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HYDROLOGIC MONITORING OF THE SEATTLE ULTRA-URBAN STORMWATER MANAGEMENT PROJECTS

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Water Resources Series
Technical Report No. 166
April 2001

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Abstract

Increased urbanization has lead to significant hydrological and ecological changes throughout the Puget Sound Lowland region. Direct physical alteration of the subsurface soil water storage capacity, natural drainage network, and land use patterns have permanently modified the hydrologic regime. The results are increased stormwater peak flow rates and volumes, increased frequency of floods, and decreased water quality in the receiving bodies of water. In an attempt to mitigate urbanization impacts, creative approaches are necessary to manage urban stormwater.

This thesis documents the monitoring of two Seattle Public Utilities (SPU) "ultra-urban" stormwater management projects. In this context "ultra-urban" is defined as any built environment within the City of Seattle, including a variety of industrial, commercial, residential, and mixed land use types. The two ultra-urban stormwater projects monitored are the Viewlands Demonstration Swale and the Street Edge Alternative (SEA) Streets Millennium Project, located in the Pipers Creek Watershed in North Seattle. The projects are designed to benefit runoff-receiving waters both in reducing stormwater quantities and improving runoff quality.

This thesis examines the hydrologic and hydraulic performance of the Viewlands swale during post-construction monitoring and theoretical pre-construction performance. The results demonstrate that the Viewland swale is limited in its ability to mitigate large amounts of stormwater (above volumes produced by a 6-month 24-hr storm), directly before reaching the natural drainage network. The main constraint on swale effectiveness is its limited soil water storage capacity and available land area. Once storage capacity is

reached over the course of a storm, the swale has minimal impact in attenuating peak flow rates or inflow volumes.

At the SEA Streets site, baseline performance is monitored and compared to the theoretical performance of both a conventional design and the constructed SEA Streets design. The dominant characteristic of the residential block is that runoff-response is precipitation-driven and fast. The post-construction performance of the SEA Streets project has yet to be determined, but the project attempts to control stormwater production at the source and in the upper watershed. Hence it focuses on the root of the problem and recognizes that the developed upper-watershed significantly impact the health of the stream.

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CHAPTER 1 – INTRODUCTION

1.1 Problem Statement

Increased urbanization has lead to significant hydrological and ecological changes throughout the Puget Sound Lowland region. Direct physical alteration of the subsurface soil water storage capacity, natural drainage network, and land use patterns occur during the clearing of native vegetation and the construction of urban and suburban infrastructure. The hydrologic regime is permanently modified as soil water storage is reduced and the impermeable area increases in the watershed. These changes result in increased stormwater peak flow rates and volumes, increased frequency of floods, and decreased water quality in the receiving bodies of water. Inevitably this leads to the fragmentation and disintegration of the riparian zone, loss of viable species habitat, and degradation of the stream ecosystem.

In an attempt to mitigate urbanization impacts, point sources of pollution have come under strict regulation in the United States through the Clean Water Act of 1972 and the Clean Air Act of 1970. Non-point sources of pollution are harder to identify and mitigate. In particular, stormwater became an important non-point pollution source due to its varied and diffused impacts throughout a watershed. Currently, Best Management Practices (BMPs) are used in watersheds to mitigate the impacts of non-point sources of pollution by delineating minimum requirements, operating procedures, and treatment protocols.

The City of Seattle has a long stormwater management history. Over the years, these efforts have become progressively more inclusive of projects intended to protect and to restore the health of the City's freshwater ecosystems. Creative approaches are necessary to manage stormwater in urban areas, since impacts from the developed watershed significantly influence the health of the stream. As such, the National Marine Fisheries Service (NMFS) requires quantitative relationships between stormwater management activities implemented in the watershed and benefits to the associated stream ecosystem.

In addition, the Washington Department of Ecology (WDOE) has indicated that the City's next stormwater National Pollutant Discharge Elimination System (NPDES) permit will include requirements to define the same types of quantitative relationships.

In the summer of 1999, the Seattle Public Utilities (SPU) established a memorandum of understanding with the University of Washington's Center for Urban Water Resources Management to assist in the evaluation of various stormwater management Capital Improvement Projects (CIP). The work under the agreement involves testing a variety of innovative "ultra-urban" stormwater management techniques and documenting their benefits with quantitative data. In this context "ultra-urban" is defined as any built environment within the City of Seattle, including a variety of industrial, commercial, residential, and mixed land use types. The first stormwater management projects proposed for testing apply mainly to single-family residential and neighborhood commercial areas.

1.2 Research Objectives

The broad objectives of the series of ultra-urban studies are to:

- Determine how effective the selected projects are in reducing peak rates and volumes of runoff;
- Evaluate receiving water ecosystem benefits that could be achieved with widespread application of these project types; and
- Develop a long-term, systematic approach to ultra-urban stormwater management in Seattle.

Ideally, evaluating a project's performance entails making hydrologic comparisons before (baseline period) and after its construction. Since there was limited opportunity for pre-construction environmental monitoring, project performance was gauged using "hindcasts" of hydrologic variables under equivalent meteorological conditions to those occurring during project monitoring.

1.3 Report Structure

The two ultra-urban stormwater management projects described in this report are the Viewlands Demonstration Swale and the Street Edge Alternative (SEA) Streets Millennium Project. The projects were designed to benefit runoff-receiving waters, both in reducing stormwater quantities and in improving runoff quality.

The following chapters describe the activities performed to establish monitoring of the first two ultra-urban projects. Chapter 2 provides background on the regional characteristics of Puget Sound, hydrologic processes in forested and urban watersheds, and pertinent flow rate measuring equations. Chapter 3 describes the Pipers Creek Watershed and the ultra-urban sites, which are contained within it. Chapter 4 discusses the monitoring and instrumentation site plans. Chapter 5 details the management and quality assurance/ quality control (QA/QC) methods applied to the data, and Chapter 6 discusses the data analysis methodology. In Chapter 7, the results of field monitoring are detailed and hydrologic comparisons are made between the collected data and the modeled pre- and post-construction conditions. Chapter 8 provides a summary, conclusions, and recommendations for future work.

CHAPTER 2 – LITERATURE REVIEW

2.1 Regional Characteristics

Seattle is located in King County, Washington. It lies between Puget Sound on the west and Lake Washington on the east. The population of Seattle is 540,500 (1999 census) and 3,190,000 in the greater metropolitan area. The city is a focal point of a highly urbanized region, with extensive suburban development to the north, south, and east.

Precipitation in the Puget Sound region is characterized by low-intensity, long-duration storms mainly in the form of rain. The average annual total precipitation for the region ranges from 34 to 38 in (864-965 mm), with maximum monthly precipitation occurring in December and January (Sea-Tac). Due to orographic effects from the Olympic mountains to the west and the Cascade Range to the east, as well as temperature regulation from the Pacific Ocean, the Puget Sound climate is characterized by mild, wet winters and dry, warm summers.

The dominant soil type of the Puget Sound Lowlands is glacial till overlain by a shallow layer of topsoil. Glacial till in the Puget Sound Lowlands has an estimated saturated hydraulic conductivity that ranges from 6 to 12 in/yr (15 to 30 cm/yr) under a 1:1 hydraulic gradient (Olmsted, 1969). This means that if the soil above a till layer remains fully saturated all year, between 15 and 30 cm (6 to 12 in), on average, will reach underground aquifers.

Due to these regional characteristics, the Puget Sound Lowlands naturally generate hydric conditions in areas of low relief with large amounts of subsurface flow, especially after extensive periods of rainfall. However human-induced changes, such as increased urbanization, have lead to drastic modification of the natural system. In attempts to mitigate the hydrologic changes caused by urbanization, restoration of the natural infiltration and storage regimes in the watershed have become major management goals.

2.1.1 Storage of water

The storage and subsequent slow release of water from the soil column is an important component of the hydrologic system. Storage is typically defined by the water balance or continuity equation, which describes the inputs, outputs, and storage for a given three-dimensional piece of land (the control volume) (Linsley et al., 1982):

$$dS/dt = I - O$$

$$dS/dt = (P + Q_{in}) - (ET + Q_{out} + L)$$

$$dS/dt = (P + Q_{in}) - (ET + Q_{direct} + Q_{indirect} + L)$$

where:

dS/dt = change of storage with respect to time

I = volume flux into the control volume (L^3/T)

O = volume flux out of the control volume (L^3/T)

P = precipitation (L^3/T)

Q_{in} = inflow (L^3/T)

$Q_{out} = Q_{surface} + Q_{subsurface}$ = outflow (L^3/T)

L = leakage to a deeper aquifer (L^3/T)

ET = evapotransporation (L^3/T)

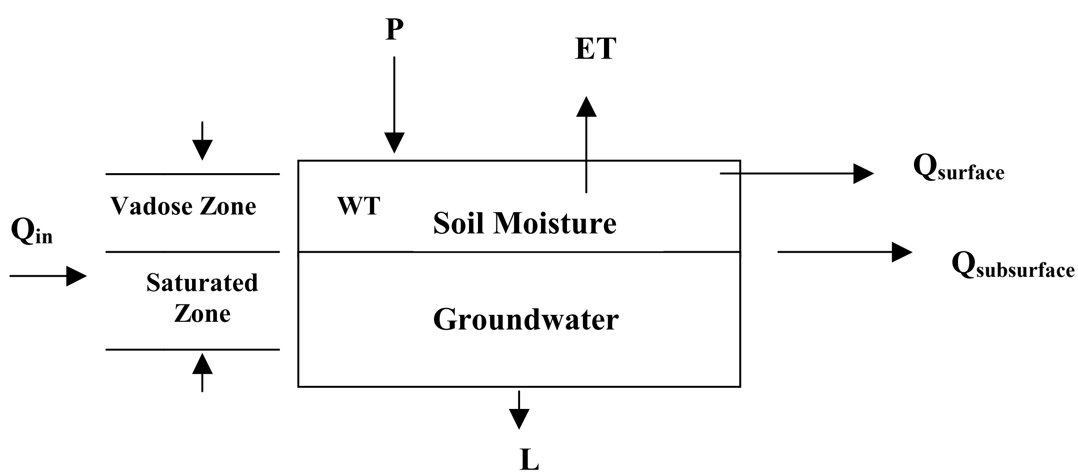

Figure 2-1: The water balance concept

The control volume can be defined at different spatial scales, from the basin scale to the plot scale (i.e. the size of a residential street or channel). Quantitative description of the physical, biological, and chemical mechanisms that define the rainfall-runoff process, is the motivating problem of hydrologists in solving the water balance. The dominant mechanisms are discussed below.

2.1.2 Natural rainfall-runoff response patterns

Consider a storm of constant a precipitation rate occurring under dry antecedent conditions on a forested, undisturbed catchment. At the start of the storm, vegetal cover intercepts precipitation. The amount of intercepted water depends on vegetation type and cover, wind speed, and evaporative loss off the wetted surface of the foliage (Linsley et al., 1982). For a well-developed forest canopy the annual interception can be up to 10 to 20 percent of the rainfall, and the storage capacity of the canopy can range from 0.03 to 0.3 in (0.8 to 7.6 mm) (Linsley et al., 1982). After interception-storage capacity is reached, rainwater is then retained in surface depressions such as puddles, ditches, and other low points in the basin. Some of the water evaporates, infiltrates to contribute to subsurface flow, or fills larger depressions that directly become surface flow.

Infiltration of rain through the subsurface increases the hydraulic residence time of the water reaching the stream system. Water can be stored in the subsurface in two ways: above the water table in the vadose zone or below the water table in the saturated zone. The vadose zone constitutes the upper part of the soil column that is partially saturated and has a negative capillary pore pressure (Freeze and Cherry, 1979). In this zone, water is stored in micropores by capillary pressure and in larger macropores. Movement down gradient occurs by gravity drainage. As water drains down through the flow paths of least resistance (the macropores), water is diverted into the micropores by capillary forces (Linsley et al., 1982).

As a storm progresses, the stored water in the vadose zone increases and the rate of infiltration decreases. The subsurface soil eventually reaches its infiltration capacity, or the maximum rate at which water can enter the soil. If the rainfall rate exceeds the infiltration rate, water ponds on the surface then travels as Horton overland (or infiltration-limited) flow. In the Puget Sound region, the relatively low rainfall intensity and the thin permeable topsoil supports principally subsurface flow to the channel. Deeper saturated subsurface flow (groundwater or baseflow) to streamflow does not fluctuate as rapidly. Basins that have relatively impermeable soils and constrained groundwater respond to storms quickly. Basins that have highly permeable soils and a large baseflow component have a slow, attenuated response to storms.

Overland flow occurs under three conditions: Hortonian overland flow; saturated overland flow where the subsurface layer becomes saturated and rain that falls on the surface travels laterally; and return flow where subsurface flow intersects the surface and travels overland. Due to variations in rainfall, infiltration, and topography, overland flow is spatially and temporally varied.

2.2 Watershed urbanization and hydrological impacts

2.2.1 Urban rainfall-runoff response patterns

Increased urbanization in a watershed leads to direct and indirect changes such as loss of soil storage, increase in impermeable area, increase in peak flow rates and volumes, and disintegration of the riparian zone and stream habitat. Initially, as vegetation is cleared, there is a loss of evapotranspiration potential, detention of water in the canopy and understory, and buffering of the soil from rainfall impact. During construction soils are compacted and regraded, which decrease the depression storage, soil hydraulic conductivity, and effective subsurface storage capacity.

Removal of permeable topsoil leaves the relatively impermeable glacial till exposed. Though till is beneficial as a foundation base due to its geomechanical properties, it can generate large volumes of surface runoff. Often during landscaping, only a 0.8 to 2 in (2 to 5 cm) layer of topsoil is replaced over the compacted till (Kolsti et al., 1995). In a three-year study of a 41.3-acre (16.7-ha) residential catchment, lawns atop glacial till (70 percent of the watershed's area) produced between 39 percent of the measured runoff in 1991 and 60 percent in 1992 (Burges et al., 1998).

The most influential result of increased urbanization is the increase in the amount of impermeable surfaces in the watershed. The total impervious area (%TIA) is defined as the sum of roads, parking lots, rooftops, and other impermeable areas in a watershed (Dunne and Leopold, 1978). Of this total, the effective impermeable area (%EIA) comprises those impermeable surfaces that are connected to a surface drainage system that flows to the natural stream network.

The distinction between %TIA and %EIA can be illustrated with a residential lot and a parking lot of the same %TIA. In the residential lot, some infiltration opportunity exists for water draining off rooftops onto surrounding lawns (%EIA < %TIA), eventually flowing to the stormwater drains. Conversely, parking lots may have up to 100 percent surface runoff (%EIA = %TIA), draining directly to stormwater drains. Hence, the transport-related impervious area (streets, parking lots, and driveways) may have a greater impact on the hydrologic regime than the rooftop-related impervious area (Schueler, 1994). In a study by the City of Olympia, the transportation system in suburban areas alone accounted for over 60 percent of the basin's %TIA (City of Olympia, 1994).

The type of land use in the watershed is also a significant factor influencing response to increased urbanization. Commercial land uses typically have 90 %TIA, negligible surface depression, and subsurface storage capacity; and so they respond rapidly to rainfall (Taylor, 1993). Residential land uses, with housing densities ranging from 1.2 to 2.9 houses per hectare, can range from 25 to 80 %TIA (Antoine, 1964). Unless a residential lawn is amended to increase the storage capacity, runoff occurs relatively quickly from the glacial till-based lawn and acts more like a parking lot than a permeable surface (Kolsti et al., 1995). Table 2-1 describes the %TIA and %EIA for various land uses (Taylor, 1983).

Table 2-1: %TIA and %EIA for various land uses (Taylor, 1993)

Code	National Standard	%TIA	%EIA	Reference
111	Low density single family (<1 unit/acre)	<15	4	a
112	Med. Density single family (1-3 unit/acre)	20	10	a
113	High density single family (3-7 units/acre)	40	25	a
114	Mobile homes	70	60	b
115	Low density multi-family (>7 units/acre)	80	72	b
120	Commercial (general)	90	85	b
121	Retail sales and services	80	72	b
123	Offices and professional services	75	66	b
124	Hotels and motels	75	66	c
131	Light industrial	60	48	d
132	Heavy industrial	80	72	c
144	Freeway right-of-way	100	99	d
151	Energy facilities	80	72	c
152	Water supply facilities	80	72	c
155	Utility right-of-way	5	1.5	c
160	Commercial facilities (general)	75	66	c
161	Educational facilities	40	27	b
162	Religious facilities	70	60	c
171	Golf courses	20	10	b
172	Parks	5	1.5	b
190	Open land (general)	2	1	c
192	Land being developed	50	37	c

193	Open space - designated	2	1	c
200	Agricultural land	5	1.5	c
300	Grassland	2	0	c
400-430	Forest lands	2	0	c
440	Clear-cut areas	5	0	c

References:

a. King County Surface Water Management (1990)
b. PEI (1990)
c. Estimate based on similar land uses
d. Alley and Veenhuis (1983)

The basin size plays a major role in influencing the degree of urbanization effects. A large basin with significant storage areas (e.g. lakes and wetlands) can attenuate runoff to a larger degree than a smaller basin. In large basins, on the order of hundreds of square miles, the travel time of the runoff to the stream system outlet is prolonged. In small basins, on the order of tens of square miles, precipitation produces runoff relatively quickly. The high percent of impervious area and short travel distances in a small basin leads to fast generation of overland flow and shortened travel times to the stream.

2.2.2 Hydrological and ecological impacts

Due to the decrease in the natural storage capacity of the subsurface, increase in impervious area, and increase in hydraulic efficiency, there is a subsequent increase in the peak flow rates produced by a storm. There is also production of new peak responses that would not have occurred in the undeveloped watershed (Booth, 1991). The runoff volumes that would have been retained in long-term subsurface storage now reach the stream system in larger quantities, at higher velocities, and faster.

As roads and drainage channels (i.e. stormwater culverts and outfalls) are constructed, hydrologic and physical barriers are created. Diversion, piping, and collection of both surface and subsurface water alter the natural flow regime. The increase of road density and artificial drainage density (DD) destroy the continuity of the natural stream network and riparian corridor. Figures 2-2 and 2-3 show the total percent impervious area (%TIA) to the sub-basin road-density and drainage density in the Puget Sound Lowland region (May et al., 1997).

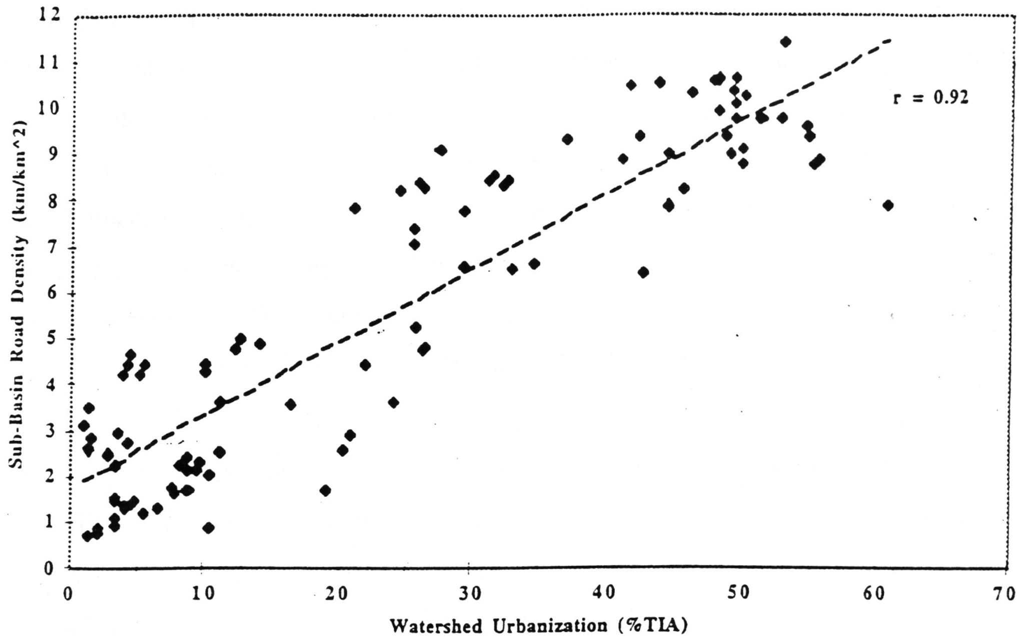

Figure 2-2: Road-density as an index of watershed urbanization (%TIA) in the Puget Sound Lowland Streams (May et al., 1997)

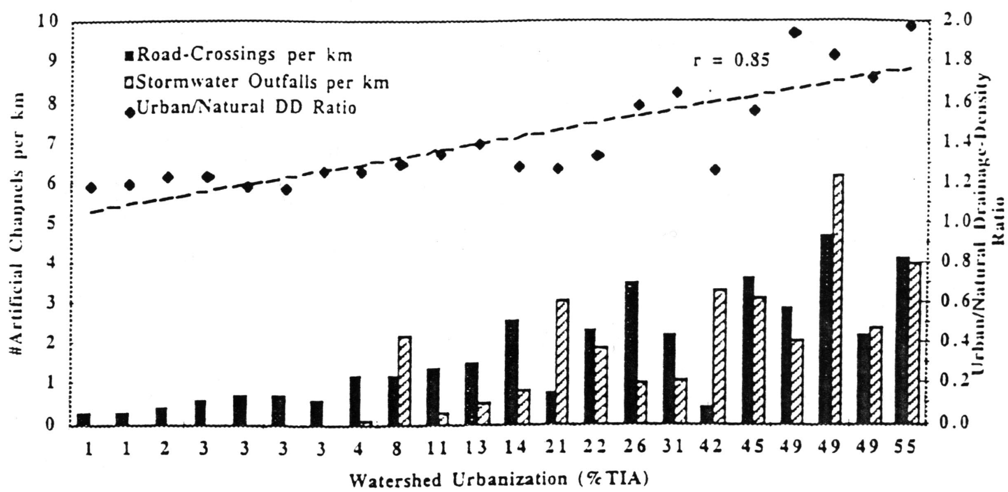

Figure 2-3: Artificial channels and changes in the natural drainage density as an index of watershed urbanization (%TIA) in the Puget Sound Lowland Streams (May et al., 1997)

Flood peak flow rates typically greater than a 5-yr recurrence interval (probability of a flood flow being equaled or exceeded in any year; 1/5 or 0.2), disturb the stream channel and can cause large-scale changes like bank erosion, wash-out of woody debris, and destruction of habitat (Booth, 1991). In undisturbed basins, these infrequent and culturally perceived "natural disasters" are natural disturbance regimes that typically have in-stream morphological benefits. As urbanization increases, peak flow rates increase as do the frequency of larger floods. The channel is unable to "recover" from the impacts of the last large flood, before another of equivalent magnitude occurs (Booth, 1991).

The large peak flow volumes and velocities lead to eroded banks that are stripped of vegetation and stability. The channel bed becomes widened, incised, and more uniform, losing the riffle/pool sequencing (Pizzuto et al., 2000). There is wash-out of large woody debris, which becomes weakly anchored and suspended above the bankfull flow during increasingly common, large floods (Booth, 1991).

All these impacts lead to lower habitat and species diversity both in the surrounding riparian zone and in the stream. For additional discussion on salmonid and macroinvertibrete impacts in the Puget Sound Lowland streams, refer to Anderson (1992), May et al. (1997), Hungington et al. (1996), and Bledsoe et al. (1989). Additional problems in urban streams result from delivery of fines and chemicals that leach from bare soil, asphalt, and lawns of the urban areas. For additional discussion on stormwater quality impacts refer to Canning (1985), Novotny and Olem (1994), and Schueler (1996).

2.3 Best-management practices

Choosing a stormwater control BMP depend on whether the objective is reducing stormwater quantity or improving stormwater quality. This report focuses mainly on a *particular set of BMP designs intended to decrease stormwater quantity. A brief discussion of the most common conveyance, detention, and infiltration systems used for water-quantity BMPs are provided in the following sections.*

2.3.1 Basin Types

Detention basins

A detention basin is a storage reservoir with a constricted outlet that releases surface flow slowly over time. Its main purpose is to attenuate the peak flow rate by increasing the hydraulic residence time of the water before it reaches the natural drainage network. This is done by mimicking the pre-development conditions and determining the volume required for a prescribed storm or a long history of storms (Ferguson, 1998); (King County Surface Water Design Manual, 1998). Conventional detention basins reduce peak flow rates but not the total volume of the surface runoff. Hence, they do not meet all the necessary requirements of reducing stormwater quantity.

There are two common types of detention basins: wet and dry extended basins. Wet detention may come in the form of a constructed pond or wetland, which acts as a temporary storage pool for surface water. Suspended particles and pollutants have time to settle out or biodegrade. A main benefit of wet detention ponds is that during the "first flush" of a storm, when the stormwater is highly polluted, the water is retained in the pond (Ferguson, 1998). Constructed ponds and wetlands are typically underlain by saturated soil or lined with concrete, so they do not achieve high infiltration rates.

Dry extended detention occurs when the conveyance or storage channels are intermittently dry between storms. Dry detention basins can be designed as both flood control and water quality control device, incorporating both detention and settling of pollutants. The major shortcoming of the dry extended basin is that during the "first flush" the residence time through the basin is short and the trapping efficiency is low (Ferguson, 1998).

Conventional detention basins, including wet and dry extended basins, fail as a mitigation strategy in a number of ways. These systems may decrease peak flow rates but do not reduce the total flow volumes through the system, and they do not match the flow durations that occurred under pre-development conditions. Booth and Jackson (1997) have noted that underestimates of the post-development conditions have occurred through the use of the Rational Method and the Soil Conservation Service (SCS) Curve-Number method. They found that by using the King County Runoff Time Series (KCRTS) model, which is based on the continuous rainfall/runoff Hydrologic Simulation Program – FORTRAN (HSPF) model, detention pond volumes would need to be up to 50 percent greater than they currently are under the SCS design to achieve stated management goals for runoff control (Booth and Jackson, 1997). In existing urban areas, the conversion of large tracts of land may be both economically and politically infeasible, since close to 10 percent of the gross area of a new development would have to be dedicated to treating stormwater (Booth and Jackson, 1997).

Infiltration basins

To achieve not only conveyance and detention objectives, a new dimension was added to stormwater basins: infiltration. Infiltration is the most complete solution to stormwater issues because it attempts to restore natural hydrologic processes (Ferguson, 1998). In addition, the upper layer of the soil column naturally filters and adsorbs chemicals, nutrients, and other pollutants.

The Washington Department of Ecology (1999) guidelines for infiltration basins are: a minimum infiltration capacity of 0.5 in/hr (1.3 cm/hr), a maximum ponding time of 24 hours, and inclusion of some form of pre-treatment (sedimentation, biofiltration, etc.). Hilding et al. (1994) performed a study on 23 infiltration basins in the Puget Sound region prior to pre-treatment requirements. The basins had a mean age of 10.6 years and a relatively high mean infiltration rate of 15.8 in/hr (40.1 mm/hr) (Hilding et al., 1994).

They found that 35 percent had heavy sediment deposition, 31-44 percent needed mowing, and 26 percent had standing water between storms. They concluded that infiltration basins were still a viable stormwater mitigation alternative, but extensive maintenance was required at an annual cost of $500 to $1000 dollars per basin (Hilding et al., 1994).

Vegetated swales

Swales are open channels with unobstructed flow. Appropriately sized vegetated swales can infiltrate, store, treat, and discharge water slowly. Vegetated swales are typically BMPs that are incorporated in residential and commercial designs because they aim to reduce peak flow rates and velocities, permit infiltration, and be aesthetically-pleasing (Ferguson, 1998). Modifying the bed gradient with grade control structures such as check dams, rip rap, and log weirs can reduce velocity through the swale. The roughness through the swale can be increased with boulders, dense vegetation, or complex bed forms (Dennison, 1996). When vegetation is used together with nonliving materials to decrease velocities and erosion potential of the stormwater as it flows through the swale, the approach is termed ***bioengineered*** (Ferguson, 1998).

Swales that are designed specifically for pollution control are termed ***biofilters***. The criterion for runoff biofiltration is a limiting velocity of 0.5 ft/s (0.15m/s) (Ferguson, 1998). At this limit, the swales hypothetically captures 63 to 83 percent of particulate pollutants, and 29 to 46 percent of metals and nutrients (Ferguson, 1998). Additionally, the swale should have a minimum hydraulic residence time of nine minutes. Descriptions of biofilter strips and highway ditches aimed at improving stormwater quality can be found in Deletic (1999), Horner (1988), and the King County Surface Water Manual (1998).

Innovative approaches in landscape architecture

Stormwater control systems become functional extensions of streams by acting as drainage corridors. Vegetated drainage corridors can attenuate, infiltrate, and treat runoff, with the added functionality of providing multiple ecological and societal benefits like providing wildlife habitat, greenways, and recreational facilities (Ferguson, 1998). They attempt to emulate natural stream systems that have the tendency to agrade, degrade, and meander toward a dynamic equilibrium within the constraints of the system.

