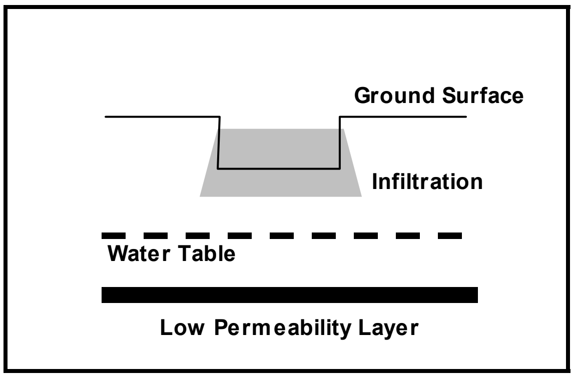
FINAL REPORT

Development of a Rational Basis for Designing Recharging Stormwater Control Structures and Flow and Volume Design Criteria



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Prepared by

Eric Winkler, Ph.D., Principal Investigator
David Ahlfeld, Ph.D., Co-Investigator
Gokhan Askar, Project Engineer
Michelle Minihane, Project Engineer

Contact:
Eric Winkler, Ph.D.
Center for Energy Efficiency and Renewable Energy
University of Massachusetts at Amherst
160 Governors Drive
Amherst, MA 01003-92650
Email: winkler@ecs.umass.edu

Center for Energy Efficiency and Renewable Energy

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Dr. Winkler conceived the project goals and served as principal investigator responsible for project management, project deliverables, editorial review and coordinated review by DEP and outside reviewers. Dr. Ahlfeld was responsible for conducting the research and supervision of the graduate engineers, Gokhan Askar and Michele Minihane, both of whom performed work as MS degree students in the Department of Civil and Environmental Engineering at the University of Massachusetts. The first chapter of this report was conceived and carried out by David Ahlfeld. Dr. Ahlfeld conceived chapter two of this report and Michele Minihane carried out computational work. Gokhan Askar carried out computational work in the Addendum to this Report "Assessment Of The Relative Importance Of Hydraulic Parameters On Infiltration Basin Performance" with guidance by Dr. Ahlfeld and Dr. Winkler.

The authors wish to acknowledge the Massachusetts Department of Environmental Protection staff, Nancy Baker and Thomas Maguire, for their review, insightful comment and overall support of this project.

PROJECT BACKGROUND

In March of 1997, the Massachusetts Department of Environmental Protection (DEP) and the Office of Coastal Zone Management (CZM) published guidance standards for managing stormwater. The guidance document includes requirements for recharge to groundwater based on soil type. The guidance document also specifies that 80% of total suspended solids (TSS) must be removed from the first one half inch of stormwater runoff in most cases, and from the first inch in areas of higher pollution potential, such as shellfish beds or recharge areas for public drinking water supplies. It has been suggested that the recharge and TSS removal standards need additional clarification beyond what is currently included in the guidance documents so that they can be successfully implemented across the Commonwealth.

Recharge Standard

The successful recharge of stormwater from infiltration structures is strongly influenced by the soil characteristics and subsurface hydrology of the site. In particular, the soil permeability and depth to seasonal high groundwater can seriously impact the performance of a recharge system. Question has been raised as to whether Best Management Practices (BMPs) designed for recharge are capable of operating effectively. Given the numerous environmental conditions that can affect performance research is needed to understand these design and operating parameters. This attempts to provide guidance beyond what is currently available for the professional community designing recharge systems and for regulatory bodies that must determine if designs are appropriate for given sites.

The objectives of the recharge portion of the project were to:

- Determine ranges of values for infiltration capacity parameters of soils that are typical in Massachusetts;
- Determine the parameters that are significant indicators of infiltration capacity; and

- Determine the combinations of parameter values that can be used to predict the limits of infiltration capacity.

In this project a detailed modeling study was conducted over a one-year period using MODFLOW and add-in packages to complete the objective stated above. The results of this work are located in the Addendum to this report entitled “Assessment of the Relative Importance of Hydraulic Parameters on infiltration Basin Performance”. The implementation of this work into a working tool follows in Chapter One of this report. It is the goal of this work to provide designers and regulators with a means of focusing attention on the portion of the site where data and analysis are needed and provide a basis for determining whether the recharge standard will be met during pre-construction review of a project.

TSS Removal Standard

The current Stormwater Guidance standards require removal of a percentage of TSS from a specific volume of stormwater. The potential for treating this volume and the majority of pollutants associated in stormwater has led to the acceptance of treating the first flush, defined as the first half inch or one inch in areas of higher pollution potential. However, some stormwater BMPs have rated removal efficiencies based on rates of flow and are typically design to perform up to a peak flow through the device. BMP performance based on peak flow and a standard based on a volume introduces a complex relationship. Rainfall intensity and buildup of sediment both influence the resulting BMP performance. For a particular BMP to meet the 80% removal standard, it should be able to remove materials under a range flow conditions and particle sizes. These constraints bear directly on the sizing of the device. If the device is suitably large, then removal efficiency will be adequate in most cases. However, devices, which are oversized, add unnecessary capital, installation and O&M cost and may increase the impact the BMP has on the installation. For these reasons, the requirement that 80% of TSS be removed from a specified volume of stormwater is viewed by some as incompatible with common flow based design standards. The relationship between the two characteristics of storm events is not easily drawn and needs to be clarified.

The objectives of the flow and volume design criteria portion of the project were to:

- Define the relationship between flow volume and flow rate as a design factor; and
- Evaluate the impact of storm intensity on performance characteristics such that designs based on volume or flow rate may be comparable.

In the second chapter of this report, a series of tables are presented defining the probability that peak flow rainfall intensities may be exceeded during the first one-half or inch of rainfall. Computational analysis of rainfall intensities from regional 15-minute rainfall data was used to develop a simple relationship between the volume criteria of the Stormwater Guidance and a flow based design practice. This relationship may be useful to both designers and regulators during the pre-construction review of stormwater management plans.

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CHAPTER 1: TOOLS FOR ASSESSING INFILTRATION PERFORMANCE OF STORMWATER MANAGEMENT BASINS

Introduction

The Massachusetts Stormwater Management Policy has the goal of improving water quality in the state. One technique for stormwater management is the use of infiltration basins. These structures are designed to capture storm water and recharge this water to the subsurface.

Two factors are important to the successful design of an infiltration basin. The first factor is the precipitation regime to which the basin will be exposed. The precipitation characteristics include the intensity of rainfall during a storm, the length of a storm and the time between storm events. The second factor is the ability of the water in the basin to drain into the subsurface once it is filled. This second factor is the focus of this document.

The basic concepts underlying the recharge of stormwater from a basin to groundwater are depicted in the following figures. The subsurface typically consists of soil grains with pore space between the grains. Figure 1.1 shows the main elements of the basin and groundwater system in schematic form. The water table location is shown as a dashed line. Below this line all the pore spaces between soil grains are filled with water. Above this line is the unsaturated zone where the pore spaces are substantially drained. As drainage from the infiltration basin progresses the water table configuration will change and portions of the unsaturated zone will become saturated. At the bottom of Figure 1.1 is a heavy line representing a low permeability layer. This layer has a significantly lower permeability than the soils above it. Many different geologic configurations may produce a low permeability layer. Typical examples are a clay layer or silt layer underlying sandy soils or bedrock underlying loamy soils. Depending on the location of the site the low permeability layer may lie anywhere from a few feet below the ground surface to several hundred feet below ground surface.

Figure 1.1 Infiltration Basin Schematic

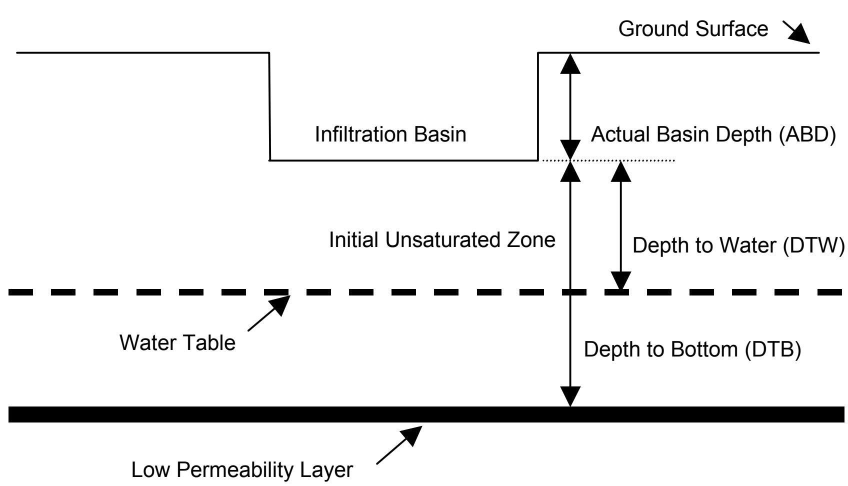


Figure 1.2 depicts the state of the basin and subsurface shortly after the end of a storm. The shaded area represents the location of the water at that time. Note that most of the water is in the basin but some has already begun to infiltrate into the subsurface. This includes water that has migrated directly below the bottom of the basin and water that has moved laterally through the basin walls. This water has begun to infiltrate during the storm.

Figure 1.2 Water Distribution Shortly After Storm

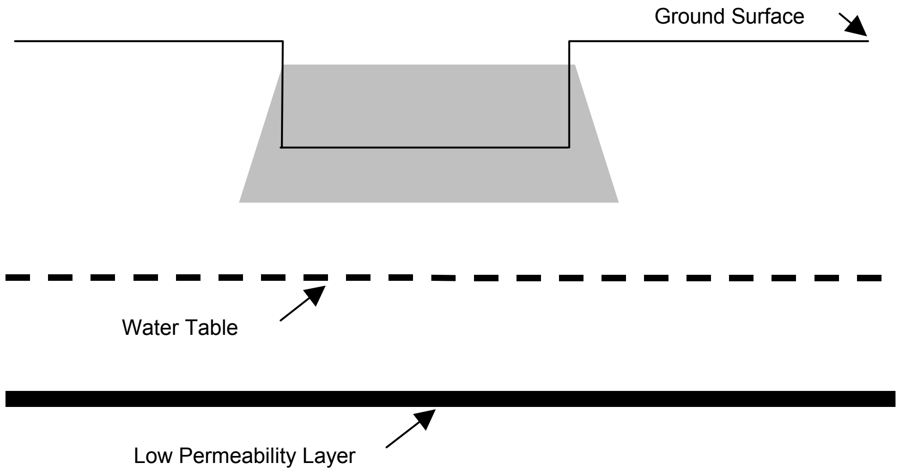


Figure 1.3 depicts the water distribution at intermediate time after the storm. Some water still remains in the basin, but most water is now located in the subsurface. The zone beneath the basin that was initially unsaturated is now fully saturated. In addition, water has continued to move laterally away from the basin. In effect, the water table has now risen so that it reaches to the bottom of the basin.

Figure 1.3 Water Distribution Some Hours After Storm

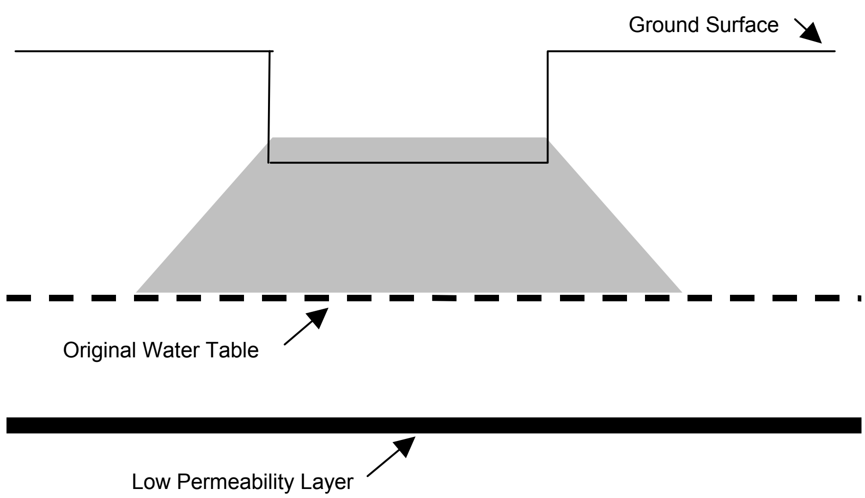
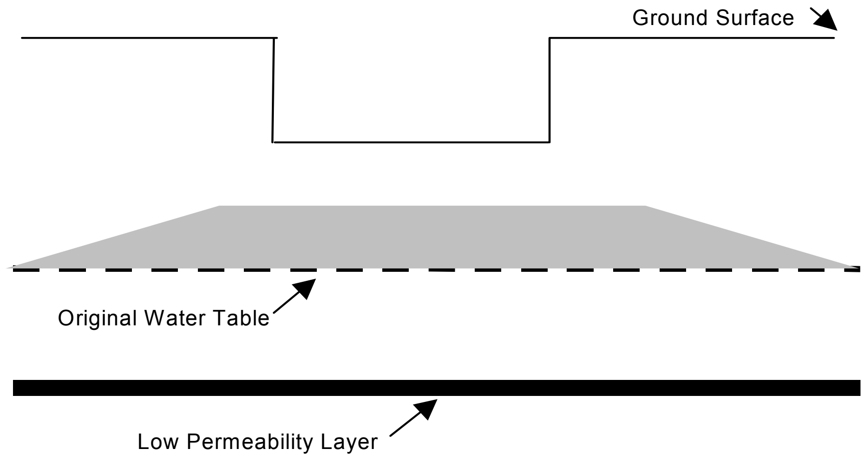


Figure 1.4 shows the water distribution in the late stages of drainage. The basin is now entirely empty, however, the infiltrated stormwater still has an impact on subsurface conditions. A mound in the water table remains as water slowly drains further from the basin.

