Department of Civil and Environmental Engineering
University of Washington
Seattle, Washington 98195

HYDROLOGIC MONITORING OF THE SEATTLE ULTRA-URBAN STORMWATER MANAGEMENT PROJECTS

by

Richard R. Horner
Heungkook Lim
Stephen J. Burges

Water Resources Series
Technical Report No. 170

November 2002

ABSTRACT

Seattle Public Utilities constructed two drainage projects in the northwestern part of the city to decrease stormwater quantities discharged to Pipers Creek, with the goal of reducing channel erosion there and water pollutant loadings to the stream. One project, the Viewlands Cascade Drainage System, replaced a narrow, partially concreted ditch with a wide series of stepped pools. The second installation, at 2nd Avenue NW and known as a Street Edge Alternatives (SEA Streets) project, involved the complete reconstruction of the street and its drainage system to reduce impervious area and install stormwater detention ponds. These projects have been monitored for flow in relation to precipitation to determine their actual benefits. Flow was sensed with shaft encoder floats and pressure transducers that recorded water depths behind V-notch weirs. Precipitation was recorded using tipping bucket gauges.

Monitoring has demonstrated that the Viewlands Cascade is capable of reducing the influent runoff volume by slightly more than one-third during the wetter months and overall for the year. Based on estimates for the ditch that preceded the Viewlands Cascade project, the new channel reduces runoff discharged to Pipers Creek in the wet months by a factor of three relative to the old ditch.

The 2nd Avenue SEA Streets project has prevented the discharge of all dry season flow and 98 percent of the wet season runoff. It can fully attenuate the runoff volume produced by approximately 0.75 inch (19 mm) of rain on its catchment. Based on estimates for a street drainage system design according to City of Seattle conventions, the SEA Streets alternative reduces runoff discharged to Pipers Creek in the wet months by a factor of 4.7 relative to the conventional street. Despite serving a catchment less than 10 percent as large as the Viewlands Cascade, the 2nd Avenue NW project retains more than one-third as much runoff volume in the wet season as Viewlands, and thus has higher efficiency on a unit area basis. However, when normalized in terms of the cost per unit catchment area served, the SEA Streets project is considerably less cost-effective than the Cascade channel.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 - INTRODUCTION

1.1. Background and Objectives

The City of Seattle has launched a program to protect and improve 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. The Washington Department of Ecology (WDOE) is moving in the same direction under the City's stormwater National Pollutant Discharge Elimination System (NPDES) permit.

In the summer of 1999, 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 (CIPs). 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.

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

1 Determine how effective the selected projects are in reducing peak rates and volumes of runoff;

2 Evaluate receiving water ecosystem benefits that could be achieved with widespread application of these project types; and

3 Develop a long-term, systematic approach to ultra-urban stormwater management in Seattle.

The first two ultra-urban stormwater management projects to be evaluated are the Viewlands Cascades Drainage System and the 2nd Avenue NW Street Edge Alternative (SEA) Streets Millennium Project. The projects were designed to reduce stormwater quantities discharged to Pipers Creek. A related goal in the case of Viewlands was to decrease the high velocities often occurring in the previous drainage ditch to prevent bypass of the drain inlet at its end, and the consequent erosion of the adjacent slope. Both projects were also expected to provide water quality benefits through enhanced pollutant capture by vegetation and soils and reduced pollutant mass loadings associated with lower flow volumes.

The Viewlands Cascade receives drainage from a catchment originally thought to be approximately 26 acres (10.5 ha) in area. That figure has been called into question recently and will be established firmly in upcoming work. Collected runoff is piped to the Cascade, where it flows through 16 stepped cells formed by log weirs to the downstream drain inlet and onward to Pipers Creek via another pipe. Construction cost was approximately $225000.