A well-designed vegetated swale can emulate certain hydraulic characteristics of a natural system and also have the specific goal of mitigating stormwater quantity impacts on the natural stream system. Therefore, a vegetated drainage corridor may meet the requirements of stormwater conveyance, detention, infiltration, and treatment, as well as allow adaptation within the channel and provide community greenspace.

2.4 Flow measurement and V-notch weirs

To determine the stormwater quantity at the project sites, flow monitoring was done using V-notch weirs as flow measuring devices. The following sections describe characteristics and the installation and maintenance requirements for V-notch weirs.

2.4.1 Introduction

A ***weir*** is defined here as an overflow structure that is built across an open channel to measure the discharge rate of water. By creating the necessary hydraulic conditions, only one depth of water can exist in the upstream pool for a given discharge (US Bureau of Reclamation, 1997). This permits a measured water depth to be converted to a corresponding volumetric flow rate.

A ***sharp-crested weir*** is characterized by a thin sharp-edged metal plate set vertically in the channel and attached to a supporting structure called a ***bulkhead.*** The point of the V is called the ***invert*** or ***crest*** of the weir. The invert is used as the zero reference elevation. V-notch weirs are the system of choice to measure volumetric flow rate that span a wide range of flows at a given location, particularly low flow rates (Roberson and Crowe, 1993). This is because a small incremental change in the discharge rate causes a relatively large change in the upstream stilling basin water depth.

A ***fully-contracted weir*** exists when the weir pool is large enough to allow free, unconstrained lateral flow to the crest of the weir. Under these conditions, the water flows uniformly and relatively slowly, so that the kinetic energy per unit width of the approach flow becomes negligible. As water approaches and nears the weir crest, it accelerates and causes a local drop or ***drawdown*** in the water level.

The water flowing along the sides and from the bottom of the weir to the outlet causes the water to spring forward and upward, forming a jet as it passes over the weir invert (US Bureau of Reclamation, 1997). The falling water that springs free of the weir plate is called the ***nappe***. For a fully-contracted weir, the weir pool geometry does not affect the nappe geometry. But for a ***partially-contracted weir***, where the sides or bottom of the weir pool are relatively close to the invert, the contraction of the weir discharge is partially-suppressed and the nappe may cling to the weir plate.

2.4.2 V-notch weir discharge relationship

The discharge equation for a V-notch sharp-crested weir is (Roberson and Crowe, 1993):

$$Q = \frac{8}{15} * C_d * \sqrt{2g} * \tan\frac{\theta}{2} * H^{5/2} \qquad (4.1)$$

Where:

Q = volumetric flow rate (L^3/T)

g = acceleration due to gravity (L/T^2)

θ = weir angle

H = piezometric head above the invert of the weir (L)

C_d = V-notch weir discharge coefficient

The V-notch weir discharge coefficient (C_d) adjusts a hydrodynamically determined flow rate for differences in fluid properties, notch angles, and heads. The minimum value of C_d for all V-notch angles (θ) is 0.581. A value of 0.58 is usually acceptable provided that $20^o < \theta < 100^o$, H > 2in (5.1 cm), and the invert is at least twice the maximum measuring head above the weir plate (Chin, 2000). For H < 2in, viscous and surface tension effects may become important and C_d can be calculated using formula (4.2) (Chanson, 1999):

$$C_d = 0.583 + \frac{1.19}{(\mathrm{Re}^* We)^{0.125}} \tag{4.2}$$

where:

Re = Reynolds Number

We = Weber Number

Equation 4.2 applies only for a fully-contracted weir, where the channel and weir plate geometry do not affect the nappe geometry.

The Reynolds number (Re) expresses the relative influences of inertia to viscous forces and it is defined when the head over the invert of the weir is the length of interest (Ackers et al., 1978) and (Chanson, 1999).

$$\mathrm{Re} = \frac{vH\rho}{\mu} \tag{4.3}$$

where:

ρ = density of the flowing fluid (M/L^3)

μ = kinematic viscosity of the fluid (L^2/T)

The Weber number (We) reflects the relative influences of inertia to surface tension forces (Ackers et al., 1978) and (Chanson, 1999):

$$We = \frac{\rho g H^2}{\sigma} \qquad (4.4)$$

where:

σ = surface tension of the fluid (M/T^2)

2.4.3 V-notch weir installation requirements

The theoretical V-notch weir discharge equation can be applied if the following conditions are met. Any deviations from the conditions cause the true flow rate to differ from the predicted flow rate (Ackers et al., 1978); (US Bureau of Reclamation, 1997):

1. Clean water not transporting the following:
    - fine sediments in suspension that can coat the weir crest;
- coarse sediments that damage the sharp edge of the weir plate;
- pollutants that can corrode the weir;
- large debris that can build up behind the weir plate (reducing P);
2. The velocity of approach is negligible:
    - Approaching flow is subcritical, fully developed, mild in slope, and free of curves;
    - The approach channel is symmetric and the flow is normal to the weir;
3. The floor and walls of the channel are remote enough that they do not affect the geometry of the nappe:
    - The upstream bottom edge of the weir plate and fastener projections should be located a distance of at least twice the maximum measuring head ($2H_{max}$) from the invert of the weir;
    - The minimum width of the approach channel (B) should be located a distance of at least ($2H_{max}$) from the invert of the weir;
    - The flow measuring devices should be located ($4H_{max}$) upstream of or a distance of ($2H_{max}$) sideways from the outer edge of the notch;
4. The upstream face of the bulkhead is plumb and normal to the axis of the channel;

5. The upstream face of the weir plate is plumb, smooth, and flush with the upstream face of the bulkhead;
6. Flow over the invert is at free-flow conditions;
7. The nappe springs clearly from the weir plate:
   - Crest thickness (t) is at a maximum of 0.04 in (1 mm);
   - The weir plate should be smooth to within 0.8 in (2 cm) of the crest;
   - The upstream top edges of the weir plate should be straight and sharp;
   - The downstream crest face is angled to 60^o from the vertical;
8. Head measurements should be greater than 0.2 ft;
9. Maximum measuring head should be less than 1.25ft (0.38 m); and
10. The maximum downstream water surface level should be at least 2.4 in (6.1 cm) below the invert elevation.

If the above design requirements are not met, equations 4-1 to 4-4 used to calculate the volumetric flow rate are inaccurate and field testing of the weir is needed.

2.4.4 Maintenance

To maintain the weirs, a number of steps should be taken. First, the weir pool should be free of weeds, debris, and trash. Therefore, the weir pool should be dredged at regular intervals. Particular care should be taken at the base of the weir, to insure that the invert of the pool is at a depth of ($2H_{max}$) below the invert of the weir.

Second, the weir plate should be resistant to corrosion or chemical deterioration. In addition, debris carried in the flow should not build up or damage the weir crest (Ackers et al., 1978). Any dents or irregularities at the crest of the invert lead to clinging of the nappe. Therefore irregularities should be carefully dressed with a fine-cut file, stroking only in the plane of the upstream face of the weir plate or on the plane of the beveled surface of the weir plate (US Bureau of Reclamation, 1997).

Finally, inspections should be made to determine if leakage occurs under or around the weir structure. If leakage is occurring, immediate remedial action must be taken to prevent undercutting of the weir structure. This may entail the use of geosynthetic liners upstream of the structure or a redesign of the structure itself. A check must then be made to make sure the weir is level and that the invert still corresponds to the zero reference of the measuring gauge.

CHAPTER 3 – WATERSHED AND SITE DESCRIPTION

3.1 Catchment description

The Pipers Creek Watershed is located in northwest Seattle, Washington (Figure 3-1). It is bordered on the west by the Puget Sound, on the east by the Densmore Creek Watershed, and on the north by the Broadview Watershed. The Pipers Creek Watershed has an area of 2.9 square miles (7.5 km^2), with an estimated road density of 22.9 miles of road per square mile of watershed (14.2 km/km^2) and a stream density of 1.63 mile of stream per square mile of watershed (1.0 km/km^2). The dominant land uses are single-family residential (86.2 %), multi-family residential (7.1 %), and neighborhood/ commercial (6.7 %). Appendix A provides the GIS specifications that were used in determining the watershed dimensions and land use characteristics.

The stream network of interest is Pipers Creeks, which flows through Carkeek Park and drains into the Puget Sound. Pipers Creek has an estimated flow length of 4.7 mi (7.6 km) and a riparian zone of 0.1 mi^2 (0.2 km^2). Carkeek Park is approximately 0.3 mi^2 (0.8 km^2) and contains areas of high landslide potential of up to 49.5 percent of the park (0.14 mi^2; 0.4 km^2). Two sites within the basin, the Viewlands Demonstration Swale and the 2nd Avenue SEA Streets Project, have been monitored to determine their hydrological behavior within the highly urbanized Pipers Creek Watershed.

3.2 Viewlands site description

The Viewlands Demonstration Swale is located in the Pipers Creek watershed adjacent to the Viewland Elementary School. The length of the channel runs along the school property on the residential block of NW 105th Street. The upstream end is bordered by Third Avenue NW and the downstream end is bordered by Carkeek Park. The Viewland Demonstration Swale site is located on a ridge above Carkeek Park, adjacent to a hill slope of high landslide potential. Approximately 26 acres (0.1 km^2) of the Pipers Creek watershed drains into the swale, which ultimately flows into Pipers Creek.

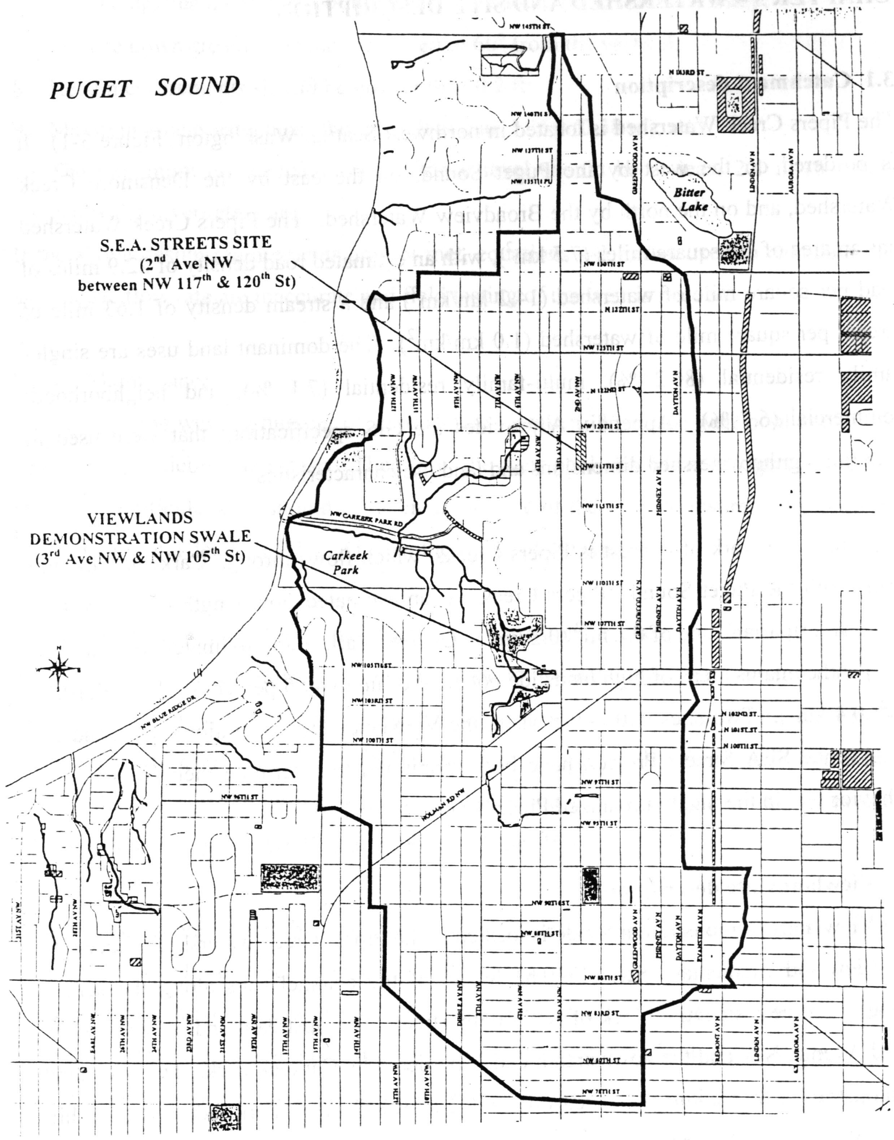

Figure 3-1: The Pipers Creek Watershed in Seattle, Washington

3.2.1 Site history

Historically, large stormwater flow rates, with an estimated maximum peak flow rate of 25 cfs for a 25-yr 24-hr storm, were discharged at velocities estimated at up to 15 ft/s into a constructed open drainage channel (Figure 3-2). The drainage channel was approximately 175 ft (53.3 m) long and 3 ft to 6 ft (0.9 to 1.8 m) wide. The first 55 ft (16.8 m) section was an open concrete channel, and the last 120 ft (36.6 m) section was a vegetated ditch. After stormwater passed through the channel, it entered a grated drain inlet box and was tight-lined down a steep bank to Pipers Creek.

The channel was designed and built as a vegetated swale in 1990, and rebuilt in 1994. Due to a lack of maintenance coupled with high flow rates and sediment loads during storms, the effluent grate was frequently blocked with debris. This led to the bioswale being overtopped and a channel being incised. Substantial soil erosion occurred from the top of the bank just west of the drain inlet down to Pipers Creek. The results were elevated flow rates, decreased water quality, and high sediment loads into Pipers Creek.

3.2.2 Design objectives for the Viewlands Demonstration Swale

Due to the interest of Peggy Gaynor, a landscape architect who resides on NW 105th Street, an alternative stormwater conveyance design was proposed to Seattle Public Utilities. The proposal replaced a planned concrete pipe design that would have conveyed the stormwater directly to the metal grate at the downstream end of the block. Though the original plan was a conventional stormwater conveyance system, it would not have reduced the high stormwater velocities or volumes expected.

Figure 3-2: Pre-construction at the Viewlands Demonstration Swale site (facing upstream towards NW 105th Street)

Figure 3-3: Post-construction at the Viewlands Demonstration Swale site (facing upstream towards NW 105th Street)

The new Viewlands Demonstration Swale is not a vegetated swale design. Instead, the channel was designed to simulate key components of a natural gravel-bed stream reach, aimed at mitigating the impacts of stormwater on urban waterways. The physical constraints of the site, particularly the limited available land area, precluded inclusion of an artificial flood plain or extended storage basin. Figures 3-2 and 3-3 compare the Viewlands site in the pre-construction phase as a vegetated swale and in the post-construction phase, as a swale that simulates a natural stream reach.

The design objectives of the Viewlands Demonstration Swale were to:

- Increase surface runoff storage via increased bed area and infiltration potential;
- Increase the hydraulic residence time through the swale;
- Decrease the volume and peak flow rates of the surface runoff discharged to Pipers Creek; and
- Facilitate the sedimentation of solids and pollutants carried by the surface runoff.

To measure the performance of the design, the stormwater flow rates have been monitored at the lower and upper ends of the swale. An assessment was made to determine if the above objectives were meet over the length of the monitoring period from July 2000 to January 2001. Because there was no baseline monitoring opportunity before swale construction, the relative success of the project was made by estimating the behavior of the former ditch, under the same meteorological conditions that were present during post-construction monitoring, and then comparing the two situations.

3.2.3 Viewland Demonstration Swale description

The gravel-bedded channel is approximately 270 ft (82.3 m) long and is designed to convey the estimated maximum peak discharge rate of 25 cfs (708 L/s) for a 25-yr 24-hr storm. Fifteen log weirs create a series of level step pools or cells, increasing the hydraulic residence time through the swale. The channel top width ranges from 8 to 12 ft (2.4 to 2.7 m), with a channel depth of approximately 3.5 ft (1.1 m). Side slopes range from 33 to 100 percent, with the banks stabilized through the use of native vegetation, boulders, and logs.

The components of the swale were designed to meet engineering requirements. For example, the boulders were sized to remain in place at the design flow rate. In addition, the logs were buried into the banks for at least a length of 4 ft (1.2 m) on each side and were placed to resist buoyancy when fully submerged.

As stormwater drains from the watershed, it is channeled via a series of open ditches and culverts to the Viewland Swale. Before the stormwater reaches the channel, a Stormceptor™ sediment trap provides partial removal of sediment and oil. The stormwater, carried by a 1.5ft (0.5m) diameter concrete pipe, drains into the first below-grade cell (Cell 1). A concrete floor in Cell 1 allows dredging of accumulated sediment and reduces the scouring effects of the concentrated flow (Figure 3-4). Cell 1 is sized to act as a stilling basin to facilitate flow measurement over a 120^{o} V-notch weir,. The V-notch weir, located at the downstream end of Cell 1, is used for influent flow monitoring. Monitoring equipment is housed in a standpipe upstream of the weir to record the water level in Cell 1. The distance from the floor of Cell 1 to the water level surface is recorded by the flow monitoring equipment, which is used to determine the weir inflow rate.

A geotextile was placed between the concrete bed and the log weir to minimize infiltration and leakage of water from the first cell. Where leaking has been observed, bentonite clay and sandbags have been placed to seal the areas. The water then flows into a plunge pool in Cell 2, where large boulders have been placed to minimize any scouring of the sediments.

Figure 3-4: View of Cell 1 that includes the stand pipe housing the flow monitoring equipment and a 120^{o} V-notch weir (facing downstream towards Pipers Creek)

Figure 3-5: View of Cell 16 that includes a 120^{o} V-notch weir and the concrete culvert with metal grating (facing downstream towards Pipers Creek)

The water eventually reaches Cell 15, which is immediately upstream of the second 120^{o} V-notch weir (Figure 3-5). Cell 15 is sized to form a level quiescent pool and the water level is recorded by monitoring equipment housed in a stand pipe. The water that flows over the V-notch weir falls into a rock-armored plunge pool (in Cell 16) and into the grated metal concrete culvert. Once in the concrete pipe, the water flows through an energy dissipation structure and to the natural stream system of Pipers Creek.

3.3 2nd Avenue NW SEA Streets Millennium Project site description

The Street Edge Alternative (SEA) Streets project is one of twelve citywide projects that are aimed at mitigating the negative impacts of urbanization on natural creek habitat. The purpose of the SEA Street redesign is to evaluate alternative means of reducing surface water runoff from non-arterial residential streets (SPU SEA Streets, 1999).

The primary criteria used in siting the SEA Streets project on a residential road were (SPU SEA Streets, 1999):

- Within the Pipers Creek Watershed;
- A non-arterial residential street;
- A through street with no dead ends;
- Does not have existing concrete curbs and sidewalks;
- Has approximately 10 to 20 homes facing the road;
- Has some direct connection to the creek watershed;
- Not near critical slope areas;
- Does not have a steep gradient;
- No Metro Transit Route runs on the street;
- Properties are above street elevation;
- Not directly served by the existing storm drainage system; and
- Potential alley access for the properties during periods of construction.

Potential blocks that met the above criteria were identified by the Seattle Public Utility's SEA Streets project team. Residents on the potential blocks were then notified and surveyed. Unanimous support was required by the residents before the project was considered for construction on the block.

3.3.1 Design objective for the SEA Streets redesign

The success of the SEA Streets Project is based on the following criteria (Seattle Public Utility SEA Streets, 1999):

Meet the following hydraulic requirements:

- Reduce impervious area and increase infiltration of rain within the boundaries of the residential block;
- Decrease the volume and peak flow rates of surface runoff discharged to Pipers Creek; and
- Decrease the production and transport of water pollutants;

Meet community satisfaction at all phases of the project; and

Minimize the maintenance requirements through proper design and resident stewardship.

Precipitation and stormwater runoff were monitored at the pre-construction phase (from March 2000 to July 2000) and the post-construction phase (January 2001 and on) of the project. Post-construction monitoring began in January 2001, and will be the subject of future reports.

Precipitation hyetograph and runoff hydrograph analysis is performed for each storm recorded at the site during baseline SEA Streets conditions. To put the hydrograph analysis into perspective, runoff volumes are estimated from both a conventional street design and a theoretical constructed SEA Streets design, under the same meteorological conditions that were present during pre-construction monitoring.

3.3.2 Pre-construction site description

The chosen location for the Street Edge Alternatives (SEA) Streets Project is the residential block of 2nd Avenue NW, between NW 117th and 120th Streets. Refer to Figure 3-6 for a view of the 2nd Avenue street at pre-construction in August 1999.

The street is 660 ft (201.2 m) long and has a 60 ft (18.3 m) right-of-way.

The project area slopes slightly toward the southwest (Figure 3-6). Runoff flows parallel to a low asphalt berm along the full length of the west side of 2nd Ave NW. It discharges flow to a ditch along the north side of NW 117th at the corner of 2nd Ave NW, which is part of the conveyance system that eventually reaches Pipers Creek (Figure 3-7). Properties on the east side of 2nd NW have substantial pervious yard and contribute less runoff than direct precipitation on the right-of-way. Properties on the west side of the block mainly drain away from the street and contribute little street runoff.

Figure 3-6: The pre-construction site at the 2nd Ave SEA Streets Project (view south-facing 2nd Ave NW from NW 120th St)

Figure 3-7: The pre-construction outflow location (at 2nd Ave NW facing east on NW 117th St)

Figure 3-8: The pre-construction location for the V-notch weir and runoff measurement system (at 2nd Ave NW facing east on NW 117th St)

To prepare for baseline monitoring, the project area was isolated hydraulically by placing two traffic speed bumps across the entrances of 2nd Ave NW at NW 117th and another at NW 120th Street. Water flow from the isolated part of the street was directed towards a constructed monitoring station, placed on the southwest corner of 2nd Ave NW and NW 117th Street. Figures 3-7 and 3-8 show the location of the monitoring station before and after construction. The flow monitoring station consisted of a concrete vault covered by a geosynthetic liner that formed a leak-proof stilling basin, a stand pipe that housed the flow monitoring equipment, and a 22.5^{o} V-notch weir. The weir outflow was then directed to the open channel near the pipe underneath 2nd Avenue (Figure 3-7).

3.3.3 Post-construction site description

Initial construction began on July 24, 2000, but was delayed while the design was modified. Work restarted on September 25, 2000 and continued through January 12, 2001. Post-construction monitoring began on January 19, 2001. The as-built project is shown in schematic plan form in Figure 3-9 and pictorially in Figure 3-10.

As part of the street redesign, the street width was decreased by approximately 6 ft (1.8 m) and parking was reduced and set perpendicular to the street (Figure 3-10). The sidewalks are approximately 5 ft (1.5 m) in width. Both the street and sidewalks meander along the length of the block, following the contours of the detention ponds. Detention ponds have been constructed in front of the residences, with a system of culverts that connect the ponds to the monitoring swale (Figure 3-11). The detention ponds are approximately 15 ft (4.6m) in width, 50 to 75ft (15 to 25m) in length, and are planted with native landscaping (Seattle Public Utilities 1999).

To determine the performance of the stormwater control designs, the installment of the monitoring stations are discussed in the following chapter.

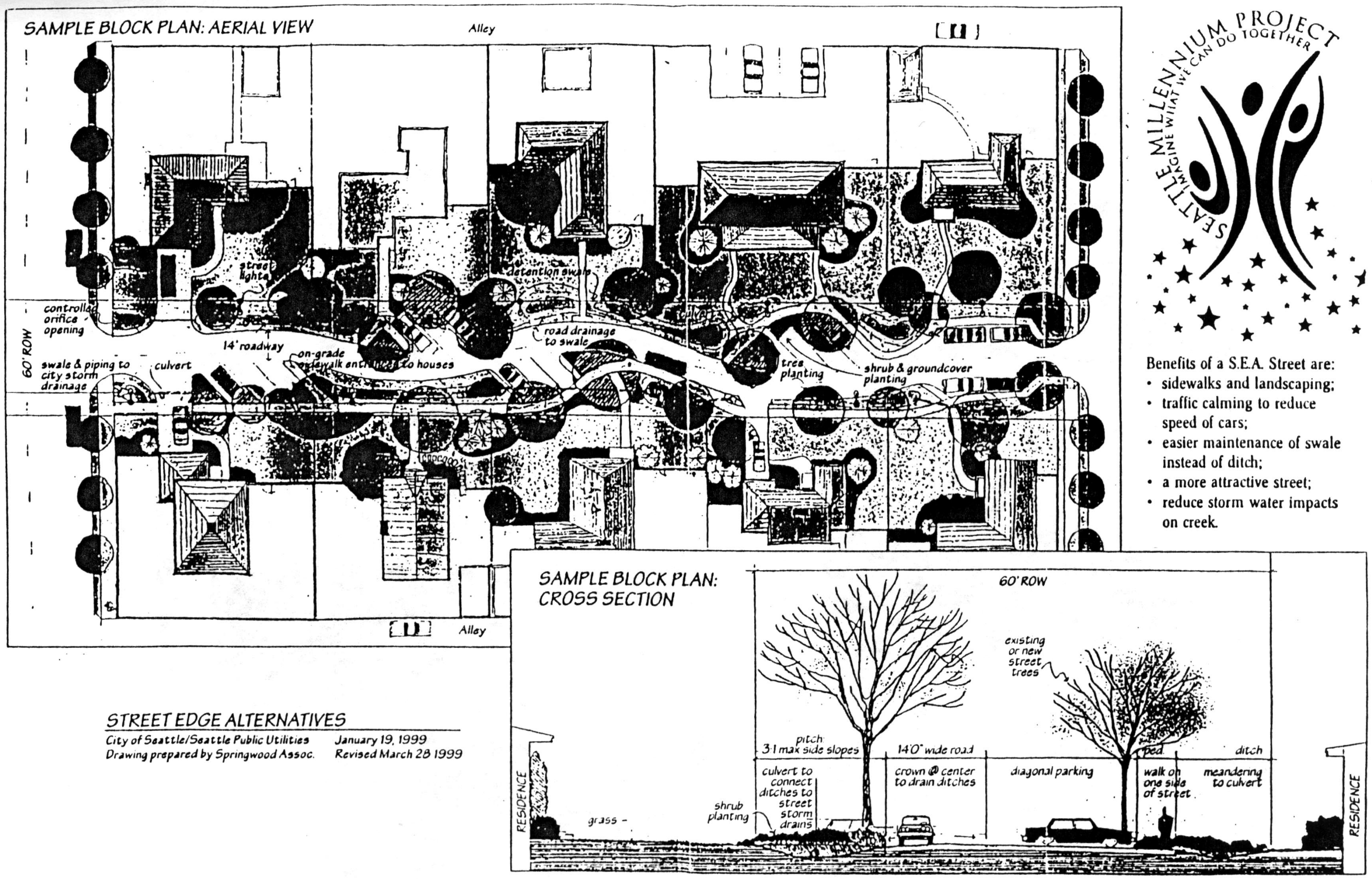

Figure 3-10: The SEA Streets project block plan

Figure 3-10: The post-construction site at the SEA Streets Project (view north-facing 2nd Ave NW from NW 117th St)

Figure 3-11: A sedimentation pond approximately 15 ft (4.6 m) wide by 75 ft (23 m) long (view north-facing 2nd Ave NW from NW 117th St)

CHAPTER 4 – MONITORING DESIGN AND IMPLEMENTATION

A major objective is to examine the performance of alternative stormwater conveyance systems relative to more conventional conveyance systems. To determine the performance of the systems, results were drawn from comparisons made between pre-construction and post-construction conditions. Monitoring systems were established to collect hydrological data that could be used to illustrate differences in response between the two systems.

4.1 Instrumentation Plan at Viewlands

At the Viewlands Demonstration Swale site, no baseline monitoring was possible and all field data were collected under post-construction conditions.

4.1.1 Meteorological monitoring

The first component of the Viewlands instrumentation plan involved collecting meteorological information for the site. A monitoring station was established to collect the necessary precipitation and flow data for evaluation of the swale, and also to collect meteorological information to perform future water and energy mass balances.

Numerous considerations influenced siting the meteorological station at the Viewlands site. The space had to be fenced and locked for security of the equipment and for safety of the children attending Viewlands Elementary school. The monitoring station had to be sited on open level land, at a distance of at least ten times the height of any nearby building, tree, or other obstruction (CM10 Manual, 1997). Each piece of equipment had to collect data without interference from another piece or from the surrounding fence. The equipment had to be located 6 ft (1.8 ft) away from the fence and have a radius of free space of at least the height of the instrument.

Taking all the requirements into consideration, the best location for the monitoring station was within the fenced playing field of the Viewland Elementary School. The fenced enclosure consists of a Campbell Scientific Tripod Weather Station with a mounted CR10X data logger, two CR10X data loggers that records the flow information from the swale, an evaporative pan station, and a series of measuring gauges.