Figure 1.4 Water Distribution Many Hours



Research has shown that three factors control the rate of drainage in most settings (Askar, 2000). These factors are; the infiltration rate (also known as the hydraulic conductivity or soil permeability), the depth to water and the depth to the low permeability layer. These are shown in Figure 1.1 as DTW and DTB. Many design procedures directly incorporate infiltration rate. However, consideration of the depth to water and the depth to the low permeability layer is less common. The intent of this document is to provide a simple means of determining the significance of these two additional factors for specific field sites.

Storage Capacity of Soils During a Storm

As suggested by Figure 1.2, some infiltration may occur during a storm event. At the end of the storm the basin and sub-basin soils have stored a volume of water that will subsequently disperse in the subsurface. This volume can be expressed as a depth times a basin area. The depth of the basin is normally considered as the distance from the bottom of the basin to the elevation of the outlet. However, if some water is stored in the soils beneath the basin, the basin depth is effectively increased. In this section a method is presented that provides a conservative estimate of the additional effective depth achieved from storage in soils beneath the basin.

The amount of infiltration that occurs before the end of the storm depends upon the rate at which water enters the basin, the duration of the storm, the infiltration rate of the soils beneath the basin and the depth to seasonal high groundwater. If the amount of infiltration is substantial then the basin capacity is effectively increased for storms of a particular magnitude.

In this section data is provided for five different soil groups and for a wide range of storm conditions. The data are presented in Figures 2.1 through 2.5. For a particular soil type, storm intensity, expressed as inflow rate, and storm duration the figures provide the effective additional depth added to the basin. The calculations assume a constant inflow rate to basin during the storm. The data, additional assumptions, and procedures used to produce these figures are provided in Appendix C.

The figures should be used as follows for a particular field site.

1) Select the appropriate soil type and identify the corresponding figure.
   a) Figure 2.1: sand,
   b) Figure 2.2: loamy sand,
   c) Figure 2.3: sandy loam,
   d) Figure 2.4: loam,
   e) Figure 2.5: silt loam.

2) Determine the Actual Basin Depth (ABD).
   The basin depth is measured from the bottom of the basin to the elevation of the outlet structure so that ABD is the approximate depth of water that would be in the basin if it were full (see Figure 1.1)

3) Determine the Maximum Additional Depth (MAD).
   The rate of infiltration into the soils will diminish once the initial unsaturated zone beneath the basin is filled. The maximum additional depth available is related to the distance to seasonal high groundwater and soil properties. MAD should be calculated as follows:
   a) for sand: MAD = 0.34 x DTW
   b) for loamy sand: MAD = 0.31 x DTW
   c) for sandy loam: MAD = 0.30 x DTW
   d) for loam: MAD = 0.21 x DTW
   e) for silt loam: MAD = 0.22 x DTW

   where DTW is the depth to water defined as the vertical distance from the bottom of the basin to the seasonal high groundwater level (see Figure 1.1).

4) Determine the Storm Duration (SD).
   Typical storm durations are provided on the figures.
   a) 30 minute,
   b) 1 hour,
   c) 2 hour,
   d) 6 hours.

5) Determine the storm Inflow Rate (IR).
   The inflow rate is the average flowrate per basin area at which water enters the basin during the storm duration, SD. IR can be computed by an appropriate rainfall-runoff method. If a constant intensity storm with negligible time of concentration and abstraction is assumed then it can be calculated as:

   $IR = P \times (A_c/A_r)$

   where

   P = precipitation rate (ft/hr)
   A_c = contributing impervious area (acres)
   A_r = area of infiltration basin (acres)[1]

6) Determine the Effective Depth Infiltrated (EDI) from the appropriate figure.
   a) Pick the point on the Inflow Rate axis corresponding to your inflow rate. If your IR value is greater than 3 ft/hr then use 3 ft/hr.
   b) Move up to the line corresponding to the storm duration.

[1] The area of basin for irregularly shaped basins such as those with sloping walls or with bowl shaped bottoms should be defined to be representative of the area within which vertical infiltration will occur. For shallow basins with relatively flat bottoms, a reasonable approximation is to set A_r equal to the area of horizontal plane located 1 foot above the lowest point in the basin.

c) Move left to the EDI axis. Identify the EDI value.

7) Determine the Actual Depth Infiltrated (ADI).
The calculations used to produce Figure 2.1 to 2.5 assume that unlimited depth is available for infiltration. If EDI exceeds the maximum available depth, MAD, then MAD is the correct depth to use.
Select ADI as the smaller of MAD and EDI.

8) Determine Effective Basin Depth (EBD)
EBD = ABD + ADI

Effective Infiltration Example I

Assume that the soil is loamy sand, depth to water is 2 feet, precipitation rate is 0.5 inches per hour, storm duration is 2 hours, contributing impervious area is 3 acres, basin depth is 4 feet and the basin area is 0.05 acres.

1) Appropriate figure is 2.2.
2) ABD = 4 feet.
3) MAD = 0.31 x 2 feet = 0.62 feet
4) IR = 0.5 inch/hr x 1/12 ft/inch x 3 acres / 0.05 acres = 2.5 ft/hr.
5) SD = 2 hours.
6) Move to 2.5 point on Figure 2.2, move up to the 2 hour line (short-dash line), move to the left to the EDI axis to an EDI value of 0.9 feet.
7) ADI = smaller of 0.62 and 0.9 = 0.62 feet
8) EBD = 4 + 0.62 = 4.62 feet

For this example, the initial unsaturated zone has entirely filled during the storm event and for this storm event the basin is capable of storing 4.62 feet of water.

Basin Limitations for Shallow Conditions

Figures 1.3 and 1.4 depict how stormwater drains from the basin after the end of the storm. The rate of this drainage depends on the infiltration rate, the thickness of the initial unsaturated zone and the thickness of the initial saturated zone. If either of these zones are thin then drainage can be slowed especially for low permeability soils. In this section a method is presented that provides a conservative estimate of the drainage that has occurred after 72 hours. This time has been identified as the target time by which the basin should be emptied for effective operation of the basin and is based on the mean duration between storm events.

In this section results are presented for five different soil groups and for a wide range of basin and subsurface geometries. The results are provided in Figures 3.1 through 3.5. The figures provide the depth of water that will be remaining in the basin after 72 hours based on soil type, depth to water and depth to low permeability layer. The data, additional assumptions, and procedures used to produce these figures are provided in Appendix C. The data is generated for basins whose shortest horizontal dimension is 40 feet. Basin significantly narrower than this may drain faster.

The figures should be used as follows for a particular field site.

1) Select the appropriate soil type and identify the corresponding figure.
    a) Figure 3.1: sand,

   b) Figure 3.2: loamy sand,
   c) Figure 3.3: sandy loam,
   d) Figure 3.4: loam,
   e) Figure 3.5: silt loam.

2) Determine the Actual Basin Depth (ABD).
   The basin depth is measured from the bottom of the basin to the elevation of the outlet structure so that ABD is the approximate depth of water that would be in the basin if it were full (see Figure 1.1).

3) Determine the Depth to Water (DTW).
   The DTW is defined as the vertical distance from the bottom of the basin to the seasonal high groundwater level (see Figure 1.1).

4) Determine the Depth to Bottom (DTB)
   The DTB is defined as the vertical distance from the bottom of the basin to the top of the low permeability layer (see Figure 1.1).

5) Determine the Saturated Zone Thickness (SZT).
   The SZT is given as:
   SZT = DTB – DTW

6) Determine the Initial Mound Size (IMS).
   The IMS is given as:
   IMS = DTW + ABD

7) Determine the Drop in Water Level (DWL) from the appropriate figure.
   a. Pick the point on the IMS axis corresponding to your IMS value.
   b. Move up to the line corresponding to the SZT value. If SZT is greater than 8 feet use the 8-foot line.
   c. Move left to the DWL axis. Identify the DWL value.

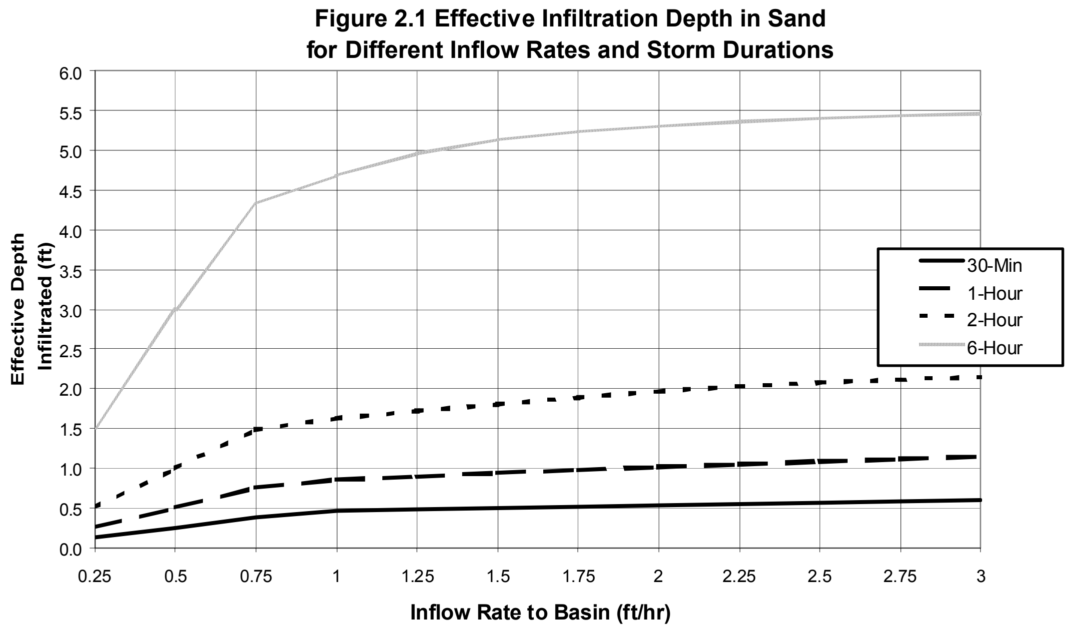
8) Determine if water remains in the basin after 72 hours.
   a. if DWL is less than ABD then water remains in the basin
   b. if DWL is greater than ABD then the basin has drained (although a mound of water may still be present in the subsurface)

Effective Infiltration Example II

Assume that the soil is loamy sand, depth to water from basin bottom is 2 feet, depth to low permeability layer is 6 feet and the basin depth is 4 feet.

1) Appropriate figure is 3.2.
2) ABD = 4 feet.
3) DTW = 2 feet
4) DTB = 6 feet
5) SZT = 6 – 2 = 4 feet
6) IMS = 4 + 4 = 8 feet
7) Move to IMS = 8 on the horizontal axis on Figure 3.2, move up to the 4 foot (dashed) line, move to the left to the DWL axis to an DWL value of 5 feet
8) Since 5 is greater than 4, water has drained from the basin after 72 hours.

