requirements are also assumed to satisfy pollutant level criteria for other “pollutants of concern”.

Stormwater Hot Spots

A stormwater hot spot is defined as a land use or activity that generates higher concentrations of hydrocarbons, trace metals or toxicants than are found in typical stormwater runoff. If a site is designated as a hotspot, it has important implications for how stormwater is managed. Stormwater runoff from hot spots cannot be allowed to infiltrate into groundwater. Additionally, a greater level of stormwater treatment is needed at hot spot sites to prevent pollutant washoff. This treatment plan typically involves preparing and implementing a stormwater pollution prevention plan that involves a series of operational practices at the site that reduce the generation of pollutants from a site or prevent contact of rainfall with the pollutants. For both present and future conditions, the following land uses and activities within the Concord-Alewife area are deemed typical stormwater hot spots:

- Vehicle fueling stations
- Vehicle service and maintenance
- Vehicle and equipment cleaning facilities
- Fleet storage areas (bus, truck, construction equipment yards)
- Outdoor liquid container storage
- Outdoor loading/unloading facilities

The following land uses and activities are not normally considered hot spots:

- Public streets
- Residential development
- Institutional development
- Office developments
- Non-industrial rooftops

Additional water quality treatment for the first half inch of runoff from urban hot spots shall meet the following criteria:

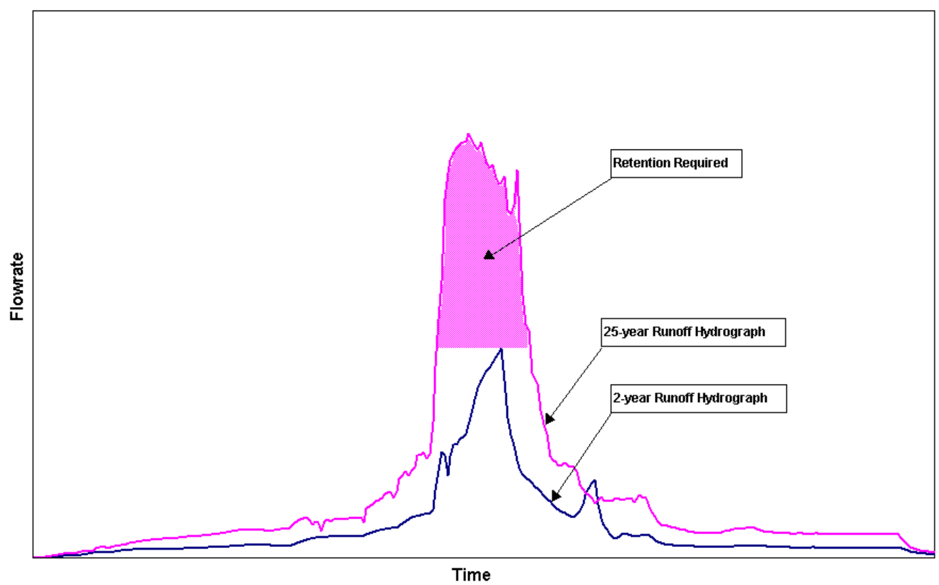
1) 98% removal of oil and grease, and
2) 90% removal of all heavy metals associated with particulate solids in excess of 10 microns.
3) The submission of a Spill Pollution Prevention Control Plan.

3.2 Stormwater Quantity Control

3.2.1 INTRODUCTION

This Section outlines the methodology for determining the required private development stormwater runoff retention/detention quantity for parcels in the Concord-Alewife area. These requirements are in addition to those required for properties within the 100 year flood plain but can be engineered to compliment each other. As a general rule, for properties discharging in to the City of Cambridge municipal drainage system the City will provide a drainage level of service up to the 2-year storm event. The stormwater runoff detention requirement states that the total volume of runoff generated between the pre-development 2-year 24-hour storm discharge and the post development 25-year 24-hour storm discharge shall be retained. Figure 3-1 illustrates this requirement with hypothetical stormwater runoff plots. The shaded area (the area between the peak 2-year runoff and the 25-year runoff) represents the quantity of stormwater retention required for achieving compliance with the City's stormwater quantity control rule.

Figure 3-1: Private Development Onsite Retention Requirement



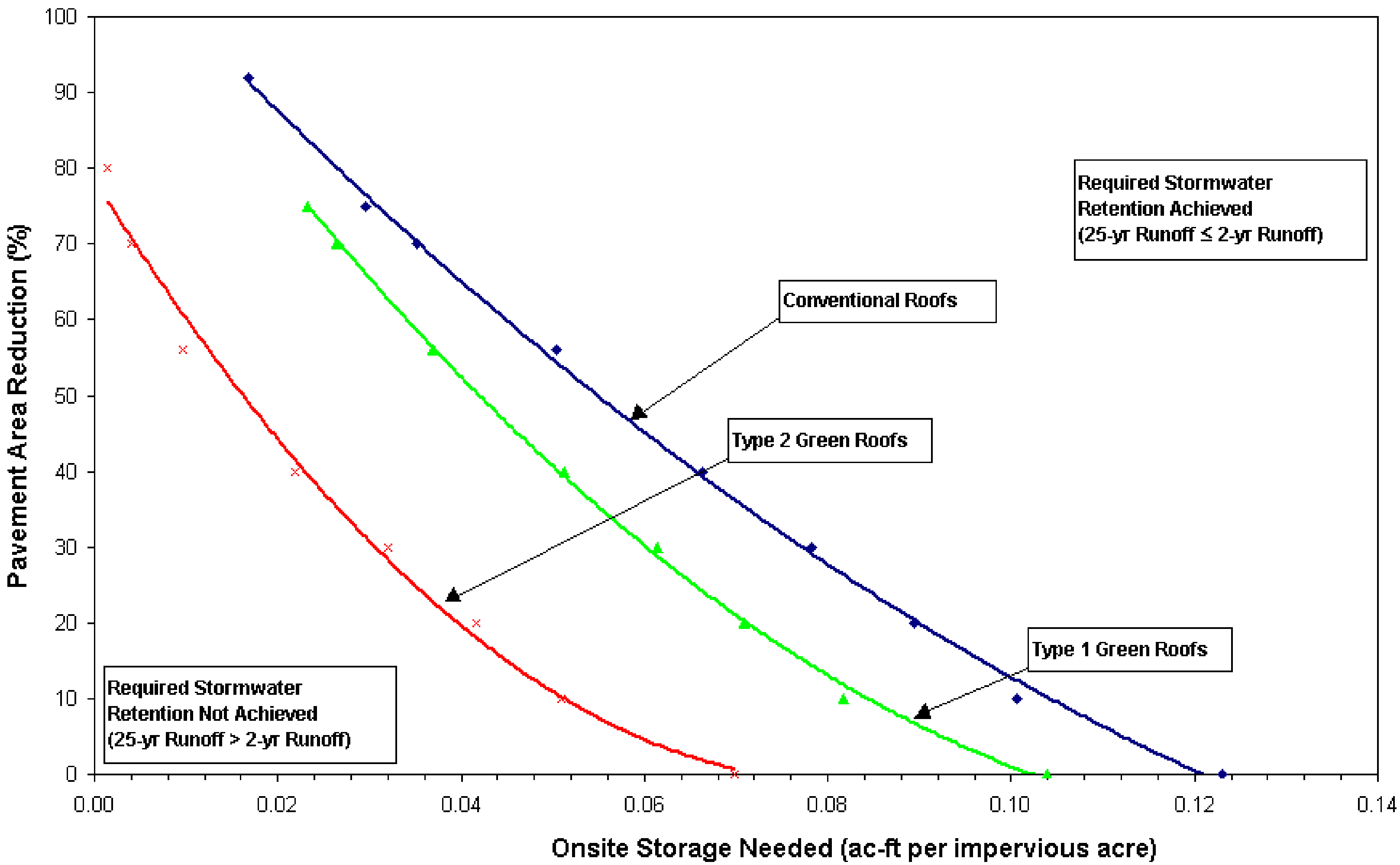
Potential impacts of redevelopment in the Concord-Alewife area have been evaluated using hydrologic and hydraulic modeling. A more detailed summary of the hydrologic/hydraulic modeling analysis is provided in *Appendix 4*. Modeling scenarios were developed based on area-specific, applicable methods of reducing runoff in the Concord-Alewife area. Overall stormwater management goals of the Concord-Alewife Rezoning Petition include increasing the minimum open space requirement and also creating a permeable space requirement. Types of runoff reduction technologies included in the analysis were green roofs, converting impermeable surfaces to permeable surfaces, and onsite storage. Storage can be developed by any combination of shallow swales, detention ponds, and underground tanks. Since groundwater levels in the study area are relatively high, the use of many typical infiltration technologies, such as infiltration trenches and basins, biofilters or porous pavement, are difficult to apply here. However, green roof technologies have been successfully implemented in Cambridge on Sidney Street and are an environmentally attractive and effective method of runoff reduction. The Sidney Street project combined green roof technology with roof top detention storage.

Figure 3-2 illustrates a simple tool used for calculating stormwater runoff retention requirements for any given parcel in the Concord-Alewife area using roof storage technologies as alternatives or in combination with traditional detention-type storage facilities. Three parameters that represent potential runoff reduction strategies are shown in Figure 3-2 and include roof type, pavement area reduction and onsite storage. Three curves are drawn which represent the relationship between the three parameters and the range of alternatives that could be used to satisfy the runoff retention requirement. The roof types specified are detailed in Table 3-1.

Table 3-1: Modeled Roof Configuration Summary

Configuration	Description
Conventional Roof	100% of the roof surface is impervious
Green Roof Type 1	100% of the roof surface is “green”; Excess runoff is conveyed directly to the collection system
Green Roof Type 2	60% of the roof surface is “green” (excess runoff to roof storage compartment); 40% of the roof surface is modeled as conventional (impervious); All runoff is conveyed into a 4” deep storage chamber on the roof through a controlled outlet down to the collection system

Figure 3-2: Private Development Stormwater Retention Requirements



The amount of pavement area reduction to achieve the required runoff retention (i.e., pavement area converted to green pervious area) is plotted on the y-axis while the corresponding amount of required on-site storage volume (per existing impervious acre) is plotted on the x-axis. The roof type is represented by the three different curves. The stormwater quantity control requirement is met if a point falls on the curve or to the upper-right hand side of the curve.

<u>Analysis Assumptions:</u>

Existing conditions pervious surface types are assumed to have low infiltration capacity (i.e., mostly hydrologic soil type D). One assumption in Figure 3-2 is that when changing an impervious surface area to a pervious surface area the pervious surface area has a composite hydrologic soil type, made up of 10%B, 60%C and 30%D. Another assumption in Figure 3-2 is that the green roofs are the "extensive" type, with 0.3 inches of initial uptake of rainwater, as documented in green roof pilot studies. The figure also assumes that the green roofs are comprised of vegetation similar to prairie grass (Manning's surface roughness value of 0.35), which allows for flow attenuation. *Appendix 3* contains two examples of how this graph can be applied in calculating the various storage quantities required.

3.2.2 CONVENTIONAL STORAGE TECHNIQUES

This section discusses traditional methods for retaining and detaining runoff. Retention systems hold the water, thus reducing the overall volume leaving the site. Detention systems involve releasing water slower than it is captured, thus reducing the peak flow rate of water leaving the site.

Surface Retention Basins

Wet ponds or retention basins are constructed stormwater ponds that retain a permanent pool of water. Besides water quantity reduction, a permanent pool enhances water quality treatment by promoting sedimentation, which removes significant loads of metals, nutrients, and organics. Dissolved contaminants are also removed by physical adsorption to the bottom or through natural chemical flocculation and settlement and through biological processes of bacterial decomposition and uptake by aquatic plants and algae. Generally, large tributary watersheds are required to maintain pool elevations. There are concerns about these ponds becoming grounds for mosquito breeding.

Surface Detention Basins

Dry ponds, or detention ponds, are basins that temporarily impound water for gradual release to the receiving stream or storm sewer system. They are generally designed to completely empty out between runoff events. They control water quantity and can limit downstream scour and loss of aquatic habitat by reducing the peak flow rate and energy of discharges to receiving water systems.

Underground Retention Tanks

Off-line underground storage tanks are commonly used in other parts of Cambridge. These tanks are generally constructed of concrete box culvert sections or of poured-in-place concrete structures. They have been used in Cambridge for both private and public projects, particularly where space is limited. Retention tanks, as opposed to detention tanks, have generally been used due to the shallowness of Cambridge storm conveyance systems. The form of retention system used on Cambridge public projects is called a burp storage system. Typically, weir overflow chambers from the conveyance system control the influent flow to retention tanks such that the mainline system does not surcharge. Stored flow is returned to the system via small post-event pumps after the storm event has passed and capacity has returned to the system. This type of underground storage technology is appropriate for the Concord-Alewife area. BMP catch basins for paved tributary area upstream of the storage tank are required (see Section 3.3.3 for further details). Furthermore, underground storage tanks must have adequate entry points for cleaning operations. Finally, for projects within the 100-year flood plain, underground retention tank systems can also be designed to allow unrestricted access by river floodwaters that would otherwise be displaced as a result of new structures being built within the flood plain, thus satisfying compensatory flood storage requirements.