The 2nd Avenue NW SEA Streets project represents a full street right-of-way redesign. The width of the 660-ft (201-m) long roadway between NW 117th and NW 120th Streets was reduced from 25 ft (7.6 m) to 14 ft (4.3 m), parking slots were provided at angles to the street, and sidewalks were added. The remainder of the 60-ft (18-m) right of way was devoted to runoff detention ponds planted with native vegetation. The original right of way covered approximately 0.91 acre (0.37 ha), about 0.38 acre (0.15 ha) of asphalt and the remainder in vegetation on the edges. Hard surface was reduced slightly to 0.31 has (0.13 ha) in the redesign, with the remainder given to ponds. The construction cost was initially bid at $244000. There were substantial additional costs for this first-of-its-type project in reaching community consensus, change orders to satisfy community concerns, etc.

The catchment area draining to the 2nd Avenue NW pond system includes properties on the east side of 2nd Avenue NW, as well as the streetscape, and totals approximately 2.3 acres (0.93 ha). Slopes are very slight toward the west and south. The catchment discharges to a ditch flowing along NW 117th Street at the southwest corner of the project.

Precipitation at the Viewlands Cascade has been monitored since January 2000. Post-construction flow monitoring began in July 2000 and has continued since then. Baseline (pre-construction) monitoring was not possible at this site, because construction began very shortly after establishment of the memorandum of understanding.

The construction schedule at 2nd Avenue NW allowed some baseline monitoring of the pre-existing street, from March 11 to July 11, 2000. At that point monitoring was suspended during construction, which lasted until the following January. Post-construction monitoring started soon thereafter and has continued since then.

A graduate thesis (Miller 2001) and a technical report in this series drawn from the thesis (Miller, Burges, and Horner 2001) document all events in the ultra-urban stormwater management studies through January 2001. These references provide more extensive background to the projects, a review of relevant literature, descriptions of the monitoring equipment and methods at both sites, data management and analysis procedures, results for the period of coverage, discussion of findings, and what conclusions could be drawn at that time.

This report updates the data at both sites from the beginning of the water year on October 1, 2000 through April 2002. It relates more recent to earlier findings and draws additional conclusions with the more complete record.

1.2. Brief Description of Instrumentation

This subsection provides a basic description of the monitoring systems established at both projects. Refer to Miller (2001) and Miller, Burges, and Horner (2001) for full details.

The log weirs at the ends of cells 1 and 15 of the Viewlands Cascades Drainage System were outfitted with V-notch weirs to serve as controls for comparative flow monitoring near the entrance and exit of the channel. Weir water levels, from which flow rates were computed, were sensed at each point with both float/shaft encoders and submersible pressure transducers.

The Viewlands site has a full meteorology station on the adjacent elementary school property. The station has three precipitation gauges, two tipping-bucket recording gauges and a non-recording collector. Mounted on a tripod are temperature and relative humidity probes, a wind anemometer, a net radiometer, a short-wave pyranometer, and a solar panel for power supply. The station also includes an evaporation pan with an anemometer and a radiometer mounted just above the water surface. Data from all flow and meteorological instruments are logged at one of three data loggers at the station for computer downloading.

With the collection of sufficient data, the downstream Viewlands flow monitoring station was decommissioned in May 2002. The upstream station will continue in operation for at least several more years to serve as the check point for rainfall-runoff mathematical modeling of the catchment now getting underway. All meteorological instruments will also continue to function to support the same purpose. The goal of this enterprise is to develop a calibrated, verified hydrologic model that can be used for future stormwater management decision making relative to small catchments in the Pipers Creek watershed.

With no runoff entering from outside its catchment, the 2nd Avenue NW SEA Streets site was equipped only with a flow monitoring station at the point where runoff exits the project. This station has a float/shaft encoder with a stilling basin and V-notch weir flow control. In an adjacent yard are a tipping-bucket recording precipitation gauge and a non-recording collector. This site has one data logger. The 2nd Avenue NW monitoring system will continue to operate for an undetermined period of time to collect more post-construction data.