Appendix 1A: Figures for Effective Infiltration Depth



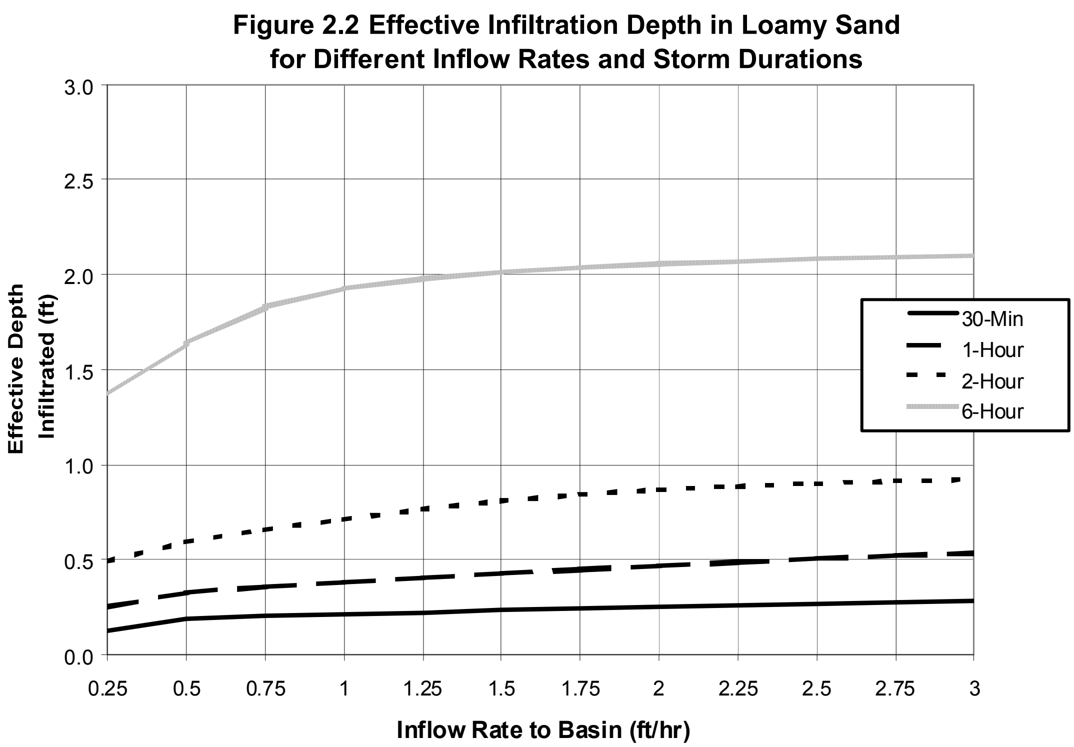


Figure 2.3 Effective Infiltration Depth in Sandy Loam for Different Inflow Rates and Storm Durations

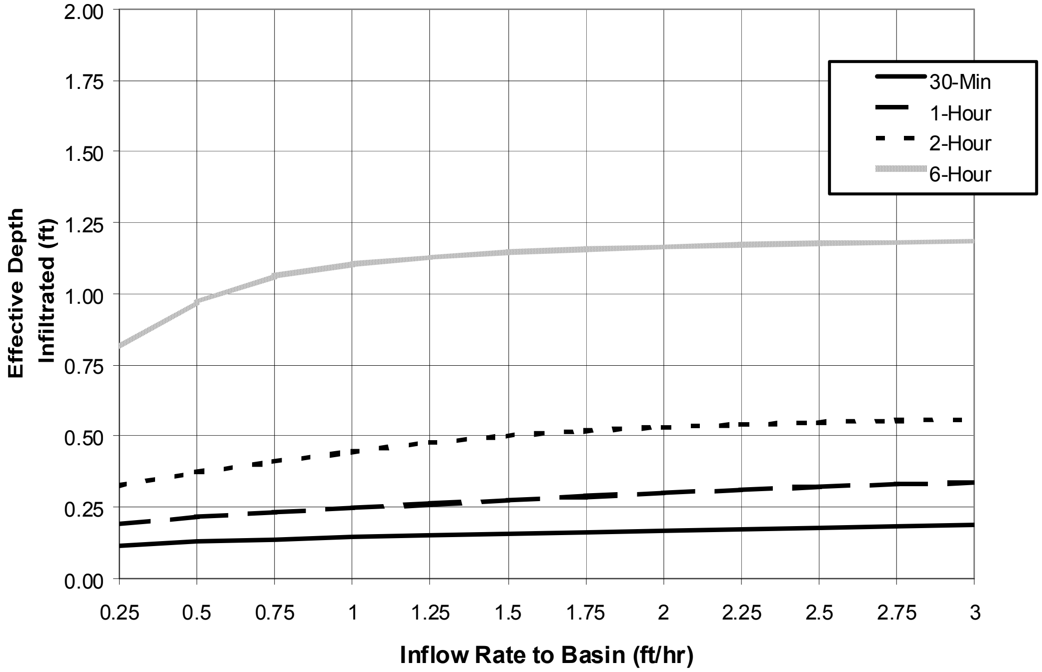
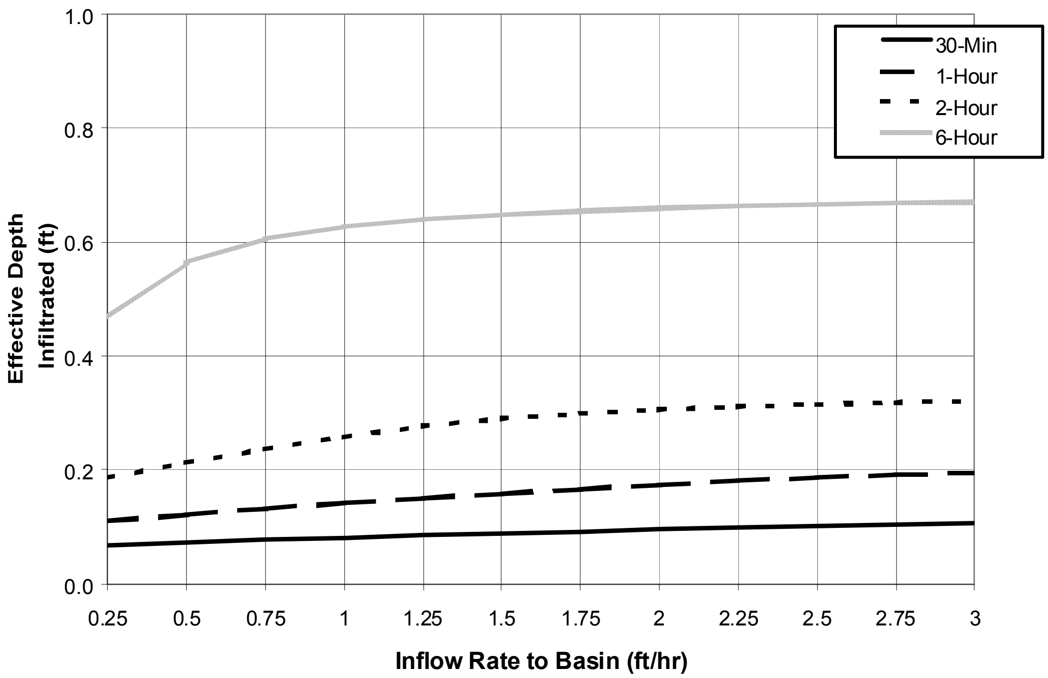


Figure 2.4 Effective Infiltration Depth in Loam for Different Inflow Rates and Storm Durations



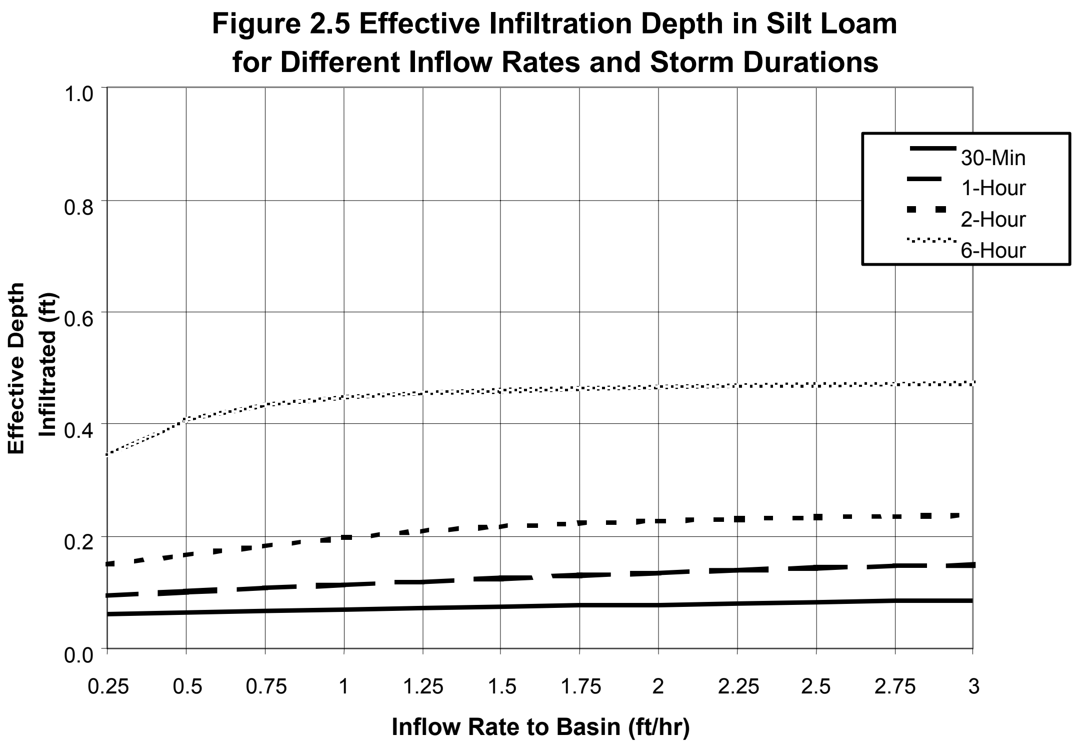


Figure 2.5 Effective Infiltration Depth in Silt Loam
for Different Inflow Rates and Storm Durations
1.0
0.8
0.6
0.4
0.2
0.0
Effective Depth
Infiltrated (ft)
0.25
0.5
0.75
1
1.25
1.5
1.75
2
2.25
2.5
2.75
3
Inflow Rate to Basin (ft/hr)
30-Min
1-Hour
2-Hour
6-Hour

Appendix 1B: Figures for Basin Water Level Decline

Figure 3.1 Basin Water Level Decline for Different Initial Mound Sizes and Initial Saturated Zone Thickness for Sand Soils

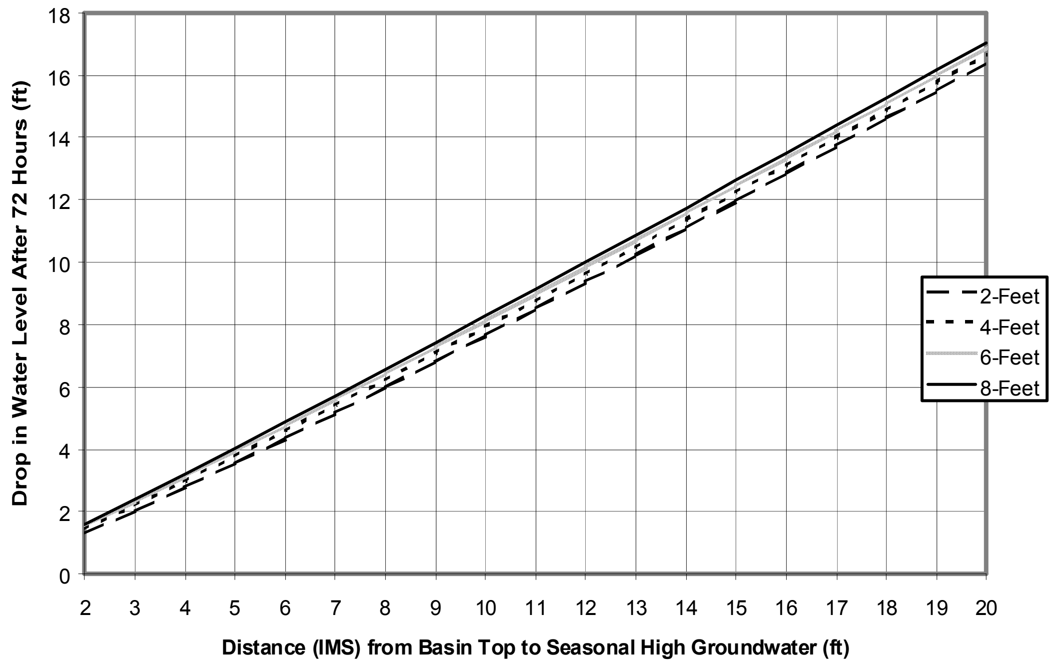


Figure 3.2 Basin Water Level Decline for Different Initial Mound Sizes and Initial Saturated Zone Thickness for Loamy Sand Soils