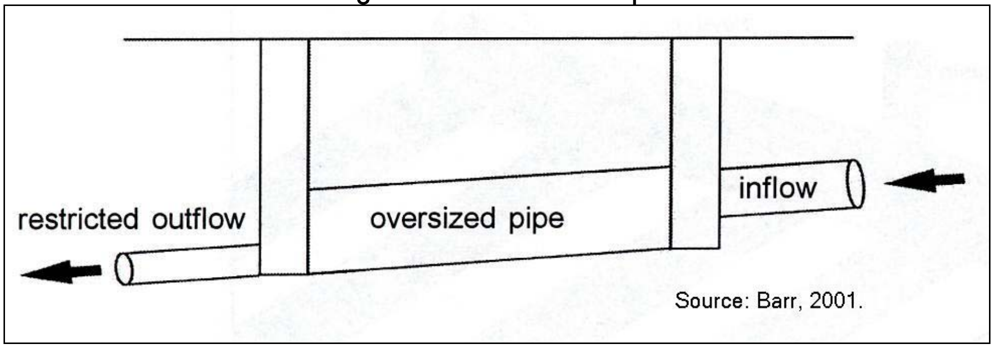
Underground Detention Tanks

Off-line underground storage tanks are used throughout Cambridge and are similarly constructed of concrete box culvert sections or poured-in-place structures. In a few instances, detention tanks, as opposed to retention tanks, have been used where there is sufficient grade to create storage that can temporarily impound peak flows, prior to conveying the flows back into the collection system via gravity. Typically, weir overflow chambers control the influent flow to detention tanks. Total wet outflow from the development is limited such that the peak flow for the 2-year, 24-hour storm is not exceeded. BMP catch basins are required when considering these systems.

Oversized Pipes

Similar to detention tanks, oversized pipes are also used to reduce peak flow rates by providing temporary subsurface storage of stormwater runoff. An oversized pipe system is a large pipe that has a small outlet at its invert (Figure 3-3). When inflow rates are larger than the outflow rates in this pipe series, runoff is detained, generally on the order of a few hours. Oversized pipes are a retrofit alternative for existing storm drainage pipes in the upper portions of the drainage system. A careful analysis of the storm drainage system must be conducted to prevent water backup and flooding. Other variations of this concept include a manifold arrangement of storm drainage pipes and underground vaults.

Figure 3-3: Oversized Pipe



Oversized pipes are an effective way to reduce peak flows from small (less than 5 acres) sites and can be used in sites with insufficient space to construct larger detention structures. They can also be used in retrofit projects. They have a higher cost than surface retention facilities. Design considerations include the following:

- Pipes should be located in areas where they can be accessed for maintenance. They should not be constructed under structures that cannot be excavated.
- Inflow and outflow rates should be defined in a drainage plan or storm sewer analysis. Generally, inlets are sized to convey frequent runoff events from paved surfaces.
- Minimum diameters should be 72 inches, because smaller sizes are difficult to clean.
- The slope of the oversized pipe should be approximately 0.2%. A slight slope must be maintained to completely drain the pipe, but steep slopes reduce the amount of storage available.
- Emergency surface overflow paths should be located and sized to convey the 100-year runoff in the event that the oversize pipe inlet or outlet becomes plugged or inoperable.

3.3 STORMWATER QUALITY CONTROL

It is envisioned that the stormwater quality standards for the Concord-Alewife area will substantially reduce solids and parking lot-related oil and grease being discharged in harmful concentrations to the municipal drainage system. Heavy metals associated with particulate solid matter will also be reduced with rigorous BMP solids control programs. It is expected that projects will broadly adhere to the site planning and BMP requirements outlined in Chapter 2, Volume 2 of DEP's Stormwater Management Policy. The design criteria presented herein are based on a number of assumptions. These include:

- The runoff generated in excess of the existing 2-year storm peak runoff rate, and less than the 25-year storm runoff hydrograph must be retained onsite
- In addition, in those areas within the 100-year flood plain, the runoff volume required to meet the compensatory storage requirements that offset the displaced volume caused by development must be retained onsite.
- The runoff volume to be treated is calculated as 0.5 inches of runoff times the total impervious area of the post-development project site.

- Site conditions are to be evaluated for each development so that only techniques suited for specific conditions are employed.

Adequate stormwater management performance depends on continuous and proper maintenance, which will be a specified prerequisite for any and all stormwater management controls.

3.4 STANDARDS

The following are the standards for controlling the quantity and quality of stormwater discharges for development and redevelopment projects in the Concord-Alewife area into the City of Cambridge municipal drainage system. When one or more of the Standards cannot be met, an applicant may demonstrate that an equivalent level of environmental protection will be provided. The standards are:

1. No new development/redevelopment (project) can discharge untreated stormwater directly to the City of Cambridge municipal drainage system. Appropriate stormwater management controls must be incorporated.
2. The post- project peak discharge runoff from a site must not exceed the pre-project peak discharge for any rainfall event. This must be verified for the 2-year, 10-year and 25-year and 100 year storm events.
3. The post- project discharge hydrograph for the 25-year 24-hour rainfall event must be less than or equal to the 2-year 24-hour rainfall event pre-project discharge hydrograph to the municipal drainage system as indicated in Section 3-2
4. Loss of annual recharge to groundwater will be minimized through the use of infiltration measures to the maximum extent possible. The annual recharge from the post –development should not be less that the annual recharge from the pre-development or existing site conditions.
5. Development/redevelopment is required to incorporate a 25% permeable area within each development area.
6. There shall be no negative impact from drainage on abutting properties.
7. For new development, stormwater management systems must be designed to remove 80% of the average annual load (post-development conditions) of Total Suspended Solids (TSS)[2]. It is presumed that this standard is met when:
    - (a) Suitable nonstructural practices for source control and pollution prevention are implemented;

[2] Total suspended solids was selected as the target pollutant constituent for a removal standard because of its widespread contribution to water quality and aquatic habitat degradation, because many other pollutant constituents including heavy metals, bacteria and organic chemicals sorb to sediment particles, and because available data sets for BMP removal efficiency reveal that TSS has been the most frequently and consistently sampled constituent.

(b) Stormwater best management practices (BMPs) are sized to capture the prescribed runoff volume; and

(c) Stormwater management BMPs are maintained as designed.

8. Redevelopment of previously developed sites must meet the Stormwater Management Standards to the maximum extent practicable.
9. Erosion and sediment controls must be implemented to prevent impacts during construction or land disturbance activities.
10. All stormwater management systems must have an operations and maintenance plan to ensure that systems function as designed.

Explanation of Standards

Selected water quality standards have been included and expanded upon, and are discussed below for informational purposes. Developers are responsible for compliance with any and all MA DEP regulations as required by law.

Standard 1 Untreated Stormwater

Treated stormwater is defined to be stormwater that meets the requirements in Standards 2 through 9.

Standard 2 Post-project Peak Discharge Rates

To meet Standard 2, controls must be developed for the 2-year, 10-year, and 25-year and 100 year 24-hour storm events. Measurement of peak discharge rates must be calculated using the point of discharge or downgradient property boundary. The topography of the site may require evaluation at more than one location if flow leaves the property in more than one direction. An applicant may demonstrate that a feature beyond the property boundary (e.g. culvert) is more appropriate as a design point.

Standard 3 Post project Discharge Hydrograph

The total volume of runoff generated between the pre-project 2-year 24-hour storm discharge and the post project 25-year 24-hour storm discharge shall be retained or recharged on site. This requirement ensures that during an event up to and equal to the 25 year 24 hour SCS event the municipal drainage system will not receive discharge in excess of the pre development 2 year 24 hour discharge. Figure 3-1 illustrates this requirement with hypothetical stormwater runoff plots. The shaded area (the area between the peak 2-year runoff and the 25-year runoff) represents the quantity of stormwater storage, or groundwater recharge required for achieving compliance with the City’s stormwater quantity control rule.

Standard 4 Groundwater Recharge

The project must maintain the same level of groundwater recharge from the site. This volume must be adequately treated before recharge.

In instances where roof runoff from conventional roofs is infiltrated, the infiltrated volume may be subtracted from the total runoff volume, and this may be subtracted from that portion of the development required as being permeable.

Standard 5 25% Pervious Area

The post-development permeable area mix must be minimally comprised of 10% NRCS Hydrologic Type B soil, 60% NRCS Hydrologic Type C soil, and the balance NRCS Hydrologic Type D soil for infiltration purposes. This soil must be a minimum of 2 feet in depth. Allowable runoff infiltration rates from the three assumed soil types are 0.25 inches of runoff, 0.10 inches of runoff and 0.0 inches of runoff, respectively. These rates are conservative and have been adopted from the DEP Stormwater Management Policy. Using the assumed soil type mix of the new post-development permeable area results in an average weighted value of 0.085 inches of runoff that can be infiltrated from roof runoff.

Standard 6 Negative impacts on Abutting Properties

There shall be no uncontrolled or negative discharge or impact onto adjacent properties

Standard 7 Removal of 80% TSS

For new development BMPs must be selected so that a total of 80% TSS removal is provided by one ore more BMPs as shown in Table 3-2 as taken and edited from DEPs Stormwater Management Policy. Use the column showing design rates for the projected removal rate, unless there is a demonstration that a higher or lower figure within the column showing the range of average TSS should be used. BMPs not listed below should be evaluated based on data on removal efficiencies provided by the applicant. The 80% TSS removal requirement applies to post-development conditions after the site is stabilized.

Table 3-2: TSS Removal Rates (adapted from Schueler, 1996, & EPA, 1993)

BMP Item	Design Rate, %
Underground Detention Tanks	70
Underground Retention Tanks	80
Extended Detention Pond	70
Wet Pond (a)	70
Constructed Wetland (b)	80
Water Quality Swale (shallow biofilter)	70
Infiltration Trench	80
Infiltration Basin	80
Dry Well	80
Sediment Trap	25
Grassed Drainage Channel	25
Deep Sump and Hooded Catch Basin	25
Street Sweeping	10

Notes:
(a) Includes wet extended detention ponds, wet ponds, multiple pond designs
(b) Includes shallow marsh, extended detention wetlands, pocket wetland, and pond/wetland designs
(c) Includes surface, underground, pocket, and perimeter designs
(d) Includes compost, peat/sand, and bio/filtration designs

Overall TSS removal for a process train of controls is computed in a multiplicative fashion. For example, TSS removal for street sweeping and catch basins is 0.10 + (1-0.10)*0.25 = 0.325 = 32.5%.

The basis of the TSS removal rates in Table 3-2 is as follows:

- Street sweeping – See schedule outline in Section 3.5
- Deep sump and hooded catch basin – See Section 3.5
- Grassed drainage channel – The temporary captured water quality volume is to be released in no less than 30 minutes. Design storm velocity should not exceed 1.5 ft/sec.
- Sediment trap - See requirements established in Section 3.5
- Detention/Retention storage facilities– The first half-inch of captured runoff shall be retained within the storage volume for no less than 12 hours. For detention facilities, several throttled outlet devices may be necessary to ensure proper treatment time as well as limit peak flows to that of the 2-year storm event rate. Storage captured within retention facilities shall be retained for an appropriate settling time but discharged to the receiving storm drain system in no less than 72 hours for mosquito control. For any pond system, an allowance of 10% of the water quality volume shall be established for forebay treatment. Adequate openings shall be provided for underground facilities to permit access for maintenance and cleaning of accumulated sediments.

Standard 8 Redevelopment

"Redevelopment" projects are defined as follows:

(1) Maintenance and improvement of existing roadways, including widening less than a single lane, adding shoulders, correcting substandard intersections and drainage, repaving; and

(2) Development, rehabilitation, expansion, and phased projects on previously developed sites, provided the redevelopment results in no net increase in impervious area.

Components of redevelopment projects that include development of previously undeveloped sites do not fall under Standard 8.

Standard 9 Erosion and Sediment Controls

Examples of BMPs for erosion and sediment control are staked hay bales, filter fences, hydro seeding, and phased development.

Standard 10 Operation and Maintenance Plans

An operation and maintenance plan (O&M Plan) should, at a minimum, identify:

(1) Stormwater management system(s) owner(s);

(2) The party or parties responsible for operation and maintenance;

(3) A schedule for inspection and maintenance; and

(4) The routine and non-routine maintenance tasks to be undertaken.

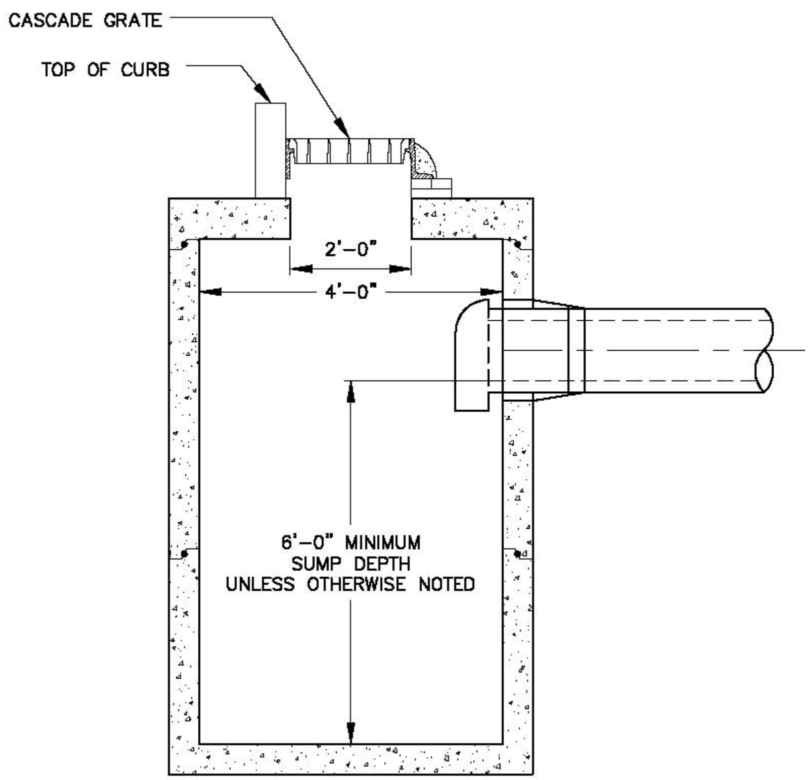
The owner of the BMP is generally considered to be the landowner of the property on which the BMP is located, unless other legally binding agreements are established with another entity. Routine maintenance during construction and post development phases of the project as defined in the project's Operations and Maintenance Plan is essential to ensure the long-term viability of the BMP.