1.3. Summary of January 2000-January 2001 Results

1.3.1. Viewlands Cascade Drainage System

For the period July 2000 to January 2001, the Viewlands flow monitoring equipment registered a peak upstream flow rate of 3.9 cfs (110 L/s), approximately one-sixth of the anticipated peak flow rate for the 25-year, 24-hour design rainfall event of 25 cfs (708 L/s). Two storms approximated the 6-month, 24-hour storm. The remaining 34 storms fell beneath this level. Due to the relatively low precipitation, assessment of the performance of the swale design based on calendar year 2000 was limited.

The estimated average water velocities through the swale ranged from a maximum of 2.7 ft/s (0.8 m/s), for the largest flow rate to a minimum of 0.11 ft/s (0.03 m/s). The minimum hydraulic residence time (channel volume/peak flow rate) ranged from 1.67 minutes, at the larger flow rates, to as much as 41 minutes. Any storms above the maximum observed peak flow rate will have a residence times of less than two minutes and a velocity greater than 3 ft/s (0.9 m/s).
Of the 36 individual storms that produced measurable runoff in the Viewlands channel, in 14 cases no inflow reached the downstream monitoring station, almost all presumably having infiltrated the soil. Regardless of soil moisture conditions, the channel retained up to approximately 1,000 ft^3 (28.3 m^3) of runoff, while high retention (75 to 99.9 percent) was achieved for inflow volumes in the range of 1,000 to 3000 ft^3 (28.3 to 85.0 m^3). The cascade system could fully attenuate runoff from an average precipitation depth of 0.22 inch (5.6 mm) during dry soil moisture conditions and 0.13 inch (3.3 mm) during wet conditions. During the dry soil period, 78 percent of the measured inflow infiltrated or was otherwise retained by the channel. Retention dropped to 34 percent during the wet soil period. Over the course of the July 2000 to January 2001 study interval, the system retained 38 percent of the total inflow. In addition to the hydrologic benefit to Pipers Creek, pollutant mass loading would decrease by at least as much, and most likely more due to contaminant capture in the channel's vegetation and soil.

The highest reductions in peak flow rates were coupled with the highest percentages of retention in the channel. For the storm hydrographs that were analyzed, there was either complete attenuation of the peak flow rates or modest (<20 percent) reductions in the peaks. 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 (after 30 minutes to 6 hours of flow), the inflow and outflow rates consistently matched one another.

For comparison to the Viewlands Cascade Drainage System, flow retention in the previous ditch was estimated for the same storms according to the volume of water estimated to infiltrate over the wetted area. Over the course of the study period, the cascade retained 73,710 ft^3 (2,090 m^3) of water. Under the same meteorological conditions, the previous ditch would have infiltrated, at most, an estimated 24,650 ft^3 (700 m^3), or 67 percent less.

1.3.2. 2nd Avenue NW SEA Streets

For the March to July 2000 pre-construction period, the 2nd Avenue NW 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 (43 L/s) for a 25-year, 24-hour rainfall event. Analysis of the storm hyetographs and hydrographs for the 35 storms during the predominantly wet soil moisture conditions indicated a rapid, precipitation-driven runoff response. As a result, the runoff hydrograph closely followed the start, rise, and fall of the precipitation hyetograph.

To put the hydrologic analysis of the baseline 2^{nd} Avenue NW conditions into perspective, runoff volumes were estimated for both a conventional street design and the SEA Streets design. The cumulative measured runoff volume from the existing street was 8601 ft^3 (244 m^3) during the study period of March 11 to July 11, 2000. The conventionally designed road with a curb/gutter/sidewalk system would have generated an estimated 14806 ft^3 (420 m^3) of runoff under the same rainfall conditions, or 72 percent more. It was estimated that, with a SEA Streets design and the same precipitation, the street right of way would have produced 4989 ft^3 (141 m^3) of runoff. This quantity is 42 percent less than the runoff from the pre-existing street and 66 percent less than from a conventionally designed road. Runoff from the east-side properties into the street was observed to be very minor and, if included, would not change these figures appreciably. Water pollutant mass loadings are expected to be lower from the innovatively designed street by at least equivalent amounts, and probably by more through pollutant trapping in the vegetated ponds.