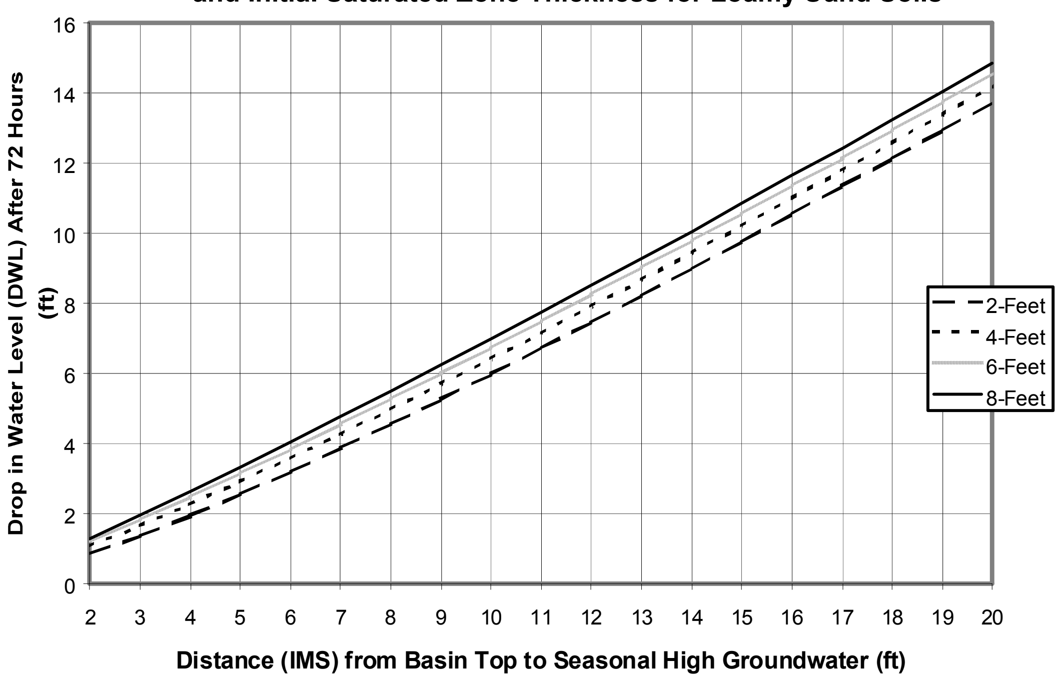


Figure 3.3 Basin Water Level Decline for Different Initial Mound Sizes and Initial Saturated Zone Thickness for Sandy Loam Soils

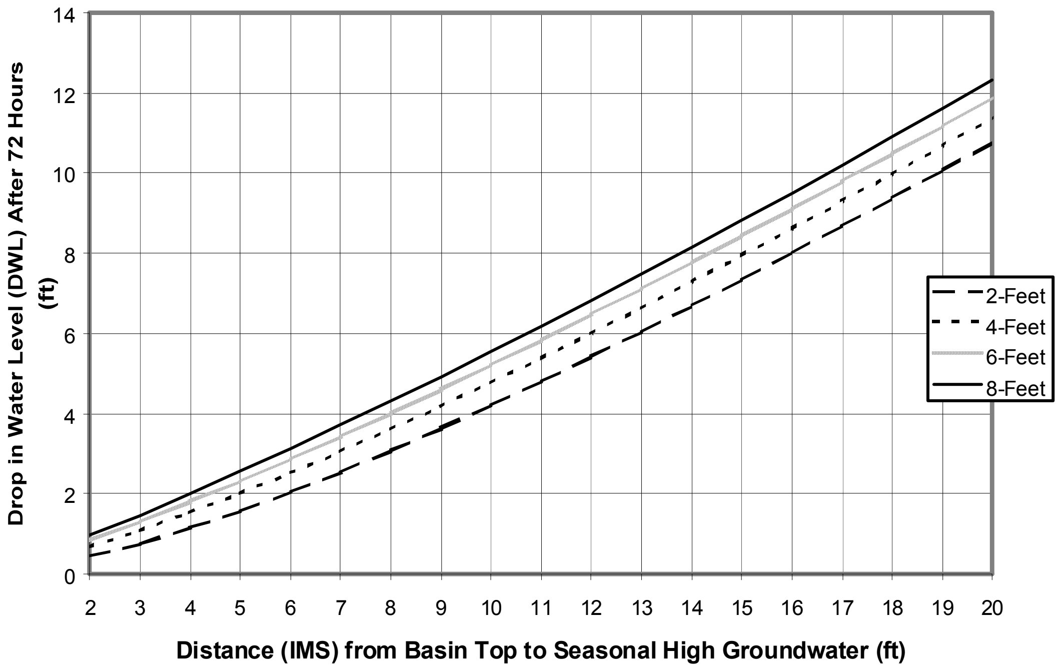


Figure 3.4 Basin Water Level Decline for Different Initial Mound Sizes and Initial Saturated Zone Thickness for Loam Soils

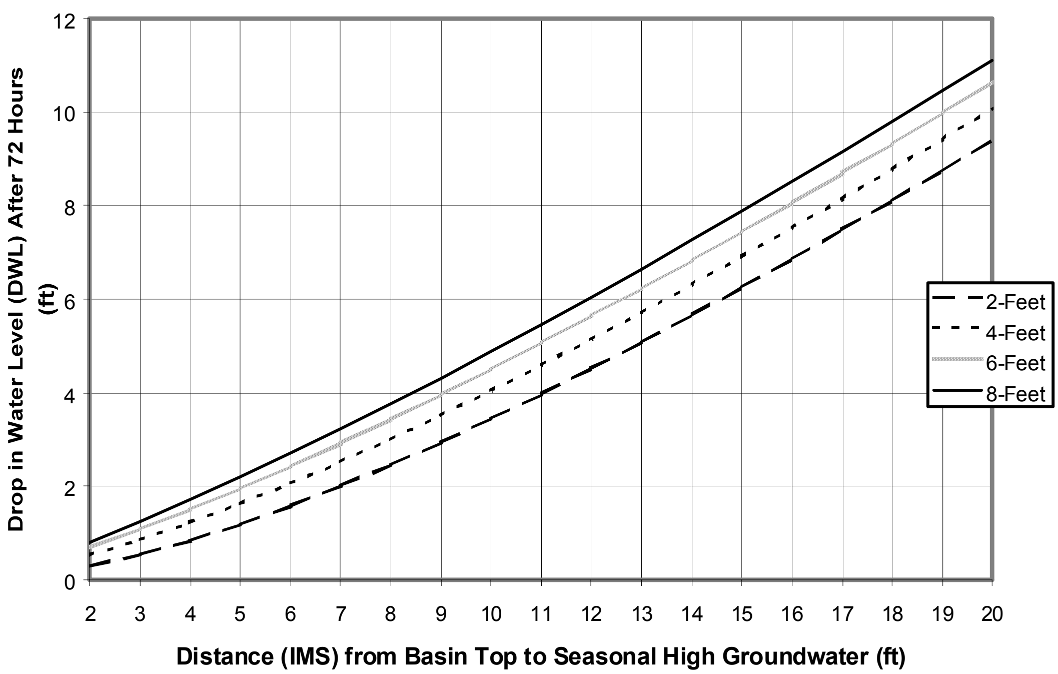
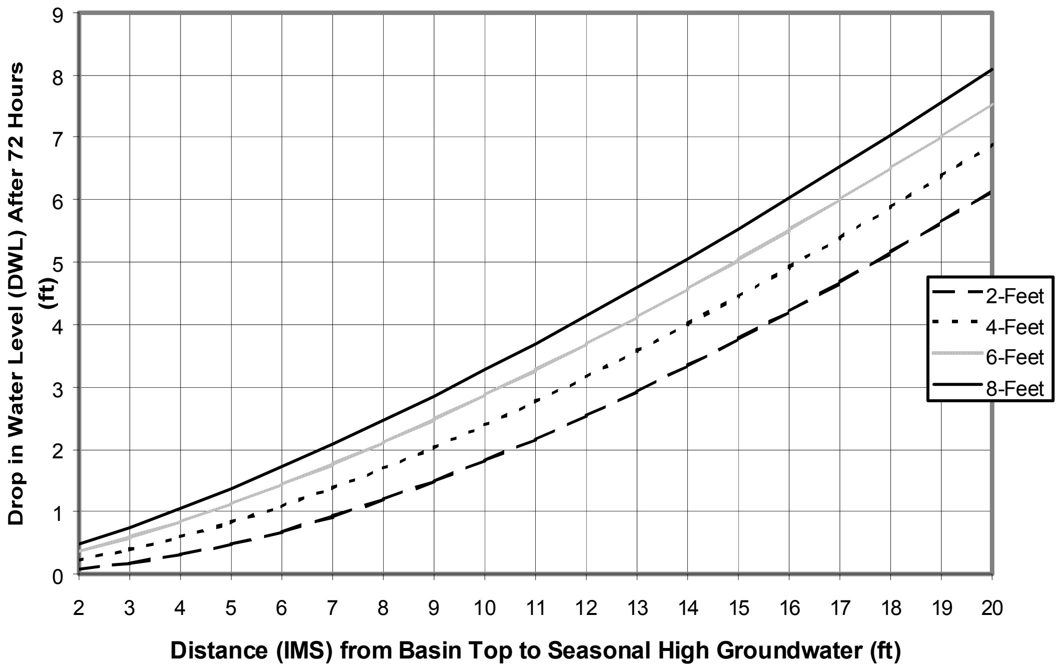


Figure 3.5 Basin Water Level Decline for Different Initial Mound Sizes and Initial Saturated Zone Thickness for Silt Loam Soils



Appendix 1C: Procedures Used to Create Figures

Figures 2.1 to 2.5 were generated using the Green-Ampt method (Dingman, 1994, Bedient and Huber, 1992). This method assumes that infiltration occurs as a sharp wetting front. The parameters that control infiltration are the infiltration rate, the soil porosity, the initial soil moisture content and the wetting front soil suction head. These are specified for each of the five soil types considered. Values for infiltration rate are taken from the Massachusetts Technical Bulletin 99-01 (MADEP and MACZM, 1999). Values for porosity and suction head are taken from Maidment (1993). Initial soil moisture content is assumed to be at field capacity. The values used are taken from Dingman (1994).

A table of the values used is provided below

Table 1: Values Used in Green-Ampt Calculations

Soil Type	Infiltration Rate (in/hr)	Porosity	Wetting Front Suction Head (ft)	Initial Moisture Content
Sand	8.27	0.437	0.16	0.10
Loamy Sand	2.41	0.437	0.20	0.13
Sandy Loam	1.02	0.453	0.36	0.15
Loam	0.52	0.463	0.29	0.25
Silt Loam	0.27	0.501	0.55	0.28

The Green-Ampt calculations account for the presence of the basin and the increasing head in the basin as it fills. The increasing head adds to the infiltration driving force and increases the infiltration rate. This in turn decreases the time to filling. The actual rate of basin filling and the maximum head that can be achieved in the basin will depend on detailed basin geometry. It is assumed that at no time does the depth of water in the basin exceed 2 feet. The rate of basin filling is based on an assumption of a basin with vertical sidewalls. The impact of these assumptions is; 1) the average rate of filling up to a depth of 2 feet should be a reasonable approximation to the actual filling rate if the basin area (A_r) is defined appropriately, 2) when the actual basin head exceeds 2 feet the model results will underestimate infiltration rate and overestimate time to filling of soil void spaces which is a conservative assumption. For the case when the inflow rate is less than the infiltration rate the unsaturated zone is presumed to fill from the impermeable layer up.

The procedure requires specification of MAD, the maximum additional depth, which is related to the depth to water and soil properties. The coefficients used to compute MAD are the porosity minus the initial moisture content for each soil type. These properties are listed in Table 1.

The figures display results over a range of basin inflow rates from 0.25 ft/hr to 3 ft/hr. As an example, this can accommodate precipitation rates as low as 0.1 inches/hr with area ratios as low as 30. It was determined that for basin inflow rates greater than 3 ft/hr the effective depth infiltrated approaches a constant value.

The figures display lines for several different storm durations. At a given inflow rate increased duration implies deeper ponding in the basin. The storm durations provided are limited to those associated with realistic ponding depths. It is assumed that these durations will be adequate for all practical design settings.

A mass balance check is incorporated into the Green-Ampt code. All simulations satisfy mass balance to within 0.001%.