CM10 tripod weather station

The CM10 Tripod Weather Station provides a support structure for mounting weather station components. It is equipped with a MSX10 Solar Panel, PS12LA Rechargeable Power Supply, CR10X Data logger, and a waterproof enclosure with desiccants (Figure 4-1). Meteorological sensors are also mounted onto the tripod and include the HMP45C Temperature and Relative Humidity probe, Met One 034A-L Windset Wind Anemometers, and Kipp & Zonen CM3 Shortwave Pyranometer (Figure 4-2).

The MSX10 Solar Panel mounted onto the tripod uses a photovoltaic power source used for recharging the PS12LA 12V batteries. The batteries are then used to power the CR10X Data logger, ensuring continuous data collection. The CR10X Data logger and PS12LA 12V Power Supply are mounted inside a waterproof enclosure. Desiccants are left in the enclosure to ensure that the wires and computer chips do not get waterlogged and short out. The mounted CR10X Data logger collects and stores all the meteorological information, which is then downloaded with a laptop computer for later data analysis in the office.

The tasks involved in setting up the CM10 Tripod Weather Station were mounting and wiring both the CR10X Data logger and PS12LA 12V Power Supply PS12LA 12V Power Supply. The HMP45C Temperature and Relative Humidity Probe, Met One 034A-L Windset Wind Anemometers, and Kipp & Zonen CM3 Shortwave Pyranometer were also attached to the tripod, wired, and programmed. The tripod was grounded for lightening strike protection.

Figure 4-1: The CM10 Tripod Weather Station located at the Viewlands swale monitoring station

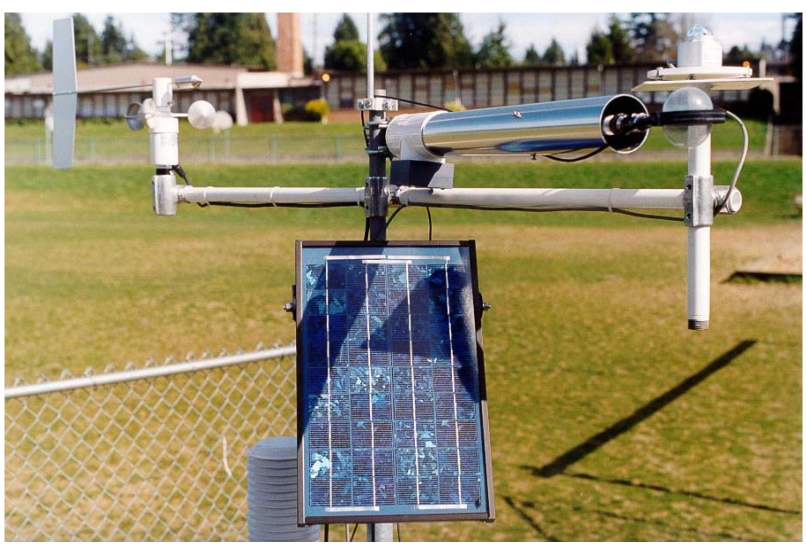

Figure 4-2: The CM10 Tripod Weather Station with a mounted solar panel, wind anemometer, net radiometer, and shortwave pyranometer

Two additional Campbell Scientific CR10X data loggers are located within the fenced meteorological station. They collect and store information from the pressure transducers and shaft encoders in the swale and in the evaporation pan collection system (Figure 4-3). The data loggers are mounted 5 ft (1.5 m) from the ground on 1.5 in (3.8 cm) diameter galvanized steel pipes set in concrete. A 12-volt marine rechargeable battery powers the instruments.

Figure 4-3: The evaporation pan collection system

Precipitation gauges

Three precipitation gauges are located at the Viewland monitoring site: two TB3 tipping-bucket precipitation gauges and one NovaLynx precipitation gauge (Figure 4-4). Each precipitation gauge is located approximately 8 ft (2.4 m) from each other to reduce interference. The lip of the buried gauges are surrounded by a furnace filter, which prevents splash of water into the gauge. Figure 4-4 shows the buried TB3 trench gauge (top) and the buried Novalynx precipitation gauge (bottom) within the station.

Figure 4-4: The buried TB3 tipping-bucket trench gauge (top) and the buried Novalynx gauge (bottom)

TB3 tipping bucket precipitation gauge

The TB3 precipitation gauges operate on a tipping bucket principle. Precipitation is collected and strained by stainless steel mesh before being passed to the tipping bucket measuring system. Tips of the bucket occur with each 0.0078 in (0.2 mm) of precipitation and are detected electronically by a reed switch. The data logger counts the number of tips, which can then be converted to a volume of water. The manufacturer-specified calibration accuracy is +/- 2 percent for precipitation intensities of 1.0 to 23.6 in/hr (25 to 600 mm/hr) (TB3 Manual, 1999).

The first TB3 precipitation gauge is housed in a trench that has side walls supported by a wooden structure. The purpose of the trench is to locate the top lip of the precipitation gauge at the same level as the ground, thereby minimizing wind effects on rainfall catch. The lip of the gauge is surrounded by a furnace filter mesh to eliminate potential splash of precipitation into the gauge.

Figure 4-5: The standing TB3 tipping bucket gauge

The standing TB3 precipitation gauge is mounted on three stainless steel threaded rods with the gauge placed 25 in (63.5 cm) above the ground (Figure 4-5). Standing precipitation gauges are the most common configuration used to collect precipitation, but they underestimate the amount of precipitation due to wind flow patterns. A wind anemometer is placed at the height of the lip of the gauge to measure the wind speed to attempt to correct the recorded precipitation.

Each TB3 precipitation gauge is connected by plastic tubing to a container that collects all the water that runs through it. This acts as a check to confirm that the tip count water volume agrees with the measured accumulated water.

One complication in collecting the cumulated water is potential evaporation losses between the storms and the time the accumulated water is measured. In addition, with the generally low precipitation intensities in the Pacific Northwest, TB3 Gauges bead water on the funnel that may evaporate, causing the amount of precipitation to be underestimated.

NovaLynx precipitation gauge

The NovaLynx precipitation gauge is a simple collection gauge. A self-draining hole is dug, filled with pea gravel, and then covered by a constructed wood frame. The lip of the gauge is placed level with the ground and surrounded by a mesh furnace filter to reduce wind and splash effects (Figure 4-4). The 8 and 1/16 inch (20.5 cm) in diameter funnel directs water into a measuring tube. A graduated cylinder is used to manually measure the water, which is then compared to those measured electronically from the TB3 Gauges.

Maintenance

The tipping buckets require periodic cleaning. The precipitation gauges need to be kept level. Manual tipping of the bucket a specified number of times and then comparing with the data record, ensures that each tip is being recorded and that the tipping mechanism is operating freely (CS410 Manual, 1999).

4.1.2 Flow monitoring

The second component of the Viewlands instrumentation plan involves collecting stormwater runoff data. The flow rate is measured at the upstream end of the swale in Cell 1 and then again at the downstream end in Cell 16 (refer back to Figure 3-5). The differences in peak flow rate, hydraulic residence time, and volume infiltrated determines if the swale meets its design objectives to mitigate some of the detrimental effects of urbanization. Flow measuring devices were placed in the upstream and downstream ends of the swale. The devices include water depth measurement via pressure transducers and float/shaft encoders, and a hydraulic flow control in the form of V-notch weirs. Considerations for the placement of the flow measuring equipment are detailed below.

Stilling basins

A stilling basin is located upchannel from each V-notch weir. Energy dissipaters were needed where the weir discharges to an erodible channel bed so plunge pools were added, which reduce kinetic energy by turbulent recirculation (Chanson, 1999). At the end of the channel, a stepped drop structure carries the outflow to Pipers Creek. It acts by spreading water laterally, leading to a local reduction of the flow velocity (Chanson, 1999).

V-notch weirs

To accommodate the anticipated range of stormwater flows, 120^o V-notch weirs were installed on the top of the first (Figure 4-6) and last (Figure 4-7) logs. The sides of the weir plates are bolted into the rock boulders and cemented into place. Each V-notch weir adds approximately 1 foot (0.3 m) of head to the cells immediately preceding them at the maximum design flow rate of 25 cfs (708 L/s). The upstream weir plate is 2.3 ft (0.7 m) high, with a top width of 13.6 ft (4.1 m) and a bottom width of 9.6 ft (2.9 m). The invert of the weir lies 2.4 inches (6.1 cm) above the log weir. The crest thickness is 0.25 in (0.6 cm), with the downstream edge angled to 60^o to the vertical (Figure 4-6). The downstream weir plate is 2.5 ft (0.8 m) high, with a top width of 8 ft (2.4 m) and a bottom width of 6.9 ft (2.1 m). The invert of the weir lies 4.9 in (12.4 cm) above the log weir. The crest thickness is 0.25 in (0.6 cm), with the beveled (60^o) edge facing upstream rather than downstream (Figure 4-7).

CS405 submersible pressure transducers

CS405 Submersible Pressure Transducers from Campbell Scientific were used. The CS405 pressure transducer is composed of a silicon pressure cell fitted into a stainless steel barrier

diaphragm. The transducer contains a strain gauge element, which measures the hydrostatic pressure.

Figure 4-6: The upstream V-notch weir, standpipe, and influent pipe (facing upstream towards NW 105th Street)

Figure 4-7: The downstream V-notch weir with Cells 15 and 16 filled (facing upstream towards NW 105th Street)

The transducers are vented at one end of the cable, so that changes in barometric pressure do not affect the water level readings (CS 405 Manual, 2000). This is performed via a vent tube open to the atmosphere, terminating in a desiccated enclosure. The desiccated enclosure acts as a protective barrier against moisture collecting in the vent tube, which may otherwise result in mildew, corrosion, dust, or formation of a liquid column (CS 405 Manual, 2000).

The data logger records the pressure information as feet-head of water at fifteen-minute increments. As the water level rises and falls in the stilling basins, the pressure transducers record the water level in the weir pools relative to a reference elevation. The invert of each weir was used as the zero elevation reference point. The CS405 Pressure Transducer has a claimed accuracy of 0.1 percent of the full-scale output or approximately 0.1 ft (3.5 cm) (CS 405 Manual, 2000). Field observations, however, show a lower accuracy range of up to +/- 0.3 ft (9.1 cm).

The pressure transducer can be suspended either inside or attached externally to a rigid conduit (like a PVC conduit). This is to insure that the transducer remains vertical and does not slump. The pressure transducers must be lowered into the water slowly so that air bubbles do not get trapped in the sensor. By gently shaking the transducer under water, any trapped air bubbles can be dislodged.

If the transducer is housed in a standpipe, fine sediments can settle out of the water column and coat the sensor head. The sensor head should be wiped clean periodically or rinsed with a mild detergent, to avoid clogging the strain gauge element. The desiccants should also be replaced periodically, as indicated by a change of color in the desiccant.
The wiring of the transducer should be limited to an 880 ft (268 m) cable length (CS 405 Manual, 2000). The cable should be buried within a protective conduit, either a PVC conduit or a flexible garden hose, to ensure that the wires are not accidentally cut. It should also be buried at a minimum depth of 6 inches to minimize temperature fluctuations. When laying the cable, care should be taken not to crimp the vent tube inside the cable. Therefore the cable should be bent no more than a radius of one inch (CS 405 Manual, 2000). The data logger collecting the flow information must also be grounded, to reduce chances of damage from lightening and electromagnetic noise.

Three CS405 submersible pressure transducers are located at the Viewland site. Two measure the upstream and downstream water levels in the Viewland bioswale and one measures the water level in the evaporative pan. A pressure transducer is housed in each stand pipe within the upstream and downstream stilling basins. For data collection purposes the reference point was established as the floor of the stilling basins (refer to Figures 3-5 and 3-6). To apply the theoretical V-notch weir flow equation, the invert of the weir is the elevation of interest. The relationship between the invert of the weir and the floor of stilling basins was determined by surveying.

The pressure transducers mounted in the stilling basins are attached to a length of PVC pipe, insuring that the transducers are upright and easy to remove. To reach the data monitoring site, electrical wiring has been laid 160 ft (49 m) from the upstream stilling basin and 100 ft (30 m) from the downstream stilling basin. The electrical wiring runs under the sidewalk and along the fence line. The pressure transducer placed in the evaporative pan is mounted against the sidewall and held in place by Velcro strips.

CS410 shaft encoders

The CS410 shaft encoder is designed as a pulley system, with a thin metal punched tape draped over notches raised partially on the pulley. A float sits at the water's surface at one end of the punched tape and a counterweight is attached at the other end. As the water level fluctuates this causes the float to move vertically and the shaft to rotate. Shaft rotation is converted to electronic signals that are measured by the data logger (CS410 Manual, 1999). The data logger records the information as a water level in feet at fifteen-minute increments.

The shaft encoder is mounted within a stand pipe with sufficient clearance to allow the pulley, tape, and counterweight to be unobstructed as they rotate and move vertically. The pulley is one foot in circumference, with 128 counts per revolution (CS410 Manual, 1999). This equates to a 0.094 in (2.4 mm) count , which is the lowest increment that the shaft encoder can sense (CS410 Manual, 1999). The error range for the shaft encoder is minimal. Manufacturer's specification claims an accuracy resolution of 0.094 in (2.4 mm) or approximately 1 count. From field observations, the CS410 shaft encoder has an error range of +/- 0.34 in (8.5 mm) or approximately 3 counts.

Two CS410 shaft encoders are located at the Viewland swale and are housed with the pressure transducers in the standpipes in Cell 1 and Cell 16. The reference elevation has been established as the bottom of the stand pipe and the relationship between the invert of the weir and the reference elevation has been determined by field measurements. Use of two kinds of stage gauges (i.e. the pressure transducer and shaft encoder) insures that if either piece of equipment malfunctions or gives spurious readings, continuous readings can still be made.

Each shaft encoder requires two pulse channels to run, but the CR10X data logger only has two of the necessary pulse channels. Only the downstream shaft encoder was connected to those channels initially. It has been collecting data since March 11, 2000. The upstream float recorder was connected to an alternative wiring configuration that was suggested by the Campbell Scientific technicians. Unfortunately, the data being collected was erroneous and the suggested wiring configuration did not work. Campbell recommended a SDM-SW8A pulse channel module to provide eight additional pulse channels, but it also did not function. The module was returned to Campbell and an additional data logger was obtained in its place. This new device has been recording data since October 1, 2000.

4.2 Instrumentation plan at the SEA Streets site

Field data were collected under both pre- and post-construction conditions. Only the pre-construction field data, and its comparison to a hypothetical conventional street design, are presented in this report.

4.2.1 Meteorological monitoring

There are two components of the SEA Streets monitoring plan: meteorological and flow monitoring. The first component of the SEA Streets instrumentation plan involves collecting precipitation information for the site. For protection of the equipment, the TB3 tipping bucket precipitation gauge and Novalynx precipitation gauges were placed in the backyard of the property on 2^{nd} Ave NW and NW 117^{th} Street. For aesthetic reasons and ease of downloading, the CR10X data logger was placed near the monitoring swale.

CR10X data logger

A CR10X Data logger collects and stores the information from the CS410 shaft encoder located in the stand pipe and the TB3 precipitation gauge located in the backyard of the property at the northwest corner of 2^{nd} Ave NW and NW 117^{th} Street. The CR10X Data logger is powered by a 12-volt marine rechargeable battery. The data logger is located at the corner of 2^{nd} Ave NW and NW 117^{th} Street, approximately 8 ft (2.4 ft) away from the stilling basin. The information is stored in 5-minute increments and downloaded to a laptop computer for later analysis. The data logger is mounted 5 ft (1.5 m) from the ground on a 1.5 in (0.46 m) diameter galvanized steel pipe set in concrete. The power supply sits in a plastic container and is chained to the steel pipe. On July 27, 2000, the data logger and power supply were dismantled and removed to prepare for construction. On January 19, 2001, the data logger was reinstalled for post-construction monitoring.

Precipitation gauges

One standing TB3 tipping-bucket precipitation gauge is located in the back yard of the property at the northwest corner of 2^{nd} Ave NW and NW 117^{th} Street. The precipitation gauge is mounted 38 inches above the ground to prevent water from lawn sprinklers entering the precipitation gauge (Figure 4-5). It is sited 8 ft (2.4 m) from the fence enclosing the back yard to prevent wind from blowing any water into the precipitation gauge from the fence and vegetation. Both sources would increase the apparent amount of precipitation recorded by the tipping precipitation gauge.

The SEA Streets standing precipitation gauge also has the same secondary accumulation system installed at Viewlands. In addition, a minimum of 6 hours is required to calibrate the gauge. A NovaLynx precipitation gauge is placed approximately 3 ft (0.9 m) away from the standing precipitation gauge to reduce potential interference affects. It was installed in the same way as at Viewlands and took about the same length of time.

4.2.2 Flow monitoring

Baseline collection of the stormwater runoff data was performed at the monitoring basin (Figure 4.8). A shaft/float water level recording system was placed in a stand pipe next to the constructed stilling basin. The resulting water depth measurements were converted to a volumetric flow rate through the installed 22.5^o V-notch weir. Considerations for the placement of the flow measuring equipment are detailed below.

Stilling basins

Flow from the street was directed towards an at-grade or near-grade concrete vault, covered by a geosynthetic liner that formed the leak-proof stilling basin (Figure 4-8). A geosynthetic liner was placed between the asphalt road and outlet of the weir to prevent any infiltration or leakage of water from the stilling basin. The sides of the stilling basin were buttressed by sandbags to create a storage volume of approximately 4.5 ft^3 (0.13 m^3). The stilling basin was covered by a lid made from plywood for safety and protection.

Figure 4-8: The constructed stilling basin and stand pipe housing the shaft encoder at the SEA Streets site

A stand pipe was located next to the stilling basin. The stand pipe provided protection of the flow monitoring equipment and dampened waves as water flowed into the stilling basin. A CS410 shaft encoder was housed in the standpipe to measure the water level. The reference point was established as the bottom of the stand pipe and the relationship between the invert of the weir and the reference point was determined by surveying.

A new stilling basin has been constructed for the constructed monitoring period, starting in January 2001. The sides of the stilling basin are buttressed by boulders and creates approximately the same storage volume of 4.5 ft^3 (0.13 m^3). The entire stilling basin is lined by a geosynthetic liner to isolate the flow and prevent leakage of water from the stilling basin. The new standpipe is placed within the stilling basin and houses the shaft encoder. Figure 4-9 shows the new basin located at SEA Streets during construction.

Figure 4-9: Construction of the new monitoring stilling basin at the SEA Streets site

V-notch weirs

Flows ranging from zero to a maximum peak discharge of 1.5 cfs for a 25-yr 24-hr storm are expected. The V-notch weir adds approximately 1 foot of head to the catch basin at the maximum design capacity. To accommodate the anticipated range of flows, a 2-ft (0.6 m) high 22.5^{o} V-notch weir was installed at the downstream side of the stilling basin. The sides of the weir plate are bolted to wood stays and concreted in place. The weir plate is 11.5 in wide, 23.5 in high, and the weir notch is 7 in above the wood frame.

CS410 shaft encoder

A single CS410 shaft encoder is located at the SEA Streets site (see the description of the shaft encoder under the Viewlands heading). The shaft encoders are supported by a wooden platform mounted near the top of the stand pipe, so that the float and counterweight hang freely from the pulley. Electrical wiring was laid 8 ft (2.4 m) to reach the data logger. On July 11th 2000, the shaft encoder was removed to make way for construction on NW 117th Street. The shaft encoder was reinstalled on January 19, 2001 for post-construction monitoring, in its new location in the stilling basin.

CHAPTER 5 – DATA RECORDING AND MANAGEMENT

The use of multiple data loggers, varying data record lengths, and the sheer quantity of data made it necessary to create a systematic approach for storing and validating the data. The approach was designed to facilitate data collection, storage at multiple sites, and quality assurance/quality control (QA/QC). The goal of creating a continuous and consistent data record, and easing the transition between graduate students taking on the project, motivated an attempt to automate the process. The following sections describe the data record structure and the data management approach.

5.1 Data records collected to date

The Viewlands CR10X data loggers record all data at 15-minute intervals. With one exception, on June 23, 2000 between 11:00 AM and 11:15 AM, the data logger mounted on the Viewlands weather station tripod and the Viewlands swale data loggers have no breaks in their record. The SEA Streets data logger recorded data at 15-minute intervals from March 4, 2000 to May 6, 2000 and then at 5-minute intervals until shutdown for construction on July 27, 2000. There is one break in the record, from April 29, 2000 at 6:00 PM to May 6, 2000 at 8:15 AM. Construction at the SEA Streets site finished on January 12, 2000 and the datalogger has been recording since January 19, 2001. Appendix B summarizes the records collected by the Viewlands meteorological station CR10X data logger, the Viewlands swale data loggers, and SEA Streets datalogger, respectively to date.

5.2 Data downloading and storage

Data are downloaded via an SC929 RS232 interface 9-pin connector, which is connected between the CR10X data logger and a laptop computer. PC208W ® software was supplied by Campbell Scientific to be the interface between the data logger and end user. Each CR10X was assigned a different identifier for recognition by the software.

The program is written so that each piece of equipment has a code that initializes data collection, performs a 5-minute or 15-minute average and then stores the data for retrieval. In addition to the sensor data, the day, time, and battery voltage are listed for easy reference. The data are first stored in the CR10X datalogger as PC208W .csi data files. They are then downloaded to a laptop computer and saved to new folders as .dat files, which includes files of the weekly downloads and continuous files with all data to date. The data records are then transferred to disks as well as onto a secondary computer. Therefore, the data exist in four different locations, ensuring accessibility if the data logger or computer malfunctions.

5.3 Quality assurance/quality control (QA/QC)

QA/QC of the meteorological data collected at the Viewlands and the SEA Streets sites will eventually be performed via comparison with the hydrologic data collected by Professor Stephen Burges at the University of Washington's Center for Urban Horticulture. Because the equipment was not installed at the same time, and the data records are not of the same length, this step must wait on further data collection in the Northwest Seattle sites. The following two subsections describe quality control efforts related to precipitation and stage measurements, respectively.

5.3.1 Precipitation

QA/QC is performed on the precipitation data in a number of ways. The first method is to calibrate the TB3 tipping bucket precipitation gauge. The manufacturer quotes 0.0079 in (0.2 mm) of precipitation per tip but also recommends that the owner perform field tests.

To calibrate the gauge, a volume of 22 ounces (657 mL) is released into the funnel at a known rate 2 in/hr (50 mm/hr). Six to eight trials are typically performed, and the average number of tips from all trials is used for calibration. From factory calibrations, the number of tips expected from a single trial is, on average, 100 to 104 tips. That number is converted to units of mm of water per tip and used to calculate the amount of water registered over a given time interval.

Additionally, each TB3 precipitation gauge has a secondary collection system consisting of a tube connected to a sealed plastic container. The volume of water collected is compared to the volume determined by the tips as recorded by the data logger. If the two values are substantially different, they are then compared to the volume collected by the NovaLynx precipitation gauge.

5.3.2 Stage measurements

QA/QC is also performed on the stage measurements. The use of both the pressure transducer and shaft encoder instruments to measure the Viewlands upstream and downstream water levels creates a redundant system. The two instruments are used as a check, which is especially desirable because the reference head for the pressure transducers fluctuates significantly.

Water level verifications are also regularly performed in the field. A tape measure is lowered manually into the stand pipes to record the water level, and the reading is compared to the recorded stage values, which can be viewed in real time on the laptop computer. If the field measurements differ from the data logger records, user specified offsets in the program are changed so that either the equipment registers the correct amount or the data record itself is corrected. In addition, stages in graduations of 0.25 in (0.64 cm) above the weir invert were inscribed on the weir plates. These markings allow visually comparison of the level of water flowing over the weirs with the data logger data in real time.

CHAPTER 6 – DATA ANALYSIS METHODS

Once the data are collected and stored, it is necessary to validate the accuracy of the data by identification and correction of all discrepancies in the data records. Equipment-specific data analysis is then conducted, specifically to establish the precipitation and flow relationships for the Viewlands and SEA Street sites. The following section describe the methods to correct the data records and perform the subsequent analysis in determining performance of the projects.

6.1 Error identification in the data record

Analysis of either the precipitation or flow data must be preceded by a review of the data record to identify potential problems. Campbell Scientific Programs (.csi files) are structured in the same format, with the first four columns named in the following order:

1: Program ID
2: Julian Day
3: Time [min]
4: Battery [V]

The remaining columns depend on how the user creates the .csi program, describing the additional meteorological and hydrological equipment.

The original .csi data record may be imported into Microsoft Excel or Access as a comma-delimited .dat file, where the necessary corrections are made. Alternatively, a program can be created to automate the steps in the analysis of the data. In transfer to the software program, care must be taken to ensure that the data columns are labeled and proper units assigned.

The first step in correcting the data record is to identify where program changes occur, as noted in the data record by a change in the program ID. Typically this shows when the user has made major changes to the program, such as adding new pieces of equipment or changing the time interval at which the data are registered by the CR10X datalogger.

The second step is to identify problems in the data record. A check must be performed to make sure 365 or 366 days are present in the data record. If blocks of data are missing, it is unlikely that the user will be able to reconstruct those sections of the data record. A check must also be performed to identify if the data have been recorded at varying time intervals (ie. 15-minutes or 5-minute intervals). The entire data record should be in a consistent time increment, preferably at 15-minute time increments for the Viewlands data and a 5-minute time increments for the SEA Streets data.

A check must also be performed to identify if data are missing or repeats, i.e. due to Daylight Savings Time (DST) changes. If DST occurs then in the data record it will seem that four 15-minute time increments are missing, then later in the data record it will seem that four 15-minute time increments repeat themselves. To correct the data record, the 15-minute increments and day must be renamed to read continuously. To avoid having to correct the data each time DST occurs and ensure a consistent data record, the laptop clock should be kept at Pacific Standard Time all year around.

Instrument signal errors, typically due to a voltage spike, show as a reading of –6999 in the data record and must be flagged. The next step is to create a QA/QC column that gives an in-depth description of all the errors identified in the data record thus far. A qualitative score is assigned to each recorded value indicating the reliability of the reading. Table 6-1 describes the labeling system.

Finally, after the errors in the data record are identified and described by the QA/QC file column, a new file is created that incorporates all the corrections. Hence, there are several levels of records: 1. as measured, 2. errors identified, 3. corrections incorporated, and 4. analysis applied (as described in the following sections).

Table 6-1: The QA/QC labeling system

QA/QC	Reading
0	Good original data
1	Bad battery value, data not replaced
2	Missing data, data not replaced, marked with a –99
3	Averaged from 5-min values
4	Averaged from 5-min values, and bad battery values for at least one entry
5	Either trench or standing gauge tips zeroed out, originals in last two columns
6	Calibration: trench or standing gauge tips zeroed out, originals in last two columns
7	Equipment not operational

6.2 Precipitation analysis

The TB3 tipping bucket gauge calibration tests must be flagged. In the data record, the calibration tests have a certain number of tips that occur every 15 minutes, during the period of the calibration test. During that specified time period, the values must be changed to zero (since they do not represent actual storms). In addition, extra tips (which are neither storms nor calibration tests) in the data record must be flagged. Typically 10 to 20 tips are manually performed on a gauge as a check that the datalogger is registering the applied tips. In addition, erroneous tips that are caused by accidentally hitting the gauge during water collection and maintenance must also be removed from the data record. The QA/QC field is then updated to reflect the changes made to the precipitation data.

Once the data record is corrected, the conversion amount from the calibration tests is applied to the data. Therefore, the amount of precipitation is determined for each 15-minute increment, 24-hours a day, for the length of the data record. These data become the basis for producing precipitation summaries and conducting storm analysis.

To create daily precipitation summaries, the amount of precipitation is summed per day. Appendix C contains the monthly precipitation record for each TB3 precipitation gauge for the year 2000 for both the Viewlands and SEA Streets sites. To create monthly summaries, the amount of precipitation is summed per month. In addition, the number of days it rained in the year is noted.

For the storm analysis, a storm can be defined that has precipitation greater than a specified threshold, produced over a given number of days, and distinct if there is an absence of precipitation of approximately 6-hours on either side of the storm. The user must concurrently review the flow record to determine which storms produced runoff in the channels. Typically, storms of less than 0.10 in (2.5 mm) over a 6-hour period and under 0.01 in/hr (0.03 cm/hr) intensity did not produce runoff in either the Viewland or SEA Streets channels and were not further analyzed. The storms above these thresholds are analyzed in relation to the flow events.

For storms producing runoff in the channel, the user must review the precipitation data record and determine the length of the storm in hours. The data records for all three TB3 precipitation gauges were simultaneously reviewed to determine the storm duration. For the specified storm duration, the total precipitation and the storm intensity were then calculated.

6.3 Flow Analysis

The inflow and outflow patterns of the Viewland bioswale were analyzed for the period of June 20, 2000 to March 1, 2001. The outflow patterns of the SEA Streets site were analyzed for the baseline period of March 11, 2000 to July 11, 2000.