CHAPTER 2 - PROBLEMS AND PROCEDURAL MODIFICATIONS FOR 2001-2002 MONITORING

2.1. 2nd Avenue NW Shaft Encoder Pulley Slippage

Instrument-recorded stages are routinely checked against manual measurements with a tape to determine if any instrument correction is needed. If there are some differences but they are consistent over time, the shaft encoder readings can be adjusted. Inconsistency signifies slipping of the pulley on its shaft. A pattern of inconsistency was noted in early 2002. Investigation traced intermittent slippage back as far as March 11, 2001. Reliable manual measurements were available to determine discharge from April 7 through November 20, 2001, after which these measurements were not considered to be reliable again until January 3, 2002. From then until the first week of April, manual measurements again were available until the pulley was tightened. The setscrew only bears against the shaft, though, less securely than a positive seating. The installation is being further improved during the dry mid-summer period in 2002, when discharge is not expected.

As reported below, discharge from the 2nd Avenue NW catchment has been very limited since completion of the project construction. Analysis indicated that it was very unlikely that any discharge occurred during the first period without manual measurements in the spring of 2001. There undoubtedly was some discharge during the second data gap in the following winter, but it was possible to estimate its amount by comparing meteorological conditions then and when discharge was known (refer to Section 3.2.2).

2.2. Missing Data

Precipitation data at 2nd Avenue NW are missing for 22 days in the six wettest months of 2001, as well as 19 days in June and July, mostly because of battery problems and data logger malfunctions. Two recording gauges only about 15 blocks away at the Viewlands station permitted very close estimation to close these data gaps.

Snowmelt on one occasion in February 2001 caused the Viewlands trench rain gauge to over-estimate precipitation. The standing gauges there and at 2nd Avenue NW allowed correction to be made.

2.3. Viewlands Water Losses

It has been observed since the beginning of monitoring that water levels in Viewlands cells 1 and 15, ahead of the upstream and downstream V-notch weirs, respectively, continue dropping after flow into the cells and over the weirs stops. This water loss is positive from a performance standpoint, since much of this water infiltrates, although some leakage can be seen under the logs. However, the loss complicates monitoring and the upstream versus downstream flow comparison. Various efforts, described by Miller

(2001) and Miller, Burges, and Horner (2001), were attempted to stop water loss, without much success. These references also discuss early tests to attempt to quantify losses. They concluded that water loss is a major factor at relatively low flow rates but that flow measurements are likely to be quite accurate above 0.25 cfs (7.1 L/s). Measured loss rates were mostly in the range 0.03-0.04 cfs (0.85-1.1 L/s). The measurements were considered to be too limited for conclusive correction of low flow rates.

Additional loss tests were performed in the first cell using a fire hydrant as a water source in September 2001. Testing was precluded at the downstream end by the danger of discharging chlorinated water to Pipers Creek. The test procedure follows.

Field Procedure

1. Place a measuring stick with fine divisions on a flat, firm platform in the first channel bay. Bring the water up to the point where water loss just begins.

2. Add flow and/or adjust the measuring stick to get the water level exactly at a mark on the measuring stick; shut off flow. Record water depth.

3. Use a stopwatch to time how long it takes for the water level to drop to another selected mark. Record that depth.

4. Immediately when the level drops to the lower selected mark, signal an observer at the hydrant to start water flow at a high rate. That observer must record the flow meter reading before starting flow.

5. Observe the water level rise. When it reaches the initial mark (recorded in step 1), immediately signal the observer at the hydrant to stop flow. Record the meter reading when flow stops.

Data Analysis

1. Subtract the reading in step 4 from the reading in step 5 to get the volume needed to replace leakage. Divide by the time recorded in step 2 to get loss rate.