3.5 TYPICAL CITY OF CAMBRIDGE BEST MANAGEMENT PRACTICE (BMP) CONTROLS

BMP Catch Basins

The City of Cambridge standard BMP catch basin (Figure 3-4) includes a 6-foot sump, floatables and oil & grease hood, and a 12-inch leader pipe connecting to a manhole on the local storm drain. All existing catch basins not satisfying the City of Cambridge BMP standards shall be replaced. Catch basin density shall not be less than 1 basin per 0.75-acre catchment.

Figure 3-4: Example BMP Catch Basin



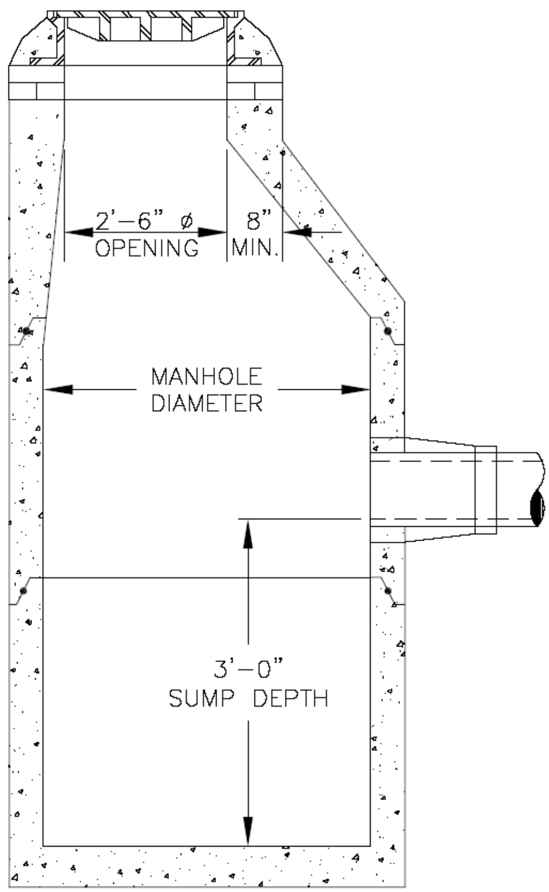
Catch basins shall be monitored for grit accumulations on a semi-annual basis and cleaned on an annual basis or when the sump accumulations reach a depth of 50%, whichever is sooner. Bacterial mosquito prevention tablets shall be installed in all catch basins during the month of July.

BMP catch basins will be required for all paved areas upstream of any storage facility intended to meet stormwater quality requirements as noted in Section 3.2.2. No fewer than two BMP type catch basins per half acre will be required for paved areas upstream of any storage facility.

Grit Sumps

At the terminus of any connection from the private development to the existing municipal collection system, a manhole with a deep grit sump (3-foot depth below invert) shall be constructed. Figure 3-5 illustrates an example grit sump structure. Grit sump manholes shall be monitored for grit accumulations on a semi-annual basis and cleaned on an annual basis or when the sump accumulations reach a depth of 50%, whichever is sooner.

Figure 3-5: Example Grit Sump Structure



Grit sump type manholes will be required for any upstream stormwater conveyance system manhole for which the peak influent flow velocity for the 3-month design storm is less than 2 ft/sec.

The necessary design rainfall distributions for the developer to perform the calculations identified throughout this document are included in Appendix 6.

Catch Basin Inserts

Catch basins receiving direct runoff from fueling areas and/or construction equipment yards shall be outfitted with inserts (Figure 3-6) having filtering capacity for particulates and the capacity to absorb oil and grease. These inserts shall be inspected on a monthly basis and the oil & grease absorption systems replaced on no less than a quarterly basis.

Figure 3-6: Example Catch Basin Insert (Flo-Gard™ High Capacity Catch Basin Filter Insert with adsorbent pouches (2002))

Outlet Conveyance Capacity

As outlined in Section 3.2 above outflow connection(s) from private developments will be limited to the pre-development 2-year, 24-hour peak flow rate for the entire area up to the 25-year 24-hour storm event. Discharges beyond the 25-year 24-hour event can be conveyed to the City's stormwater drainage system through overflow connections or other means. Flow restrictors such as orifices, vortex throttles (minimum 8-inch opening), or Hydroslides are acceptable. A comprehensive review of these technologies can be found in Chapter 9 of MOP No. 17 (ASCE-WEF, 1999

Parking Lot Mechanical Sweeping

All parking lots shall be mechanically swept on no less than a quarterly basis, starting in the spring after snowmelt.

3.6 TREATMENT REQUIREMENTS AT STORMWATER HOT SPOTS

It is envisioned that high rate physical treatment devices (i.e. stormceptor and vortechnics, etc) will be used to treat flows from hot spots to provide high levels of oil and grease and fine solids removal. Removal of fine solids down to the 10-micron size will ensure capture of solids larger than “fine silt”. Substantial removal of the heavy metals associated with particulates will result. The private developer must provide prior treatment performance monitoring data to DPW prior to acceptance.

SECTION 4 – SANITARY SEWER REQUIREMENTS IN THE CONCORD-ALEWIFE AREA.

Sanitary sewer controls are required to the maximum extent practicable on all development projects in the Concord/Alewife area. All private sewer connections in this area will connect into the City's sewer system, which discharges into the MWRA Alewife Brook Conduit. The MWRA interceptor system in this area is a Combined Sewer Overflow (CSO) system that collects flow from The City of Cambridge and The City of Somerville combined sewer systems as well as those separated systems in Belmont and Arlington. This system discharges combined sewer overflows to the Alewife Brook during significant wet weather events.

Future private development sewer discharges will not be allowed to exacerbate existing combined sewer overflow volumes or frequencies. Dry weather flow rates and volumes will be restricted and all existing extraneous private property inflow sources to the sewer system such as sump pumps, roof drains, area drains, and catch basins shall be identified and removed to the maximum extent practicable for new and redeveloped properties.

Any new development proposing to add additional wastewater to an existing sewer connection shall be required to reduce infiltration/inflow sources at an equivalent rate such that there is no net additional flow to the system. Typically, this will require a 4 to 1 ratio of inflow/infiltration (I/I) removal at the 3-month 24-hour event or the storage of sanitary flow from the property for a period of 8 hours multiplied by a safety factor of 1.5, or a combination of both strategies to ensure that there is no adverse impact on CSO events. The City Engineer reserves the right to determine the appropriate means of mitigation. The applicant shall provide sufficient calculations and /or measurements for approval by the City Engineer. Appropriate means of controlling sediment and odor control during activation shall also be provided for approval.

Appendix 1 - Common Pollutants

Sediment (Suspended Solids)

Eroded soils are a common component of urban stormwater and are a pollutant in their own right. Excessive sediment can be detrimental to aquatic life by interfering with photosynthesis, respiration, growth and reproduction. Sediment particles transport other pollutants that are attached to their surfaces including nutrients, trace metals and hydrocarbons. High turbidity due to sediment decreases the value of surface waters for recreational use and can decrease light penetration for submerged aquatic plants. Sediments also fill streams and storm drainpipes causing flooding and property damage. It can also reduce the capacity of the waterway blocking navigation. Erosion from construction sites, exposed soils, street runoff, poor maintenance of drainage systems and stream bank erosion are the primary sources of sediment in urban runoff. In addition, the energy from light reflecting off of suspended sediment can increase water temperatures (Kundell and Rasmussen, 1995).

Nutrients

Runoff from developed land often has elevated concentrations of both phosphorus and nitrogen, which can enrich streams, lakes, reservoirs and estuaries. This process is known as eutrophication. Significant sources of nitrogen and phosphorus include fertilizer, atmospheric deposition, animal waste, organic matter, sewer overflows and leaks, detergents, and the dry and wet fallout of materials in the atmosphere. Excessive nutrients also promote alga growth, which can reduce light penetration to submerged vegetation thus depleting oxygen from bottom waters. In addition nitrification of ammonia by microorganisms cans consume dissolved oxygen.

Organic Carbon/Organic Enrichment

Organic matter, washed from impervious surfaces during storms, can present a problem in slower moving downstream waters. Some sources include organic material blown onto the street surface, and attached to sediment from stream banks, or from bare soil. In addition, organic carbon is formed indirectly from algal growth within systems with high nutrient loads. As organic matter decomposes, it can deplete dissolved oxygen in waters. Declining levels of oxygen in the water can have an adverse impact on aquatic life.

Bacteria/Pathogens

Pathogens are waterborne disease causing organisms and include a broad range of bacteria and viruses that are difficult to identify. Pathogens enter surface waters from a variety of sources including sanitary and combined sewer overflows, wastewater, illicit connections to the storm drain system, pet waste and warm blooded wildlife. Indicator bacteria such as coliform (E. coli) and fecal streptococci (Enterococci) are used as indicators of potential pathogens.

Hydrocarbons

Vehicles leak oil and grease that contain a wide array of hydrocarbon compounds, some of which can be toxic to aquatic life at relatively low concentrations. Sources are automotive (primarily due to leaking engines), and some areas that produce runoff with high runoff concentrations include gas stations, commuter parking lots, convenience stores, residential parking areas, and streets (Schueler, 1994). Other sources include illegal disposal of motor oil in storm drains and illegal disposal of grease from restaurant grease traps to storm drains.

Trace Metals

Cadmium, copper, lead and zinc are routinely found in stormwater runoff. Many of the sources are from automotive vehicles. For example, one study suggests that 50% of the copper in Santa Clara, CA comes from brake pads (Woodward-Clyde, 1992). Other sources of metals include paints, road salts, and galvanized pipes. These metals can be toxic to aquatic life at certain concentrations, and can also accumulate in the bottom sediments of waterways. Other sources include industrial and commercial sites, rooftops and painted areas, improperly disposed household chemicals, landfills, hazardous waste sites and atmospheric deposition.

Pesticides

A modest number of currently used and recently banned insecticides and herbicides have been detected in urban and suburban streamflow at concentrations that approach or exceed toxicity thresholds for aquatic life. Key sources of pesticides include application to lawns and highway median and shoulder areas.

Chlorides

Salts that are applied to roads and parking lots in the winter months appear in stormwater runoff and meltwater at much higher concentrations than many freshwater organisms can tolerate. One study of four Adirondack streams found severe impacts to macroinvertebrate species attributed to chlorides (Demers and Sage, 1990).

Thermal Impacts

Runoff from impervious surfaces may increase temperature in receiving waters, adversely impacting aquatic organisms that require cool water conditions (e.g., alewife). Data suggest that increasing development can increase stream temperatures by between five and twelve degrees Fahrenheit, and that the increase is related to the level of impervious cover in the drainage area (Galli, 1991). Since warm water can hold less dissolved oxygen than cold water, this thermal pollution further reduces oxygen levels in depleted urban streams.

Trash and Debris (Objectionable sediments/solids)

Considerable quantities of trash and debris are washed through the storm drain networks and from Combined Sewer Overflows (CSO) into streams. The trash and debris accumulate in streams and lakes and detract from their natural beauty. Depending on the type of trash, this material may also lead to increased organic matter or toxic contaminants in water bodies.

Snowmelt Concentrations

The snow pack can store hydrocarbons, oil and grease, chlorides, sediment, and nutrients. In cold regions, the pollutant load during snowmelt can be significant, and chemical traits of snowmelt change over the course of the melt event. Oberts (1994) and others have reported that 90% of the hydrocarbon load from snowmelt occurs during the last 10% of the event. From a practical standpoint, the high hydrocarbon loads experienced toward the end of the season suggest that stormwater management practices should be designed to capture as much of the snowmelt event as possible.

Appendix 2 - LOW IMPACT DEVELOPMENT (LID) IN THE CONCORD - ALEWIFE AREA

Introduction

Techniques termed "Low-Impact Development Techniques", (LID), have gained popularity in recent years. These techniques have been shown to decrease surface runoff from urban areas and decrease the flooding and water quality impacts of urbanization on receiving water bodies.

LID is an innovative stormwater management approach that is modeled after nature: manage rainfall at the source using uniformly distributed decentralized micro-scale controls. LID employs natural and built features that reduce the rate of runoff, filters out pollutants and increases groundwater recharge through small, cost-effective landscape features. In urban environments these measures help to improve the water quality of receiving streams/rivers, protect threatened aquatic resources, reduce the potential for flooding by stabilizing the flow rates of nearby rivers, improve project aesthetics, and reduces the size and cost of traditional BMPs and end-of-pipe (EOP) treatments.