The first step in the flow analysis is to determine the error range of the CS405 submersible pressure transducers and the CS410 shaft encoders. At the Viewland site, the upstream pressure transducer wavers +/- 0.59 in (1.5 cm). The downstream float recorder wavers significantly at +/- 7.2 in (18.3 cm) and was therefore not used in the analysis. The upstream and downstream shaft encoders waver +/- 0.34 in (0.86 cm). Since the upstream pressure transducer and downstream float recorder have been in operation for the greatest length of time and have the greatest accuracy, the flow analysis is based on their data records. At the SEA Streets site, the shaft encoder wavers +/- 0.34 in (0.86 cm). Table 6-2 summarizes the error range for each flow monitoring equipment.

Table 6-2: The error range for weir stage measurements

	Viewlands Upstream PT	Viewlands Upstream SE	Viewlands Downstream **PT**	Viewlands Downstream SE	SEA Streets SE
Error Range: (+/-) [ft]	0.049	0.028	0.600	0.028	0.028

*PT = pressure transducer
SE = shaft encoder

To correct the flow data record, the head/stage is set to zero if the readings are in the error range of each piece of equipment. This initial analysis must be cross-referenced with the precipitation data record to verify that no storms occurred during this time period, hence distinguishing the rainfall and runoff response from instrument noise.

The second step is to compare the recorded flow data with field observations. The recorded flow data are continuous at 15-min, whereas the field check observations are single values corresponding to tape measurements of the actual water levels in the stilling basins. The difference between the field observation and the recorded stage at that time step is used to readjust the datum for the recorded shaft encoder and pressure transducer stage. Typically, the shaft encoder and pressure transducer data are adjusted once per week.

The user must determine where in the data record the application of the field correction will begin and end. During periods of little rain, the corrections are typically carried forward through the period of a week. During periods of frequent storms, the corrections are typically applied to the storms that immediately precede and occur when the field measurement is made.

The upstream and downstream cells were surveyed to establish the relationship between the bottom of the stilling basin (the datum for the flow records) and the invert of the weir (datum for the flow rate estimates). On a per storm basis, the maximum inflow and outflow rates, inflow and outflow volume over the weir and the volume infiltrated (for only the Viewland bioswale) are determined. The flow rate is then used to produce hydrographs per storm. Table 6-3 shows the calibration value to establish the new datum as the invert of the weir, and the equations used to calculate the head over the weir, flow rates, and volumes for the pre-construction phase of the project.

Table 6-3: Relationships and equations used in the flow analysis

	Viewlands Site Upstream Weir	Viewlands Site Downstream Weir	SEA Streets Site Weir
Calibration Constant: [ft]	1.82	0.828	1.125
Head over Weir (H_w): [ft]	H_w = Stage Record - Calibration	H_w = Stage - 0.828 (upstream) H_w = Stage - 1.82 (downstream)	H_w = Stage - 1.125
Flow rate (Q): [cfs]	$Q = \text{Constant}* H_w^{5/2}$	$Q = 4.33* H_w^{5/2}$	$Q = 0.4938* H_w^{5/2}$
Volume (V): [ft^3]	V = Sum of Q * Time Increment * Unit Conversion	V = Sum of Q * 15min * 60s/min	V = Sum of Q * 15min * 60s/min

6.3.1 Viewland weir leakage

Over the course of swale operation from October 1999 to the summer of 2000, scouring occurred downstream of each log weir. Gravel originally placed at the downstream side of each log migrated approximately 6 to 12 in (15 to 31 cm) (Figure 6-1). Scour pools formed beneath the invert of the weir plate due to the concentrated flow, eroding the channel bed to maximum depth of 6 to 8 in (15 to 20 cm). In addition, the geotextile that was placed between the concrete bed of the first cell and the log weir to prevent leakage deteriorated over time. The undercutting of the log weirs and the environmental deterioration of the geotextile led to leakage under and around the stilling basins in Cells 1 and 15.

Figure 6-1: Scouring downstream of each log weir from 6 to 8 in (15.2 to 20.3 cm)

Weir tests were performed to determine the quantity of runoff that was being lost to leakage. Substantial amounts of leakage would cause the measured stage readings to underestimate the amount of runoff entering and exiting the system. Two sets of tests were undertaken and are described in the following sections.

Weir flow rate tests – September 2000

Weir tests were performed at the upstream weir (Cell 1) on September 6, 2000 and on the downstream weir (Cell 15) on September 21, 2000 to determine leakage. Five trials were performed at various weir stage heights. Water was discharged from a City of Seattle water utility fire hydrant into the upstream side of the weir being tested. A meter connected to the fire hydrant measured the inflow rate, which was adjusted by a valve on the hydrant. The cells were then filled and steady-state was reached before beginning each trial.

The stage over the weir was known from increments marked onto the weir plate. The outflow rate was determined by physically collecting the water that flowed over the weir into a large plastic sheet that covered the downstream cell. Therefore the volume and time of the trial was known, and an outflow rate could be determined. The difference between the inflow and outflow rates indicated the steady-state leakage rate for that stage and set of antecedent swale soil conditions.

Weir flow rate tests - November 2000

Weir tests were performed on the upstream weir on November 25, 2000 and on the downstream weir on November 26, 2000. Storms occurred during both tests, and nine trials were performed at various weir stage heights. The inflow volume was measured at the concrete pipe feeding the weir pool. Water was collected in a calibrated 5-gallon bucket over a given time period, which was recorded using a stopwatch.

The stage over the weir notch was known from increments marked on the weir plate. From the first set of tests in September 2000, the measured flow rates over the weir coincided closely with those predicted by the 120^{o} V-notch weir flow equation. The feasibility of this relationship is shown in Figure 7.2 (in the following chapter). Therefore, the outflow rate was calculated from the V-notch weir equation using the observed stage. The difference between the measured inflow and calculated outflow rates determined the leakage rate.

6.4 Relationship between precipitation and flow

The relationship between precipitation and flow is established by identifying and analyzing trends in a storm. For each storm that produces runoff, the storm duration, amount of precipitation and average storm intensity are determined. The amount of time leading up to a storm and the time between the start of a storm and runoff response are determined.

Maximum flow rates and volumes are determined at both sites. At the Viewlands site, the maximum downstream flow rate and volume are determined. The difference between the upstream inflow volume and the downstream outflow volume equals the amount of water entering channel storage and/or infiltrating into the channel banks and bed. From this, a percent reduction of the inflow rate is determined at varying temporal scales. The amount of infiltration is calculated over the course of a year and partitioned into dry and wet antecedent soil moisture conditions.

6.5 C-program

To expedite data analysis, a series of C-programs and UNIX shell scripts were created to process each data record produced by the CR10X data loggers. Generic C-programs were produced to review and identify problems in the precipitation and flow data records. The following were identified or checked:

1. changes in the Program ID;
2. continuity of the data record;
3. consistency of time-increments; and
4. instrument malfunctions.

The programs make the necessary corrections, producing a continuous 15-minute data series. This is performed by inserting void values in place of missing ones and averaging the 5-minute time step to 15-minute time step. In addition, a quality assurance/quality control (QA/QC) column is created, which reflects any changes made to the original data.

The new output files (as identified by the filename extension .out) are created for each of the three CR10X data logger records at the Viewland site and for the CR10X data logger record at the SEA Streets site. The output files are structured in the same manner, with the first six columns containing the following parameters:

1. Year
2. Month
3. Day
4. Julian Day
5. Time [min]
6. QAQC
7. Program ID
8. Battery [V]

The remaining columns in the output files contain instrument data, and depend on how the user creates the .csi program describing the corrected meteorological and hydrological data.

For the precipitation and flow analysis, a generalized averaging program was written, which reads time series of flow or precipitation data and calculates hourly, daily, weekly, monthly, and event summaries. For the application, the user extracts the corrected information for each TB3 tipping bucket precipitation gauge, creating a new file containing all three gauge records, for input to an averaging program.

The user also identifies the individual storms and records their starting and stopping times in an event file, which will be read by the averaging program. For each input data column, corresponding to each precipitation gauge, a number of output files are produced:

1. hourly_avg.out
2. daily_avg.out
3. weekly_avg.out
4. monthly_avg.out
5. event_avg.out

The first four output files average the data at various time increments (from 15-minute time increments to hourly, daily, weekly, and monthly time increments). The last output file produces the following columns per storm:

1. percent coverage for the storm period
2. precipitation sum
3. maximum precipitation
4. storm intensity

The output files are then exported into Excel (or any other plotting software package) for creation of hydrographs and further analysis.

6.6 Comparison with theoretical pre-construction conditions

6.6.1 Viewlands site

The purpose of the pre-construction analysis for the Viewlands site is to compare how the old concrete/vegetated channel would have responded under the same meteorological conditions that occurred from June 2000 to January 2001. Project performance is judged by the new swale's ability to out-perform the old channel in decreasing storm water quantities.

To determine the potential volume infiltrated and/or detained in the old vegetated channel, an area-to-area analysis was performed. The measured volume that was infiltrated/detained in the new swale was apportioned over the area wetted during the course of a storm. Given the same meteorological conditions but a different geometry for the old channel, the wetted area and potential volume infiltrated were estimated for each storm.

Assuming that the old vegetated channel infiltrated and stored water in the subsurface under the same dynamics as the new Viewland swale, the upper range of water potentially infiltrating in the old channel was defined. During construction, the existing material was excavated to a depth of 16 to 20 in (41 to 51 cm) below grade and was replaced with 8 to 10 in (20 to 25 cm) of native backfill covered by 8 in (20 cm) of stream bed gravel. From a geotechnical investigation performed at the Viewlands site on May 27, 1999, a sieve analysis revealed the native soil type was "well-graded gravel with silt and sand", which is typically less permeable than a gravel bed layer. Therefore, assuming similar infiltration and storage dynamics for the native soils would tend to overestimate the potential volume infiltrated through the vegetated portion of the old ditch.

The first step in the pre-construction analysis was to determine the potential infiltration area in the new swale. The new swale was designed to accommodate low flow through the center of the channel and high flow over the entire width of the channel bed. Figure 6-2 shows a photograph of the typical bed cross-section for one cell in the swale. The bed slope is 0.048 and the Manning's roughness coefficient (n) is 0.04, for a constructed rock-lined channel with some weeds (King County Surface Water Design Manual, 1998). The low flow bed width is 7 ft (2.1 m), with a 1:1 side slope that produces a depth of 0.9 ft (0.27 m) and accommodates a maximum low flow rate at 48 cfs (1,359 L/s). The high flow bed width is approximately 11 ft (3.4 m) with a minimum boulder height of 1.7 ft (0.52 m) defining the high flow depth.

Figure 6-2: Typical bed cross-section for the constructed Viewlands swale

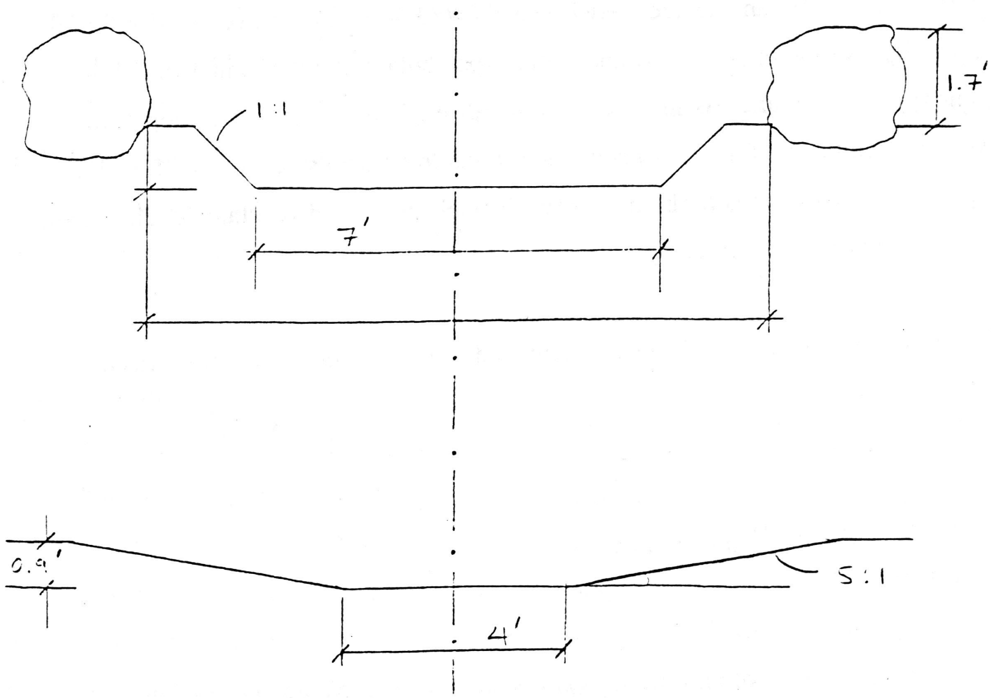

Figure 6-3: The channel cross-sections for the new swale (top diagram) and for the vegetated section of the old channel (bottom)

The second step in the analysis was to determine the potential infiltration area in the old channel, which was composed of two sections. The first section was a parabolic concrete channel with a length of 62.7 ft (19.1 m), an average top width of 3.3 ft (1.0 m), and average depth of 0.9 ft (0.27 m). Above the concrete section was a sediment bank that could accommodate additional flow in cases of large storms. The average top width of the sediment bank was 5.2 ft (1.58 m) and the average depth was 1.7 ft (0.52 m). It was assumed that no infiltration occurred over the length of the concrete section.

The second section of the channel was a vegetated ditch, composed of vegetation, sediment deposits, and gravel. Surface flow and infiltration occurred over the length of the vegetated ditch. The bed slope is 0.048 and the Manning's roughness coefficient (n) is 0.035 for a constructed channel with a stony bottom and weedy banks (King County Surface Water Design Manual, 1998). From surveyed field data, the vegetated ditch had a length of 128 ft (39 m), an average top width of 11 ft (3.4 m), and an average depth of 0.7 ft (0.2 m). By plotting the field data and fitting a parabola to it, the channel was shown to have a nearly trapezoidal cross-section. The trapezoidal cross-section was approximated to have a bed width of 4 ft (1.22 m) and a horizontal to vertical side slope of 5:1. Figure 6-3 shows the cross-sections for both the new channel and the old vegetated ditch channel section to scale (1 inch = 3 feet).

The flow rate through an open channel is estimated using Manning's equation was used:

$$Q = \frac{1.49}{n} * A * R_h^{2/3} * S_f^{1/2} \tag{6.1}$$

-where:

Q = flow rate (L^3/t)

A = area (L^2)

R_h = hydraulic radius (L)

S_f = channel slope (at uniform flow) or head loss with change in elevation

-for a trapezoidal cross-section:

$$R_h = \frac{A}{P_w} = \frac{(by + my^2)}{(b + 2y(1+m^2)^{1/2})} \qquad (6.2)$$

-where:

P_w = wetted perimeter (L)

b = bed width (L)

m = side slope

The volume that was infiltrated into the new swale for each storm was measured over the June 2000 to January 2001 study period. By knowing the peak inflow rate (Q), the swale bed slope (S_f), and roughness coefficient (n), the Manning's equation was used to back-calculate the maximum possible steady-state water depth (y), the wetted perimeter (P_w), and wetted infiltration area (A_{inf}) for each storm in the new channel. Given the same meteorological conditions but a different geometry for the old channel, Manning's equation was again used to back-calculate the (y), (P_w), and (A_{inf}) for each storm. Finally, an plan area-to-plan area comparison was made to apportion the measured volume of water that infiltrated over the wetted area in the new swale with the wetted area of the old vegetated ditch, for each storm. The infiltrated volume was then summed for the old swale and compared to the total measured volume for the new swale over the course of the study period.

6.6.2 SEA Streets site

The relative magnitude of the runoff measured during baseline SEA Streets conditions was compared to the estimated runoff volumes for both a conventional street design and for the constructed SEA Streets design. Table 6-4 describes the dimensions of baseline SEA Streets, constructed SEA Streets, and conventional roadway designs. Figure 6-4 shows the minimum requirements for a conventional street design. The constraining dimensions of all three roadways were based on the SEA Streets length of 660 ft (201 m) and a 60 ft (18 m) right-of-way.

Table 6-4: Residential Roadway Dimensions

	SEA Streets Roadway (Baseline)	SEA Streets Roadway (Constructed)	Conventional Roadway
Length [ft]	660	660	660
Right-of-way [ft]	60	60	60
Roadway width [ft]	25	14	25
Concrete sidewalk width (total) [ft]	-	10	13
Concrete curb/ gutter width (total) [ft]	-	4	6
Tree/grass edge or sedimentation ponds width (total) [ft]	35	32	16

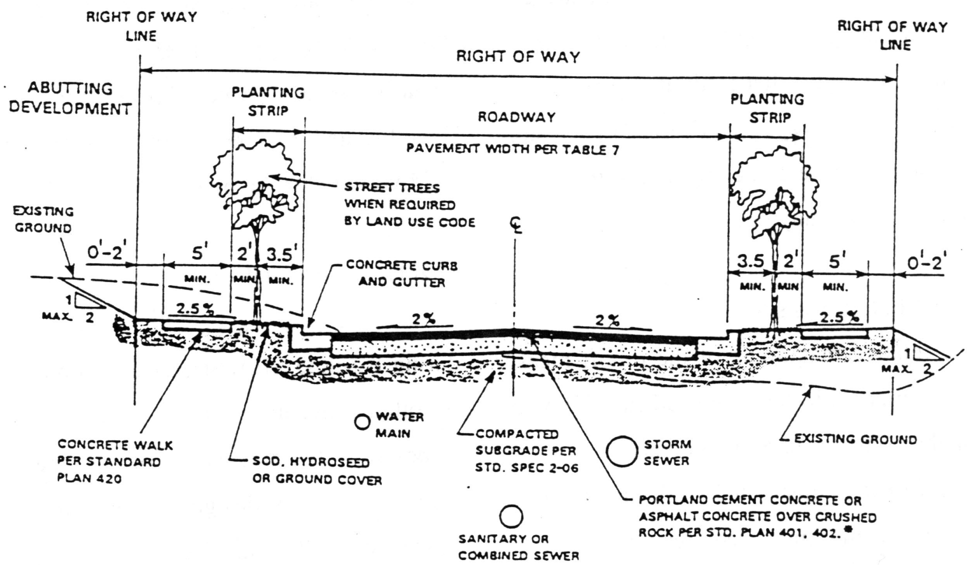

Figure 6-4: Minimum dimensions for a conventional street design

The following equation was used to determine runoff volumes for both the conventional street and the constructed SEA Street roadway designs:

$$V = D_r * A * C_a \tag{6.3}$$

-where:

V = total runoff volume (L^3)

D_r = precipitation depth (L)

C_a = area-weighted runoff coefficient

A = contributing area (L^2)

Table 6-5 describes both the range and average values for the runoff coefficient given different materials. For the concrete surfaces a median value of 0.875 was used. For the tree/grass edge the road grade is approximately 2.5 percent, so the lower range of the runoff coefficient was used at 0.18.

Table 6-5: Runoff Coefficients (Bedient and Huber, 1992)

Runoff Coefficients	Range	Average
Asphalt Street	(0.7 – 0.95)	0.825
Concrete Street	(0.8 – 0.95)	0.875
Grass lawns – heavy soil (Average 2-7%)	(0.18 – 0.22)	0.2

Instead of using literature values for the runoff coefficient of asphalt, it was determined from the measured flow data collected during the pre-construction baseline period. Only storms preceded by three dry days were used in the analysis. This assumed that drier soil conditions would cause the precipitation to initially infiltrate into the grass lawns, producing runoff predominantly from the asphalt street. The asphalt runoff coefficient was then back-calculated using ten storms.

For the constructed SEA Street site, it was assumed that no runoff would occur from the sedimentation ponds and all water falling on the sidewalks would either drain into the grass edge or the sedimentation ponds. Hence, only the asphalt and concrete edges of the street would contribute to runoff.

CHAPTER 7 – PRECIPITATION AND FLOW ANALYSIS

7.1 Viewlands Demonstration Swale

7.1.1 Characterization of precipitation patterns

Analysis of precipitation record

Precipitation was monitored continuously at the Viewlands site from January 2000 to January 2001 and at the 2nd Avenue SEA Streets site from March 2000 to July 2000. Precipitation is also monitored at the Seattle-Tacoma International Airport, located approximately 30 miles to the south. Comparison with the Sea-Tac precipitation record provides an opportunity to put the Viewlands and SEA Streets precipitation data into perspective. Table 7-1 shows the monthly distribution of precipitation recorded by the TB3 tipping bucket trench gauge at the Viewlands site for the year 2000 and the Sea-Tac precipitation record for the years 1999 and 2000, and the 51-yr precipitation mean.

Table 7-1: Monthly precipitation totals

					Millimeters								
Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Total**
Viewlands 2000	62	115	73	33	68	31	12	11	33	81	83	67	**669**
SeaTac 1999	174	177	93	38	54	47	30	23	4	57	244	129	**1070**
SeaTac 2000	96	133	72	38	83	41	6	8	31	76	83	NA	**667**
SeaTac 51-yr Mean	141	107	94	64	42	38	20	27	47	89	149	149	**967**
					Inches								
Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Total**
Viewlands 2000	2.4	4.5	2.9	1.3	2.7	1.2	0.5	0.4	1.3	3.2	3.3	2.6	**26.3**
SeaTac 1999	6.8	7.0	3.7	1.5	2.1	1.9	1.2	0.9	0.2	2.3	9.6	5.1	**42.1**
SeaTac 2000	3.8	5.3	2.8	1.5	3.3	1.6	0.2	0.3	1.2	3.0	3.3	NA	**26.3**
SeaTac 51-yr Mean	5.5	4.2	3.7	2.5	1.7	1.5	0.8	1.1	1.9	3.5	5.9	5.9	**38.2**

Notes: NA indicates data was not available

Precipitation at the Viewlands site was comparable in volume and pattern to the Sea-Tac record for the overlapping year 2000. Precipitation records at the airport indicate a 1999 calendar year precipitation amount of 42.1 in (1070 mm). This is 11 percent above the 51-year mean precipitation amount of 38.2 in (967 mm). The Viewlands trench gauge recorded a 2000 calendar year precipitation amount of 26.3 in (669 mm). The Viewlands record is 31 percent below the 51-year precipitation mean and 24 percent below the amount recorded at Sea-Tac in 1999.

For the period of July 2000 to January 2001, the Viewlands flow monitoring equipment registered a peak upstream flow rate of 3.9 cfs (110.4 L/s), approximately one-sixth of the anticipated peak flow rate of 25 cfs (707.9 L/s) for a 25-yr, 24-hr storm. Due to the unseasonably low precipitation, accurate assessment concerning the efficacy of the swale design based on the weather patterns of the calendar year 2000 will be limited.

Calibration of TB3 precipitation gauges

Table 7-2 summarizes four calibration tests that were performed on the TB3 tipping bucket precipitation gauges. As shown in Table 7-2, the calibration amount [mm/tip] in all cases is within five percent of the manufacturer's specification of 0.2 mm/tip. The field calibration data were therefore used to adjust the equipment-specific data record, multiplying all precipitation depths in tips by the calibration factor. Additional information on the TB3 calibration tests are given in Appendix D.

Table 7-2: TB3 tipping bucket precipitation gauge calibration tests

	Viewlands Trench Gauge	Viewlands Trench Gauge	Viewlands Standing Gauge	SEA Streets Standing Gauge
Test date:	1/17/00	7/18/00	7/20/00	7/25/00
Average tips:	103	110	106	109
Vol collected [mL]:	657	657	657	657
Conversion from [mL/tip] to [mm/tip]:	0.032	0.032	0.032	0.032
[mm/tip]:	**0.203**	**0.190**	**0.197**	**0.192**
[inches/tip]:	**0.0080**	**0.0075**	**0.0078**	**0.0076**

Accuracy of the TB3 tipping-bucket precipitation gauges

The buried trench gauge at the Viewland site should collect more water than the standing precipitation gauge. The lip of the trench precipitation gauge is at ground level, minimizing wind effects. From January1, 2000 through December 31, 2000, the trench gauge accumulated 26.3 in (66.8 mm) of precipitation while the standing precipitation gauge collected 28 in (71.1 mm) of precipitation, which is six percent higher. Due to breaks in the SEA Street standing precipitation gauge record, it is difficult to make a direct comparison with the Viewlands precipitation gauges. In general, the SEA Streets gauge recorded less precipitation than either of the Viewlands gauges.

The discrepancy in the amount of precipitation collected between the precipitation gauges could be due to a number of factors. The baked surface coating on the TB3 funnel beads water, especially at low storm intensities. Evaporation occurs from the beaded water and that amount of precipitation is not accounted for in the data record. Water drops may also be blown from the instrumentation tower into the standing precipitation gauge. From field observation and the wind anemometer data record, the wind blows strongly across the school grounds. The maximum-recorded wind speed is 18 ft/s (5.5 m/s) at 10 ft (3 m) elevation. The standing precipitation gauge may be close enough to the instrumentation tower to be affected at high wind speeds, but there is no direct observational evidence of water drops being blown into the gauge.

The trench gauge data record is used because the recorded precipitation depths are closet to the accumulated amounts collected by the secondary collection system and by the Novalynx precipitation gauge. Refer to Appendix C for a comparison of the monthly precipitation records for the TB3 tipping bucket precipitation gauges.

7.1.2 Characterization of flow patterns

Two problems arose while monitoring the runoff in the Viewlands channel: development of a clinging nappe at the invert of the V-notch weir and leakage under the weir. Both problems caused underestimation of the recorded flow as it traveled into and out of the Viewland swale.

Viewlands V-notch clinging nappe

For a properly installed and sized vertical sharp-crested V-notch weir, the lower nappe is deflected upwards and forward. This causes a trajectory that springs clear of the weir plate immediately downstream of the sharp-edged invert (Chanson, 1999). Theoretically, the pressure on the crest invert is atmospheric, allowing free flow to occur. If the nappe is not properly ventilated and the water flows down the weir plate, the discharge will be underestimated (US Bureau of Reclamation, 1997).

The clinging nappe phenomenon depends on inflow conditions, weir pool geometry, crest thickness, and the roughness of the weir plate. A clinging nappe was observed at both the upstream and downstream Viewlands V-notch weirs. During field tests on September 6, 2000 and September 21, 2000, the nappe did not spring clear of either weir plate, even at a maximum test head of 4 in (10.2 cm). A discussion of the hydraulic and installation conditions that contributed to the clinging nappe phenomena is given below.

The stormwater draining into the Viewland channel carries pollutants, fine suspended sediments that coat the weir, and large debris that build up behind the invert. Coating of the weir increases the roughness of the weir plate in turn, allowing the formation of a boundary layer of slower moving fluid adjacent to the vertical face of the weir plate (Ackers et al., 1978). The fluid layer closest to the weir plate is retarded, affecting the upward contraction of the nappe and subsequently the discharge. For accurate measurements, the V-notch plate should be smooth within a distance of 0.8 in (2 cm) of the crest. Beyond this distance, the influence of the surface finish is negligible.

Unfortunately due to daily environmental stresses, the weir plates at Viewlands are no longer smooth.

V-notch weirs are designed, tested, and calibrated in the laboratory setting with a vertical upstream face flush with a bulkhead. At the Viewland site, the bottom of the weir plate is not flush with the log weir but is mounted onto its top middle section. In addition, the invert of the upstream weir is 2.4 in (6.1 cm) from the base of the weir plate, and the invert of the downstream weir is 4.9 in (12.4 cm) from the base. The maximum upstream head observed to date has been 11.5 in (29.2 cm). As discussed in Chapter 2, 23 in (58.4 cm) should be the minimum distance from the invert to the weir plate, not 2.4 in. Figure 7-1 shows the environmental deterioration of the V-notch weir plate and improper placement on the log weir that resulted from site geometric constraints.

Figure 7-1: Environmental deterioration and geometric constraints of the V-notch weir

The combination of these conditions prevent the necessary surface and crest contraction that would cause the nappe to spring clearly. The weirs were calibrated in the field in an attempt to quantify the affects of the clinging nappe and the weir leakage.

Viewland weir leakage

<u>Weir flow rate tests – September 2000</u>

Both the upstream and downstream tests showed that substantial leakage was occurring in the joint between the weir plate and the log, where the weir plate was bolted to the rocks, among the boulders, along the underside of the log, and in the cracks of the cement mortar. The greatest amount of leakage occurred through the large side boulders, where soil had been eroded.

The upstream weir test, performed on September 6, 2000 showed that the most significant leakage occurred at flow rates less than 0.15 cfs (4.2 L/s). At the upstream weir, flows of approximately 0.05 cfs (1.4 L/s) exhibited an 88 percent loss of inflow. At flows of approximately 0.14 cfs (4.0 L/s), 29 percent of the inflow leaked under or around the weir. The loss dropped to 14 percent at flows of approximately 0.18 cfs (5.1 L/s). Figure 7-2 plots the results of the September 2000 test for the upstream V-notch weir, as well as theoretical V-notch weir flow equation values. The theoretical weir equation values closely agreed with the five measured flow rates up to approximately 0.25 cfs (7.1 L/s).