Figures 3.1 to 3.5 are generated using the analytical solution to the decay of a rectangular mound (Polubarinov-Kochina, 1962). This solution assumes an initial mound with a rectangular profile. The initial height of the mound is H1. Surrounding the mound is a uniform thickness water level of H0, which extends to infinity. The governing equation that is solved assumes transient one-dimensional flow with a constant thickness for the saturated zone. The hydraulic conductivity and storage coefficient are assumed constant.

The parameters used include the hydraulic conductivity, specific yield, saturated zone thickness and mound dimensions. For each soil type the hydraulic conductivity is assumed to be the infiltration rate listed in Table 1 and the specific yield is the porosity minus the initial moisture content listed in Table 1. The width of the mound, which corresponds with the width of the shortest dimension of the basin, is assumed to be 40 feet. The mound width does effect drainage predictions. For all soils except sand, basins wider than 40 feet have nearly the same behavior to that calculated for 40 feet. As can be seen from Figure 3.1 sand drains adequately for most practical drainage basin designs (e.g. basin depth less than 6 feet, depth to water greater than 2 feet). For basins much narrower than 40 feet the drop in water level may be somewhat higher than that predicted in these results. The mound height H1 is set to the IMS value plus the SZT value defined above. This implies that the mound is treated as having a height equal to the depth of water in the basin and the distance to the top of the low permeability layer. The H0 is a variable and equal to the dimension SZT defined above. The saturated thickness is set equal to the average of the H0 and H1 values.

The use of this model provides a simple way to produce the results depicted in Figures 3.1-3.5. However, it does involve certain assumptions. To test the validity of these assumptions the model results are compared with those produced by Askar (2000) who used a detailed three-dimensional numerical model to model the mound decay problem.

Askar (2000) modeled both the basin filling and the mound decay problems and reports the depth of water in the basin 72 hours after the initiation of basin filling. The analytical solutions report time from the end of filling. Hence, the reported time for analytical solutions is added to the time estimated by Askar (2000) to fill the basin.

Table 5.4 in Askar (2000) simulates a case with DTW of 2 feet, ABD of 4 feet, basin width of 25 feet (on short side), and varying SZT values. For these cases the basin required about 9 hours to fill. The comparison provided in Table 2 shows the analytical solution at 63 hours after the end of filling or 72 after the beginning of filling. The analytical solution tends to slightly over predict the mound height but is in general good agreement with the numerical solution.

Table 2: Comparison of Analytical Solution with Numerical Solution for Sandy Loam

SZT	Initial Mound Height	Analytical Mound Height at 72 hours (ft)	Askar (2000) Mound Height at 72 hours (ft)
4	6	3.48	3.07
6	6	3.13	2.94
8	6	2.88	2.88

Table 5.6 in Askar (2000) simulates a case with SZT of 8 feet, ABD of 3 feet, basin width of 25 feet (on short side), and varying DTW values. For these cases the basin required about 6 hours to fill. The comparison provided in Table 2 shows the analytical solution at 66 hours after the end of filling or 72 after the beginning of filling. The analytical solution tends to under predict the mound height.

Table 3: Comparison of Analytical Solution with Numerical Solution for Silt Loam			
DTW	Initial Mound Height	Analytical Mound Height at 72 hours (ft)	Askar (2000) Mound Height at 72 hours (ft)
2	5	3.55	3.84
4	7	4.81	5.51
6	9	6.01	7.34

Table 5.8 in Askar (2000) simulates a case with SZT of 8 feet, ABD of 4 feet, basin width of 25 feet (on short side), and varying DTW values. For these cases the basin required about 9 to 10 hours to fill. The comparison provided in Table 2 shows the analytical solution at 62 hours after the end of filling or 72 after the beginning of filling. The analytical solution tends to slightly under-predict the mound height but is in general good agreement with the numerical solution.

Table 4: Comparison of Analytical Solution with Numerical Solution for Sandy Loam			
DTW	Initial Mound Height	Analytical Mound Height at 72 hours (ft)	Askar (2000) Mound Height at 72 hours (ft)
2	6	2.88	2.88
3	7	3.29	3.49
4	8	3.69	4.17

Appendix 1D: References

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Appendix 1E: Glossary of Terms

ABD	Actual Basin Depth (see Figure 1.1)
ADI	Actual Depth Infiltrated
DTW	Depth to Water (see Figure 1.1)
DTB	Depth to Bottom (see Figure 1.1)
DWL	Drop in Water Level
EBD	Effective Basin Depth
EDI	Effective Depth Infiltrated
IMS	Initial Mound Size
IR	Inflow Rate to basin
MAD	Maximum Additional Depth
SD	Storm Duration
SZT	Saturated Zone Thickness

Appendix 1F: Computer Code Used for Green-Ampt Calculations

```
!  PROGRAM SIBPGA
!
!    This program Simulates Infiltration with a
!       Basin with Ponding using Green-Ampt
!
!  written by David Ahlfeld
!      University of Massachusetts, Amherst
!
!  in support of Massachusetts Dept. of Env. Protection project 99-06/319
!
! Variable Definitions
!
!     Input Physical Parameters
! phi = porosity of soil
! psif = effective tension at wetting front
! theta = volumetric water content
! khsat = saturated vertical hydraulic conductivity (ft/hr)
! rinf = rate of inflow to basin (ft/hr)
!
!     Input Control Variables
! tstopv = the times at which the location of the wetting front is recorded
(hr)
! nz     = number of depth increments
! tinc   = time increment to use in numerical calculation (hr)
! tend   = ending time (hr)
!
!     Iteration Measures
! fcum = cumulative infiltration depth (ft)
! tp = time to ponding (hrs)
! yt = depth of ponding (ft)
! zf = depth of wetting front (ft)
! time() = time required for wetting front to reach indicated depth(hr)
!
        program greenampt

      real phi, theta, khsat, rinf, tinc, tend, psif, tp, zf,yt
        REAL TIME(6),vphi(5),vpsif(5),vtheta(5),vkhsat(5),tstopv(6)
        CHARACTER NAMEi(5)*15, NAME*15
        call setparam(vphi,vpsif,vtheta,vkhsat,namei)
        OPEN(10,"Report.out")
        OPEN(12,"Data.out")
! set iteration and output parameters
        tstopv(1) = 0.5
        tstopv(2) = 1.
        tstopv(3) = 2.
        tstopv(4) = 6.
        tstopv(5) = 12.
        tstopv(6) = 24.
        nz   = 6
      tend = 100.
! loop over physical parameters
        do 300 iset =1,5
! set physical parameters
        phi = vphi(iset)
      psif = vpsif(iset)
```

```
      theta = vtheta(iset)
      khsat = vkhsat(iset)
        name = namei(iset)
        do 90 i=1,nz
          time(i) = 0.0
90     continue

!
! loop over inflow rate to basin (ft/hr)
!
        WRITE(12,*)name,"   30-Min","   1-Hour","   2-Hour","   6-Hour","   12-
Hour","  24-Hour"
        WRITE(10,*)"Begin ", Name
        do 200 rinf = 0.25,5,0.25
        if (rinf.lt.khsat) then
! ponding will never occur - calculate time to fill from below
          do 60 i=1,nz
            time(i) = rinf*tstopv(i)
60        continue
          WRITE(12,8000)rinf,(time(i),i=1,nz)
          GOTO 200
        end if

        call grnampt (phi, psif, theta, khsat, rinf, tp, zf, yt, nz,&
                     time, tstopv, tend, tinc)
!
        WRITE(12,8000)rinf,(time(i),i=1,nz)
        do 110 i=1,nz
          time(i) = 0.0
110     continue
! compute mass balance check on solution
          xbal = yt + zf*(phi-theta) - rinf*tend
          xbal = xbal/(rinf*tend)
          if (xbal.gt.0.00001) then
            WRITE(10,9100) yt,zf*(phi-theta),rinf*tend,xbal
          end if
200     continue
300     continue
!
8000    FORMAT(f6.2,9e12.4)
9100    FORMAT('Poor Mass Balance',/,t10,"Pond Depth",t40,e12.6,/,   &
          t10,"Infiltration Depth",t40,e12.6,/,                     &
          t10,"Total Depth",t40,e12.6,/,                            &
          t10,"Percent Imbalance",t40,e12.6)

        end program
!
        subroutine grnampt(phi, psif, theta, khsat, rinf, tp, zf, yt,&
         nz, time, tstopv, tend, tinc)
        Real phi, theta, khsat, rinf, tinc, tend, psif, tp, zf,yt
        REAL time(nz), tstopv(6)
! compute time to ponding
!       Dingman (eq 6-27)
!
      tp = khsat*abs(psif)*(phi-theta)
      tp = tp/(rinf*(rinf-khsat))
! set time step increment (hrs)
```

```
        tinc = MIN(tp/10.,0.01)
!
! compute depth of wetting front at ponding
!
      zf = tp*rinf/(phi-theta)
      yt = 0.0
      fcum = tp*rinf
        WRITE(10,9010)rinf,tp,zf
        iint = 0
        IF(tp.gt.tstopv(1))then
! it has filled to some depth
           IF(tp.lt.tstopv(2))then
             iint = 1
           ELSEIF(tp.lt.tstopv(3))then
             iint = 2
           ELSEIF(tp.lt.tstopv(4))then
             iint = 3
           ELSEIF(tp.lt.tstopv(5))then
             iint = 4
           ELSEIF(tp.lt.tstopv(6))then
             iint = 5
           ELSEIF(tp.gt.tstopv(6))then
             iint = 6
           endif
        endif
        do 60 i=1,iint
          time(i) = rinf*tstopv(i)
60      continue
       if (tp.gt.tstopv(nz)) then
! it has already filled to capacity
            return
        endif
! loop through time
!
        i = iint
      do 100 t = tp,tend,tinc
! compute infiltration rate during this time increment
          ytm = MIN(yt,2.)
        ft = khsat*(1 + (abs(psif)+ytm)/zf)
! update the cumulative infiltration at end of step
        fcum = fcum + ft*tinc
! update the wetting depth
        zf = fcum/(phi-theta)
! update the ponding depth
        yt = yt + (rinf - ft)*tinc
! output
          if (t.gt.tstopv(i+1)) then
            WRITE(10,9020) zf,t,yt
            i = i + 1
            time(i)=fcum
          end if
          if (i.eq.6) then
            return
          end if
100     continue
9010    FORMAT('Inflow Rate',t40,e12.6,/,t10,'Time to Ponding',t40,e12.6,/,&
          t10,'Depth of Front at Ponding',t40,e12.6)
```

```
9020    FORMAT(t10,'Time to Fill to Depth of ',f6.1,' is ',f8.4,/,&
               t10,'Pond Depth at this time is ',f8.4)
!
        return
        end

        subroutine setparam(vphi,vpsif,vtheta,vkhsat,namei)
        REAL vphi(5),vpsif(5),vtheta(5),vkhsat(5)
        CHARACTER NAMEi(5)*15

!   phi values and psif values taken from Maidment, Handbook of Hydrology,
!      Table 5.5.5 after conversion to ft
!   theta values are assumed to represent field capacity and are taken
!      from Figure 6-4 in Dingman
!   khsat values are in inches/hr.  Convert to ft/hr. These values are taken
!      from MA Draft Technical Bulletin 99-01
! set sand
        vphi(1)   = 0.437
        vpsif(1)  = 0.16
        vtheta(1) = 0.10
        vkhsat(1) = 8.27/12.
        namei(1)  = "Sand"
! set loamy sand
        vphi(2)   = 0.437
        vpsif(2)  = 0.20
        vtheta(2) = 0.13
        vkhsat(2) = 2.41/12.
        namei(2)  = "Loamy Sand"
! set sandy loam
      vphi(3)   = 0.453
        vpsif(3)  = 0.36
        vtheta(3) = 0.15
        vkhsat(3) = 1.02/12.
        namei(3)  = "Sandy Loam"
! set loam
        vphi(4)   = 0.463
        vpsif(4)  = 0.29
        vtheta(4) = 0.25
        vkhsat(4) = 0.52/12.
        namei(4)  = "Loam"
! set silt loam
        vphi(5)   = 0.501
        vpsif(5)  = 0.55
        vtheta(5) = 0.28
        vkhsat(5) = 0.27/12.
        namei(5)  = "Silt Loam"

         return
           end
```

CHAPTER 2: DEVELOPMENT OF FLOW AND VOLUME DESIGN CRITERIA

Project Overview

The Massachusetts Stormwater Management Policy has the goal of improving water quality in the state. One aspect of this policy is a requirement that treatment be applied to the stormwater runoff that occurs early in a storm event. This “first flush” water must be treated due to the fact that it typically contains the greatest concentration of pollutants. The Massachusetts Policy provides an explanation of the standards for water quality (standards 4-6) that stipulates that the first 1.0-inch of runoff must be treated for critical areas and the first 0.5 inches must be treated for all other areas.