LID methods integrate stormwater management early in site planning activities with an emphasis on prevention and minimization rather than mitigation. LID runoff control objectives to mimic pre-development site hydrology as much as possible are accomplished by:

- Recreating natural landscape features
- Reducing impervious areas
- Increasing drainage flow paths
- Facilitating detention opportunities

LID incorporates a set of overall site design strategies, as well as, highly localized, small – scale, decentralized source control techniques. These features not only include open space, but also rooftops, streetscapes, parking lots, sidewalks, and medians. Rather than collecting runoff in piped drainage systems and controlling flow downstream in a large stormwater management facilities LID's decentralized approach disperses flows and manages runoff closer to where it originates. Because LID embraces a variety of useful techniques for controlling runoff, designs can be customized according to resource protection requirements, as well as, site constraints. New projects, redevelopment projects, and capital improvement projects can all be viewed as candidates for implementation of LID.

Studies have found that, in an urban environment, it is not practical to attempt reaching a 100 percent (%) natural hydrologic regime even with widespread use of LID techniques. Many LID techniques rely on infiltration practices, which are not effective during the winter season and have decreased efficiency during snowmelt or rain-on-snow events.

The effectiveness of LID's as "stand-alone" stormwater volume controls for a watershed is strongly dependent on the infiltration nature of local soils and average groundwater levels. Unfortunately, groundwater levels throughout the Concord-Alewife area are generally within 3 feet +/- of the ground surface. Because of the poorly draining soils and high groundwater levels in the Concord-Alewife area the successful application of a number of highly efficient LID technologies such as deep bed biofilters, infiltration basins, infiltration trenches and porous pavement will be limited. However, other LID applications can be used in the Concord-Alewife area such as shallow vegetated swales, vegetated rooftops, reduction in impervious areas, etc. Since LID applications emulate the natural hydrologic regime more effectively than current piped stormwater conveyances and EOP structures, use of both conventional technology and LIDs are envisioned for the Concord-Alewife area.

The best applications for LID techniques in the Concord-Alewife area include:

- Parking lot retrofits – particularly where runoff is diverted to porous landscaping and where parking lot sizes are reduced.
- Use of shallow vegetated swales in place of curb and gutter to increase detention storage and to reduce the size or extent of piped stormwater systems.
- Use of vegetated areas to reduce impervious areas and to disconnect paved areas.
- Utilization of "green roof" technology to dampen peak flows from roof areas.

Table A2-1: Comparative Assessment of Select LID Techniques

Item	BMP Options	Treatment Focus	Longevity	Potential Effectiveness for LID	Concord-Alewife Area Applicability
A	Revegetation of paved areas and other features	Reduction in Impervious Area	50+ years	Very High (preserves the most natural hydrology)	Moderate High
B	Reduction in parking lot size	Reduction in Impervious Area	50+ years	High (minimizes overall impervious area)	Moderate
C	Disconnection of roof area	Reduction in Impervious Area	50+ years	High (maximizes infiltration potential)	Moderate
D	Shallow vegetated swales	Stormwater Conveyance	20+ years	High (infiltration and storage potential)	Low-Moderate
E	Vegetated Roofs	Stormwater retention and flow management	20+ years	High	Moderate
F	Grass Channels and Filter Strips	Pre-treatment for stormwater infiltration or retention	Unknown, but may be limited	Low Best for pre-treatment for other LID	Moderate
*	Infiltration Trenches	Stormwater infiltration	50% failure rate within 5 years for trenches	Moderate (infiltrates but high failure rate)	Not Applicable
*	Infiltration Basins	Stormwater infiltration	60 to 100% failure within 5 years for basins	Moderate (infiltrates but high failure rate)	Not Applicable
*	Permeable Pavement	Stormwater Infiltration	Unknown, but may be limited	High (maximizes infiltration)	Not Applicable

* No further discussion is provided in this document, as this LID is not applicable for the Concord-Alewife area

Details of Appropriate LIDs for Concord-Alewife Area

REVEGETATION OF PAVED AREAS AND OTHER FEATURES

Reduction in impervious area is the first and most important step in reducing runoff peaks and volumes in urban and suburban areas. Related to this, is minimizing directly connected impervious areas, so that runoff from impervious surfaces is not discharging into the storm drain system which leads to surface water bodies. The benefits of reducing impervious area include the following:

- Creation of open space.
- Increased infiltration and decreased runoff rate and volume.
- Decreased volume of water to be treated for water quality improvements before discharge.
- Decreased peak runoff rates and volumes on downstream conveyances and detention facilities.
- Decreased extent of curb and gutter.
- Smaller stormwater drainage systems.
- Decreased pavement for street cleaning and on-going maintenance.

Several approaches for reducing or disconnecting impervious area are recommended for the Concord-Alewife area. These are summarized in Table A3-2. Each approach is described in more detail in the following sub-sections.

Table A2-2: Applications for Reducing Impervious Areas

Approach	Ideal Application
ITEM A: Reduce Parking Lot Size	Parking lots of any size Retrofitted parking
ITEM B: Disconnect Impervious Areas	Parking lots, roadways, walkways in residential and commercial developments
ITEM C: Create New Vegetation	New and redevelopment of residential and commercial projects
ITEM D: Vegetated Rooftops	New buildings

ITEM A: REDUCE PARKING LOT SIZE

Developers can reduce paved parking by looking for opportunities to share parking with other facilities that may have different peak parking needs. Employers can incorporate facilities and programs to encourage alternative means of transportation to reduce the need for parking. Where flexibility allows, specific parking lot design considerations include:

- Consider one-way traffic flow, rather than two-way flow through parking lots.
- Reduce stall width to minimum allowed by zoning.
- Shorten stall lengths to minimum allowed by zoning allowing vehicles to overhang pervious areas.
- Size more of the required stalls, at least 30 percent, for compact cars.

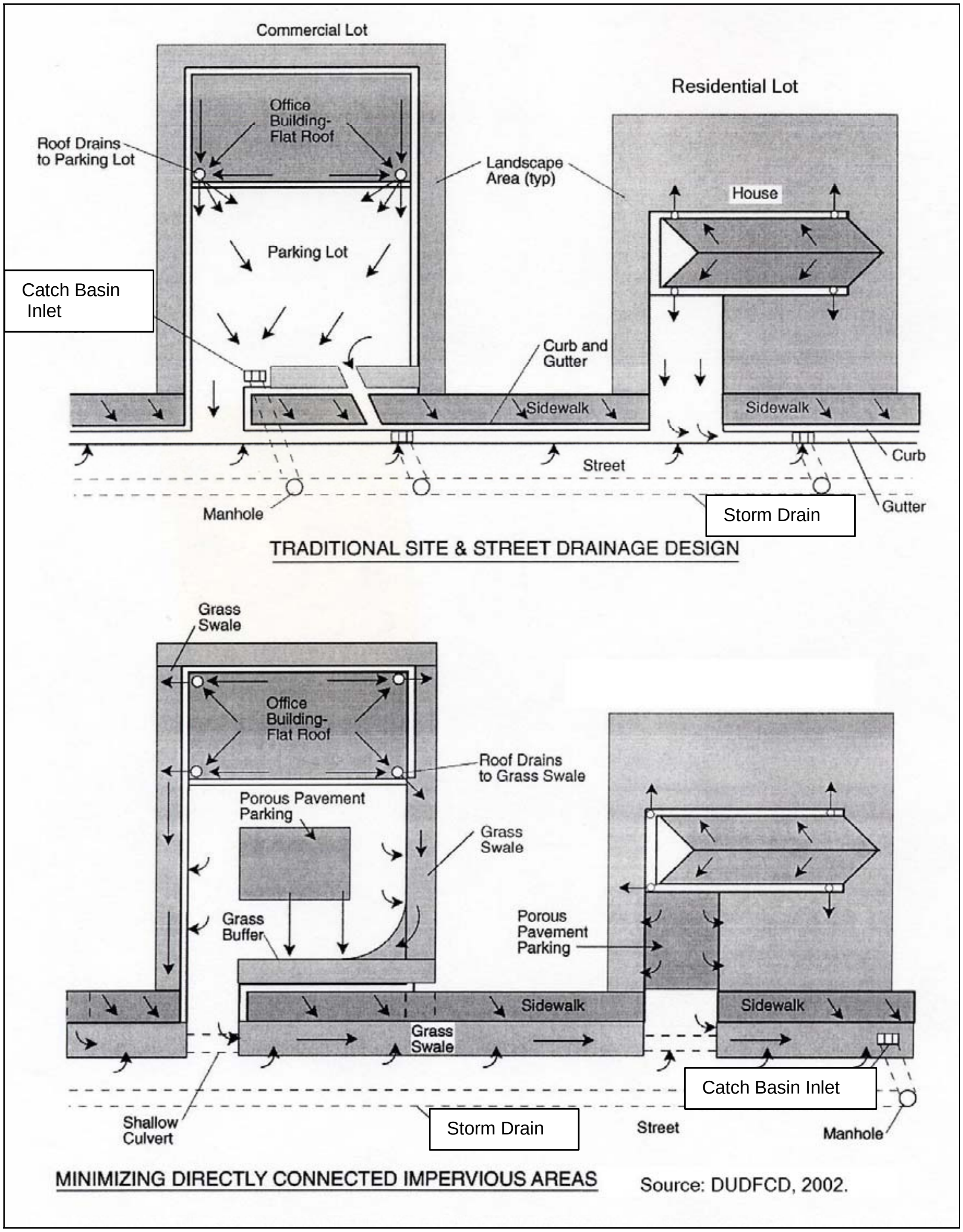
- Reduce the number of stalls by careful consideration of required parking lot size.
- Use 90-degree stall angle; it has the least pavement per vehicle as compared to 30-degree, 45-degree, or 60-degree stall angles.
- Reduce paved areas that do not contribute to parking lot functions.

ITEM B: DISCONNECT IMPERVIOUS AREAS

Areas of pavement can be disconnected to reduce the volume of water discharging to a single point. Areas of pavement that can be interrupted include parking lots, traffic lanes (by medians), and paved walkways.

An example of an integrated design for reducing connected impervious areas is shown on Figure A3-1. This figure shows use of grassed swales and grass buffers. Grass buffers, or more generally vegetative buffers, are discussed below.

Figure A2-1: Examples of Minimizing Directly Connected Impervious Areas – Residential and Commercial

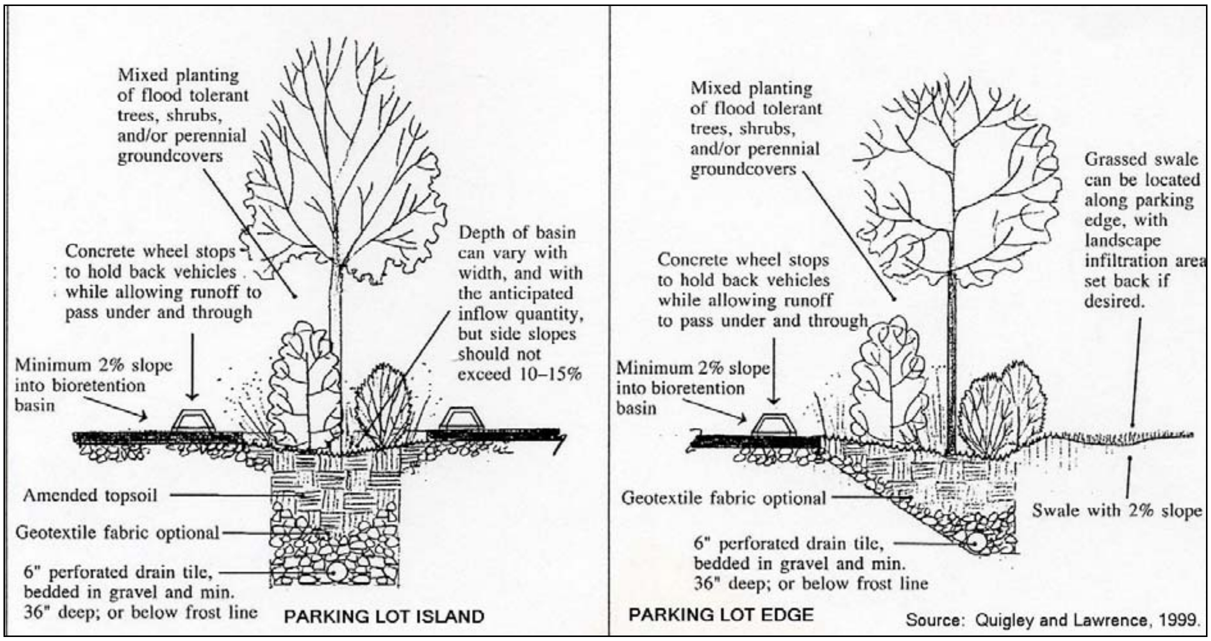


Vegetative buffers and porous landscaping are ideal for providing breaks in paving. Other considerations for vegetative buffers and grass channels and filter strips are described below.

B.1: VEGETATIVE BUFFERS

Figure A3-2 shows an example of a parking lot island planting incorporating vegetative buffer concepts. Recessed vegetative buffers can be used in place of the typical landscaped islands that are curbed and set higher than the paved parking lot grade. Pavement is graded so that the surface flow is towards rather than away from the islands. A bypass should be included in the design that can handle runoff in excess of the design flow and direct it towards an overflow structure.

Figure A2-2: Examples of Vegetative Buffers in Parking Lot Setting



The success of vegetative buffers is extremely dependent on both a designer developing proper installation specifications and a contractor properly implementing them. Poor construction techniques can cause the best-designed facility to fail prematurely. Construction technique and inspection are critical to ensure proper landscaping, soil mixtures, and grading around the facility, as well as the use of approved materials. Keep in mind that the plant and soil components are crucial elements of the facility and are the key to the vegetative buffer's basic function. Considerations for vegetative buffers and porous landscaping are provided below (source: LID Center, 2003).