The downstream weir test performed on September 23, 2000 showed that flows of approximately 0.07 cfs (2.0 L/s) exhibited an 87 percent loss of inflow. The loss dropped to 20 percent at flows of approximately 0.18 cfs (5.1 L/s). Leakage under the downstream weir at flow rates below 1 cfs (28.3 L/s) did not infiltrate into the gravel bed, but became outflow from the system. This was due to the slight bed slope from the bottom of the weir to the outflow grate. Details of the weir tests are given in Appendix E.

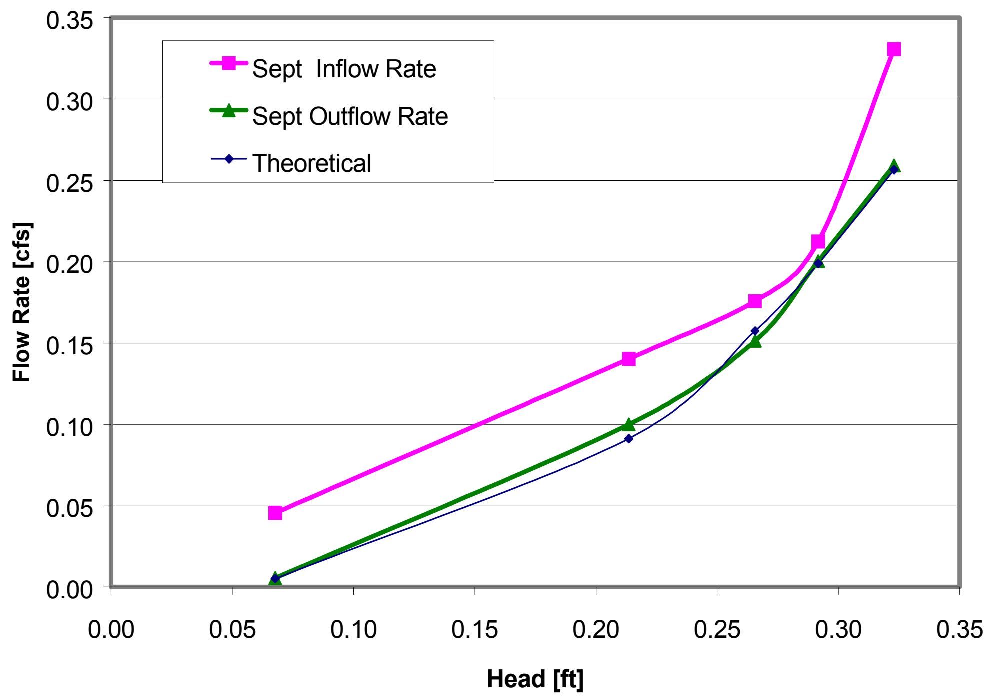

Figure 7-2: September 2000 tests – measure of the inflow rates to and from the upstream V-notch weir

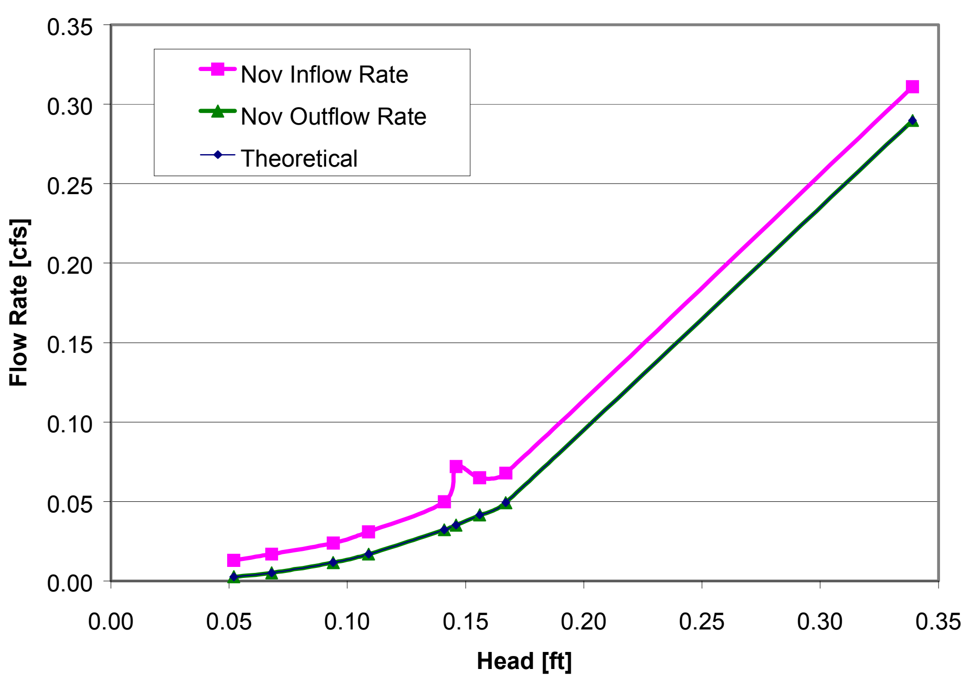

Figure 7-3: November 2000 tests – measure of the inflow rates to and from the upstream V-notch weir

Remedial measures – October 2000

In October 2000, remedial measures were taken to mitigate the leakage occurring at both the upstream and downstream V-notch weirs. Large boulders were placed beneath the invert of the weirs on the downstream side, to dissipate the energy of the stormwater flowing over the weir. Permeable geotextile fabrics were filled with bentonite and placed within the cracks of the boulders on the upstream side of the weirs, specifically near where the weir plate was connected to the boulders.

Figure 7-4 shows the remedial measures taken in Cell 1 to reduce the leakage under the upstream weir. Figure 7-5 shows the remedial measures taken in Cell 15 to mitigate leakage under the downstream weir. From the figures, one can see that heavy polyethylene sheeting was placed over the log weir that supports the V-notch weir, extending 12 ft (3.7 m) along the cell floor. The aprons are held with tack strips of wood and covered along the log with a bentonite compound to prevent leakage on the edge. Plastic sheets line the side walls upstream from the V-notch weir to a little above the water line and are secured with sand bags.

Weir flow rate tests - November 2000

A second set of tests at both weirs confirmed that leakage still occurred but to a lesser degree. The most significant leakage occurred at flow rates less than 0.1 cfs (2.8 L/s). At the upstream weir, low flows of approximately 0.05 cfs (1.4 L/s) exhibited a 36 percent loss of inflow. The loss dropped to 7 percent at flows of approximately 0.3 cfs (8.5 L/s). Figure 7-3 shows the results of the November 2000 test for the upstream V-notch weir. Comparison of Figures 7-2 and 7-3, plotted using the same scale and over the same interval, shows the decrease in leakage between the two upstream weir tests. The downstream weir tests were conducted at flow rates greater that 0.1 cfs (2.8 L/s), so leakage rates of less than 10% occurred.

Figure 7-4: Remedial measures taken in October 2000 at the upstream V-notch weir

Figure 7-5: Remedial measures taken in October 2000 at the downstream V-notch weir

Results

The leakage rates observed during the second set of weir tests were less than for the first set. One explanation was that the first set of tests were performed during a period of dry antecedent soil moisture conditions. There had been 8 days of no rain prior to the upstream weir test on September 6, 2000 and 15 days of no rain prior to the downstream weir test on September 23, 2000 test. It rained the 2 days prior to and during the upstream and downstream weir tests on November 25 and 26, 2000. In addition, remedial measures were performed in October that attempted to reduce the leakage (as described in the previous section).

The series of weir tests were preliminary measures performed to quantify the volume of leakage. Continued tests are needed to determine the leakage rates at both high flows and at varying swale soil moisture conditions. Over the course of the study period, the volume lost to leakage caused an underestimation of the measured flow entering and exiting the Viewlands swale. The most substantial leakage occurred below flow rates of 0.15 cfs. At these low levels, the majority of the inflow leakage will be stored in the swale and will never be recorded downstream.

Subsequent flow rate analysis have been determined using the theoretical V-notch weir flow equations. The preliminary flow rate tests of the upstream weir, shown in Figure 7-2, indicate that for flow rates above approximately 0.25 cfs (7.1 L/s), the data are likely to be quite accurate. The weirs require additional field testing, and all volumetric flow rates and volumes will need to be reassessed, when a complete set of weir tests have been performed.

7.1.3 Hydrograph Analysis

The main purpose of this report is to examine and evaluate the hydrologic response of two Seattle Public Utilities "ultra-urban" stormwater conveyance systems. The evaluation of the stormwater systems is primarily performed by comparisons made of the patterns and magnitudes of precipitation-runoff response and by hydrograph analysis. A number of aspects of each storm were considered and include the following:

1. antecedent precipitation conditions;
2. lag time (time from the first precipitation until water reaches the weir);
3. storm duration;
4. precipitation depth;
5. storm intensity;
6. upstream and downstream peak flow rates;
7. upstream and downstream accumulated runoff volumes; and
8. infiltrated runoff volumes.

The reduction of the peak flow rate and the inflow volume between the upstream and downstream weirs defines the performance of the swale. Appendix F provides the storm record for the Viewlands site.

Antecedent soil moisture conditions

<u>Dry (June 20 – October 9, 2000)</u>

Hydrologic patterns for the seven storms that occurred during predominantly dry swale soil moisture conditions are assessed. The precipitation record and field observations corroborate the predominantly dry state of the soil in the swale between the June 20 to October 9, 2000 period. This is evident by the long times between storms, from a maximum of 26 days to a minimum of 8 days. During the dry soil period, 78 percent of the measured inflow infiltrated and/or was detained by the swale.

Figures 7-6 to 7-9 illustrate the precipitation and subsequent runoff response in the Viewland swale. At the top of each figure is the precipitation hyetograph in units of inches per 15-minute increment (right vertical axis). At the bottom of the figure is total runoff rate measured at the upstream and downstream weirs in units of cubic feet per second (left vertical axis). The precipitation-runoff response is plotted in 15-minute increments (bottom horizontal axis).

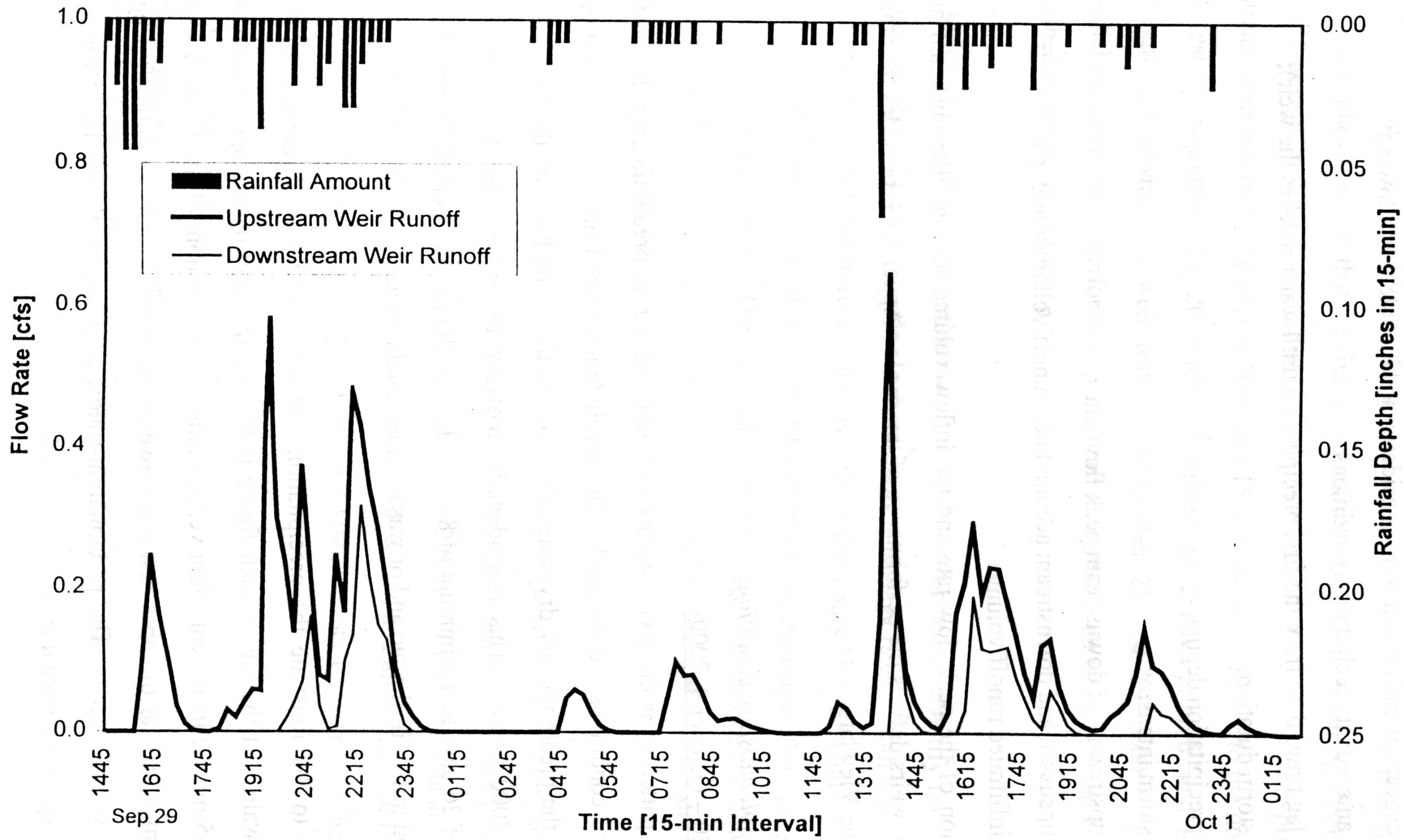

Figure 7-6: Precipitation hyetograph and runoff hydrograph for the Viewlands swale
September 29 (2:45 pm) - October 1, 2000 (1:45 am)

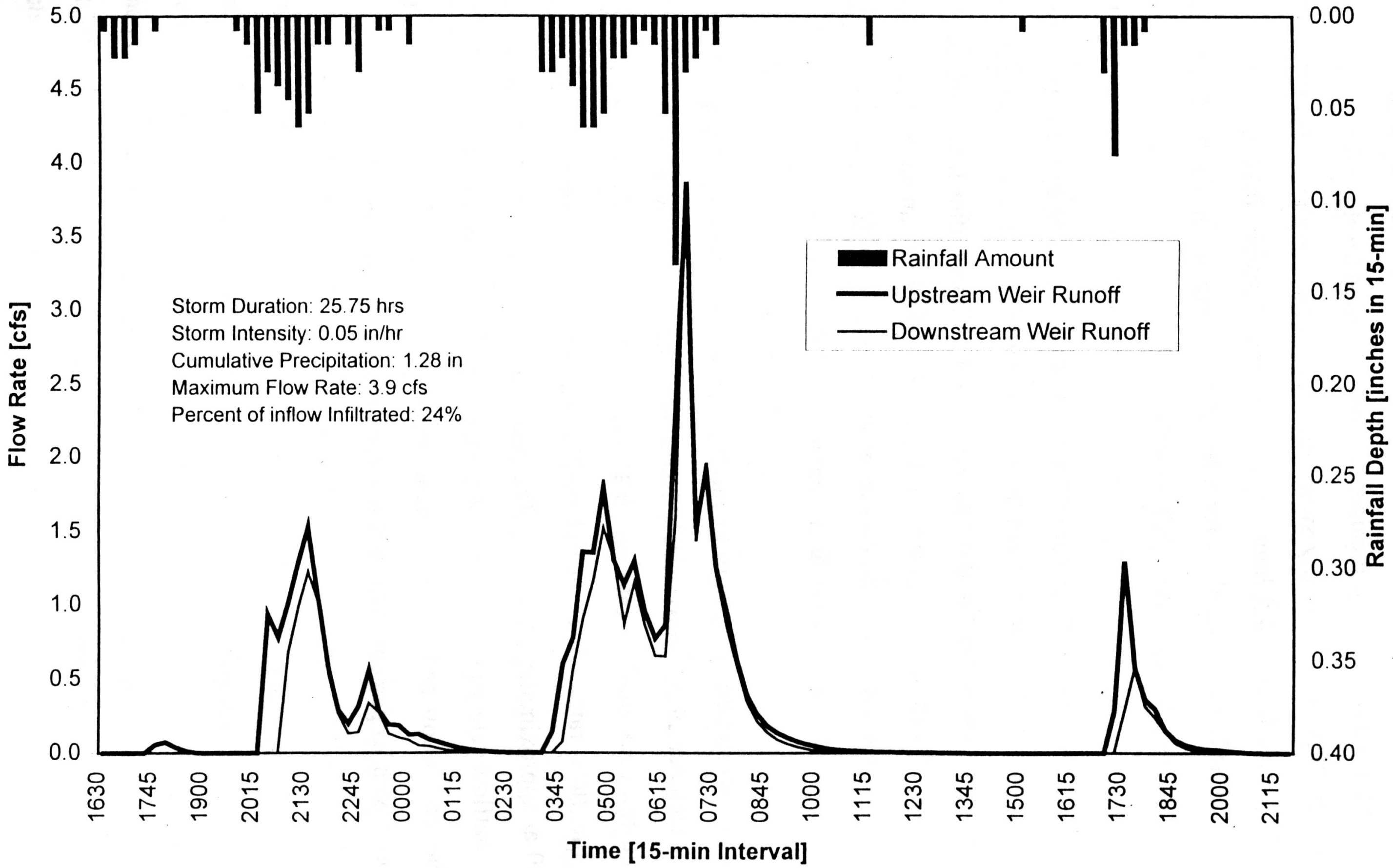

Figure 7-7: Highest-ranked precipitation and runoff for the Viewlands swale
October 19, 2000 (4:30 pm) - October 20, 2000 (9:30 pm)

A representative storm for the dry soil period occurred between September 29 and October 31 storm (Figure 7-6). No precipitation was recorded for the nineteen days prior to September 29, 2000, indicating dry swale soil conditions likely to be far below field capacity. The storm lasted 34.5 hours, produced approximately 0.84 in (2.1 cm) of precipitation, and had the third-highest recorded storm precipitation depth. The average precipitation intensity was 0.024 in/hr (0.061 cm/hr).

Figure 7-6 shows the precipitation hyetograph and runoff hydrograph for the storm. There was a lag time of one hour between the start of rainfall and flow over the upstream weir. Flow was measured over the downstream weir 4.25 hours after it was initiated at the upstream weir. As shown on Figure 7-6, the swale effectively attenuated the inflow volume and peak flow rate over the course of the storm, especially for precipitation depths less than 0.01 in (0.025 cm) (which went to depression storage).

There were two distinct periods of outflow production during the storm. The first period occurred on September 29 at approximately 10:00 pm. The peak inflow rate was 0.58 cfs (16.4 L/s) and the peak outflow rate was 0.32 cfs (9.1 L/s), resulting in a 45 percent decrease in the flow rate. The second period that produced outflow occurred on September 30 at approximately 2:00 pm. The peak inflow rate was 0.65 cfs (18.4 L/s) and the peak outflow rate was 0.19 cfs (5.4 L/s), resulting in a 70 percent decrease in flow rate. The total inflow and outflow volume was 9,050 ft^3 (256 m^3) and 2,480 ft^3 (70 m^3) respectively; with 73 percent of the inflow volume infiltrated/detained by the swale.

Wet (October 9, 2000 – January 31, 2001)

Twenty-nine storms occurred during predominantly wet swale soil moisture conditions. During the wet soil period, 34 percent of the measured inflow infiltrated and/or was detained by the swale. The storm that started on October 13, 2000 marked the beginning of wet swale soil moisture period. There were 4 days of no precipitation prior to October 13. The storm lasted 12 hours, with a lag time of 10.5 hours before runoff response in the channel. The inflow volume was 700 ft^3 (20 m^3) and it infiltrated completely.

October 16 - 20

The storms that occurred on October 16 – 20, 2000 delivered 1.86 in (4.72 cm) of precipitation over four days. The first discrete storm started on October 16 and delivered an inflow volume of 6,060 ft^3 (172 m^3). From the wet antecedent swale soil conditions caused by the October 13 storm, only 24 percent infiltration occurred. The second storm started on October 17, 2000 and delivered a precipitation depth of 0.13 in (0.33 cm) and an inflow volume of 740 ft^3 (21 m^3). The channel attenuated 100 percent of the inflow but resulted in saturated soil conditions that affected flow attenuation of the next storm. The third storm began on October 19, 2000 at 4:15 pm (Figure 7-7). It took 1.25 hours before flow over the upstream weir and 3.25 hours before flow over the downstream weir. The storm lasted 25.75 hours, had an average storm intensity of 0.05 in/hr (0.13 cm/hr), and delivered 1.28 in (3.25 cm) of precipitation.

This storm was the largest storm monitored, producing the highest maximum upstream and downstream flow rate of 3.9 cfs (110.4 L/s) and 3.8 cfs (107.6 L/s), respectively. The total inflow volume was 35,460 ft^3 (1004 m^3), and 24 percent was infiltrated. Figure 7-7 shows that the initial inflow on the rising limbs of the hydrograph went to storage and that attenuation of the peak flow rates occurred at both the start and tail ends of the storm. At the peak flow rate on October 20, 2000 at 6:45 am, only a 2 percent attenuation of the peak flow rate occurred. Following the peak, the downstream hydrograph closely matched the upstream hydrograph.

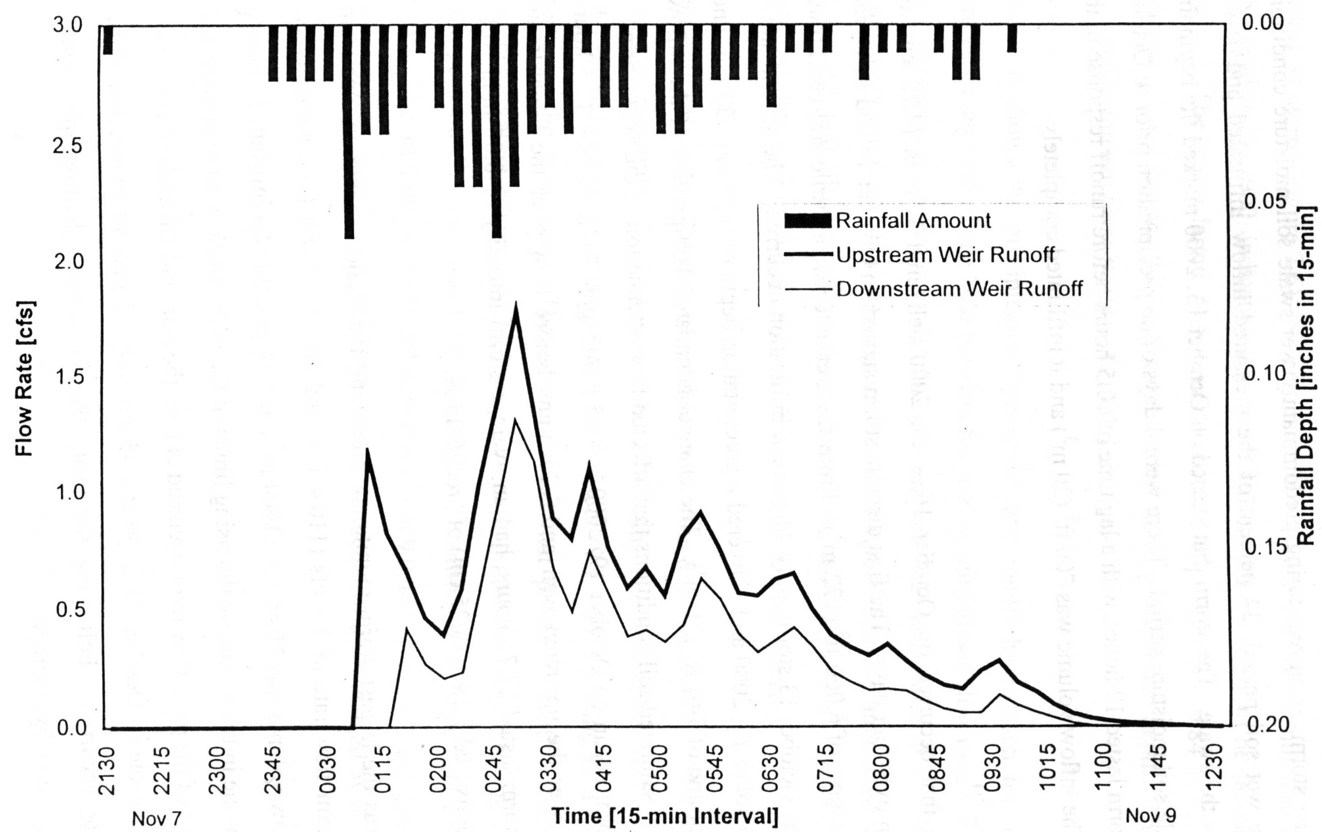

Figure 7-8: Precipitation hyetograph and runoff hydrograph for the Viewlands swale
November 7, 2000 (9:30 pm) - November 9, 2000 (12:30 pm)

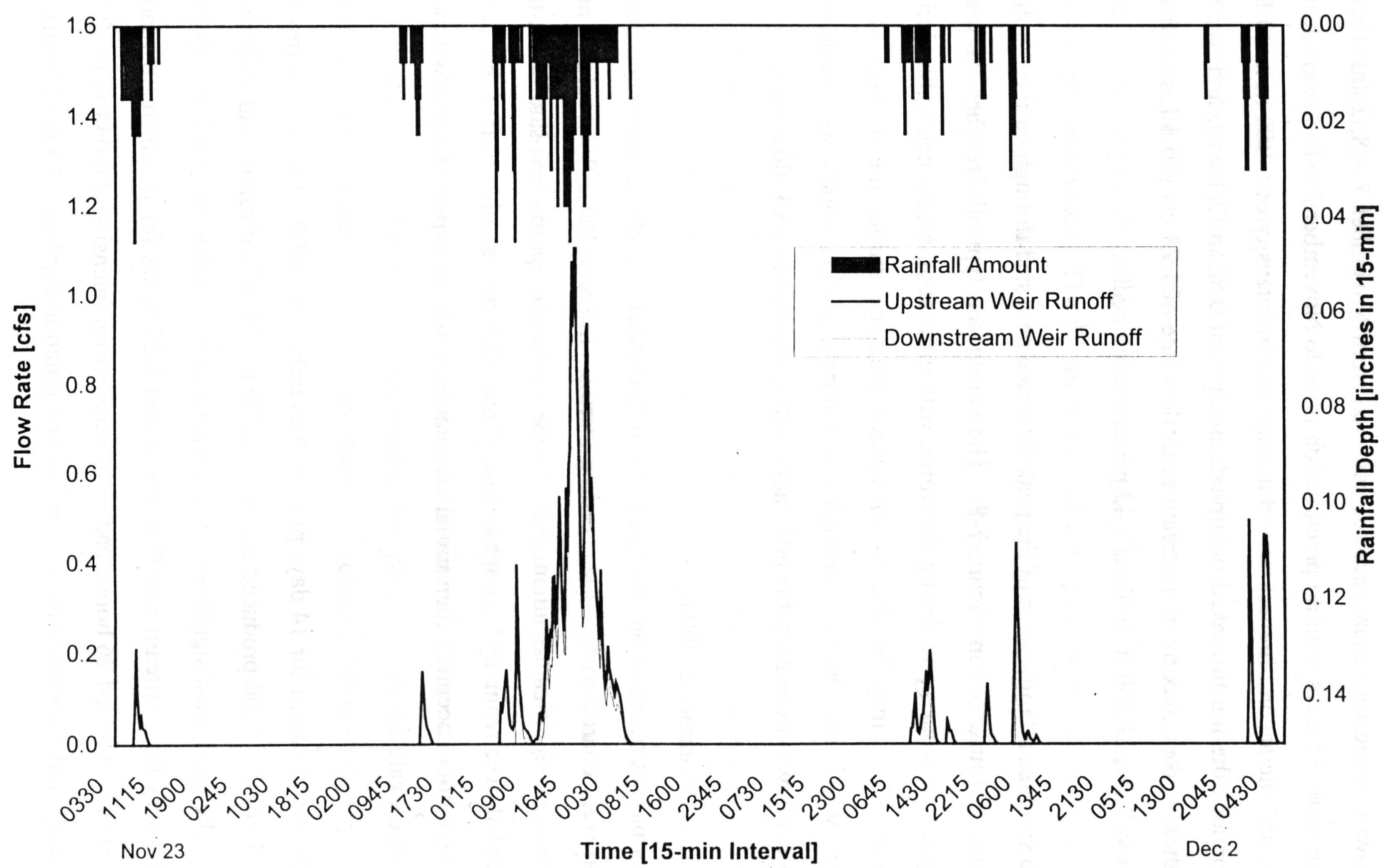

Figure 7-9: Precipitation hyetograph and runoff hydrograph for the Viewlands swale November 23, 2000 (3:30 am) - December 2, 2000 (3:30 am)

November 7 - 8

One storm with interesting characteristics occurred from November 7 – 8, 2000 (Figure 7-8). There were 3.5 days with no precipitation prior to November 7. The storm lasted 12.5 hours, had the second-highest ranked average storm intensity of 0.066 in/hr (0.167 cm/hr), produced the fourth-ranked precipitation depth of 0.82 in (2.1 cm), and had the second-highest ranked maximum upstream peak flow rate of 1.78 cfs (50.4 L/s). Of the total inflow volume, 21,260 ft^3 (602 m^3), 42 percent infiltrated.