The water quality volume can be derived simply by multiplying the depth of runoff (0.5 or 1.0 inches) by the impervious area at the site. If the treatment device is a detention pond or basin then this volume can be used for basin sizing. However, some treatment devices are based on flow-through technology. These devices do not detain water but instead perform treatment as the stormwater passes through. Sizing of these devices typically requires a design flow rate. The setting under consideration is one in which stormwater drains from an impermeable area, such as parking lot or rooftop, into a collection point where it is subject to treatment.

For purposes of sizing of flow-through devices we assume that the designer seeks a design flow rate Q_d, which can be computed as q_dA where A is the site specific impermeable area and q_d is the design flow rate per unit area or the design flux. In this report we analyze for values of the design flux, q_d.

This document is intended to provide a relationship between a specified storm depth and a resulting flow rate. Because of the wide variability of individual storm depths, durations and intensities there is no single relationship between these variables. For low intensity storms the flow rate through the collection point is small. For storms with high intensities the flow rate at the collection point may be large during the first flush period. Hence, the design flow rate should be based on these high intensity storms. However, use of very high values of q_d may lead to unnecessary over-design of flow-through devices because these intensities may occur rarely. In this document we present a range of design flux based on the probability that the design flux will be exceeded in any given storm.

In addition to the selected probability level, the design flux depends upon the three factors. The first factor is the depth of stormwater that must be treated (0.5 inches or 1.0 inches). The second factor is the location of the structure. The analysis of the data has indicated that the pattern of storm intensities is different for Cape Cod and mainland regions of Massachusetts. The third factor is the method used for estimating storm intensity. The first method is the average intensity over the entire storm up to the point at which the design depth is achieved. The second is the maximum 15-minute intensity observed during the storm up to the point when the design depth is achieved. The appropriate definition may depend upon the specific characteristics of the flow-through device and the ability of the system to attenuate peaks in flow with appropriate storage.

The results presented here are based on analysis of rainfall data at stations in Massachusetts with records of 15-minute storm data extending from 1971 to 2000. Complete details of the analysis can be found in the following sections of this chapter.

The results of this work are presented in the four tables that follow. Each table indicates the probability that the storm intensity will be less than the indicated design flux. Considering Table 1, the 0.5-inch design flux for the Cape Cod area at the 95% probability level using the average storm intensity is 0.39 inches/hr. This can be interpreted to mean that there is a 95% probability that the actual average storm intensity for any given storm will be less than or equal to 0.39 inches/hr during the first 0.5 inches of precipitation. One minus the probability value provides the likelihood that the storm intensity will be exceeded in any given storm event. This implies that there exists a 5% probability that the average intensity will exceed 0.39 inches/hr during the first 0.5 inches of precipitation.

Example

To use these results in practice the user needs to identify the appropriate storm depth, either 0.5 inch or 1.0 inch, and the location of the structure under consideration, either Cape Cod or mainland. These two factors will indicate which of the Tables is appropriate. The user then selects the probability level that should be achieved and the storm intensity estimation method.

As an example, if the design specifications are again for a 0.5-inch storm in a Cape Cod area and the average storm intensity method is used with a 0.95 probability then a design flux, q_d, of 0.39 inches/hr would be selected. If the system drains an area, A, of 2 acres then the design flow rate, Q_d, would be:

$$Q_d = q_d A = 0.39 \text{ in/hr X 2 acres X 453 gpm/ac-in/hr} = 353 \text{ gpm}$$

A design flow rate of 353 gpm would be accommodated in 95% of storms and exceeded in 5% of storms.

Flow Volume Probability Tables

Table 1: Design Flux (inches/hour) During the First 0.5 inch of Rainfall for Cape Cod Areas Based on Probability Level and Storm Intensity Estimation Method.

	Probability That Storm Intensity is Less Than Indicated Design Flux			
Storm Intensity Estimation Method	**0.75**	**0.90**	**0.95**	**0.99**
Average Storm Intensity (in/hr)	0.18	0.29	0.39	1.05
Maximum Storm Intensity (in/hr)	0.37	0.69	0.97	1.83

Table 2: Design Flux (inches/hour) During the First 0.5 inch of Rainfall for Mainland Areas Based on Probability Level and Storm Intensity Estimation Method.

	Probability That Storm Intensity is Less Than Indicated Design Flux			
Storm Intensity Estimation Method	**0.75**	**0.90**	**0.95**	**0.99**
Average Storm Intensity (in/hr)	0.19	0.37	0.76	1.39
Maximum Storm Intensity (in/hr)	0.37	0.71	1.14	2.23

Table 3: Design Flux (inches/hour) During the First 1.0 inch of Rainfall for Cape Cod Areas Based on Probability Level and Storm Intensity Estimation Method.

	Probability That Storm Intensity is Less Than Indicated Design Flux			
Storm Intensity Estimation Method	**0.75**	**0.90**	**0.95**	**0.99**
Average Storm Intensity (in/hr)	0.15	0.28	0.38	0.93
Maximum Storm Intensity (in/hr)	0.59	1.03	1.40	2.27

Table 4: Design Flux (inches/hour) During the First 1.0 inch of Rainfall for Mainland Areas Based on Probability Level and Storm Intensity Estimation Method.

	Probability That Storm Intensity is Less Than Indicated Design Flux			
Storm Intensity Estimation Method	**0.75**	**0.90**	**0.95**	**0.99**
Average Storm Intensity (in/hr)	0.16	0.30	0.52	1.74
Maximum Storm Intensity (in/hr)	0.57	1.12	1.67	3.04

Background

Increased concerns about non-point pollution and the impact of urban drainage on water quality have prompted the development of new technologies and regulatory guidelines for treatment of stormwater (Field, et al., 1998). These guidelines are often applied on a statewide, county or local basis. The requirements for treatment and the volume of water requiring treatment vary from one locale to another. One approach is to require that water quality treatment be applied to the rainwater that falls early in a storm event. This “first flush” water must be treated to meet certain water quality requirements.

The use of a first flush standard presumes that most of the pollutant load is carried by the early flow. However, the amount of pollutant transported from the ground surface to the receiving waters can depend on a number of factors including the antecedent conditions, individual storm intensity pattern and site-specific drainage characteristics. Some recent studies have suggested the pollutant loading may follow a different pattern than water loading. Lee and Bang (2000) studied urban stormwater runoff in nine watersheds in Korea and reported that for watersheds less than 100 hectares the peak in pollutant load preceded the stormwater flow peak. The reverse was observed for larger watersheds. Bertrand-Krejewski, et al. (1998) examined several first flush definitions for 12 sewer systems in France and found a wide variation in the timing of the pollutant load. Delectic (1998) studied water quality for two catchments in Europe and reported only a slight first flush effect for suspended solids and conductivity. Despite the imperfection of a first flush standard it is used and valued for its relative simplicity.

Structural stormwater BMP technologies can be broadly classified as detention or flow-through devices. Volume-based designs typically consist of a detention pond or similar structure where stormwater is collected. Sizing of a detention pond can be accomplished by determining the volume that must be detained and the amount of sedimentation that must occur to achieve treatment objectives (Haan et al., 1994). Flow-through technologies are based on treatment of stormwater as it passes through the system. These technologies typically have little or no storage capacity relative to the volume of water that passes through them during a storm event. Sizing of these devices generally requires a design flow rate.

In this work, we analyze the relation between a first flush depth and resulting flow rate based on historical precipitation patterns in Massachusetts. The Massachusetts Stormwater Management Policy (1997) includes a requirement that, on an average annual basis, the first 1.0-inch of runoff must be treated for critical areas and the first 0.5 inches must be treated for all other areas. This report is intended to provide a relationship between these storm depths and corresponding flow rates to a treatment or collection structure. The results are presented in terms of the probability that the flow rate per unit area will be less than a specified value. The methodology described here may be transferable to other locales with similar guidelines for stormwater treatment.

Analysis Procedure

The setting under consideration is one in which stormwater drains from an impermeable area, such as parking lot or rooftop, into a collection point where it is subject to treatment. Relating the rate at which water flows to the collection point and the volume of water collected requires that a time during which the flow accumulates be specified. In the most general form the volume of accumulated storm water, V, is given by

$$V = \int_{t=0}^{t=T} Q(t)dt \qquad (1)$$

where *Q(t)* is the instantaneous inflow rate to the collection point, and *T* is the time required to reach a volume *V*. By dividing both sides by the impermeable area that drains to the treatment device, *A*, the relation can be described in terms of accumulated depth, *d*.

$$d = \int_{t=0}^{t=T} q(t)dt \tag{2}$$

$$d = V/A \tag{3}$$

$$q(t) = Q(t)/A \tag{4}$$

where *q(t)* is the flow rate per unit area. Both *q(t)* and *Q(t)* may vary with time.

For purposes of sizing of a treatment device we assume that the designer seeks a design flow rate Q_d, which can be computed as $q_d A$ where *A* is the site specific impermeable area and q_d is the design flow rate per unit area or the design flux. In this report we analyze for values of the design flux, q_d. We assume that the interception storage and evaporation are negligible during the storm event. This implies that depth of accumulated inflow, *d*, is the same as the depth of rainfall.

McKay and Wilks (1995) report that, for Massachusetts, the average intensity over 1 hour is approximately 2.2 in/hr for a storm with a 100-year return period. For storm events such as this, with extremely high intensities, the design depth may be achieved rapidly requiring a high value of q_d. Storms with moderate intensities will correspond to lower values of q_d that may not be sufficiently protective of water quality for higher intensity events. Because of the variability in characteristics of individual storms there exists a relation between the value of q_d specified at a site and the reliability of the system. In this paper we derive this relation based on a statistical analysis of historical rainfall data.

Following the standards used by Massachusetts, the depth, *d*, is specified as either 0.5 inches or 1.0 inch. The task is to relate q_d to *d* for those conditions found in Massachusetts. We report results for two definitions of q_d. The first is the average intensity over the entire storm up to the point at which the design depth is achieved. The second is the maximum 15-minute intensity observed during the storm up to the point when the design depth is achieved. The appropriate definition may depend upon the specific characteristics of the flow-through device and the ability of the system to attenuate peaks in flow with appropriate storage.

Data Used in Analysis

Historical precipitation records of accumulated precipitation depth at 15-minute intervals were used for this analysis. The data was obtained from the National Climatic Data Center (NCDC) of the National Oceanic and Atmospheric Administration and spanned the period 1971 to 2000. The data is quality controlled by NCDC using spatiotemporal techniques in a variety of ways. Any storm records that contained an NCDC error flag where not used in this analysis.

For a given storm, the NCDC 15-minute data indicates the time of the beginning of the storm and the depth that accumulates at each 15-minute interval that follows for the duration of the storm. The average intensity in each interval is determined by dividing the depth accumulated during the 15-minute interval by the elapsed time. To estimate the time at which the design depth is achieved it is necessary to make an assumption about the variability of the intensity within the 15-minute interval. For this purpose we assume that the rainfall intensity during each 15-minute interval is constant.

The general procedure used here is to calculate an intensity measure for each storm at each station in the record. We organize our analyses of these intensity measures by assuming that they are each realizations from one of two populations. The two populations considered are distinguished by the geographic location of the stations as described below. Within each population all storms, regardless of station or year of occurrence are combined and analyzed to infer frequency distributions and the probability that a specified intensity will be exceeded.

The results are presented as a frequency distribution of the design flux at the collection point. However, the results are derived from the frequency distribution of rainfall intensities over the drainage during the time required to achieve the design depth. We are, in effect, assuming that the hydrograph at the collection point has the same shape and peak magnitudes as the hyetograph over the drainage area. This implies that the topography, geometry and surface characteristics of the area that drains to the collection point produce a surface routing with only a shift in the timing of peaks but not a change in magnitude.

Determining Average Intensity

To determine the average storm intensity up to the time when the design depth is reached we define $q(t)$ as a constant, q_c. Under these circumstances Equation (2) reduces to

$$d = q_c T \tag{5}$$

It is then sufficient to find T for a given value of d and solve for the constant flux, q_c. The procedure is to search the historical record for every storm in which the total storm depth exceeds the target depth, determine the time at which the target depth was reached and solve for q_c. The resulting values of q_c are assembled into a frequency distribution. This in turn is used to produce relationship between the constant flux and reliability.