Drainage Area:

- Limit drainage area to less than 2 to 3 acres; preferably less than 1 acre.

Ponding Depth:

- Maximum 3 to 4 inches recommended for soils with low infiltration rates, or high hydraulic loadings (combine with a smaller drainage area).
- Ponding depth may be increased if using sandy soils and underdrains to increase filtration.
- If space is limited, depth may be increased up to 1-foot, as long as the drainage area is ¼-acre or less.
- Any pooled water should be drawn down within 4 to 6 hours after a storm event.

Plants:

- Use species able to tolerate expected pollutant loadings, highly variable soil moisture conditions, and ponding water fluctuations.
- Use a minimum of three species of shrubs to ensure diversity.
- Avoid species that require regular maintenance.
- Do not plant shrubs within 15 feet of perforated pipes.
- Check water tolerances of existing plant materials prior to inundation of area.
- Do not block maintenance access to structures with trees or shrubs.
- Decrease the areas where turf is used. Use low maintenance ground cover to absorb run-off.
- Select plants that can thrive in on-site soil with no additional amendments, or a minimum of amendments.
- When planting a mix of plant species, plant individuals of same species in clumps (e.g., groups of three to five) rather than alternating species on a plant-by-plant basis.
- Maintain and frame desirable views. Be careful not to block views at entrances, exits, or difficult road curves. Screen or buffer unattractive views into the site.
- Use plants to direct pedestrians and to prohibit pedestrian access to pools or slopes that might be unsafe.
- Carefully consider the long-term vegetation management strategy of the planting, keeping in mind the maintenance requirements for future owners. Provide a planting surface that can withstand the compaction of vehicles using maintenance access roads.
- Select salt tolerant plant material in areas that might receive wintertime salt applications (roads and parking lots).

Soil:

- Homogeneous mix of 50% construction sand, 20% to 30% topsoil with less than 5% maximum clay content, and 20% to 30% organic leaf compost.
- PH between 5.5 and 6.5.
- Minimum depth of 2 to 2.5 feet, without large tree plantings.
- If shallow rooted plants are used, soil depth may be reduced to 1.5 feet.
- Soil infiltration rate should exceed 1.5 inches/hour.
- Have soil tested to determine if there is a need for amendments. It is often necessary to test the soil in order to determine the following:
  - PH – whether acid, neutral, or alkali
  - Major soil nutrients – nitrogen, phosphorus, potassium
  - minerals – such as chelated iron, lime
- Areas that have recently been involved in construction can become compacted. If compaction has occurred, soils should be loosened to a minimum depth of 2 inches, preferably to a 4-inch depth. Hard soils might require disking to a deeper depth. The soil should be loosened regardless of the ground cover. This will improve seed contact with the soil, providing greater germination rates, allowing the roots to penetrate into the soil. If the area is to be sodded; disking will allow the roots to penetrate into the soil. Providing good growing conditions can prevent weak or patchy plantings.
- Whenever possible, topsoil should be spread to a depth of 4 inches (2 inch minimum) over the entire area to be planted. This provides organic matter and important nutrients for the plant material and allows the stabilizing materials to become established faster, while the roots are able to penetrate deeper and stabilize the soil, making it less likely that the plants will wash out during a heavy storm.
- If topsoil has been stockpiled in deep mounds for a long period, test the soil for pH and microbial activity. If the microbial activity has been destroyed, it will be necessary to inoculate the soil after application.

Mulch:

- Maximum 2 to 3 inches deep.
- Should be fresh, not aged.
- Apply uniformly; do not pile around the base of trees.

Groundwater:

- Water table depth at least 2 feet below the lowest part of the facility (or an underdrain may be used).

Pollutant Concerns:

- Primary pollutant concerns in ultra-urban areas are metals from traffic, buildings, and rooftops, oils from automobiles, and sediment from street and lot sanding.

Underdrain:

- Recommended where the in-situ soil infiltration rate is less than 1-inch per hour (if an underdrain is not being used, soils investigation/geotechnical reports are required).
- Build with an accessible cleanout well.
- Do not locate within the groundwater zone of saturation.
- Must have a hydraulic capacity greater than the planting soil infiltration rate.

Inflow and Overflow:

- Design for overflow is necessary, since drainage areas in commercial and institutional settings are highly impervious
- Special design considerations are necessary to direct the impervious drainage area's runoff to the vegetative buffer.
- Water may be diverted into the vegetative buffer through the use of an inlet deflector block, which has ridges to channel the runoff into the landscaped area.
- In a paved area with no curb, pre-cast car stops can be installed along the pavement perimeter to protect the vegetative buffer.
- Parking lot runoff may be captured through the use of vegetated soil/gravel trenches integrated into the parking area at strategic locations.
- When inflow exceeds 3 cubic feet per second, the designer should evaluate the potential for erosion.

Location:

- Avoid locating the vegetative buffer near building areas.
- Locate away from travelled areas, such as public pathways, to avoid compaction.
- For parking lot islands, a buffer (2 feet recommended) may be used to minimize the possibility of drainage seeping under the pavement section and creating frost heave during winter (alternatively, a geotextile filter fabric curtain wall along the perimeter of the vegetated island may be used).

B.2: GRASS CHANNELS AND FILTER STRIPS

Grass channels and unlined ditches are designed to handle channelized flow at design storm flow rates. Grass channels are vegetated and constructed to promote a dense turf by placing proper soil mixtures (sandy loam) and seed mixtures. Swale size should be matched to the design flow rate, which in turn is influenced by the size and land use of the tributary area. In general, swales are appropriate for smaller parking lots or subdivisions of larger parking lots.

Grass channels are modifications of traditional ditches that provide some water quality treatment. Grass channels have a broad, mildly sloped channel, and a thick vegetative cover. The design objective is to maintain a minimum residence time of ten minutes.

Filter strips are designed to promote sheet flow at the design storm flow rate. Because flow rates must be low to be sustained as sheet flow, filter strips are only recommended for very small parking lots or portions of larger lots. The discharge end of the filter strip is typically a stream buffer or other open space. Filter strips are vegetated and constructed to promote a dense turf by placing proper soil mixtures (sandy loam) and seed mixtures. Filter strips provide a buffer; usually grass, between development and streams or stormwater conveyance systems. They provide some pollutant removal and infiltration and reduce the velocity of overland flow before it reaches the streams. These systems are often part of a riparian buffer system, including a forested buffer at the stream edge (Schueler, 1995). The use of filter strips is limited by the amount of space they need. They can also be overwhelmed by too much, or concentrated, runoff, which can cause gullies and allow bypassing of the filtering media.

Grass Channel Design Considerations:

- Side slopes flatter than 3:1.
- Longitudinal slope between 1% and 4%.
- Non-erosive for the two-year storm.
- Water quality volume retained or infiltrated in 24 hours.
- Small fore bay at the inlet as pre-treatment.
- Check dams can be used to maintain the longitudinal slope in a swale.
- Maintain a dense vegetative cover.
- Design slope so that velocities are non-erosive for the 2-year storm.
- Water quality volume retained or infiltrated in 24 hours.
- Provide an underdrain and prepared soil bed if infiltration rate of underlying soil is less than 1-inch per hour to promote filtration.
- Small fore bay or filter strip at the inlet as pre-treatment.

Vegetated Filter Strip Considerations:

- Greater than 25 feet long.
- Slope between 2% and 6%.
- Maintain a dense vegetative cover.

- Maximum contributing length 75 feet for impervious drainage; 150 feet for pervious drainage.
- Sized to temporarily pond the 2-year 6-hour storm.

ITEM C – CREATE NEW VEGETATION

Vegetation helps prevent erosion, filters runoff, and allows stormwater to filter into the ground, which ultimately results in lower stormwater management costs. Bioretention can be used, which includes porous backfill under the vegetated surface, and an underdrain that encourages infiltration and water quality filtering while avoiding extended ponding. Vegetated buffers trap and filter sediments, nutrients, and chemicals from surface runoff and shallow groundwater. For existing permeable areas, soil amendments can increase the soil's infiltration capacity and help reduce runoff from the site. They have the added benefit of changing physical, chemical and biological characteristics so that the soils become more effective at maintaining water quality.

ITEM D – VEGETATED ROOFTOPS

Also known as "green roofs" or "extensive roof gardens," these roof treatments use vegetation and lightweight soil mixtures to absorb, filter, and detain rainfall. Green roofs are widely used in Europe, and some North American cities, including Chicago, New York, Toronto, Vancouver, and Portland, have undertaken green roof initiatives.

Research and demonstrations projects indicate that, compared to traditional roofing materials, such as tar or shingles, green roof systems detain, filter, and slowly release storm water, reducing the peak flows and overall volume of runoff. If widely implemented, green rooftops have the potential to reduce storm water runoff and nonpoint source pollution problems in urban environments.

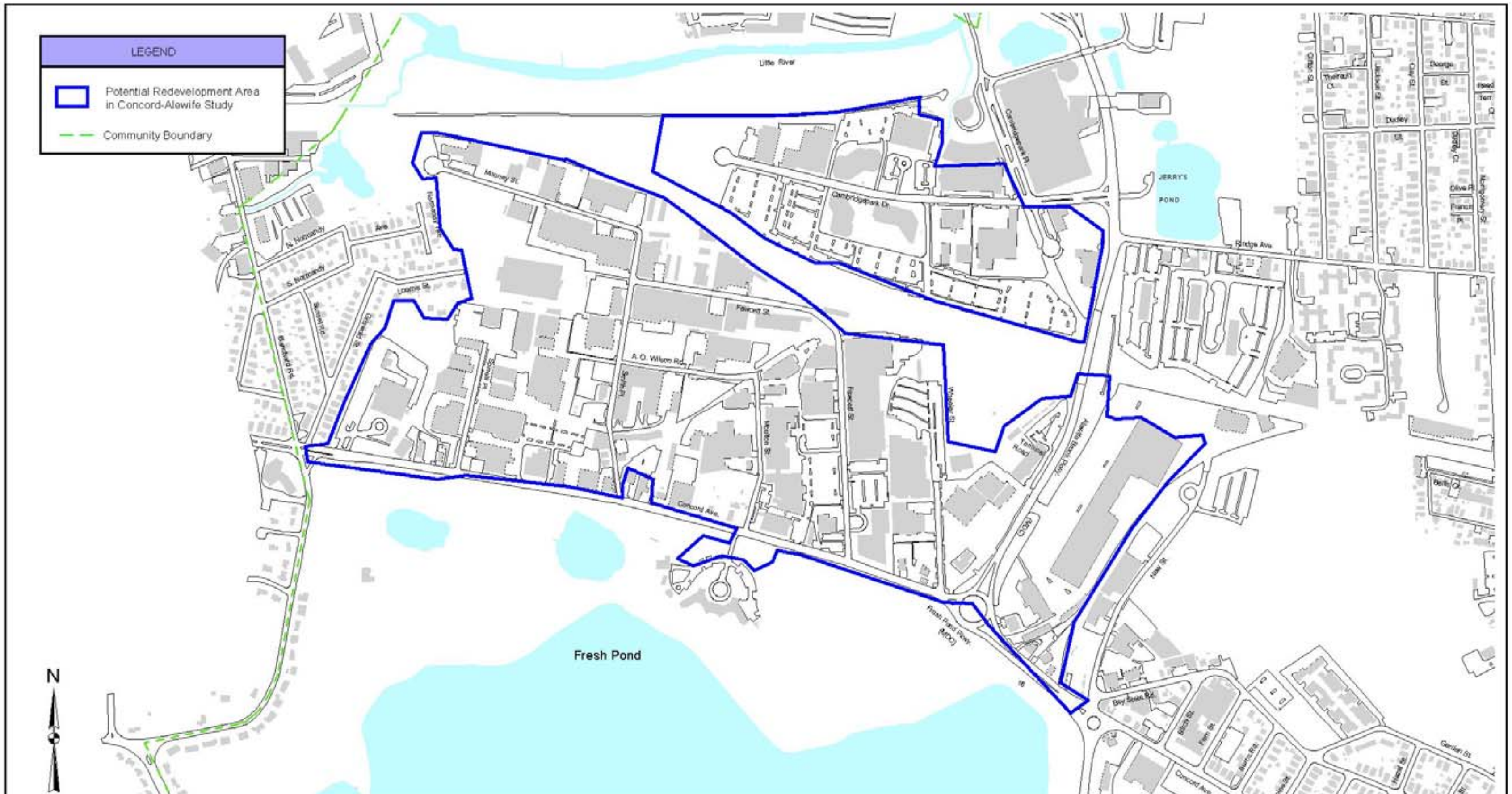
Appendix 3 – TECHNICAL BASIS FOR STORMWATER QUANTITY GUIDELINES FOR THE CONCORD-ALEWIFE AREA

Introduction & Methodology

Potential impacts of re-development in the Concord-Alewife area are being evaluated using hydrologic and hydraulic modeling. The purpose of this memo is to outline a draft methodology for determining the required private onsite runoff retention for the 2-year to 25-year storm. The study area for this analysis is shown in Figure 1.