Addition to storage and mitigation of the peak flow rate persisted throughout the duration of the storm, as illustrated in Figure 7-8. The subsurface typically reaches its water holding capacity at some point during the storm, with infiltration nearly negligible during the recession. This may be due to subsurface drainage during the 4 days of no precipitation, which may be long enough (even during predominantly wet swale soil conditions) to allow for considerable infiltration and mitigation of peak flow rates.

November 23 – December 2, 2000

The five storms that occurred on November 23 to December 2, 2000 demonstrated many features that were characteristic of succeeding storms. Initially, the inflow volume and the flow rate were significantly attenuated. As the swale soil approached saturation, the upstream and downstream hydrographs closely matched one another. If a break of greater than six hours occurred then initial attenuation could be expected for new runoff delivered to the swale.

There was no precipitation for 14 days prior to November 23, 2000. The two storms on November 23 and 25, 2000 produced less than 1,000 ft^3 (28 m^3) of runoff and infiltrated completely. The third most significant storm started on November 26, 2000 at 2:45 pm. It took 0.75 hours for upstream runoff response and 3.25 hours for downstream runoff response. This storm lasted 26 hours, had an average storm intensity of 0.044 in/hr (0.11 cm/hr), and delivered the second-highest ranked precipitation depth of 1.15 in (2.9 cm). Infiltration and attenuation of the peak flow rates occurred on the rising limb of the hydrograph (Figure 7-9). However, the peak upstream flow rate was 1.11 cfs (31.4 L/s) and the peak downstream flow rate was 1.08 cfs (30.6 L/s) with only a 2 percent reduction. The storm produced a total inflow volume of 23, 450 ft^3 (664 m^3), and infiltrated 20 percent of the inflow. Once the soil column was saturated there was little addition to storage on the falling limb of the hydrograph. Two subsequent storms took place on November 29 and December 2, 2000. Both were small storms whose runoff was initially attenuated. The fourth storm had a 13 percent reduction in the maximum peak flow rate and had a 67 percent of the water infiltrated. For the fifth storm, the maximum peak flow rate was reduced by 19 percent and 48 percent of the inflow infiltrated.

Results

These series of storms demonstrate the characteristic performance of the swale. A smaller storm with less than 1,000 ft^3 (28 m^3) of inflow and/or a 0.15 in (0.4 cm) precipitation depth, will infiltrate completely but bring the subsurface to near-saturation. A subsequent storm may have initial infiltration and attenuation of peak flow rates for the first hour. The subsurface quickly becomes saturated, however, causing the upstream and downstream flow rates to be nearly identical. If the swale has 2-4 days of no rain to allow subsurface drainage, then the cycle begins anew. Typically, sufficient subsurface water-holding capacity only occurs during predominantly dry swale soil conditions with infrequent storms. Due to the unseasonably low precipitation that occurred during the study, the swale frequently had the opportunity to “recover” between storms.

7.1.4 Hydrologic comparisons

From the study period of June 20, 2000 to January 31, 2001, 36 storms were recorded that delivered water to the Viewland swale. The maximum peak flow rate of 3.9 cfs (110.4 L/s) was recorded at the Viewlands swale on October 19, 2000. Over the course of the study period, the average swale water velocities ranged from a maximum of 2.7 ft/s (0.82 m/s) for the 3.9 cfs storm to 0.11 ft/s (0.03 m/s). The minimum swale flow-through times (channel volume/peak flow rates), ranged from 1.67 minutes (for the 3.9 cfs storm) to 41 minutes. Table 7-3 lists storm dates, a qualitative assessment of soil antecedent moisture conditions in the swale, and the total precipitation depths recorded, and the duration of the storm systems.

Table 7-3: Division of the precipitation record into discrete storms

Storm System Date	Antecedent Soil Moisture	Precipitation Depth [in]	Duration [days]
Jul 3 - 4	dry	0.15	2
Jul 22 – 23	dry	0.28	2
Aug 18	dry	0.28	1
Aug 29	dry	0.15	1
Sep 9 – 10	dry	0.24	2
Sep 29 – Oct 1	dry	0.84	3
Oct 9	dry	0.35	1
Oct 13 - 14	wet	0.19	2
Oct 16 - 20	wet	1.86	5
Oct 27 – 29	wet	0.76	3
Nov 4	wet	0.31	1
Nov 7 - 9	wet	0.92	3
Nov 23 – Dec 2	wet	2.16	10

Dec 8	wet	0.10	1
Dec 14 - 16	wet	0.96	3
Dec 21 - 25	wet	0.96	5
Dec 31	wet	0.17	1
Jan 4 -5	wet	0.78	2
Jan 8 - 9	wet	0.23	2
Jan 13 -14	wet	0.08	2
Jan 17 - 21	wet	1.16	5
Jan 28 - 29	wet	0.40	2

Percent infiltration

Of the 36 individual storm that produced measurable runoff in the Viewland swale, runoff infiltrated completely for 14 storms. Five of the storms with 100 percent infiltration occurred during dry antecedent soil moisture conditions. The other nine occurred during wet antecedent soil moisture conditions. Water from the 13 storms that produced an inflow volume less than 1,000 ft^3 (28 m^3) infiltrated completely. Generally, storms that produce less than 1,000 ft^3 of inflow typically have a high swale infiltration rate of greater than 90 percent.

Table 7-4 describes the average and approximate range of inflow rates and precipitation depths classified by varying rates of infiltration. Included in the table are the number of storms recorded at the given infiltration range. The threshold is 1,000 ft^3 for complete infiltration and approximately between 1,000-3000 ft^3 for high infiltration (75 – 99.9%), regardless of the swale soil moisture conditions. For low infiltration (0 – 24.9 %), the total inflow volume exceeded approximately 15,000 ft^3 (425 m^3).

Table 7-4: Classification of the average and approximate range of inflow rate and precipitation depth by percent infiltration of runoff

Percent Infiltration [%]	Number of Storms	Inflow Volume (Average and Range) [ft^3]	Precipitation Depth (Average and Range) [in]
100	14	570 (<1,000)	0.17 (< 0.2)
(75 – 99.9)	4	1,490 (1,000 - 3000)	0.17 (< 0.2)
(50 – 74.9)	5	4,670 (3,000 - 6000)	0.45 (0.2 – 0.5)
(25 – 49.9)	9	8,010 (6,000 – 15,000)	0.52 (0.5 – 0.8)
(0 – 24.9)	4	21,630 (> 15,000)	0.91 (> 0.8)

For the 14 storms where 100 percent of the inflow was infiltrated, the swale effectively attenuated an average precipitation depth of 0.17 in (0.43 cm). During dry soil moisture conditions, the swale held an average precipitation depth of 0.22 in (0.56 cm), and during wet soil moisture conditions it held an average precipitation depth of 0.13 in (0.33 cm). The threshold for the precipitation depth corresponds to high attenuation (75-100% infiltration) of approximately 0.2 in (0.51 cm), regardless of the soil moisture conditions.

Peak flow rate

For the storm hydrographs that were analyzed, there was either complete attenuation of the peak flow rate or there was modest (<20 %) reduction in the peak flow rates, with only eight exceptions. Table 7-5 describes the average percent infiltration into the swale classified by varying rates of reduction in peak flow rates. Included in the table are the number of storms recorded at the specified range.

Table 7-5: Classification of percent infiltration by percent reduction in peak flow rate

Percent Reduction in Peak Flow Rate [%]	Number of Storms	Percent Infiltration (Average) [%]
100	14	100
(75 – 99.9)	1	79
(50 – 74.9)	1	51
(40-49.9)	4	43
(30-39.9)	0	-
(20-29.9)	2	25
(10-19.9)	5	14
(0 – 9.9)	10	4

The reduction in peak flow rates was associated with antecedent swale soil moisture conditions and the duration of flow in the channel. Antecedent swale soil moisture conditions are indicative of the amount of subsurface storage capacity available at the start of the storm. The soil water storage capacity quickly tapered off after the start of inflow into the channel. The greatest reduction in peak flow rates occurred within the first 30 minutes of measurable swale inflow. After saturation of the subsurface (within 30 minutes to 6 hours), the swale acted primarily as a conveyance system and less as a detention or infiltration system.

6-month, 24-hour storms

Two storms exceeded a 6-month 24-hour storm, which is defined by Seattle Public Utilities as having a precipitation depth of 1.08 in (2.74 cm). The storm on October 19, 2000 produced 1.28 in (3.25 cm) in 25.75 hours. This equated to a 24-hour precipitation depth of 1.14 in (2.9 cm) and an average storm intensity of 0.047 in/hr (0.12 cm/hr). The storm on November 26, 2000 produced a storm depth of 1.15 in (2.9 cm) in 26 hours.

This also equated to 1.14 in (2.9 cm) in 24-hrs and an average storm intensity of 0.047 in/hr (0.12 cm/hr). For these storms, the percent reduction in peak flow rates was 2 percent and 2.7 percent and the percent of inflow infiltrated was 24 percent and 19.6 percent, respectively.

The remaining 34 storms fell below the 6-month, 24-hour storm threshold. Hence the swale has yet to experience runoff from storms up to the design maximum of a 25-year, 24-hour storm.

7.1.5 Water Quality

Physical and chemical constituents in water are customarily quantified in terms of their concentration (mass/unit volume) and mass loading (mass flux/unit time). Mass loading is the product of concentration and volumetric flow rate. Pollutants such as metals, synthetic organic chemicals, nutrients, and pathogens are associated with the sediments to varying degrees, and thus would decrease as a function of the captured sediment. While re-entrainment of trapped pollutants can and does occur, field observations of the Viewlands swale suggest a net reduction of masses and concentrations of oils, sediments, and associated pollutants occurred.

From observation, water entering Cell 1 was turbid and oily, carrying suspended fines, large gravel, small boulders, asphalt, leaf litter, and trash. After a storm ceased, Cell 1 was filled to the weir invert and an oily film formed on the water surface. Water typically remained in Cell 1 for at least 2 to 3 days, eventually infiltrating into the ground and/or evaporating. During this time, solids settled out and oils deposited on the sediment surface. Over the course of one year, Cell 1 accumulated a layer of sediment 1 to 2 ft (0.3 to 0.6 m) thick.

Cells 2 to 5 accumulated substantial amounts of fine sediment. During small storms, water infiltrated rapidly and surface flow never reached the downstream cells. Abundant plant growth occurred in these upper cells, which helped to uptake nutrients, filter solids and fines, and provide surfaces for oil deposition. Infiltrated water in the upper cells appeared to reach the impermeable glacial till layer approximately 6 ft (1.8 m) in depth, spread laterally, and reemerge in the lower four cells (Cells 13 to 16). The water was filtered by the swale bed and subsurface layers and reemerged clear.

Overall, the swale appeared to be effective at retaining large particles and oil in Cell 1, removing fines in Cells 2 to 5, and reducing turbidity as the water reemerges from the soil into Cells 13 to 16 (especially at the observed low flow rates). But improvements in water quality decreased under saturated swale soil conditions and large inflow rates, which occurred predominantly from runoff produced by storms that exceeded the 6-month 24-hour storm.

This analysis will be made more formal when additional flow data and future water quality monitoring are available. However, the overall 38 percent flow volume decrease from inlet to outlet over the course of the hydrologic monitoring period reliably indicates a likely proportional reduction of mass loading to the receiving water occurred. Declines of this magnitude, most likely supplemented by pollutant mass reductions, represent a substantial potential ecological benefit to pollutant sink areas, such as water bodies with relatively slow water exchange (i.e., many lakes, Puget Sound) and marine and freshwater sediments.

7.1.6 Comparison with pre-construction conditions

The purpose of the preconstruction analysis for the Viewlands site is to estimate how the previous concrete/vegetated channel would have responded under the same meteorological conditions that occurred from June 2000 to January 2001. Project success is based in part, by the new swale's ability to outperform the old channel in effectively decreasing storm water quantities.

The pre-construction analysis for the Viewland swale was based on making an area-to area-comparison of the measured volume of water infiltrated over the wetted area for each storm in the new swale; then apportioning the volume over the wetted area of the vegetated portion of the old ditch. The main assumption is that the infiltration rate through the subsurface (predominantly native glacial till) and the storage volume was the same for both conditions. Hence, all water that could possible infiltrate was stored in the subsurface. In making this assumption, the upper limit of infiltration for the old ditch was actually determined.

The maximum surface area for the new swale at low flow rates is approximately 2,580 ft^2 (240 m^2) and at high flow is approximately 2,980 ft^2 (277 m^2). The maximum surface area for the old vegetated ditch was approximately 1,430 ft^2 (133 m^2). There was 45 percent less potential surface infiltration area in the old vegetated ditch than in the Viewland swale at low flow and 52 percent less area at high flow.

Over the course of the study period, the new swale infiltrated 73,710 ft^3 (2,090 m^3) of water. Under the same meteorological conditions, but different channel dimensions and infiltration area, the old vegetated ditch would have infiltrated at maximum, 24,650 ft^3 (700 m^3). This is 67 percent less infiltrated water than in the new system. Table 7-6 shows the five storms of highest peak inflow rate, the total infiltrated volume over the course of the storm, and the percent difference in infiltrated volume between the old vegetated ditch and new swale. Appendix G details the pre-construction methodology.

Table 7-6: Comparison of current swale and previous ditch performance in relation to infiltrated volume of stormwater for the five highest-ranked storms

Date	Maximum Flow Rate [cfs]	Current Swale: Infiltrated Volume [ft^3]	Previous Ditch: Infiltrated Volume [ft^3]	Percent Difference [%]
1. 10/19-20/2000	3.88	8515	3436	60
2. 11/7-8/2000	1.78	9016	3239	64
3. 1/4-5/2001	1.54	3495	1233	65
4. 11/4/2000	1.45	1814	635	65
5. 11/26-27/2000	1.11	4593	1557	66

7.2 SEA Streets Site

7.2.1 Characterization of flow patterns

The SEA Streets project evaluation depends on three scenarios:

1. Performance during baseline data collection prior to construction;
2. Comparison of baseline versus conventional street design performance; and
3. Comparison of baseline versus constructed performance.

Baseline conditions have been monitored from March 11, 2000 to July 11, 2000. Hydrograph analysis of the precipitation-runoff response was the main method of evaluating the pre-construction hydrologic regime of the SEA Streets site. Comparisons of the baseline design to both a conventional street and constructed SEA Streets design was evaluated by estimating the volumes that might have been produced under the same meteorological conditions that occurred during the study period.

Hydrograph analysis

Comparisons were made of the patterns and magnitude of precipitation-runoff response. The discussion of the storm hydrographs will focus on the 35 storms that took place during predominantly wet antecedent street moisture conditions at the SEA Street site. From the SEA Streets precipitation record, there were 18 days on which precipitation occurred in March, 12 days in April, 18 days in May, and 9 days in June (Appendix H contains a complete description of all the recorded SEA Street storms). Table 7-7 lists storms, a qualitative assessment of street antecedent moisture conditions, and the recorded precipitation depths.

The minimum precipitation depth that produced runoff was 0.04 in (0.1 cm). Any precipitation below this amount either went to depression storage or was retained in the lawns. The maximum observed precipitation depth was 0.63 in (1.6 cm). This cannot be considered representative of the region, due to the limited sampling period and drier-than-normal conditions for year 2000.

Table 7-7: Division of the precipitation record into discrete storms

Storm System Date	Antecedent Soil Moisture	Precipitation Depth [in]	Duration [days]
Mar 9 - 23	wet	1.76	15
Mar 27 - 29	wet	0.28	3
Apr 4 - 6	wet	0.37	3
Apr 13 - 14	wet	0.4	2
Apr 21 - 25	wet	0.43	4
May 8 - 11	wet	0.84	3
May 18 - 22	wet	0.51	5
May 26 - 31	wet	0.76	6
Jun 6 - 18	wet	0.97	13

June 6 - 18

The storm system of June 6 – 18 demonstrated many features that were characteristic of the precipitation-runoff response patterns at SEA Streets. The runoff hydrograph closely follows the start, rise, and fall of the precipitation hyetograph. The series of storms produced approximately 0.97 in (2.46 cm) of precipitation in four distinctive storms. There were four days of no precipitation prior to the first storm on June 6, 2000. Figure 7-10 shows immediate runoff response to the precipitation in all four storms.

The third storm on June 11, 2000 had the highest-ranked maximum peak flow rate of 0.083 cfs (2.4 L/s), total runoff production of 1,910 ft^3 (54 m^3) and maximum storm duration of 23.25 hours. The maximum observed flow rate of 0.083 cfs (2.4 L/s) is significantly below the anticipated 1.5 cfs (42.5 L/s) peak flow rate expected from a 25-yr, 24-hr storm. Hence, the street has yet to experience any substantial storms. Figure 7-11 shows the hydrograph for June 11, 2000.

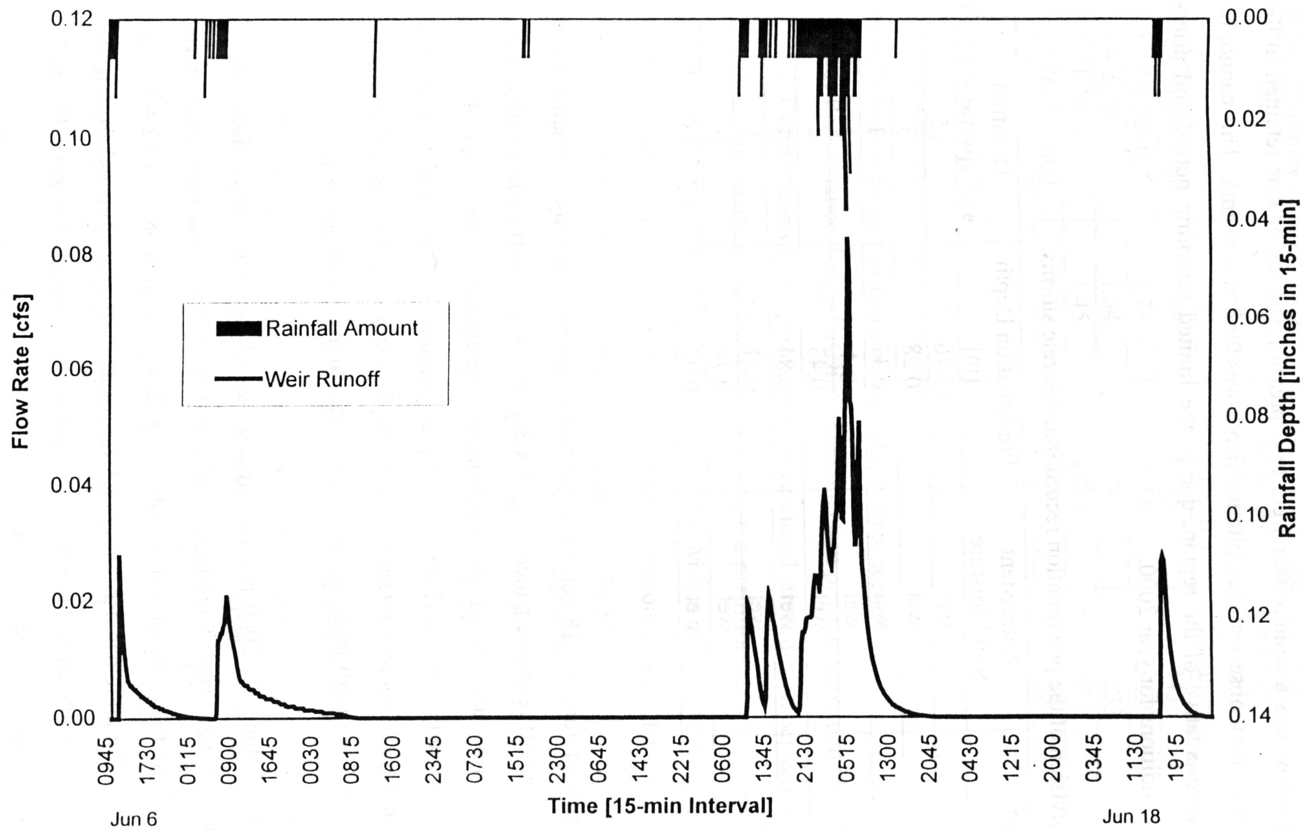

Figure 7-10: Precipitation hyetograph and runoff hydrograph for the SEA Streets Project
June 6, 2000 (9:45 am) - June 18, 2000 (12:45 am)

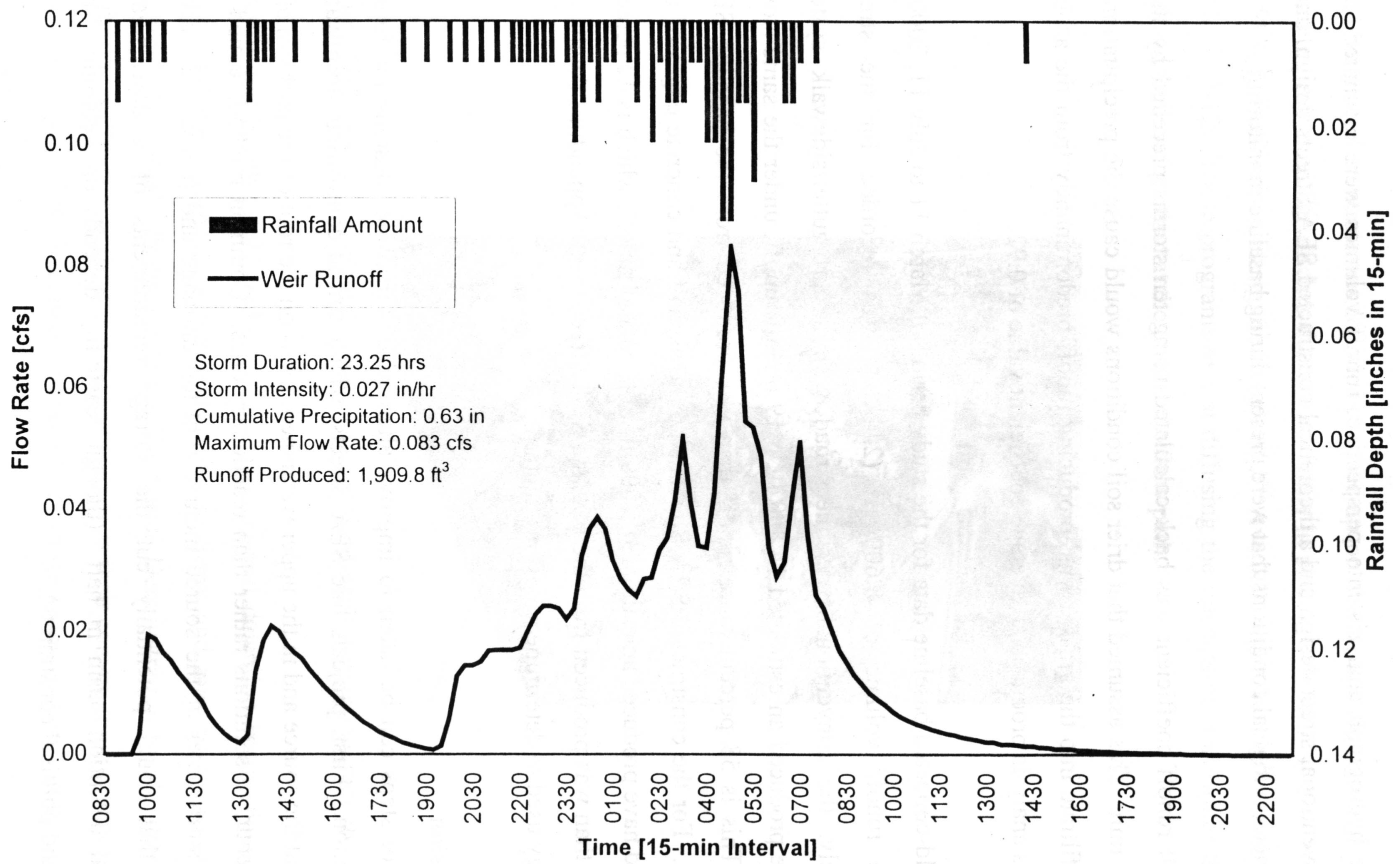

Figure 7-11: Highest-ranked precipitation and runoff for the SEA Streets Project
June 11, 2000 (8:30 am) - June 12, 2000 (9:00 pm)

7.2.2 Comparison with constructed conditions

To put the hydrograph analysis into perspective, runoff volumes were estimated from both a conventional street design and a theoretical constructed SEA Streets design, under the same meteorological conditions that were present during baseline monitoring.

The asphalt runoff coefficient was back-calculated using ten storms preceded by three days of no rain. This assumed that drier soil conditions would cause the precipitation to initially infiltrate into the grass lawns, producing runoff predominantly from the asphalt street. This analysis produced an average coefficient value of 0.92.

For the field-collected baseline data for the study period of March 11 to July 11, 2000, a cumulative runoff volume of 8,600 ft^3 (244 m^3) was recorded for the street. Alternatively, the conventionally-designed road with a curb/gutter/sidewalk system would have produced an estimated 14,800 ft^3 (419 m^3) of runoff, under the same storm patterns. This is 58 percent more runoff than was produced from the baseline SEA Streets site. For the constructed SEA Streets site, the asphalt and concrete edges of the street would have produced an estimated 5,000 ft^3 (142 m^3) of runoff, which is 42 percent less runoff than was produced from the baseline SEA Streets site. Appendix I details the methodology used to determine runoff production.

7.3 Discussion

A number of steps can be taken to improve alternative stormwater designs for future implementation. First, projects like SEA Streets, which aim at controlling stormwater production at the source and in the upper watershed, focus on the root of the problem and act as preventative solutions rather than reactionary ones. Common methods aimed at controlling stormwater at the source include roof-top drainage and porous sidewalks/ driveways that could potentially cut the storage requirements of a downstream conventional detention basin in half (though feasibility depends on geologic, soil, economic, and political constraints).

Since more control can be made on city right-of-ways, similar smaller swale designs could be placed at numerous locations higher up on the watershed, replacing the open concrete-lined drainage ditches that are currently in place. One such system is located at the SEA Streets project along the length of 2nd Ave NW and NW 120th Street, as shown in Figure 7-12. These programs aim at diffusing the impacts of stormwater throughout the watershed, rather than at the end of the flow path.

Figure 7-12: Swale system that replaced a concrete-lined open drainage ditch

Jackson and Booth (1997) indicated that conventional detention basins do not provide enough storage volume to accommodate design flows and may be up to an order of magnitude too small. Projects like the Viewlands Demonstration Swale attempt to use alternative methods to accomplish the same goals as their conventional predecessors. Yet they are constrained by available land area and storage volume, which ultimately hinders their effectiveness.

A coupling of the two methods may be necessary to achieve the kind of mitigation that is necessary in dealing with urban-elevated stormwater volumes. First a conventional analysis must take place to determine the necessary size of the detention basin (regardless of whether the end design is a swale design or conventional design). Runoff from storms less than the 6-month, 24-hour storm might be accommodated by a swale the size of the Viewlands Swale. For runoff from larger storms, an open or underground detention vault sized to adequate standards is needed to detain water for slow release. Such vaults could be designed to be a part of the swale or may by placed in locations like street intersections in the upper watershed.

If coupling an underground detention basin with a swale is economically infeasible, swales (like at Viewlands) that are implemented immediately upstream of the natural drainage network should focus on increasing subsurface storage capacity and hydraulic residence time. This may be done during construction, by replacing the native soil with backfill mixed with sand or gravel to increase the hydraulic conductivity of the soil. In addition, the depth of flow should be greater than 0.9 ft (0.27 m) to create larger standing pools of water behind each log weir to increase the hydraulic residence time (especially if the channel bed slope is fixed).

CHAPTER 8 – SUMMARY AND CONCLUSIONS

8.1 Summary of Project Goals

The preceding chapters detailed the development of the monitoring efforts of two Seattle Public Utilities' "ultra-urban" stormwater projects. This report examined the hydrologic and hydraulic performance of the Viewlands Swale during post-construction monitoring, and compared it to the estimated response of the replaced channel. At the SEA Street site, baseline performance was monitored and compared to the simulated performance of both a conventional roadway design and a constructed SEA Streets design. The monitoring program yielded one year of precipitation data, seven months of post-construction flow data for the Viewlands site, and five months of baseline pre-construction flow data for the SEA Streets site.

8.2 Data accuracy and analysis

Precipitation and runoff data were collected for a number of storms. Once problems with establishing and operating the monitoring stations were solved, the focus was on creating reliable and accurate data records. This was accomplished by establishing an extensive quality assurance/quality control (QA/QC) system of multiple checks for precipitation, runoff measurements, and data analysis.