2. Plot loss rate versus depth and investigate how corrections should be made to compensate for loss.

The full range of loss rates in three test series was 0.012-0.13 cfs (0.34-3.7 L/s). One of the series exhibited some relationship to cell depth in the approximate range 1.4-1.9 ft (0.43-0.58 m), although a rather inconsistent one between two replicates. Averages of the replicates in this series ranged 0.020-0.085 cfs (0.57-2.4 L/s). These data were considered to be insufficient for conclusively correcting inflows. Both the 2000 and 2001 test results are considered qualitatively in this report to judge the effect of water losses on measurements. Loss testing is being attempted again with a different procedure during the summer of 2002.

2.4. Viewlands Weir Calibration

Weir equations assume ideal conditions that are seldom fully achieved in the field. Potential non-ideal conditions include insufficient stilling of the flow prior to measurement, inadequate approach height between the channel bottom and V-notch invert, and clinging of the water to the weir plate instead of springing free. The Viewlands weirs exhibit the latter two problems. The vertical distances between the inverts and the logs in which the weirs are set are less than the desired 2 ft (0.61 m). The weir nappes are observed to cling to the plates at their outside edges, although not across their full widths. In addition, the high potential flows at Viewlands required V-notch angles of 120^0, above the 90^0 maximum commonly observed. Potential inaccuracies from these causes are accentuated at relatively low flows.

To investigate the cumulative effect of these concerns and possible correction, low-flow calibration tests were performed in conjunction with the 2001 water loss tests at Viewlands according to the following procedure.

Field Procedure

1. Turn on the water fully and fill the first channel bay rapidly until the level approaches the weir V-notch. At that point slow the flow down and bring the level to the point where flow passes over the weir. Further decrease the flow until the rotating dial on the hydrant flow meter is barely moving. Let the water flow for about 15 minutes to stabilize the level.

2. Firmly install the water collection box and check that it is level, but exclude flow with the swivel mechanism and box cover.

3. Read the stage in the first bay from the markings on the weir plate about every five minutes. When the stage is stable for three successive readings, record the stage depth and begin the test.

4. Station one observer at the hydrant flow meter with a stopwatch. During the test that observer will take three replicate readings of flow rate. These readings do not have to be exactly coordinated with the upstream observer's work but should be during the same time period. Take the readings by starting the stopwatch as the rotating dial passes a number and stopping it when it passes another number. For each reading, record the number of cubic ft flowing during that period and the time registered on the stopwatch. These readings can be taken over intervals of 3 cubic ft.

5. Station another observer at the weir with a stopwatch. Record the exact time immediately before flow is introduced to the collection box. Introduce flow by rotating the swivel mechanism and removing the box cover, simultaneously starting the stopwatch.

6. Carefully observe when water first overflows the collection box and stop the stopwatch. Record the time elapsed. Also record the stage depth. If it has changed since the test began, discard the results and repeat steps 3-7.

7. Increase the flow rate at the hydrant until the rotating dial is moving noticeably faster. Observe the stage in the first channel bay from the markings on the weir plate. If the stage does not increase after 5-10 minutes, increase the flow rate slightly and repeat this step until there is a definite stage increase.

8. Repeat steps 3-8 until the flow rate is at a maximum (when it does not change appreciably as the hydrant valve is opened further).

<u>Data Analysis</u>

1. Compute hydrant flow rates by dividing each flow quantity in cubic ft by the seconds elapsed.

2. Compute weir flow rates by dividing the volume of the collection box in cubic feet by the seconds elapsed during collection.

3. Compare hydrant and weir flow rates and determine if there is a consistent difference between them. Consistently higher hydrant measurements may signify water losses. In this case they should be considered as the primary basis to correct shaft encoder and pressure transducer readings at low flows. If there is no consistency, both hydrant and collection box measurements should be considered and a decision made about how to correct after further analysis.

4. Associate the test periods with the instrumentation records by using the recorded time at the start of the tests. Compare instrument and manually recorded stages and flow rates and assess how best to correct instrument readings at low flows.

5. After a correction procedure is decided upon, correct all flows in the low-flow range recorded during the entire monitoring program.