Modeling scenarios were developed based on applicable methods of reducing runoff in the Concord-Alewife Area. Overall stormwater management goals of the Concord-Alewife Rezoning Petition include increasing the minimum open space requirement for all uses to 15%, and also creating a permeable requirement of 25%. Types of runoff reduction technologies included in the analysis were green roofs (two variations), converting impermeable surfaces to permeable surfaces, and onsite storage. Since groundwater levels in the study area are relatively high, the use of many typical infiltration technologies, such as biofilters or porous pavement, is not applicable. However, green roof technologies have been successfully implemented in Cambridge (Sidney Street in Cambridgeport) and are an environmentally attractive and effective method of runoff reduction.

The analysis is based on the private development rule that the total volume of runoff generated between the 2-year storm peak discharge (present conditions) and the 25-year storm (CAM 004 model future conditions) shall be retained. The 2-year, 24-hr NRCS design storm event was simulated under existing conditions (2003) with a receiving water boundary condition varying in elevation from 0.5 to 3.0 feet NGVD. The 25-year, 24-hr NRCS design storm event was then simulated under future proposed conditions with a receiving water boundary condition varying in elevation from 0.5 to 6.4 feet NGVD. To obtain the runoff hydrographs for the Concord-Alewife study area, flow hydrographs in the modeled pipe network were strategically chosen and algebraically summed. Flows were summed as follows:



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MWH

Concord-Alewife Study

Figure 1: Potential Redevelopment Area in Concord-Alewife Study

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Existing Conditions:

Concord-Alewife Runoff = Flows from Wheeler Street outlet + Mooney Street Flows (towards Spinelli drain) + Wheeler Street Flows + Terminal Rd Flows – Flows from Drain Vault 5 – Sherman Street Flows

Future Conditions:

Concord-Alewife Runoff = Flows to Alewife Wetland Detention System + Flows from Wheeler Street outlet + Mooney Street Flows (towards Spinelli drain) + Wheeler Street Flows + Terminal Rd Flows – Flows from Drain Vault 5 – Sherman Street Flows

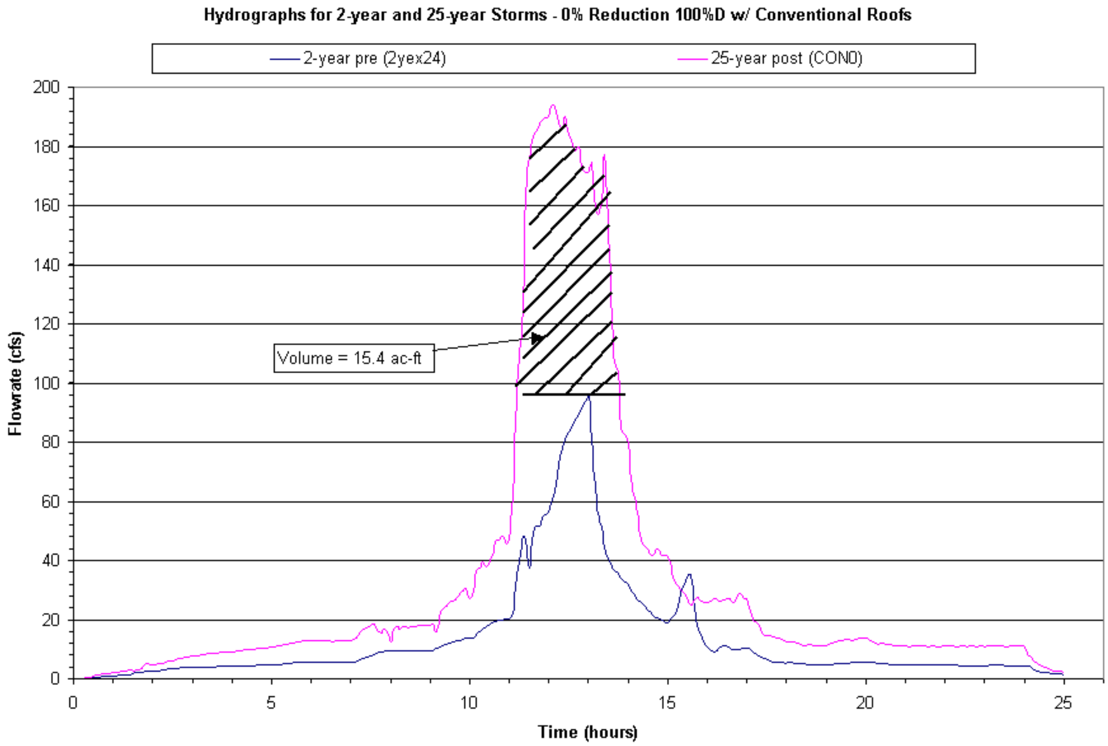
Runoff from roof surfaces, pervious ("green") surfaces, and pavement surfaces on private property was modeled. Public street areas were not included in this analysis. The total modeled study area contains 169 acres of private property with 149 acres of impervious area (roof + pavement).

Table 1 summarizes the roof configurations evaluated. For each type of roof configuration, simulations were performed for the 25-year storm under future conditions in the Concord-Alewife study area, with varying levels of impervious area reductions (e.g. private property pavement area reductions). The composite 25-year runoff hydrograph for the study area was plotted for each scenario and compared with the 2-year present conditions hydrograph. This is illustrated in Figure 2. The runoff volume difference between the 2-year peak discharge and the 25-year hydrograph was computed. This difference was assumed to be the quantity of storage required in order achieve the runoff retention goal. For the example in Figure 2, 15.4 ac-ft of storage is required to meet the goal for the study area.

TableA3.1: Modeled Roof Configuration Summary

Configuration	Description
Conventional Roof	
Green Roof Type 1	100% of the roof surface is "green"; Excess runoff is conveyed directly to the collection system
Green Roof Type 2	60% of the roof surface is "green" (excess runoff to roof storage compartment); 40% of the roof surface is modeled as conventional (impervious); All runoff is conveyed into a 4" deep storage chamber on the roof through a controlled outlet down to the collection system

Figure A3.2: Example 2y-25y Onsite Retention Requirement Calculation



Conventional roofs were modeled as impervious surfaces with a Manning's roughness ("N") value of 0.011. All green roofs were modeled as impervious surfaces with a Manning's roughness value of 0.35, which approximates the attenuation achieved by prairie grass type vegetation on the green roof. An initial loss of 0.3 inches was modeled based on results of pilot test data using extensive green roofs.

Green roofs with pervious surfaces were also evaluated; however, the impervious surface method was chosen to be conservative. Type 1 green roofs assumed that the entire roof area was utilized for green roof technology. Type 2 green roofs assumed that only the perimeter of the roof area (approximately 60% of the total roof surface) was assumed to be "green" and was pitched towards the center of the roof area, which acted as a detention basin (4 inches deep). Runoff from the "green" portion of the roof was assumed to be conveyed into the detention basin. In addition, the detention basin contained a throttled underflow to the collection system. The underflow for each model subcatchment was modeled as an orifice with a maximum discharge value that was adjusted such that approximately 80%-100% of the detention basin volume was utilized during the storm event. This type of green roof was successfully implemented on Sidney Street in Cambridgeport.

For the condition of no pavement reduction, all open space was assumed to have default type D soil parameters. For any scenario that includes pavement area reduction to increase permeability, the pervious surfaces were assumed to have a mix of soil types. The Horton equation parameters for this mix of soils was computed by assuming 10% B type soils, 60% C type soils and 30% D type soils. This blend of soil types was assumed typical for the Concord-Alewife area. This composite parameter set was then reduced by 10%.

Study Area Model Results

Figure 3 summarizes the results of the analysis. Three curves are drawn which represent the relationship between pavement area reduction and onsite storage required, for each type of modeled roof configuration. The amount of pavement area reduction to achieve the required runoff retention (i.e., pavement area converted to green pervious area) is plotted on the y-axis while the corresponding amount of required on-site storage volume is plotted on the x-axis. The runoff reduction goal is achieved if a point falls on the curve, or to the upper-right hand side of the curve. This plot represents the range of alternatives that could be used to satisfy the runoff retention requirement.

For a conventional roof configuration, if no reduction in pavement area is achieved upon development of the Concord-Alewife area, then a minimum of 15.4 ac-ft of storage would be required. As pavement area is reduced, less onsite storage is required. As expected, lower quantities of runoff were simulated for scenarios with green roof configurations, as compared to scenarios with conventional roofs. Therefore, the utilization of green roof technology may reduce the amount of required onsite storage in a development, as compared with using conventional roofs. Using Type 2 green roofs allowed for the least quantity of required onsite storage.

The curves in Figure 3 illustrate required storage volumes for the entire study area. In order to facilitate the use of these results for a subset of the study area, the required storage volumes were normalized by total impervious area and are plotted in Figure 4. The amount of pavement area reduction to achieve the required runoff retention (i.e., pavement area converted to green pervious area) is plotted on the y-axis while the corresponding amount of required on-site storage volume per impervious acre is plotted on the x-axis.

For example, if a potential development in the Concord-Alewife area has 5.0 acres of impervious surface and conventional roofs, and no pavement area reduction is proposed, then approximately 0.5 ac-ft of onsite storage would be required (0.1 ac-ft per impervious acre x 5.0 acres). Table 2 provides a summary of the model results that were used to plot the curves in Figures 3 and 4.

Figure 3: Private Development Requirements

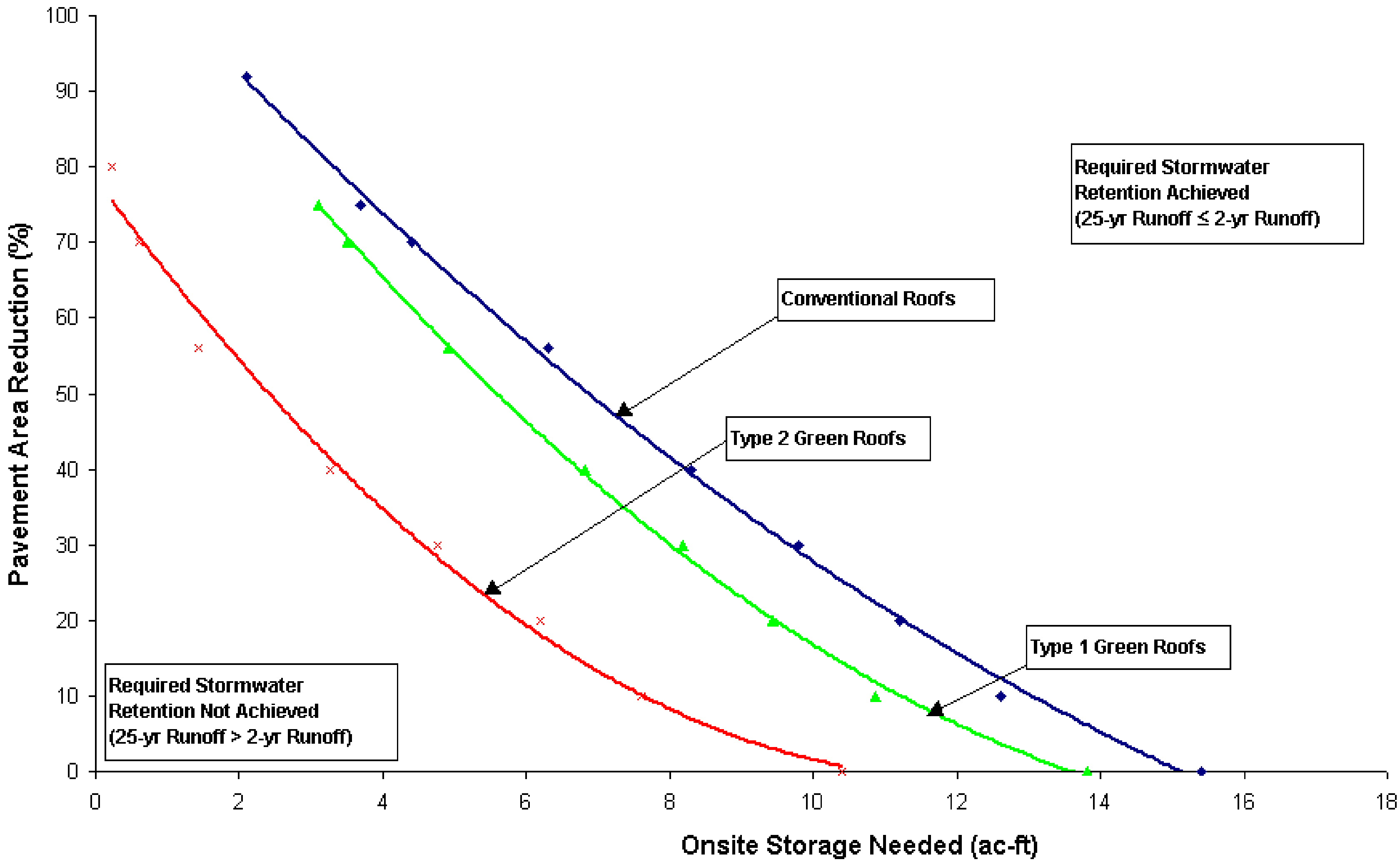


Figure 4: Private Development Requirements (Normalized)