Attempts were made to quantify errors that arose from both the equipment and from environmental changes in the systems being measured. Inflow to and outflow from the Viewlands weirs were measured to quantify the volume of leakage occurring under and around the V-notch weirs. Over the course of the study period, the volume lost to leakage caused an underestimation of the measured flow entering and exiting the Viewlands swale. The most substantial leakage occurred below flow rates of 0.15 cfs. At these low levels, the majority of the inflow leakage was stored in the swale and was never recorded downstream. Continued tests are needed to determine the leakage rates at both high flows and at varying swale soil moisture conditions.

8.3 Performance of the project sites

8.3.1 Viewlands Demonstration Swale

For the period of July 2000 to January 2001, the Viewlands flow monitoring equipment registered a peak upstream flow rate of 3.9 cfs (110.4 L/s), approximately one-sixth of the anticipated peak flow rate for the 25-yr 24-hr design storm of 25 cfs (708 L/s). Only two storms were recorded that were large enough to be approximately equivalent to the 6-month 24-hr storm. The remaining 34 storms fell beneath this level. Due to the unseasonably low precipitation, assessment of the performance of the swale design based on calendar year 2000 is limited.

The estimated average water velocities through the swale ranged from a maximum of 2.7 ft/s (1.2 m/s), for the larger flow rates (3.9 cfs), to a minimum of 0.11 ft/s (0.03 m/s). The minimum hydraulic residence times (channel volume/peak flow rates) ranged from 1.67 minutes, for the larger flow rates, to 41 minutes. Any storms above the maximum observed peak flow rate will have short residence times of less than two minutes and high velocities greater than 3 ft/s (0.9 m/s).

Of the 36 individual storms that produced measurable runoff in the Viewland swale, 14 storms produced inflow that was completely infiltrated. Regardless of soil moisture conditions, 1,000 ft^3 (28.3 m^3) was the threshold for complete infiltration, while high infiltration (75 to 99.9%) was achieved for inflow volumes in the range of between 1,000 to 3000 ft^3 (28.3 to 85.0 m^3). The swale could fully attenuate runoff from an average precipitation depth of 0.22 inches (0.56 cm) during dry swale soil moisture conditions and 0.13 inches (0.33 cm) during wet conditions. During the dry swale soil period, 78 percent of the measured inflow infiltrated and/or was detained by the swale. During the wet soil period, 34 percent of the measured inflow infiltrated and/or was detained by the swale. Over the course of the study period, 38 percent of the total inflow was detained in the system.

The highest reductions in peak flow rate were coupled with the highest percentage of infiltration into the swale. For the storm hydrographs that were analyzed, there was either complete attenuation of the peak flow rates or there were modest (<20 %) reductions in the peak flow rates. Peak flow rate reductions were associated with antecedent swale soil conditions and the duration of flow in the channel. Once the subsurface soil void space was saturated (between 30 minutes to 6 hours), the inflow and outflow rates consistently matched one another (shown in the Chapter 7 hydrographs).

The pre-construction analysis for the Viewlands site involved a plan area-to-plan area comparison of the volume of water infiltrated over the wetted area for each storm in the current swale and previous vegetated ditch. Over the course of the study period, the current swale infiltrated an estimated 73,710 ft^3 (2,090 m^3) of water. Under the same meteorological conditions, but different channel dimensions and infiltration area, the previous vegetated ditch would have infiltrated at maximum an estimated 24,650 ft^3 (700 m^3. This is 67 percent less water detained than in the new swale system.

8.3.2 SEA Streets site

For the period of March 2000 to July 2000, the SEA Streets flow monitoring equipment registered a peak flow rate of 0.083 cfs (2.4 L/s), less than one-tenth of the anticipated peak flow rate of 1.5 cfs (42.5 L/s), for a 25-yr, 24-hr storm. Analysis of the storm hyetographs and hydrographs was performed on the 35 storms that took place during predominantly wet antecedent soil moisture conditions. The dominant hydrologic characteristics of the residential block were that the runoff response was precipitation-driven and rapid. As a result, the runoff hydrograph closely followed the start, rise, and fall of the precipitation hyetograph.

To put the hydrograph analysis of the baseline SEA Streets conditions into perspective, runoff volumes were estimated for both a conventional street design and for the SEA Streets constructed design. For the field-collected baseline data for the study period of

March 11 to July 11, 2000, a cumulative measured runoff volume of 8,600 ft^3 (244 m^3) was produced from the street. Alternatively, the conventionally designed road with a curb/gutter/sidewalk system would have produced an estimated 14,800 ft^3 (419 m^3) of runoff under the same storm patterns. This is 58 percent more runoff than was produced from the SEA Street site during baseline collection. For the constructed SEA Streets site, the asphalt and concrete edges of the street would have produced an estimated runoff volume of 5,000 ft^3 (142 m^3). This is 42 percent less runoff than was produced from the baseline SEA Streets site, and 66 percent less runoff than would be produced from the conventionally designed system.

8.4 Conclusions

From the data collected and reported here, the Viewlands Demonstration Swale was able to attenuate runoff volumes generated from the 6-month, 24-hr duration storm. The main constraint on swale performance is in its limited soil water storage capacity. An analogy can be drawn between the Viewland swale and a small detention basin, with a pipe carrying flow into and out of the swale. The immediate benefits in the reduction of peak flow rates and runoff volumes come at the start of inflow into the swale. Once the soil water storage capacity was reached, the inflow and outflow rates matched. At that point, the swale acted more as a wide conveyance system rather than as a detention/infiltration basin.

In terms of water quality observations, the swale appeared to be effective at retaining large particles and oil in Cell 1, removing fines in Cells 2 to 5, and reducing turbidity as the water reemerged from the soil into Cells 13 to 16, especially at the observed low flow rates. Improvements in water quality decrease under saturated swale soil conditions and large inflow rates, which occur predominantly from runoff produced by storms exceeding the 6-month 24-hour storm. This analysis will be made more formal when additional flow data and future water quality monitoring are available. However, the overall 38 percent flow volume decrease from inlet to outlet over the course of the hydrologic monitoring period reliably indicates a likely proportional reduction of mass loading to the receiving water occurred.

From the data collected during baseline monitoring of the SEA Streets site, the precipitation-runoff response for the residential street was established. The runoff from the road was precipitation-driven and response at the monitoring swale was rapid. The main constraint on attenuation of flow rates and stormwater volumes was the limited infiltration opportunity on the residential block due to the large tracts of impermeable area. The constructed SEA Streets site included conversion of 28 percent of the impermeable roadway to sedimentation ponds. Continued post-construction monitoring and analysis are needed to quantify the benefits of the street redesign on the reduction of peak flow rates and runoff volumes.

8.4 Recommendations for future research

There are many aspects yet to be investigated with the "ultra-urban" projects. The data reported here show the limitations in the current designs, but there are many areas for potential improvement in both the monitoring and designing of alternative stormwater techniques. The following are observations that should be addressed and/or performed in future works:

1. Continued monitoring is needed to determine how both projects respond to storms greater than the 6-month, 24-hr storm;

2. Improvement of future designs of swales is needed to address the given geologic and land area constraints. The upper attenuation limits of the swale should be assessed, and if feasible, the design should be a coupled with a conventional detention system that is aimed at attenuating stormwater volumes produced by the large storms.

3. Investigate source control methods in the upper watershed, which may include both residential-scale and potential City right-of-way runoff control locations.

4. Conduct water quality monitoring to quantify the ecological benefits of the Viewlands swale and the SEA Streets redesign project.

5. At the Viewlands site, the recorded meteorological data (solar radiation, evaporation, temperature, relative humidity, etc.) should be used to construct mass water and energy balances for the site. This would provide information on the hydrologic dynamics of the highly urban Pipers Creek Watershed.

The above recommendations are aimed at further establishing the behavior of both projects during storms of greater frequency, magnitude, and soil moisture conditions then have occurred over the study period. The data analyzed to date indicate that the Viewlands swale will be limited in attenuating runoff volumes produced by storms that exceed the 6-month, 24-hr precipitation depth of 1.08 inches. The performance of the constructed SEA Streets project has yet to be determined, although its fundamental design attempts to mitigate the problems of urbanization. By attempting to diffuse the impacts of stormwater throughout the upper developed watershed, downstream stormwater detention facilities and natural drainage networks will not be forced to bear the "burden" of urbanization.

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Model MSX10 Solar Panel, Instruction Manual (1994)

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Model 260-2520 Forestry Rain Gauge, Instruction Manual (1997)

APPENDIX A

Appendix A – ArcView GIS delineated watershed

From the Washington State Geospatial Data Archive (http://wagda.lib.washington.edu), the City of Seattle's metadata sets were available and gave a near complete description of the Pipers Creek watershed. Shapefiles described the following features:

- Drainage basin boundaries;
- Natural stream and the major stormwater drainage networks;
- Street network;
- Riparian corridors;
- Potential landslide areas;
- Land use zones;
- Land and water body boundaries; and
- City parks (where I extracted Carkeek Park).

City of Seattle data parameters:

1. Coordinate System:
    - Washington State Plane, North Zone:
        a. 1^{st} Standard Parallel (D M S): 47 30 00
        b. 2^{nd} Standard Parallel(D M S): 48 44 00
        c. Central Meridian(D M S): -120 50 00
        d. Origin (latitude) (D M S): 47 00 00
        e. False Easting (m): 500,000

2. Datum:
    - Horizontal: North American Datum of 1983, 1991 adjustment
    - Vertical: North American Datum of 1998

3. Projection:

- Lambert Conformal Conic Projection

APPENDIX B

Appendix B: CR10X data logger records

The post-construction monitoring period for the Viewlands site occurred from January 2000 to January 2001. Over that time period, three CR10X data loggers were in operation and collected hydrologic and meteorologic data continuously at 15-minute increments. The baseline monitoring period for SEA Streets site occurred from March 2000 to July 2000. The following tables give a description of each piece of equipment connected to the data logger, the start of its record, and problems encountered.

CR10X Data logger on CM10 tripod weather station

Gauge	Start of Record	Problems
1. TB3 Trench Rain Gauge [No. of tips]	1/1/2000	None
2. TB3 Standing Rain Gauge [No. of tips]	1/1/2000	None

CR10X data logger measuring flow in the Viewlands swale

Gauge	Start of Record	Problems
1. Upstream Shaft Encoder [ft]	9/26/2000	Prior technical problems with Campbell Scientific
2. Downstream Shaft Encoder [ft]	3/11/2000	Switched to a new CR10X Datalogger on 10/1/2000
3. Upstream Pressure Transducer [ft]	6/20/2000	None
4. Downstream Pressure Transducer [ft]	6/20/2000	None
5. Evaporative Pan Pressure Transducer [ft]	6/20/2000	None

CR10X data logger at the SEA Streets site

Gauge	Start of Record	Problems
TB3 standing rain gauge	3/4/00 - 7/27/00	None; Pre-construction

[in]	1/19/00	Post-construction
Shaft encoder [ft]	3/11/00 - 7/11/00 1/19/00	None; Pre-construction Post-construction

APPENDIX C

Appendix C: TB3 monthly precipitation record

The tables below contain the monthly precipitation record for each TB3 precipitation gauge for the year 2000 for both the Viewlands and SEA Streets sites. To create the monthly summaries, the amount of precipitation was summed per month. The number of days it rained is also noted in the tables below.

Month	Day Start	Day End	Viewland Trench Total [in]	Viewland Trench Max [in]	Viewland Standing Total [in]	Viewland Standing Max [in]	SEA St Standing Total [in]	SEA St Standing Max [in]	No of Rain Days
Jan	1	31	2.44	0.60	2.77	0.67			15
Feb	32	60	4.53	0.89	5.12	1.07			17
Mar	61	91	2.86	0.41	3.17	0.48	2.15	0.45	18
Apr	92	121	1.32	0.46	1.43	0.54	1.29	0.39	12
May	122	152	2.68	0.44	2.61	0.42	2.14	0.42	18
Jun	153	182	1.21	0.45	1.37	0.50	1.06	0.39	9
Jul	183	213	0.47	0.28	0.50	0.27	0.44	0.28	6
Aug	214	244	0.44	0.28	0.44	0.27			3
Sept	245	274	1.29	0.43	1.29	0.43			11
Oct	275	305	3.19	0.82	3.33	0.82			13
Nov	306	335	3.26	1.08	3.32	1.12			12
Dec	336	366	2.63	0.53	2.64	0.51			19
			26.32		**27.98**		**7.09**		**153**

MONTH	Day Start	Day End	Viewland Trench Total [mm]	Viewland Trench Max [mm]	Viewland Standing Total [mm]	Viewland Standing Max [mm]	SEA St Standing Total [mm]	SEA St Standing Max [mm]	No of Rain Days
Jan	1	31	62.07	15.21	70.36	16.95			15
Feb	32	60	115.01	22.72	130.09	27.20			17
Mar	61	91	72.62	10.34	80.42	12.22	54.63	11.50	18
Apr	92	121	33.47	11.76	36.27	13.60	32.78	9.97	12
May	122	152	67.95	11.16	66.23	10.64	54.44	10.54	18
Jun	153	182	30.83	11.36	34.89	12.81	27.03	9.97	9
Jul	183	213	11.97	7.03	12.61	6.90	11.12	7.09	6
Aug	214	244	11.21	7.22	11.23	6.90			3
Sept	245	274	32.86	11.02	32.72	11.04			11
Oct	275	305	80.91	20.70	84.56	20.89			13
Nov	306	335	82.81	27.54	84.36	28.38			12
Dec	336	366	66.86	13.49	67.01	13.01			19
			668.56		**710.74**		**179.98**		**153**

APPENDIX D

Appendix D – TB3 precipitation gauge calibrations

Calibration of the TB3 tipping bucket precipitation gauges was performed as a quality assurance/quality control measure (QA/QC). The manufacturer quotes 0.0079 in (0.2 mm) of precipitation per tip as the calibration constant. It is also recommended that the owner perform field calibrations to validate this number. To calibrate the gauge, a volume of 22 ounces (657 mL) is released into the funnel at a known rate of 2 in/hr (50 mm/hr).

Six to eight trials were typically performed, and the average number of tips from all the trials was used to determine the calibration constant for each precipitation gauge. From factory calibrations, the number of tips expected from a single trial was, on average, 100 to 104 tips. That number was then converted to units of mm of water per tip. The calibration constant is then used to calculate the amount of water registered over a given time interval (i.e. a storm).

The series of calibration tests provide the following information:

1. Location of the gauge;
2. Date of test;
3. Calibration specifications;
4. Trial number;
5. Start time of the trial;
6. End time of the trial;
7. Measured water volume; and
8. Number of tips registered per trial.

APPENDIX D

D1 – Viewlands buried rain gauge calibration

January 17, 2000

Calibration:
Known volume: 657mL
Known flow rate: 50mm/hr

Known total tips: 100-104

Julian day 17:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	11:17am	11:43am	667	102
2	11:55am	12:20pm	657	96.7+
3	12:23pm	12:48pm	565**	96.7+
4	12:52pm	1:17pm*	668	96.7+
5	2:40pm	3:05pm	661	102.3++
6	3:11pm	3:38pm	661	102.3++
7	3:45pm	4:07pm	666	102.3++
8	4:17pm	4:42pm*	665	100
		Average:	**663.6**	**102.5**

Site Conditions:

* The exact end time was not written down in the field notes. It takes approximately 25 minutes to go through a cycle.

** The collection bucket was not sealed and in place. Water was not collected at the beginning of Trial 3.

+ For Trials 2, 3, and 4, the number of tips is an average of the total tips counted by the datalogger, over the entire time interval of the three trials. Due to overlapping time intervals, the exact number of tips for each trial could not be determined.

++ The same was done for Trials 5, 6, and 7.

1. The average measured volume is calculated for only seven trials, excluding Trial 3.

APPENDIX D

D2 – Viewlands buried rain gauge calibration

July 18, 2000

Calibration:
Known volume: 657mL
Known flow rate: 50mm/hr

Known total tips: 100-104

Julian day 200:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	1:48pm	2:17pm	634**	109
2	2:21pm	2:45pm	666	110
3	2:50pm	3:15pm*	669	111
4	3:35pm	4:00pm*	661	109
		Average:	**657.5**	**109.75**

Site Conditions:

* The exact end time was not written down in the field notes. It takes approximately 25 minutes to go through a cycle.

** The collection bucket was emptied in Trial 1, but I didn't shake out the collection tube thoroughly before emptying it.

1. The average measured volume is calculated for all four trials.

2. The day was cloudy in the morning and sunny in the afternoon.

APPENDIX D

D3 – Viewlands buried rain gauge calibration

July 29, 2000

Calibration:
Known volume: 657mL
Known flow rate: 50mm/hr

Known total tips: 100-104

Julian day 211:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	11:47am	12:12pm	620.3*	109
2	12:15pm	12:40pm	620.3	108++
3	12:45pm	1:10pm	620.3	108++
4	1:27pm	1:52pm	664**	111#
5	2:38pm	3:03pm	664	110
6	3:41pm	4:05pm	664	112
		Average:	**664+**	**109.67**

Site Conditions:

* Water leaked from the main container of the rain gauge. Therefore the collected volume is completely wrong for the first three trials.

** The water was collected and measured after the third trial and the sixth trial. Therefore for the second cycle the average volume is 664mL.

\+ The average measured volume only accounts for the last three trials.

++ The trials overlapped a time interval.

\# There is an erroneous tip at 1:24pm. It must be minused of the tips for Trial 4.

1. The day was sunny and windy.

APPENDIX D

D4 – Viewlands standing rain gauge calibration

July 20, 2000

Calibration:
Known volume: 657mL
Known flow rate: 50mm/hr

Known total tips: 100-104

Julian day 202:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	1:53pm	2:17pm	658.75	105
2	2:34pm	2:56pm	658.75	108
3	3:01pm	3:28pm	658.75	107
4	3:31pm	3:55pm	658.75	105
		Average:	**658.75****	**106.25**

Site Conditions:

* The water was collected and measured after the fourth cycle. Therefore each cycle has on average, 658.75mL.

** The average measured volume is calculated for all four trials.

1. The bottom connection at the brass T is leaking water.
2. It was sunny all through the day.

APPENDIX D

D5 – Viewlands standing rain gauge calibration

July 21, 2000

Calibration:
Known volume: 657mL
Known flow rate: 50mm/hr
Known total tips: 100-104

Julian day 203:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	11:30am	12:00pm	664.75	105
2	12:00pm	12:30pm	664.75	105
3	12:30pm	1:00pm	664.75	107
4	1:00pm	1:30pm	664.75	106
		Average:	**664.75***	**105.75**

Site Conditions:
* The water was collected and measured after the fourth cycle. Therefore each cycle has on average, 664.75mL.

1. The average measured volume is calculated for all four trials.
2. The bottom connection at the brass T is leaking water.
3. It was sunny all through the day.

APPENDIX D

D6 – SEA Streets standing rain gauge calibration

July 25, 2000

Calibration:

Known volume: 657mL

Known flow rate: 50mm/hr

Known total tips: 100-104

Julian day 207:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	1:53pm	2:17pm	663.3	107
2	2:20pm	2:43pm	663.3	110
3	2:45pm	3:13pm	663.3	109
		Average:	**663.3***	**108.67**

Site Conditions:

* The water was collected and measured after the third cycle. Therefore each cycle has on average, 663.3mL.

1. The bottom connection at the brass T is leaking water.
2. Cloudy throughout the morning and partly sunny in the afternoon.

APPENDIX D

D7 – SEA Streets standing rain gauge calibration

July 29, 2000

Calibration:
Known volume: 657mL
Known flow rate: 50mm/hr
Known total tips: 100-104

Julian day 209:

Trial	Start Time	End Time	Measured Volume [mL]	Tips
1	9:46am	10:09am	663.3*	107
2	10:15am	10:39am	663.3	109
3	10:45am	11:09am	663.3	109
4	11:15am	11:39am	668.3*	109
5	11:50am	12:18pm**	668.3	109
6	12:26pm	12:52pm	668.3	109
7	1:10pm	1:54pm**	659	109
8	2:00pm	2:24pm	663	108
		Average:	**664.5**	**108.63**

Site Conditions:

* The water was collected and measured after the third cycle. Therefore, the first three cycles has on average, 663.3mL. The second three cycles have on average, 668.3mL.

** Trial 5 and 7 ran longer than the normal 25 minute cycles. The siphon must have been clogged, which lowered the flow rate.

*** There is a tip between 1:55pm and 2:00pm that is part of Trial 7.

1. The average measured volume is calculated for all eight trials.
2. The bottom connection at the brass T is leaking water.
3. Cloudy throughout the morning and partly sunny in the afternoon.

APPENDIX E

E – Viewlands V-notch weir flow tests

Weir tests were performed to determine the quantity of runoff that was being lost to leakage. Significant amounts of leakage would cause the measured stage readings to underestimate the amount of runoff entering and exiting the system. Two sets of tests were undertaken in September 2000 and November 2000.

Weir flow rate tests – September 2000

Weir tests were performed on the upstream weir on September 6, 2000 and on the downstream weir on September 21, 2000. Five trials were performed at various weir stage heights. Water was discharged from a City of Seattle water utility fire hydrant into the upstream side of the weir being tested. A meter connected to the fire hydrant measured the inflow rate, which was adjusted by a valve on the hydrant. The cells were then filled and steady-state was reached before beginning each trial.

The stage over the weir was known from increments marked onto the weir plate. The outflow rate was determined by physically collecting the water that flowed over the weir into a tarp that covered the downstream cell. Therefore the volume and time of the trial was known, and an outflow rate could be determined. The measured flow rate for the known value of steady-state head was compared with the theoretical weir flow equation. The plots agreed well and are shown on Figure 7.2. Therefore, for subsequent tests the outflow over the weir notch could be determined from the weir plate stage markings. The difference between the inflow and outflow rates indicated the steady-state leakage rate for that stage and set of antecedent swale soil conditions.

Weir flow rate tests - November 2000

Weir tests were performed on the upstream weir on November 25, 2000 and on the downstream weir on November 26, 2000. Storms occurred during both tests, and nine trials were performed at various weir stage heights. The inflow volume was measured at the concrete pipe feeding the weir pool. Water was collected in a calibrated 5-gallon bucket over a given time period, which was recorded using a stopwatch. The inflow rate was then converted from gallons per second to cubic feet per second. The stage over the weir was known from increments marked onto the weir plate. The outflow rate was then calculated from the 120^{o} V-notch weir plate equation. The difference between the measured inflow and calculated outflow rates determined the leakage rate.

Appendices

The following series of weir flow measurements provide information on the following:

1. Location of the weir;
2. Date of the test;
3. Trial number;
4. The steady-state stage;
5. Metered inflow rate;
6. Measured outflow rate;
7. Leakage rate through the cell; and
8. The leakage rate as a percent loss of the inflow.

E1 – Viewlands V-notch weir flow measurements

Upstream Weir - September 6, 2000

Trial	Stage [ft]	Inflow Rate [cfs]	Outflow Rate [cfs]	Leakage Rate [cfs]	Leakage Rate as % of Inflow Rate
1	0.07	0.046	0.006	0.040	**87.8**
2	0.21	0.140	0.100	0.040	**28.7**
3	0.27	0.176	0.151	0.024	**13.9**
4	0.29	0.213	0.200	0.012	**5.7**
5	0.32	0.331	0.259	0.072	**21.6**

Notes:

1. The weir tests were conducted by Adrienne Miller, Rich Horner, and Shanti Colwell.
2. Started the water, but waited until transient flow ceased and the stage level remained constant (as checked by the increments on the weir).
3. STAGE was noted using the feet increments drawn onto the weir plate, which delineated the height above the invert of the weir.
4. INFLOW RATE was calculated from the metered fire hydrant.
5. OUTFLOW RATE was calculated from the volume of water collected in a tarp, which flowed over the weir for a given interval of time.
6. There is significant leakage, which decreases at higher flow rates. The only discrepancy is at the maximum flow rate, where the leakage is higher than for the previous trials.
7. The leakage is occurring in the joint between the weir plate and log, where the weir plate is connected to the rocks, among the boulders, the underside of the log, and in the cracks of the cement.
8. The amount of water that was used during the trials, filled up the next two cells downstream from the weir. The water never reached the bottom cell.

E2 – Viewlands V-notch weir flow measurements

Upstream Weir - November 25, 2000

Trial	Stage [ft]	Inflow Rate [cfs]	Outflow Rate [cfs]	Leakage Rate [cfs]	Leakage Rate as % of Inflow Rate
1	0.17	0.068	0.049	0.019	**27.9**
2	0.16	0.065	0.042	0.023	**35.4**
3	0.15	0.072	0.035	0.037	**51.4**
4	0.14	0.050	0.032	0.018	**36.0**
5	0.11	0.031	0.017	0.014	**45.2**
6	0.09	0.024	0.012	0.012	**50.0**
7	0.07	0.017	0.005	0.012	**70.6**
8	0.05	0.013	0.003	0.010	**76.9**
9	0.34	0.311	0.290	0.021	**6.8**

Notes:

1. The measurements were performed by Dan Smith, from the King County Department, during natural rain events.
2. STAGE was noted using the feet increments drawn onto the weir plate, which delineates the height above the invert of the weir.
3. INFLOW RATE was calculated from the volume of water collected in a graduated bucket, at the concrete pipe feeding the weir pool.
4. OUTFLOW RATE was calculated using the observed stage readings and the theoretical 120^{o} V-notch weir plate equation: $Q = 4.35*H^{5/2}$.
5. There is significant leakage, which decreases at higher flow rates.
6. The leakage is occurring in the joint between the weir plate and log, where the weir plate is connected to the rocks, among the boulders, the underside of the log, and in the cracks of the cement.

E3 – Viewlands V-notch weir flow measurements

Downstream Weir - September 23, 2000

Trial	Stage [ft]	Inflow Rate [cfs]	Outflow Rate [cfs]	Leakage Rate [cfs]	Leakage Rate as % of Inflow Rate
1	0.08	0.067	0.009	0.058	**86.5**
2	0.23	0.178	0.143	0.035	**19.7**

Notes:

1. The weir tests were conducted by Adrienne Miller, Shanti Colwell, and Andy Wood.
2. Started the water, but waited until transient flow ceased and the stage level remained constant (as checked by the increments on the weir).
3. STAGE was noted using the feet increments drawn onto the weir plate, which delineated the height above the invert of the weir.
4. INFLOW RATE was calculated from the metered fire hydrant.
5. OUTFLOW RATE was calculated from the volume of water collected in a tarp, which flowed over the weir for a given interval of time.
6. The water level dropped an inch in 5 minutes. Observed before any tests were performed.
7. There is significant leakage, which decreases at higher flow rates.

8. The leakage is occurring in the joint between the weir plate and log, where the weir plate is connected to the rocks, among the boulders, the underside of the log, and in the cracks of the cement.
9. The pump died during Trial 2, so we had to bale out the water by hand in the last cell.

E4 – Viewlands V-notch weir flow measurements

Downstream Weir - November 26, 2000

Trial	Stage [ft]	Inflow Rate [cfs]	Outflow Rate [cfs]	Leakage Rate [cfs]	Leakage Rate as % of Inflow Rate
1	0.31	0.263	0.237	0.025	**9.5**
2	0.28	0.182	0.182	0.000	**0.0**
3	0.27	0.177	0.158	0.019	**10.7**

Notes:
1. The measurements were performed by Dan Smith, from the King County Department, during natural rain events.
2. STAGE was noted using the feet increments drawn onto the weir plate, which delineates the height above the invert of the weir.
3. INFLOW RATE was calculated from the volume of water collected in a graduated bucket, at the concrete pipe feeding the weir pool.
4. OUTFLOW RATE was calculated using the observed stage readings and the theoretical 120^{o} V-notch weir plate equation: $Q = 4.35*H^{5/2}$.
5. There is significant leakage, which decreases at higher flow rates.
6. The leakage is occurring in the joint between the weir plate and log, where the weir plate is connected to the rocks, among the boulders, the underside of the log, and in the cracks of the cement.

APPENDIX F

F – Viewlands storm record

The relationship between precipitation and flow is established by identifying and analyzing trends in a storm. At the Viewlands site, the maximum upstream and downstream volumes are determined. The difference between the upstream inflow volume and the downstream outflow volume equals the amount of water entering channel storage and/or infiltrating into the channel banks and bed. From this, a percent reduction of the inflow rate is determined at varying temporal scales. The amount of infiltration and/or detention through the swale is calculated over the course of a year and partitioned into dry and wet antecedent soil moisture conditions.

For each storm that produced runoff in the swale during the monitoring period from June 2000 to January 2001, a number of parameters were found and are listed below.

Appendix F1 presents the following information:

1. The storm number;
2. Year the storm started and ended;
3. Start and end month;
4. Start and end day;
5. Start and end julian day; and
6. Start and end time (in 15-minute increments).