It was evident that hydrant flow rates were consistently higher than measured weir overflow rates and that water losses continued throughout the testing. Therefore, it was decided to take the hydrant meter as the basis for inflow to the channel. Water loss rates were calculated as the difference between hydrant and measured outflow rates. Figure 2-1 plots inflow, outflow, and loss rates for the two test series intended to calibrate the weir. Also graphed are outflow rates according to the shaft encoder.

Directly measured and shaft encoder flow rates were close over the range tested, deviating slightly more at the higher rates than the lower ones, although all rates were very low relative to weir capacity. The results show that water losses represent a greater monitoring problem than non-ideal weir conditions at low flow rates. The lowest flow rates in Figure 2-1 could actually be more than double those measured without losses.

Both losses and weir calibration are being addressed again with refined procedures in the summer of 2002.

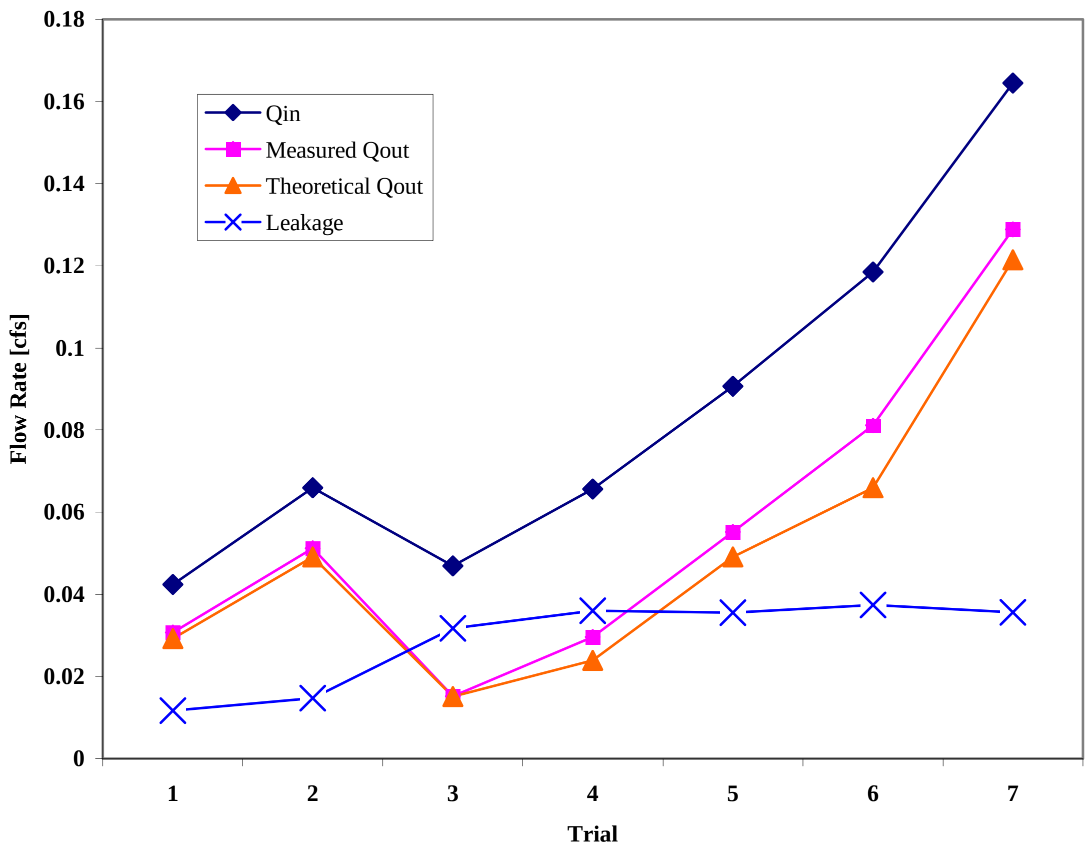


Figure 2-1. Viewlands Upstream Weir Flow as Measured Directly and by Shaft Encoder in Comparison to Inflow from Fire Hydrant in Weir Calibration Tests (water loss is taken as the difference between inflow and measured outflow)

CHAPTER 3 - PRECIPITATION AND FLOW ANALYSIS

3.1. <u>Precipitation Analysis</u>

Table 3-1 presents monthly and yearly precipitation totals from the onset of monitoring in January 2000 through April 2002 at the project stations and Seattle-Tacoma International Airport, as well as the antecedent 1999 year at the airport. Calendar-year 2001 precipitation was close to the long-term mean, while 1999 was above the average and 2000 was 25 percent below.