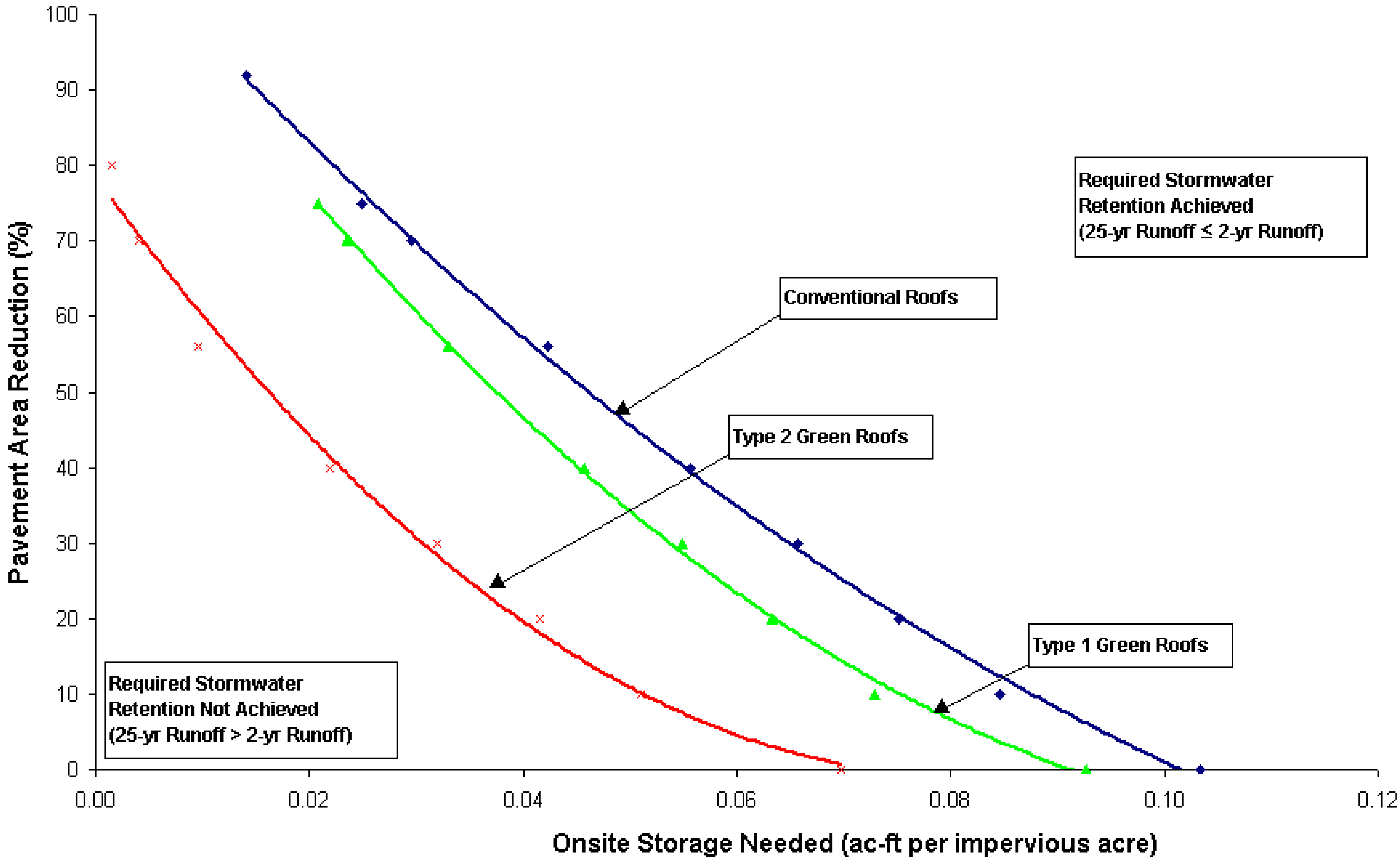


Table A3. 2: Study Area Model Results Summary

Pavement Area Reduction (%)	Volume Difference Between 2y-25y Hydrographs (ac-ft)	Volume Difference Per Impervious Acre* (acre-ft/impervious acre)
Conventional Roof Configuration		
0	15.4	0.103
10	12.6	0.085
20	11.2	0.075
30	9.8	0.066
40	8.3	0.056
56	6.3	0.042
70	4.4	0.030
75	3.7	0.025
92	2.1	0.014
Type 1 Green Roof Configuration		
0	13.8	0.093
10	10.9	0.073
20	9.4	0.063
30	8.2	0.055
40	6.8	0.046
56	4.9	0.033
70	3.5	0.024
75	3.1	0.021
Type 2 Green Roof Configuration		
0	10.4	0.070
10	7.6	0.051
20	6.2	0.042
30	4.8	0.032
40	3.3	0.022
56	1.4	0.010
70	0.6	0.004
80	0.2	0.001

*Note: Computation used 149 acres of private property impervious area

Single Subcatchment Model Results

The procedure of calculating required retention requirements using the normalized method (Figure 4) was evaluated for a single subcatchment in the model. Similar to the procedure used to create the plots in Figure 3 for the entire study area, plots were also created by modeling only the single subcatchment. The subcatchment used has a total private property area of 8.55 acres, with 7.44 acres of impervious surface (roof + pavement). The 25-year runoff hydrograph for the subcatchment was plotted for each scenario and compared with the 2-year present conditions hydrograph. The runoff volume difference between the 2-year peak discharge and the 25-year hydrograph was computed. This difference was assumed to be the quantity of storage required in order to achieve the runoff retention goal. These model results were compared with results from computing runoff retention requirements using the normalized curves in Figure 4. Figure 5 presents this comparison.

As Figure 5 shows, using the normalized curves from Figure 4 results in similar required storage volumes as explicitly modeling the individual subcatchment. The variations in results between the two computation methods appear relatively consistent for each roof configuration type. This variation may be due to the fact that the normalized method uses results from an analysis of the entire Concord-Alewife catchment area, and these results are being compared with an analysis of just one single sub-basin within this catchment area. Assuming that the single subcatchment used in this analysis is a representative sample of the entire study area, scale factors could be developed to account for the variation. For example, for the conventional roof configuration and zero pavement area reduction, the scale factor can be computed as 0.95/0.77 = 1.23. An average for all conventional roof scenarios results in a scale factor of 1.19. Using a similar methodology, average scale factors for Type 1 and Type 2 green roof configurations were computed as 1.12 and 1.00. The average scale factor for Type 2 green roof configurations amounted to slightly less than one; therefore, to be conservative a factor of 1.0 was chosen. If the normalized approach is used to compute runoff reduction requirements for a given parcel with conventional roof types, then the computed volume difference can be multiplied by 1.19 to account for the average variation between using the normalized approach and the explicit hydrologic/hydraulic modeling approach. Figure 6 illustrates a revised version of Figure 4, after scaling factors have been applied to the normalized plot. This plot is recommended to be used for the computation of runoff reduction requirements.

<u>Example Computation with Normalized Curve Approach:</u>

Assume a 2.0-acre parcel is to be developed in the Concord-Alewife area. This parcel has 1.8 acres of impervious surface (1.0 ac pavement + 0.8 ac roofs) and green roof Type 2 technology has been chosen as part of the solution to satisfy the 2y-25y runoff reduction requirement. First, the parcel must satisfy open space and pervious surface zoning requirements. According to these requirements, 25% of the parcel must have pervious surface, and 15% must be open space (half of the open space can be impervious surface and half can be pervious). Therefore, for this parcel, the 1.8 acres of impervious surface must be at least reduced to 1.5 acres, leaving 0.5 acres, or 25% (i.e. 0.5 ac/2.0 ac), pervious. It is assumed for this example that the pervious area also counts as open space, and therefore satisfies both the open space requirement *and* the pervious

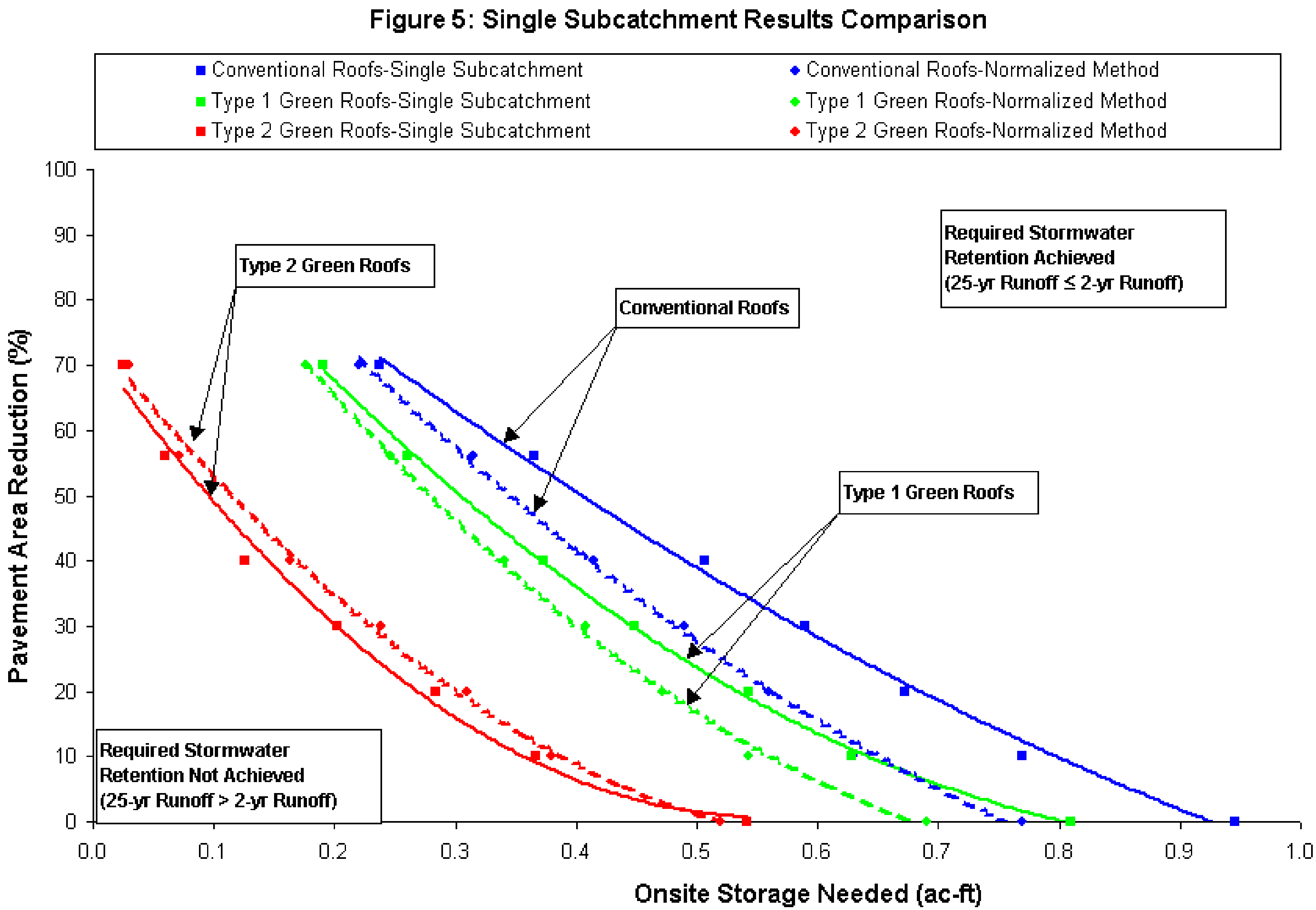


Figure 5: Single Subcatchment Results Comparison
Conventional Roofs-Single Subcatchment
Type 1 Green Roofs-Single Subcatchment
Type 2 Green Roofs-Single Subcatchment
Conventional Roofs-Normalized Method
Type 1 Green Roofs-Normalized Method
Type 2 Green Roofs-Normalized Method
Type 2 Green Roofs
Conventional Roofs
Type 1 Green Roofs
Required Stormwater Retention Achieved (25-yr Runoff ≤ 2-yr Runoff)
Required Stormwater Retention Not Achieved (25-yr Runoff > 2-yr Runoff)
Pavement Area Reduction (%)
Onsite Storage Needed (ac-ft)
0
10
20
30
40
50
60
70
80
90
100
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0