We define $T_{0.5}$ and $T_{1.0}$ as the time required for a given storm to reach 0.5 inches and 1 inch respectively. It is assumed that the rainfall intensity is constant over each 15-minute interval with the intensity based on the accumulated depth at the end of the interval. Using this assumption, the accumulated precipitation can be estimated on a continuous basis. The intensity, in inches per hour, during the j^{th} 15-minute interval is computed as

$$I_j = \frac{d_j}{0.25} \tag{6}$$

where I_j is the average intensity during the j^{th} interval and d_j is the depth that has accumulated during the j^{th} interval. The value of $T_{0.5}$ is found by first determining the interval during which the accumulated depth exceeds 0.5 inches. That is, the value of n such that

$$d_1 + d_2 + \ldots + d_{n-1} + d_n \geq 0.5 \tag{7}$$

Since the target depth is reached at some time during the n^{th} interval an interpolation must be performed. This is conducted by assuming that the intensity is constant during this time interval. The additional depth, d_n^t, required during the n^{th} time interval to achieve the target depth is given by

$$d_n^t = 0.5 - (d_1 + d_2 + \ldots + d_{n-1}) \tag{8}$$

and the time required, T_n, to achieve this additional depth is given by

$$T_n = \frac{d_n^t}{I_n} \tag{9}$$

Finally, the value of $T_{0.5}$ is computed as

$$T_{0.5} = T_n + 0.25(n-1) \tag{10}$$

A similar procedure is used to compute $T_{1.0}$.

The 0.5 inch and 1.0 inch constant intensity fluxes for each storm are computed by

$$q_c^{0.5} = \frac{0.5}{T_{0.5}} \tag{11}$$

$$q_c^{1.0} = \frac{1.0}{T_{1.0}} \tag{12}$$

For each storm, the time in which the depth has been achieved is determined and the constant flux computed. Accumulating this data produces a histogram of fluxes. Normalizing by the number of storms in the record that exceed 0.5 or 1 inch yields the probability that the required flux will be within the indicated histogram bin. Summing these probabilities yields the cumulative probability that a constant flux will be less the indicated value.

Determining Maximum Observed Intensity

From the analysis described above the value of $T_{0.5}$ and $T_{1.0}$ for each storm can be determined. In addition the determination of the intensity during each 15-minute interval of the storm has been described. The procedure to be used is to record the maximum intensity of each storm, compute the associated flux and assemble these fluxes into a histogram. The fluxes determined by maximum observed intensity, $q_m^{0.5}$ and $q_m^{1.0}$, are defined as

$$q_m^{0.5} = T_m^{0.5} I_m \tag{13}$$

$$q_m^{1.0} = T_m^{1.0} I_m \tag{14}$$

where the m subscript indicates the time and intensity associated with the time interval associated with the maximum intensity.

Results of Data Analysis

Clustering Station Data

Examination of the distributions of constant intensity fluxes for the various stations in this study showed that many of them have a similar structure and revealed some outliers. The average number of 0.5-inch storms per year is about 22.9. Three stations had an average number well below this value. These stations, Knightville Dam, Lanesboro and Norwood, were discarded for this reason.

The remaining stations were grouped based on their mean constant intensity flux. Two distinct groups were hypothesized based on this examination: stations on Cape Cod and all other stations (referred to as mainland stations). For each of these groups a series hypothesis tests were conducted. The null hypothesis is that the mean flux at each station is the same as the mean of all fluxes at all stations within the group. The hypothesis was tested using the student t statistic assuming an unknown variance. The hypothesis was accepted at the 1% significance level for all three stations in the Cape Code group Edgartown, Hyannis, and Provincetown. The hypothesis was accepted at the 1% significance level for all stations in the mainland group expect for West Brimfield, East Brimfield Lake and Groveland. These three stations were discarded from the remaining analysis.

Table 1 shows the final list of stations used, grouped into Mainland and Cape Cod groups, along with the length of record at each station and the numbers of 0.5 inch and 1.0 inch storms over the entire record at each station and the total number of storms in each of the geographic regions. Figure 1 shows the locations of the stations used in the analysis.

Table 1: Massachusetts Precipitation Stations Used in Analysis with Length of Record and Number of Storms in Record

Station Name	First Year of Record	Last Year of Record	Number of 0.5" Storms	Number of 1" Storms
Mainland Group				
Amherst	1979	2000	516	196
Barre Falls Dam	1971	2000	749	277
Becket 2 SW	1978	1989	281	91
Bellingham	1984	2000	440	168
Birch Hill Dam	1984	1997	352	128
Bridgewater	1971	2000	649	266
Buffumville Lake	1971	2000	807	334
Littleville Lake	1971	2000	747	294
Marblehead	1984	2000	409	150
Petersham 3 N	1971	1997	720	250
Rockport 1 ESE	1971	1983	272	92
Mainland Group Totals			5942	2246
Cape Cod Group				
Edgartown	1991	1997	164	57
Hyannis	1984	1999	360	130
Provincetown	1984	2000	432	155
Cape Cod Group Totals			956	342

Constructing Frequency Diagrams

Once the outlier stations were eliminated, and the remaining stations clustered, a series of cumulative frequency diagrams were constructed. Diagrams were created for average and maximum intensity and for 0.5 inch and 1.0 inch storms. The bin intervals used in constructing the frequency diagrams were based on consideration of the resolution of the data. Most of the data used in this analysis is reported with a resolution of one tenth of an inch per 15-minute interval. Once multiplied by four, to yield units of inches/hour, the resolution possible is in increments of 0.4 inch/hr. Hence, for the tenth inch data, maximum intensities can only be 0.4, 0.8, 1.2, etc. inch/hr. For the maximum intensity frequency diagrams the interval used were 0-0.2, 0.2-0.6, 0.6-1.0 and continuing with interval widths of 0.4 in/hr. That data collected at a resolution of one hundredth of an inch produced some maximum occurrences in the 0-0.2 interval. The average intensity computed from the procedure above can result in many possible values even when the individual 15-minute data has limited resolution. The bin interval used for average intensity storms was 0.1 in/hr.

Figures 2 through 5 show the histograms and cumulative frequency diagrams for 0.5 inch, 1.0 inch for both mainland and Cape Cod areas. The solid line in each figure is the cumulative frequency for the mainland groups. The dashed line is the cumulative frequency for the Cape Cod group. The figures can be interpreted as providing the frequency of occurrence of storms with a rainfall intensity less than the indicated value. For example, Figure 2 indicates that for the mainland group of stations the intensity is less than or equal to 0.37 inches/hr in 90% of storms.

Discussion of Results

The intensities are lower for the Cape Cod group than the mainland group at a given frequency level indicating that Cape Cod stations tend to have lower mean intensities. Examination of the difference in the curves across the figures indicates that the maximum intensities are generally two to three times higher than the average intensities. As the first flush depth increases the portion of the storm that is examined for maximum intensity increases. The data reflects this in the increase in storm maximum intensity with larger first flush. For the Cape Cod group, the average intensity is largely unaffected by the first flush depth and corresponding length of storm. However, for the mainland group, the longer 1.0 inch storms tend to have lower average intensities than the 0.5 inch storms indicating that storms with greater depth tend to have small intensities.

Designers of stormwater treatment systems can use these results to relate depth-based first flush regulatory requirements to design flows. To use these results in practice the user needs to identify the appropriate storm depth, either 0.5 inch or 1.0 inch, the location of the structure under consideration, either Cape Cod or mainland, and the storm intensity estimation method, either average or maximum. These three factors will indicate which of figures 2 – 5 is appropriate and which curve should be used. The analyst then selects the appropriate frequency level. As an example, if the design specifications are again for a 0.5-inch storm in a Cape Cod area and the average storm intensity method is used with a 0.95 frequency then a design flux, q_d, of 0.39 inches/hr would be selected. If the system drains an area, A, of 2 acres then the design flow rate, Q_d, would be q_dA or 0.39 in/hr times 2 acres which is 353 gpm. The user can conclude that a design flow rate of 353 gpm has been accommodated in 95% of past storms and exceeded in 5% of storms.

Conclusion

Frequency analysis has been conducted on 15-minute precipitation data from 14 stations in Massachusetts with data spanning 29 years. Data was available from over 9000 individual storm events. These events were divided into storms that reached 0.5 inch and 1.0-inch depths and those located in the Cape Cod area and on the mainland. Within each of these data groups all storms were assumed to be realizations from a single population. The frequency of storms of specified intensities were identified for each of these populations. The results of the analysis have been presented in the form of cumulative frequency diagrams.

The results of this work can be used for relating a specified storm depth to a flow rate at a stormwater treatment collection point. This may useful for the design of flow-through treatment devices. The methodology used here may be transferable to other regions. As with any analysis based on historical data, the utility of these results for predicting future performance of treatment systems is only as good as the ability of past precipitation data to represent future precipitation patterns. Any long term trend or changes in precipitation patterns reduces the reliability of past data for predicting future behavior.

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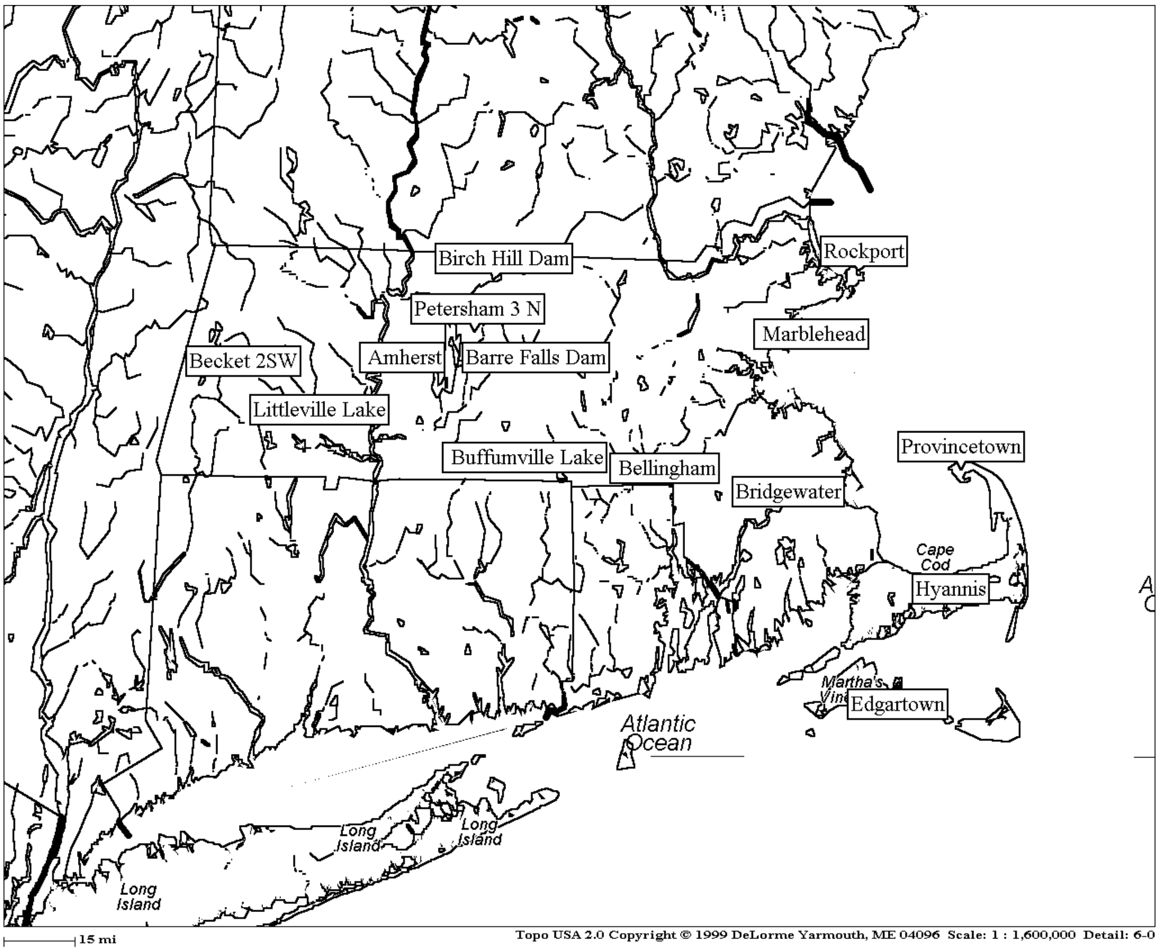
Appendix 2A: Figures