Appendix F2 presents the following information:

1. Lead time to storm (during which no rain occurred);
2. Lag time between start of precipitation and flow over the upstream weir;
3. Lag time between upstream and downstream weir response;
4. Storm duration;

5. Storm precipitation sum;
6. Average precipitation intensity;
7. Maximum upstream flow rate;
8. Maximum downstream flow rate; and
9. Percent reduction in the peak flow rates;

Appendix F3 presents the following information:

1. Cumulative upstream inflow volume;
2. Cumulative downstream outflow volume;
3. Infiltrated and/or detained volume through the swale;
4. Percent reduction in inflow volume;
5. Maximum water depth;
6. Maximum flow cross-sectional area;
7. Maximum water velocity;
8. Minimum hydraulic residence time through the swale;
9. Maximum wetted perimeter; and
10. Maximum wetted surface area of the swale.

F1 – Viewlands storm record

Event No	Start Year	Start Month	Start Day	Start Julian Day	Start Time	End Year	End Month	End Day	End Julian Day	End Time
1	2000	7	3	185	1745	2000	7	4	186	0045
2	2000	7	22	204	1300	2000	7	23	205	0145
3	2000	8	18	231	0430	2000	8	18	231	1230
4	2000	8	29	242	1545	2000	8	29	242	2015
5	2000	9	9	253	2015	2000	9	10	254	0845
6	2000	9	29	273	1430	2000	10	1	275	0100
7	2000	10	9	283	0630	2000	10	9	283	1415
8	2000	10	13	287	1500	2000	10	14	288	0300
9	2000	10	16	290	0130	2000	10	16	290	1645
10	2000	10	17	291	1900	2000	10	17	291	2215
11	2000	10	19	293	1615	2000	10	20	294	1800
12	2000	10	27	301	1845	2000	10	28	302	0600
13	2000	10	28	302	1700	2000	10	29	303	0945
14	2000	11	4	309	0300	2000	11	4	309	0730
15	2000	11	7	312	2115	2000	11	8	313	0945
16	2000	11	8	313	1615	2000	11	9	314	0300
17	2000	11	23	328	0315	2000	11	23	328	1145
18	2000	11	25	330	0915	2000	11	25	330	1330
19	2000	11	26	331	0245	2000	11	27	332	0445
20	2000	11	29	334	0400	2000	11	30	335	1115
21	2000	12	1	336	1615	2000	12	2	337	0430
22	2000	12	8	343	1800	2000	12	8	343	2300
23	2000	12	14	349	1315	2000	12	14	349	2115
24	2000	12	15	350	2345	2000	12	16	351	2315
25	2000	12	21	356	1100	2000	12	22	357	1745
26	2000	12	22	357	2345	2000	12	24	359	0400
27	2000	12	25	360	0400	2000	12	25	360	0800
28	2000	12	31	366	0600	2000	12	31	366	1030
29	2001	1	4	4	0230	2001	1	5	5	0245
30	2001	1	8	8	1245	2001	1	8	8	1400
31	2001	1	9	9	1545	2001	1	9	9	2100
32	2001	1	13	13	2230	2001	1	14	14	0630
33	2001	1	17	17	1345	2001	1	17	17	2215
34	2001	1	18	18	0900	2001	1	19	19	0000
35	2001	1	20	20	2215	2001	1	21	21	1500
36	2001	1	28	28	2115	2001	1	29	29	1130

F2 – Viewlands storm record

Event No	Time to Storm [hr]	Time to Up Response [hr]	Time to Dw Response [hr]	Storm Duration [hr]	Storm Sum [in]	Average Intensity [in/hr]	Max Flow Up [cfs]	Max Flow Dwn [cfs]	% Reduc in Flow [%]
1	0.00	6.25	0.00	7.00	0.15	0.021	0.22	0.00	100.0
2	444.25	7.00	0.00	12.75	0.28	0.022	0.35	0.00	100.0
3	626.75	6.00	2.00	8.00	0.28	0.036	0.01	0.00	99.9
4	267.25	2.50	0.00	4.50	0.15	0.033	0.09	0.00	100.0
5	264.00	3.00	0.00	12.50	0.24	0.019	0.39	0.00	100.0
6	461.75	1.00	4.25	34.50	0.84	0.024	0.65	0.32	51.5
7	197.50	1.75	0.25	7.75	0.35	0.045	0.99	0.59	40.73
8	96.75	10.50	0.00	12.00	0.19	0.016	0.23	0.00	100.0
9	46.50	10.75	1.00	15.25	0.45	0.029	0.54	0.53	2.3
10	26.25	2.25	0.00	3.25	0.13	0.039	0.25	0.00	100.0
11	42.00	1.25	3.25	25.75	1.28	0.050	3.88	3.80	2.0
12	168.75	2.50	1.75	11.25	0.55	0.049	0.63	0.61	3.8
13	11.00	5.75	2.75	16.75	0.21	0.013	0.15	0.08	44.4
14	137.25	1.00	3.25	4.50	0.31	0.068	1.45	1.45	0.0
15	85.75	3.50	0.50	12.50	0.82	0.066	1.78	1.31	26.5
16	6.50	1.00	0.00	10.75	0.10	0.009	0.25	0.00	100.0
17	336.25	3.50	0.00	8.50	0.25	0.030	0.21	0.00	100.0
18	45.50	3.50	0.00	4.25	0.13	0.032	0.16	0.00	100.0
19	13.25	0.75	3.25	26.00	1.15	0.044	1.11	1.08	2.7
20	47.25	4.75	3.75	31.25	0.37	0.012	0.45	0.39	12.5
21	29.00	8.00	2.75	12.25	0.26	0.021	0.50	0.41	18.5
22	157.50	4.75	0.00	5.00	0.10	0.021	0.22	0.00	100.0
23	134.25	1.50	0.75	8.00	0.43	0.053	1.07	0.97	9.4
24	26.50	2.00	2.25	23.50	0.53	0.023	0.35	0.30	12.0
25	107.75	1.00	15.25	30.75	0.54	0.018	0.32	0.31	1.5
26	6.00	6.25	8.75	28.25	0.31	0.011	0.22	0.20	7.4
27	24.00	1.50	2.00	4.00	0.11	0.028	0.12	0.03	79.3
28	142.00	1.00	0.75	4.75	0.17	0.036	0.34	0.19	45.2
29	5.75	2.00	2.25	24.25	0.78	0.032	1.54	1.46	5.6
30	82.00	1.75	0.00	1.25	0.04	0.036	0.001	0.000	100.0
31	25.75	3.50	1.00	5.25	0.19	0.036	0.44	0.25	43.2
32	97.50	0.75	0.00	8.00	0.08	0.010	0.13	0.00	100.0
33	79.25	5.25	0.00	8.50	0.10	0.011	0.13	0.00	100.0
34	10.75	1.00	2.75	15.00	0.27	0.018	0.49	0.37	24.3
35	46.25	2.75	1.25	16.75	0.79	0.047	0.68	0.55	18.9
36	174.25	1.00	0.75	14.25	0.40	0.028	0.77	0.69	10.2

F3 – Viewlands storm record

Event No	Up Vol [ft³]	Down Vol [ft³]	Infil Vol [ft³]	%Reduc Vol [%]	Water Depth [ft]	C-S Area [ft²]	Velocity (V) [ft/s]	Hyd Res Time [min]	Wetted Perim [ft]	S-Area Infil [ft²]
1	558.7	0.0	558.7	100.0	0.04	0.25	0.87	5.15	7.10	1917
2	624.4	0.0	624.4	100.0	0.05	0.33	1.05	4.28	7.13	1926
3	111.6	0.0	111.5	100.0	0.01	0.05	0.29	15.48	7.02	1895
4	293.8	0.0	293.8	100.0	0.02	0.15	0.63	7.15	7.06	1906
5	1867.5	0.0	1867.5	100.0	0.05	0.35	1.10	4.09	7.14	1928
6	9052.2	2481.9	6570.3	72.6	0.07	0.48	1.35	3.33	7.19	1942
7	2638.5	813.1	1825.4	69.2	0.09	0.62	1.59	2.82	7.25	1957
8	699.5	0.0	699.5	100.0	0.04	0.25	0.89	5.06	7.10	1918
9	6059.5	4612.9	1446.6	23.9	0.06	0.43	1.25	3.60	7.17	1936
10	741.1	0.0	741.1	100.0	0.04	0.27	0.93	4.83	7.11	1920
11	35456.5	26941.4	8515.1	24.0	0.20	1.44	2.70	1.67	7.56	2042
12	6072.3	3294.4	2777.9	45.7	0.07	0.47	1.33	3.37	7.19	1941
13	1199.1	269.7	929.4	77.5	0.03	0.20	0.75	6.00	7.08	1911
14	3629.9	1816.1	1813.7	50.0	0.11	0.79	1.85	2.44	7.31	1974
15	21261.9	12246.3	9015.6	42.4	0.12	0.89	2.00	2.25	7.35	1985
16	745.2	0.0	745.2	100.0	0.04	0.27	0.93	4.85	7.11	1920
17	638.1	0.0	638.1	100.0	0.03	0.25	0.87	5.18	7.10	1917
18	512.5	0.0	512.5	100.0	0.03	0.21	0.78	5.77	7.08	1913
19	23453.0	18860.5	4592.6	19.6	0.09	0.67	1.66	2.71	7.27	1962
20	3644.3	1192.1	2452.2	67.3	0.05	0.38	1.16	3.87	7.15	1932
21	3431.4	1786.4	1645.0	47.9	0.06	0.41	1.22	3.70	7.16	1934
22	341.9	0.0	341.9	100.0	0.04	0.25	0.88	5.11	7.10	1917
23	5159.9	3686.7	1473.1	28.5	0.09	0.65	1.64	2.74	7.26	1960
24	4769.2	3185.2	1584.0	33.2	0.05	0.33	1.05	4.28	7.13	1926
25	4456.7	2735.3	1721.4	38.6	0.04	0.31	1.02	4.42	7.13	1924
26	2978.4	2121.1	857.3	28.8	0.04	0.25	0.88	5.13	7.10	1917
27	737.5	46.5	691.0	93.7	0.03	0.18	0.70	6.41	7.07	1909
28	1364.6	272.3	1092.3	80.0	0.05	0.33	1.04	4.31	7.13	1925
29	21565.3	18070.4	3494.9	16.2	0.11	0.82	1.89	2.38	7.32	1978
30	1.4	0.0	1.4	100.0	0.00	0.01	0.11	41.16	7.00	1891
31	2658.8	640.8	2018.0	75.9	0.05	0.38	1.16	3.87	7.15	1931
32	479.4	0.0	479.4	100.0	0.03	0.18	0.72	6.26	7.07	1910
33	339.3	0.0	339.3	100.0	0.03	0.18	0.71	6.35	7.07	1910
34	4366.7	1694.1	2672.6	61.2	0.06	0.41	1.21	3.72	7.16	1934
35	18463.1	12399.4	6063.7	32.8	0.07	0.49	1.37	3.28	7.20	1943
36	5451.8	2952.6	2499.2	45.8	0.08	0.53	1.44	3.12	7.21	1948

APPENDIX G

G – Viewlands pre-construction analysis

The purpose of the pre-construction analysis for the Viewlands site is to compare how the old concrete/vegetated channel would have responded under the same meteorological conditions that occurred from June 2000 to January 2001.

To determine the potential volume infiltrated and/or detained in the old vegetated channel, a plan area-to-plan area analysis was performed. The measured volume that was infiltrated/detained in the new swale was apportioned over the area wetted during the course of a storm. Given the same meteorological conditions but a different geometry for the old channel, the wetted area and potential volume infiltrated were estimated for each storm. It is assumed that similar infiltration and storage dynamics occurred for both the new and old channels, which would tend to overestimate the potential volume infiltrated through the vegetated portion of the old ditch.

The first step in the pre-construction analysis was to determine the potential infiltration area in the new and old channels. The new swale was designed to accommodate low flow through the center of the channel and high flow over the entire width of the channel bed. To determine the potential infiltration area in the old channel, which was composed of two sections. No infiltration occurred over the length of the first concrete section. The second section of the channel was a vegetated ditch, composed of vegetation, sediment deposits, and gravel. Surface runoff and infiltration occurred over the length of the vegetated ditch. Appendix G1 details the dimensions and parameters used in determining the maximum low flow rate, high flow rate, and potential infiltration area of the new Viewlands swale. Appendix G2 details the dimensions and parameters used in determining the maximum flow rate and potential infiltration area of the old vegetated ditch.

The volumetric flow rate of water is given by Manning's equation:

$$Q = \frac{1.49}{n} * A * R_h^{2/3} * S_f^{1/2} \qquad \text{(G.1)}$$

-where:

Q = flow rate (L^3/t)

A = area (L^2)

R_h = hydraulic radius (L)

S_f = channel slope (at uniform flow) or head loss with change in elevation

-for a trapezoidal cross-section:

$$R_h = \frac{A}{P_w} = \frac{(by + my^2)}{(b + 2y(1+m^2)^{1/2})} \qquad \text{(G.2)}$$

-where:

P_w = wetted perimeter (L)

b = bed width (L)

m = side slope

By knowing the peak inflow rate (Q), the swale bed slope (S_f), and roughness coefficient (n), the Manning's equation was used to back-calculate the maximum water depth (y), the wetted perimeter (P_w), and wetted infiltration area (A_{inf}) for each storm in the new channel. Given the same meteorological conditions but a different geometry for the old channel, Manning's equation was again used to back-calculate the (y), (P_w), and (A_{inf}) for each storm (as shown in Appendix F3). Finally, a plan area-to-plan area comparison was made to apportion the measured volume of water that infiltrated over the wetted area in the new swale with the wetted area of the old vegetated ditch, for each storm. The infiltrated volume was then summed for the old swale and compared to the total measured volume for the new swale over the course of the study period.

APPENDIX G

G1 – Viewlands new swale dimensions and parameters

1. Determination of maximum low flow rate:

7	ft	bed width (b)
0.9	ft	maximum depth (y)
1		side slope (m)
0.04		roughness coefficient (n) (rock-lined with weeds)
0.048		slope (Sf)

For a trapezoidal cross-section:

7.11	ft2	Area (A)
8.8	ft	Top water width (B)
9.55	ft	Wetted perimeter (Pw)
0.74	ft	Hydraulic radius (Rh)
47.8	**cfs**	Maximum low flow rate (Qlow)

2. Determination of maximum high flow rate:

11	ft	average high flow width (b=B)
1.7	ft	minimum boulder height (y)
0.04		roughness coefficient (n)
0.048		slope (Sf)

For a rectangular cross-section:

18.7	ft2	Area (A)
37.4	ft	Wetted perimeter (Pw)
0.5	ft	Hydraulic radius (Rh)
96.1	**cfs**	Maximum flow rate (Qrect)

For the high flow condition:

143.9	cfs	Maximum high flow rate (Qhigh)

3. Determination of maximum area of infiltration at low flow:

9.55	ft	Wetted perimeter (Pw)
270	ft	length of channel (L)
2577.3	ft	Area of infiltration at low flows (Ainf)

4. Determination of maximum area of infiltration at low flow:

11.05	ft	Wetted perimeter (Pw)
270	ft	length of channel (L)
2982.3	ft	Area of infiltration at low flows (Ainf)

APPENDIX G

G2 – Viewlands old vegetated ditch dimensions and parameters

1. Determination of maximum flow rate:

4	ft	bed width (b)
0.7	ft	maximum depth (y)
, 5		side slope (m)
0.035		roughness coefficient (n) (stony bottom and weedy banks
0.048		slope (Sf)

For a trapezoidal cross-section:

5.25	ft2	Area (A)
11	ft	Top water width (B)
11.14	ft	Wetted perimeter (Pw)
0.47	ft	Hydraulic radius (Rh)
29.7	cfs	Maximum low flow rate (Qlow)

2. Determination of maximum area of infiltration:

11.14	ft	Wetted perimeter (Pw)
128	ft	length of channel (L)
1425.7	ft	Area of infiltration (Ainf)

APPENDIX H

H – SEA Streets storm record

The relationship between precipitation and flow is established by identifying and analyzing trends in a storm. For each storm that produced runoff from the residential street during baseline monitoring from March 2000 to July 2000, a number of parameters were found and are listed below.

Appendix H1 presents the following information:

1. The storm number;
2. Year the storm started and ended;
3. Start and end month;
4. Start and end day;
5. Start and end julian day; and
6. Start and end time (in 15-minute increments).

Appendix H2 presents the following information:

1. Lead time to storm (during which no rain occurred);
2. Lag time between start of precipitation and flow at the monitoring basin located at 2nd Ave NW and NW 117th Street;
3. Storm duration;
4. Storm precipitation sum;
5. Average precipitation intensity;
6. Maximum flow rate;
7. Measured cumulative runoff volume from the baseline monitored street;
8. Estimated cumulative runoff volume from a conventionally-designed street; and
9. Estimated cumulative runoff volume from a constructed SEA Streets design.

APPENDIX H

H1 – SEA Streets storm record

Event No	Start Year	Start Month	Start Day	Start Julian Day	Start Time	End Year	End Month	End Day	End Julian Day	End Time
1	2000	3	9	69	0215	2000	3	9	69	0430
2	2000	3	10	70	1800	2000	3	10	70	2300
3	2000	3	13	73	1315	2000	3	13	73	2230
4	2000	3	14	74	1300	2000	3	14	74	1930
5	2000	3	15	75	2330	2000	3	16	76	0900
6	2000	3	16	76	1600	2000	3	16	76	1915
7	2000	3	17	77	1330	2000	3	17	77	1630
8	2000	3	18	78	0415	2000	3	18	78	1415
9	2000	3	22	82	0330	2000	3	22	82	1115
10	2000	3	23	83	1615	2000	3	23	83	2145
11	2000	3	27	87	2200	2000	3	28	88	0030
12	2000	3	28	88	1345	2000	3	29	89	0230
13	2000	4	4	95	0600	2000	4	4	95	1930
14	2000	4	6	97	0015	2000	4	6	97	1200
15	2000	4	13	104	1245	2000	4	14	105	0015
16	2000	4	21	112	1945	2000	4	21	112	2330
17	2000	4	23	114	1130	2000	4	23	114	1415
18	2000	4	23	114	2000	2000	4	23	114	2145
19	2000	4	25	116	0115	2000	4	25	116	1415
20	2000	5	8	129	0630	2000	5	8	129	1630
21	2000	5	9	130	0415	2000	5	9	130	2230
22	2000	5	11	132	1345	2000	5	11	132	2115
23	2000	5	18	139	1630	2000	5	18	139	2345
24	2000	5	21	142	1600	2000	5	21	142	1730
25	2000	5	22	143	0100	2000	5	22	143	0500
26	2000	5	26	147	0000	2000	5	26	147	1030
27	2000	5	27	148	0330	2000	5	27	148	1130
28	2000	5	28	149	1430	2000	5	29	150	0415
29	2000	5	29	150	2145	2000	5	30	151	0045
30	2000	5	31	152	0015	2000	5	31	152	1030
31	2000	6	6	158	1000	2000	6	6	158	1130
32	2000	6	7	159	0145	2000	6	7	159	0830
33	2000	6	11	163	0815	2000	6	12	164	0730
34	2000	6	14	166	1415	2000	6	14	166	1715
35	2000	6	18	170	0730	2000	6	18	170	1115

APPENDIX H

H2 – SEA Streets storm record

Event No	Time to Storm [hr]	Time to Respond [hr]	Storm Duration [hr]	Storm Sum [in]	Average Intensity [in/hr]	Data Record SEA Streets Max Flow Rate [cfs]	Data Record SEA Streets Runoff Vol [ft^3]	Conventional Sreet Runoff Vol [ft^3]	Constructed Sea Streets Runoff Vol [ft^3]
1	196.50	1.50	2.25	0.05	0.023	0.009	21.5	123.8	42.7
2	37.75	1.00	5.00	0.17	0.033	0.021	119.1	389.2	134.2
3	62.25	4.00	9.25	0.45	0.049	0.054	467.5	1061.4	365.9
4	14.50	0.50	6.50	0.19	0.029	0.056	249.0	442.2	152.5
5	28.00	0.50	9.50	0.20	0.021	0.014	152.6	477.6	164.7
6	7.00	0.50	3.25	0.11	0.033	0.008	48.6	247.7	85.4
7	18.25	1.00	3.00	0.06	0.020	0.003	22.7	141.5	48.8
8	11.75	0.75	10.00	0.17	0.017	0.009	96.8	389.2	134.2
9	85.25	2.25	7.75	0.29	0.038	0.054	227.4	689.9	237.9
10	29.00	2.00	5.50	0.07	0.012	0.016	26.4	159.2	54.9
11	96.25	0.50	2.50	0.08	0.033	0.026	94.9	194.6	67.1
12	13.25	0.25	12.75	0.20	0.015	0.022	279.8	459.9	158.6
13	147.50	1.00	13.50	0.26	0.020	0.051	488.8	619.1	213.5
14	28.75	4.00	11.75	0.11	0.009	0.016	180.3	247.7	85.4
15	168.75	3.00	11.50	0.40	0.035	0.027	550.1	937.6	323.3
16	187.50	1.25	3.75	0.06	0.016	0.009	56.2	141.5	48.8
17	36.00	0.50	2.75	0.04	0.014	0.007	23.9	88.4	30.5
18	5.75	0.25	1.75	0.08	0.047	0.046	184.3	194.6	67.1
19	27.50	0.75	13.00	0.25	0.019	0.035	408.4	583.8	201.3
20	304.25	1.50	10.00	0.07	0.007	0.003	22.4	159.2	54.9
21	11.75	0.75	18.25	0.35	0.019	0.040	403.0	831.4	286.7
22	39.25	1.00	7.50	0.42	0.055	0.038	82.4	972.9	335.4
23	163.25	0.75	7.25	0.39	0.054	0.026	305.1	919.9	317.2
24	64.25	0.50	1.50	0.08	0.055	0.014	61.4	194.6	67.1
25	7.50	3.25	4.00	0.04	0.009	0.005	13.2	88.4	30.5
26	91.00	3.25	10.50	0.23	0.022	0.022	317.4	548.4	189.1
27	17.00	0.50	8.00	0.14	0.017	0.041	240.2	318.4	109.8
28	27.00	5.50	13.75	0.14	0.010	0.047	195.4	336.1	115.9
29	17.50	0.75	3.00	0.14	0.045	0.045	213.1	318.4	109.8
30	23.50	0.75	10.25	0.11	0.011	0.012	353.5	265.3	91.5
31	143.50	1.00	1.50	0.05	0.030	0.028	184.8	106.1	36.6
32	14.25	1.25	6.75	0.08	0.011	0.021	396.5	176.9	61.0
33	95.75	1.00	23.25	0.63	0.027	0.083	1909.8	1485.9	512.3
34	54.75	0.75	3.00	0.07	0.023	0.027	204.0	159.2	54.9
35	86.25	1.25	3.75	0.14	0.038	0.000	0.71	336.1	115.88
							8601.2	**14806.2**	**4989.1**

APPENDIX I

I – SEA Streets road design analysis

The relative magnitude of the runoff measured during baseline SEA Streets conditions was compared to the estimated runoff volumes for both a conventional street design and for the constructed SEA Streets design. The constraining dimensions of all three roadways were 660 ft (201 m) in length and a 60 ft (18 m) right-of-way. The following equation was used to determine runoff volumes for both the conventional street and the constructed SEA Street roadway designs:

$$V = D_r * A * C_a$$

-where:

V = total runoff volume (L^3)

D_r = precipitation depth (L)

C_a = area-weighted runoff coefficient

A = contributing area (L^2)

Instead of using literature values for the runoff coefficient of asphalt, it was back-calculated from the measured flow data collected during the pre-construction baseline period. Only storms preceded by three dry days were used in the analysis. This assumed that drier soil conditions would cause the precipitation to initially infiltrate into the grass lawns, producing runoff predominantly from the asphalt street.

For the constructed SEA Street site, it was assumed that no runoff would occur from the sedimentation ponds and all water falling on the sidewalks would either drain into the grass edge or the sedimentation ponds. Hence, only the asphalt and concrete edges of the street would contribute runoff. Appendix I1 to I3 details the dimensions and parameters used in estimating the runoff volume for both roadway designs. Refer to Appendix H2 for a complete list of the estimated runoff volumes per storm.

APPENDIX I

I1 – SEA Streets baseline roadway design

1. Pre-construction Roadway Dimensions:

660	ft	SEA Street Length
60	ft	Right-of-way
25	ft	Standard asphalt residential roadway width
17.5	ft	Grass/tree edge (on both sides of street)

2. Literature Values for the Runoff Coefficients: (Bedient et al, 1992)

0.7	0.95	Asphalt streets - range
	0.825	Asphalt streets - average
0.18	0.22	Grass Lawns - Heavy Soils range (2-7 percent slope)
	0.2	Grass Lawns - Heavy Soils average (2-7 percent slope)

3. Areas of each type of material:

16500	ft2	Asphalt
23100	ft2	Grass Lawns
39600	**ft2**	**Total**

Event Number	Time to Storm [hrs]	Time to Respond [hrs]	Storm Sum [in]	Depression Storage [in]	Runoff Depth [in]	Volume Inflow [ft^3]	Area of Asphalt [ft2]	Runoff Coeff
1	196.50	1.50	0.05	0.01	0.043	21.5	16500	0.36
9	85.25	2.25	0.29	0.01	0.284	227.4	16500	0.58
11	96.25	0.50	0.08	0.01	0.073	94.9	16500	0.95
13	147.50	1.00	0.26	0.01	0.254	488.8	16500	1.40
15	168.75	3.00	0.40	0.01	0.390	550.1	16500	1.03
16	187.50	1.25	0.06	0.01	0.050	56.2	16500	0.81
20	304.25	1.50	0.07	0.01	0.058	22.4	16500	0.28
23	163.25	0.75	0.39	0.01	0.382	305.1	16500	0.58
26	91.00	3.25	0.23	0.01	0.224	317.4	16500	1.03
33	95.75	1.00	0.63	0.01	0.624	1909.8	16500	2.23
								0.92

APPENDIX I

I2 – SEA Streets conventionally-designed roadway

1. Convential Roadway Dimensions:

660	ft	SEA Street Length
60	ft	Right-of-way
25	ft	Standard asphalt residential roadway width
6.5	ft	Concrete side walk (on both sides of street)
3	ft	Concrete curb and gutter (on both sides of street)
6	ft	Grass/tree edge (on both sides of street)
2	ft	Grass/tree edge on inside of sidewalk (on both sides of street)

2. Runoff Coefficients: (Bedient et al, 1992)

0.7	0.95	Asphalt streets - range
	0.825	Asphalt streets - average
	0.92	Asphalt street - found from field data
0.8	0.95	Concrete streets - range
	0.875	Concrete streets - average
0.18	0.22	Grass Lawns - Heavy Soils range (2-7 percent slope)
	0.2	Grass Lawns - Heavy Soils average (2-7 percent slope)

*Will use the asphalt value found from the field data.
*Will use the average value for the concrete street.
*Will use the lower end of the range for the lawns since SEA Streets is flat.

3. Areas of each type of material:

16500	ft^2	Asphalt
12540	ft^2	Concrete
10560	ft^2	Grass Lawns
39600	**ft^2**	**Total**

4. Determination of area-weighted runoff coefficient:

Ca = sum of (area*C)/sum of (area)

Ca = 0.71

5. Determination of Inflow Volume

Inflow Volume = Ca * rainfall depth [in] * Area [ft2] * Conversion factor [1ft/12in]

Conversion Factor = 0.083

K = constant

K = (Ca*A*Conversion factor)

K = 2344.16

APPENDIX I

I3 – SEA Streets constructed roadway

1. Constructed SEA Streets Roadway Dimensions:

660	ft	SEA Street Length
60	ft	Right-of-way
14	ft	Standard asphalt residential roadway width
5	ft	Concrete side walk (on both sides of street)
2	ft	Concrete curb on roads edge (on both sides of street)
32	ft	Sedimentation ponds (on both sides of street)

2. Runoff Coefficients: (Bedient et al, 1992)

0.7	0.95	Asphalt streets - range
	0.825	Asphalt streets - average
	0.92	Asphalt street - found from field data
0.8	0.95	Concrete streets - range
	0.875	Concrete streets - average
0.18	0.22	Grass Lawns - Heavy Soils range (2-7 percent slope)
	0.2	Grass Lawns - Heavy Soils average (2-7 percent slope)

*Will use the asphalt value found from the field data.
*Will use the average value for the concrete street.
*Will assume that there will be no runoff from the sedimentation ponds

3. Areas of each type of material:

9240	ft^2	Asphalt
4620	ft^2	Concrete
1320	ft^2	Concrete - contributing to runoff
21120	ft^2	Sedimentation Ponds
34980	**ft^2**	**Total**

4. Determination of area-weighted runoff coefficient:
   Ca = sum of (area*C)/sum of (area)
   Ca = 0.92

5. Determination of Inflow Volume
   Inflow Volume = Ca * rainfall depth [in] * Area [ft2] * Conversion factor [1ft/12in]
   Conversion Factor = 0.083
   K = constant
   K = (Ca*A*Conversion factor)
   K = 808.23