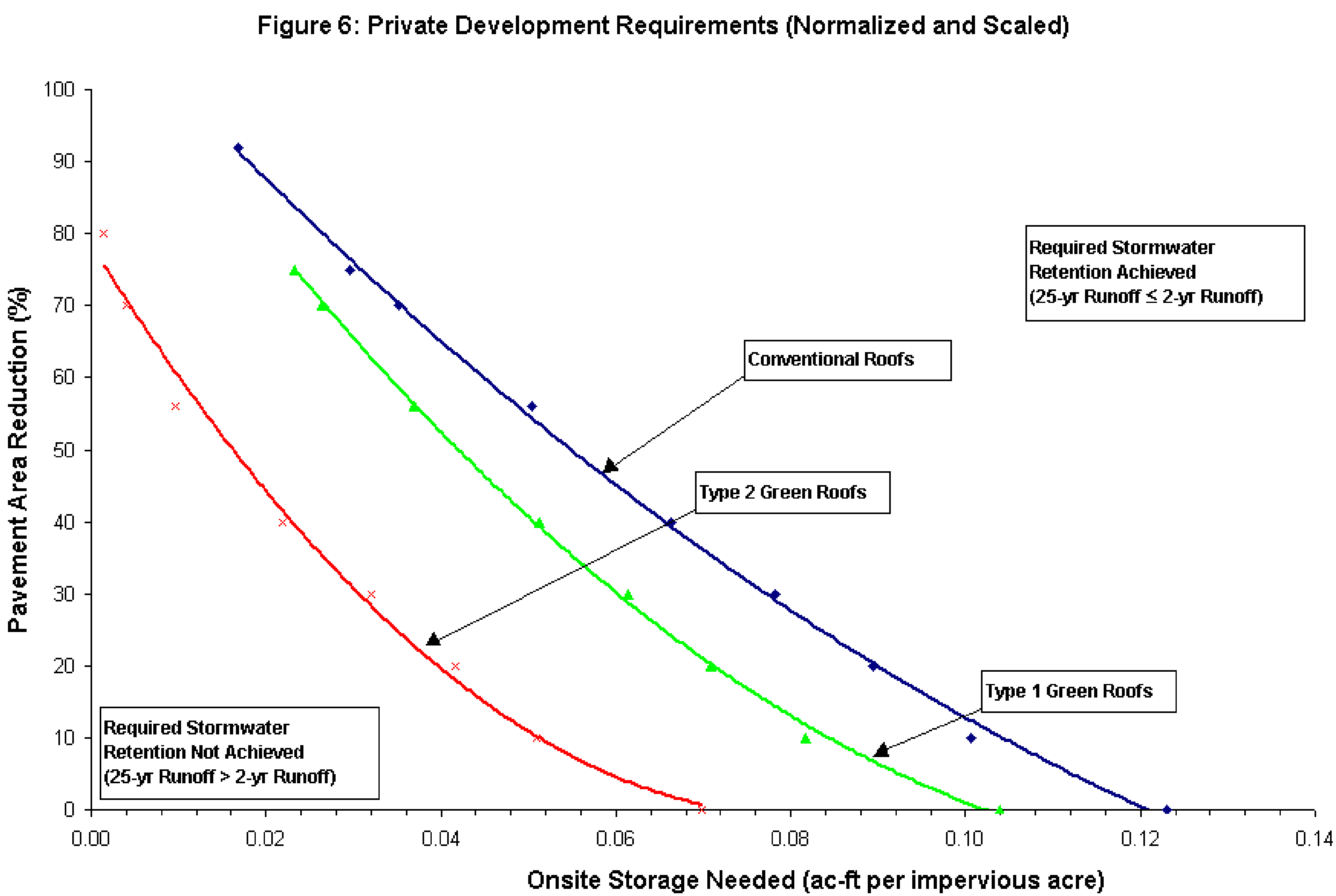


Figure 6: Private Development Requirements (Normalized and Scaled)
Required Stormwater Retention Achieved (25-yr Runoff ≤ 2-yr Runoff)
Conventional Roofs
Type 2 Green Roofs
Type 1 Green Roofs
Required Stormwater Retention Not Achieved (25-yr Runoff > 2-yr Runoff)
Pavement Area Reduction (%)
Onsite Storage Needed (ac-ft per impervious acre)
100
90
80
70
60
50
40
30
20
10
0
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14

surface requirement. It is also assumed for this example that the 0.3 acres of impervious area reduction included only pavement area.

Reducing the pavement area of the parcel 0.3 acres (e.g. from 1.0 ac to 0.7 ac) is equivalent to a 30% (1.0-0.7 / 1.0) reduction. The developer has now satisfied both open space and perviousness requirements. The red curve in Figure 4-2 corresponding to Type 2 green roofs shows that for a pavement reduction of 30%, approximately 0.031 ac-ft per impervious acre would be the required onsite storage to meet the 2y-25y runoff retention requirement.

Therefore, the actual required onsite storage for this parcel is computed by:

(Onsite storage per impervious acre from Figure 6) x (Number of pre-development impervious acres in the parcel)

= (0.031 ac-ft / imp. acre) x (1.8 acres) = 0.056 ac-ft, or approximately 18,246 gallons

Thus, for this example, the developer has satisfied open space and pervious surface zoning requirements AND satisfied the 2y-25y runoff retention requirement, by doing the following:

- Converting the roof area in the parcel to Type 2 green roofs;
- Converting 0.3 acres of pavement surface area to pervious surface; and
- Providing 18,246 gallons of onsite storage.

The onsite storage could be implemented in a variety of ways depending on the location and other environmental factors, including shallow swales, ponds and underground tanks. However, the storage volume required for this example assumes that the configuration of the storage device is such that the total volume of runoff generated between the 2-year storm PEAK discharge and the 25-year storm is retained. In order to size the storage device at a volume of 18,246 gallons while satisfying this condition, the device would have to incorporate a control feature such as an inlet weir or an outlet throttle that would allow the storage volume to be utilized at the proper time during the storm event. Figures 7 and 8 show two graphical examples of actual required storage volume that satisfies the 2y-25y runoff retention requirement. Figure 7 shows the hypothetical quantity of storage required for a device that is not configured to capture runoff during a specific time during the storm event. Figure 8 shows the hypothetical storage volume that is required for a device that successfully incorporates controls that allow for proper timing of storage utilization.

It should be noted that the developer could have also satisfied the 2y-25y runoff retention requirement by converting more pavement area to pervious surface, requiring less onsite storage. For example, if the developer had chosen to convert 50% of the pavement area to pervious area (instead of the 30%), then the onsite storage requirement would have been only 0.017 ac-ft, or 5,539 gallons. Several combinations of roof type, pavement area reductions, and onsite storage can achieve the same runoff retention goal.

Figure A3.7: Storage Requirement For a Device With No Controls Incorporated

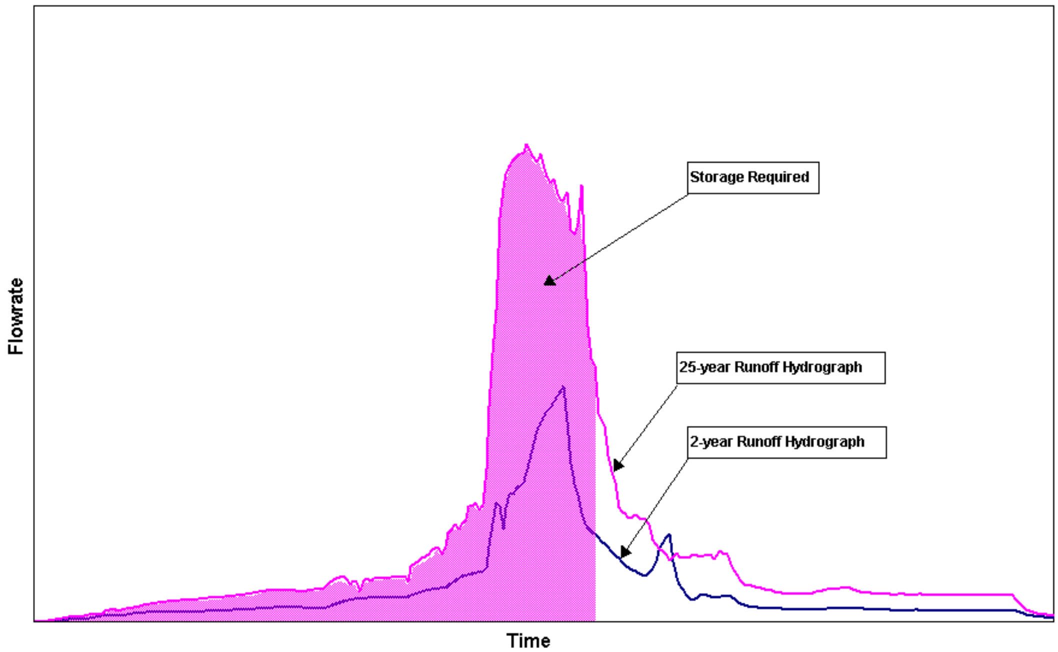
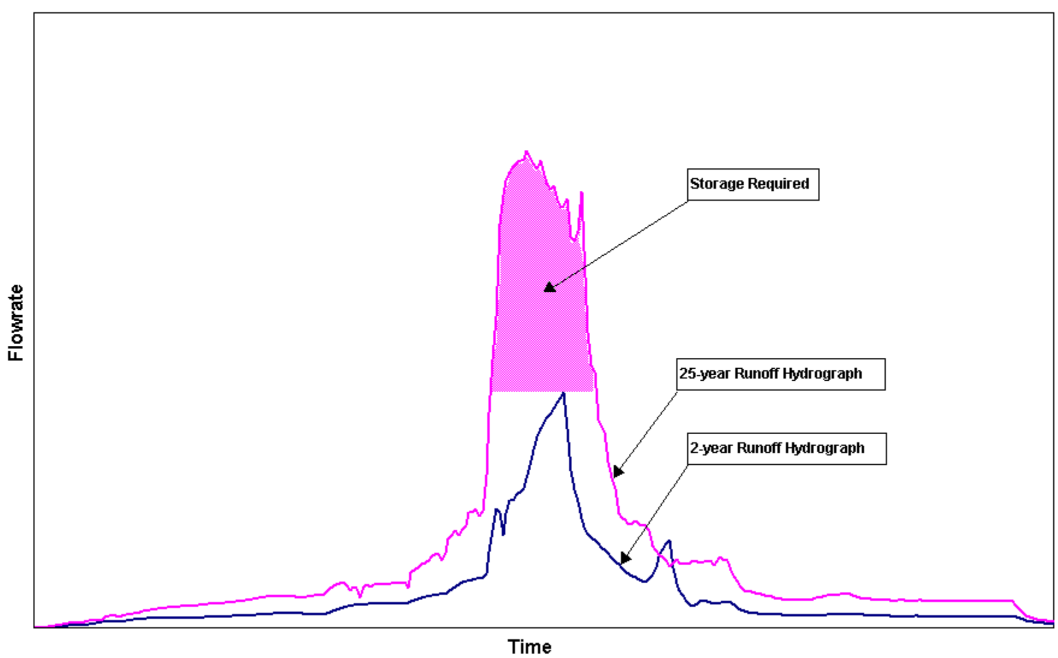


Figure A3.8: Storage Requirement For a Device With Adequate Controls Incorporated



Conclusions

It is recommended to use the normalized method for computing private development runoff retention requirements in the Concord-Alewife area. Using a hydrologic/hydraulic model to compute runoff retention requirements each time a development is proposed would be cost-prohibitive. The simplified method of using normalized values allows for a timely and reasonable computation of a potential developer's alternatives for achieving 2y-25y runoff retention goals.

Appendix 4 – Example Calculation for Water Quality Control Runoff Volume

<u>Example:</u>

Pre-development condition - 10 acres of pavement and roof

Post-development condition - 2.5 acres of permeable area, 3.0 acres roof (50% green –type 2 technology), 4.5 acres of pavement

Initial *roof runoff* volume requiring treatment = 3.0 ac * 0.5 inch = 1.5 ac-inch.

> →Initial abstraction of water due to vegetation on green roof = 0.3 inches * 1.5 ac = 0.45 ac-inch
>
> →Infiltration into post-development permeable area: 2.5 ac * 0.085 inches = 0.21 ac-inch

Adjusted *roof runoff* volume requiring water quality treatment = 1.5 ac-inch – 0.45 ac-inch - 0.21 ac-inch = 0.84 ac-inch

Pavement runoff volume requiring treatment = 0.5 ac-inch * 4.5 ac = 2.25 ac-inch

Total net runoff volume (roof + pavement) requiring water quality treatment = 0.84 ac-inch + 2.25 ac-inch = 3.09 ac-inch

Note that the system used to satisfy the stormwater retention requirement of between the 2-year and 25-year storm may also be used to offset the water quality volume requiring treatment, providing it satisfies the other stormwater quality treatment requirements.

Appendix 5 – NRCS Design Storm Distributions

Hour	Rainfall Intensity (in/hr)						
	3-month	6-month	1-year	2-year	5-year	10-year	25-year
1	0.019	0.024	0.028	0.034	0.042	0.050	0.060
2	0.019	0.024	0.028	0.035	0.042	0.050	0.060
3	0.022	0.028	0.033	0.040	0.049	0.060	0.070
4	0.022	0.028	0.033	0.040	0.049	0.060	0.070
5	0.026	0.031	0.037	0.046	0.056	0.070	0.080
6	0.032	0.039	0.047	0.058	0.070	0.080	0.100
7	0.029	0.035	0.042	0.052	0.062	0.070	0.090
8	0.054	0.067	0.080	0.098	0.119	0.140	0.170
9	0.051	0.063	0.075	0.092	0.111	0.130	0.160
10	0.076	0.094	0.112	0.137	0.166	0.200	0.240
11	0.113	0.139	0.165	0.202	0.245	0.290	0.350
12	0.417	0.512	0.611	0.748	0.906	1.060	1.290
13	0.484	0.594	0.710	0.868	1.052	1.240	1.500
14	0.125	0.154	0.183	0.224	0.272	0.320	0.390
15	0.080	0.098	0.118	0.144	0.174	0.200	0.250
16	0.051	0.063	0.075	0.092	0.111	0.130	0.160
17	0.054	0.067	0.080	0.098	0.119	0.140	0.170
18	0.026	0.031	0.038	0.046	0.056	0.060	0.080
19	0.022	0.028	0.033	0.040	0.049	0.060	0.070
20	0.029	0.035	0.043	0.052	0.063	0.070	0.090
21	0.022	0.028	0.033	0.040	0.049	0.060	0.070
22	0.023	0.028	0.034	0.041	0.050	0.060	0.070
23	0.021	0.026	0.031	0.038	0.046	0.050	0.070
24	0.020	0.024	0.029	0.036	0.043	0.050	0.060
TOTALS	**1.8**	**2.3**	**2.7**	**3.3**	**4.0**	**4.7**	**5.7**
MAX	**0.48**	**0.59**	**0.71**	**0.87**	**1.05**	**1.24**	**1.50**