Figure 1: Location of Stations Used in Frequency Analysis Study



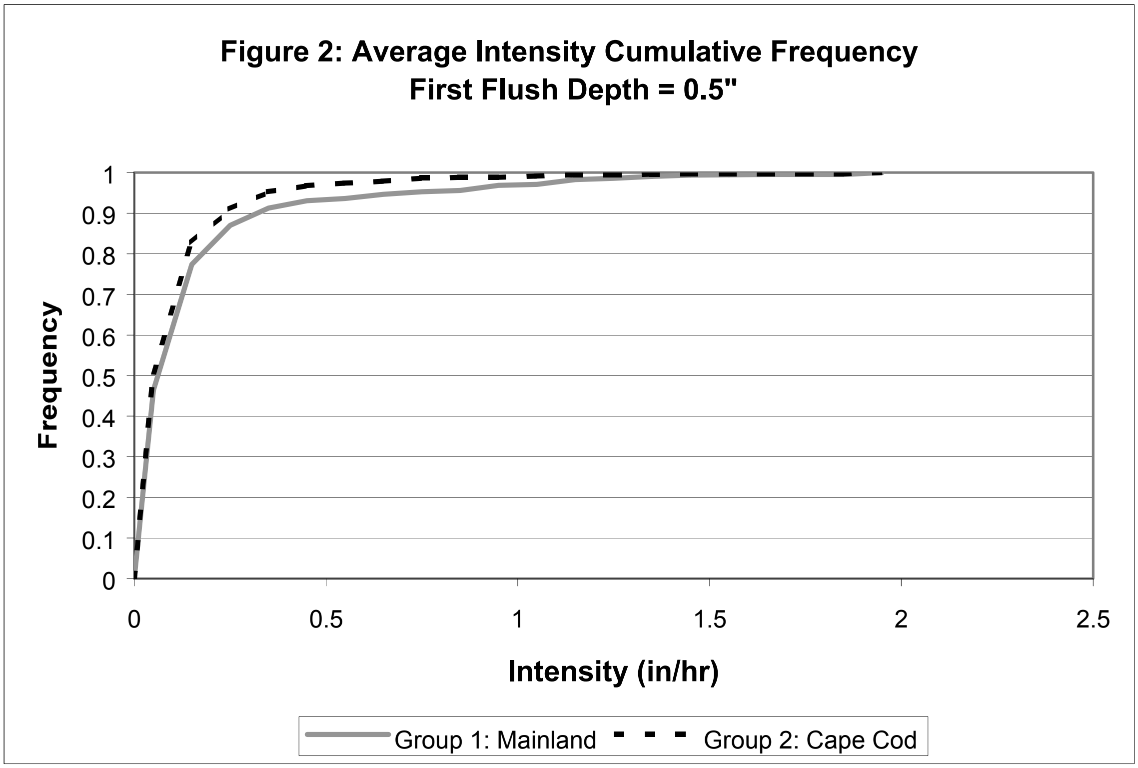


Figure 2: Average Intensity Cumulative Frequency
First Flush Depth = 0.5"
Frequency
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.5
1
1.5
2
2.5
Intensity (in/hr)
Group 1: Mainland
Group 2: Cape Cod

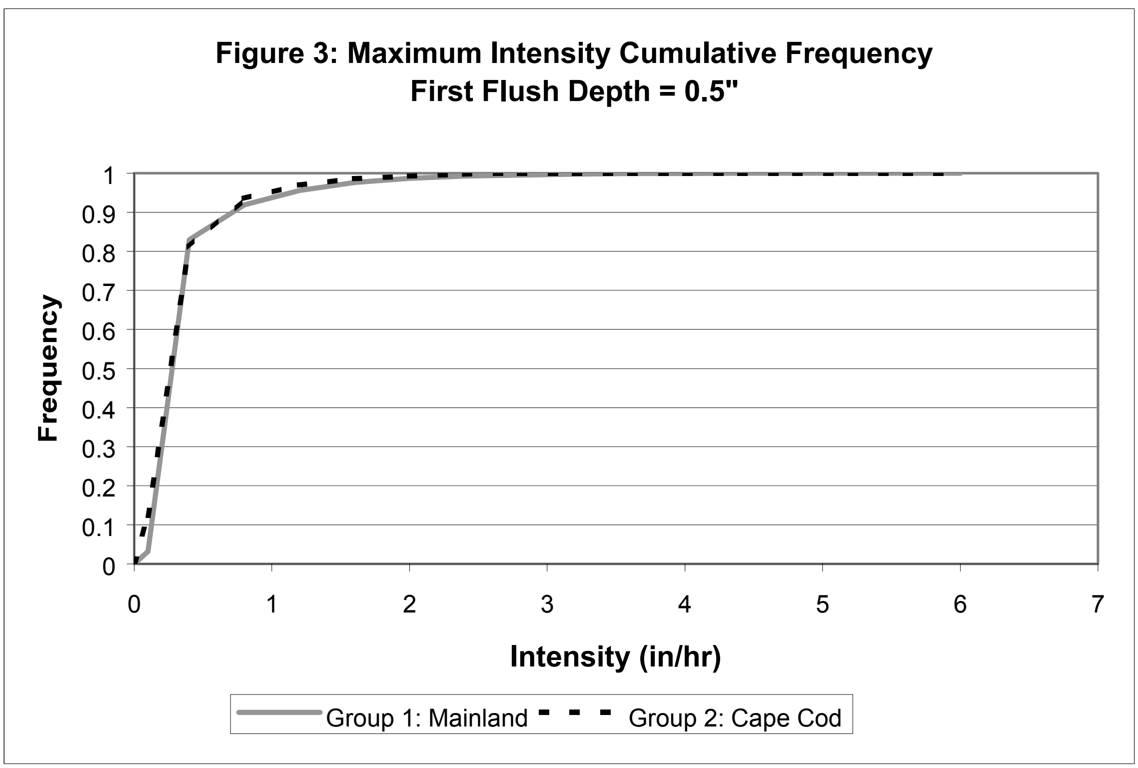


Figure 3: Maximum Intensity Cumulative Frequency
First Flush Depth = 0.5"
Frequency
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
5
6
7
Intensity (in/hr)
Group 1: Mainland
Group 2: Cape Cod

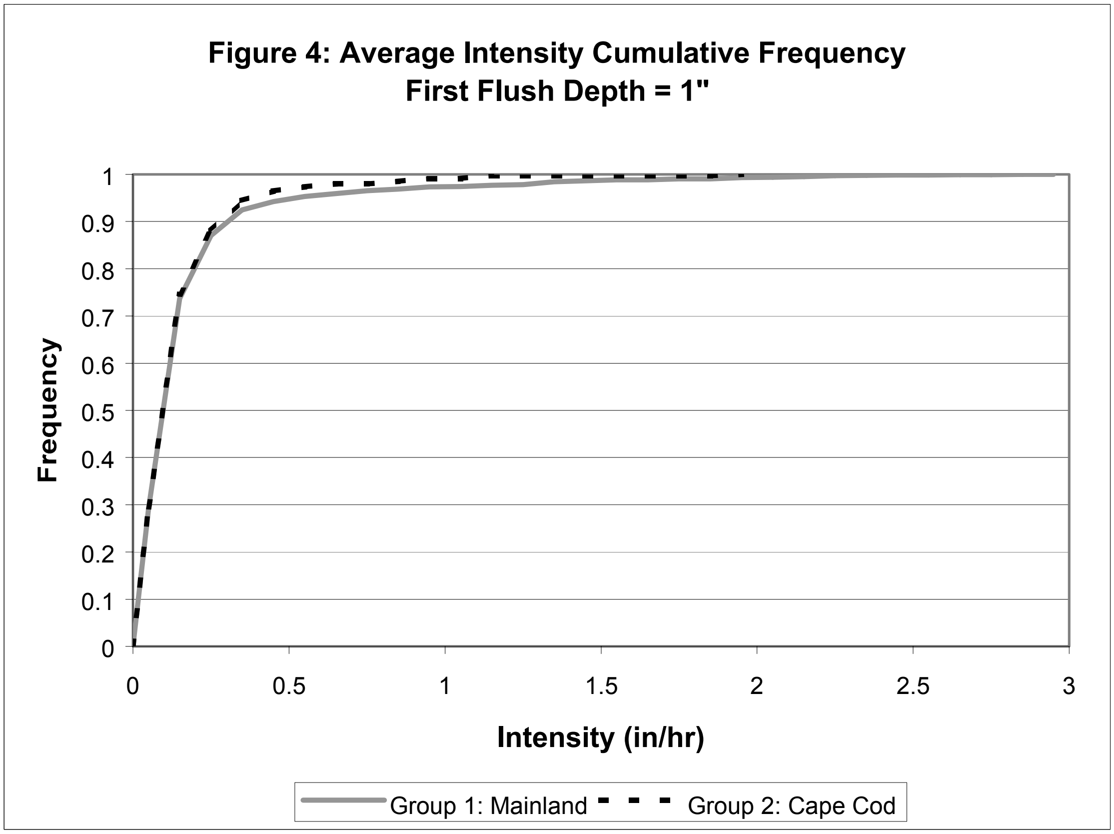


Figure 4: Average Intensity Cumulative Frequency
First Flush Depth = 1"
Frequency
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.5
1
1.5
2
2.5
3
Intensity (in/hr)
Group 1: Mainland
Group 2: Cape Cod

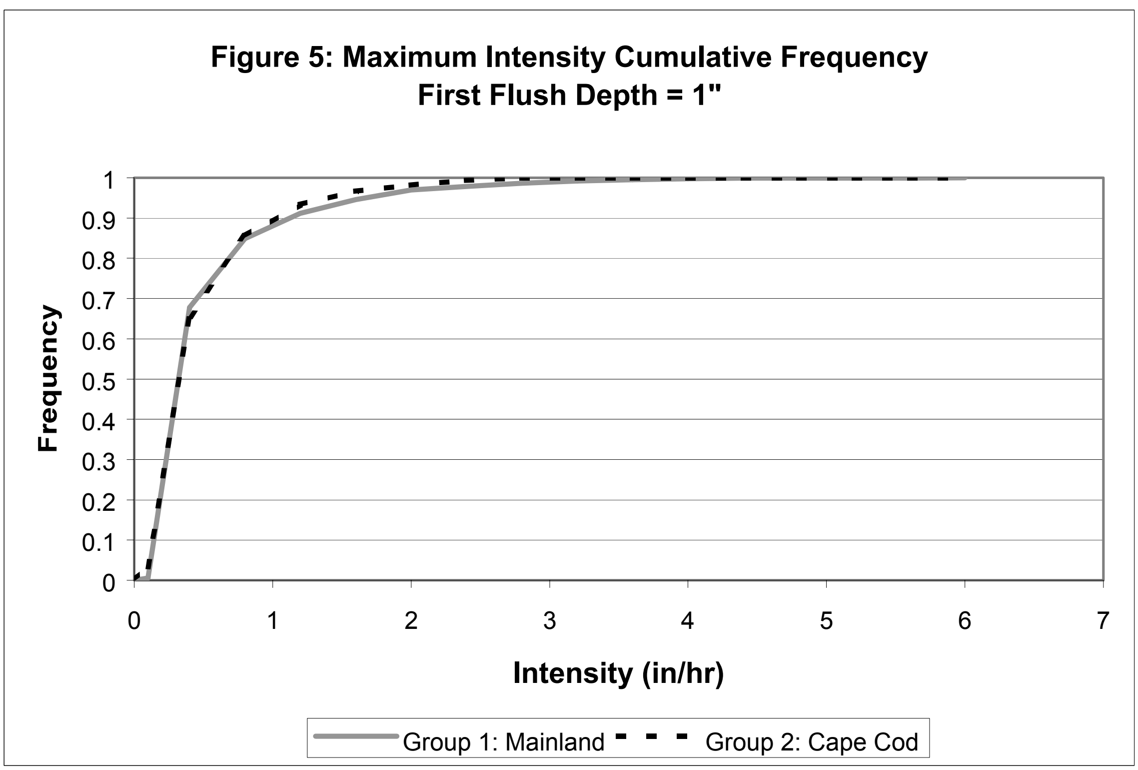


Figure 5: Maximum Intensity Cumulative Frequency
First Flush Depth = 1"
Frequency
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
1
2
3
4
5
6
7
Intensity (in/hr)
Group 1: Mainland
Group 2: Cape Cod

Appendix 2B. Glossary of Terms

A	Drainage area
d	Depth
I	Rainfall intensity
$q_c^{0.5}$	Constant flux 0.5 inch depth
$q_c^{1.0}$	Constant flux 1.0 inch depth
q(d)	Design flow rate per unit area
Q(d)	Design flow rate
q(t)	Flow rate per unit area
Q(t)	Instantaneous inflow rate
T	Time
V	Volume