Table 3-1. Precipitation Summary for Full Monitoring Period

							Millimeters						
Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Total**
Viewlands 2000[a]	62	115	73	33	68	31	12	11	33	81	83	67	**669**
Viewlands 2001[a]	90	56	74	63	32	90	20	59	10	89	229	144	**957**
Viewlands 2002[a, b]	153	105	72	66									
2nd Ave NW 2000[c]			77	33	54	27	11						
2nd Ave NW 2001[d]	86	52	66	54	28	81	22	53	10	87	231	141	**911**
2nd Ave NW 2002[b]	156	101	78	64									
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	64	**730**
SeaTac 2001	69	53	69	80	35	77	26	59	21	80	235	150	**954**
SeaTac 2002	152	106	72	108									
SeaTac 52-y 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[a]	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**
Viewlands 2001[a]	3.5	2.2	2.9	2.5	1.3	3.5	0.8	2.3	0.4	3.5	9.0	5.7	**37.7**
Viewlands 2002[a, b]	6.0	4.1	2.8	2.6									
2nd Ave NW 2000[c]			3.0	1.3	2.1	1.1	0.4						
2nd Ave NW 2001[d]	3.4	2.0	2.6	2.1	1.1	3.2	0.9	2.1	0.4	3.4	9.1	5.6	**35.9**
2nd Ave NW 2002[b]	6.1	4.0	3.1	2.5									
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	2.5	**28.8**
SeaTac 2001	2.7	2.1	2.7	3.2	1.4	3.1	1.0	2.3	0.8	3.1	9.3	5.9	**37.6**
SeaTac 2002	6.0	4.2	2.8	4.3									
SeaTac 52-y mean	5.4	4.2	3.7	2.5	1.7	1.5	0.8	1.1	1.9	3.5	6.0	5.9	**38.2**

[a] All Viewlands readings are from the trench recording gauge, except for February 2001, when the standing recording gauge reading is used because of snowmelt that produced an inaccurate standing gauge total.

[b] Results reported through Apr.

[c] Monitoring performed only from March through July.

Based on the airport station, wet season (October-March) totals were:

52-y mean—28.7 inches (729 mm);
1999-2000—17.4 inches (442 mm), 61 percent of 52-y mean;
2000-2001—16.3 inches (414 mm), 57 percent of 52-y mean; and
2001-2002—31.3 inches (794), 109 percent of 52-y mean.

Initial monitoring occurred during relatively dry winters. The most recent winter approximates typical conditions in the region, and thus provides a better opportunity to assess performance capabilities of the drainage projects.

The October 2000 to March 2001 wet season had two storms approximating the 6-month, 24-hour rainfall event for the region and one that exceeded 24 hours duration and the precipitation total associated with the 1-year, 24-hour event. In contrast, the following winter period had three storms exceeding 24 hours with rainfall between the 6-month, 24-hour and 1-year, 24-hour totals, plus three additional events lasting over 24 hours and exceeding the 1-year, 24-hour total. Furthermore, August 2001 had an unusually large summer storm also longer than 24 hours and with more rain than the 1-year, 24-hour rainfall. However, since the outset of monitoring, there have been no very large storms with infrequent return periods.

Monthly precipitation totals were generally consistent among measuring stations. Mean and maximum monthly differences for all months in the record were:

SeaTac averaged 0.1 inch (2.5 mm) more than Viewlands, with a maximum difference in any month of 1.7 inch (43 mm) more;
SeaTac averaged 0.2 inch (5.0 mm) more than 2nd Avenue NW, with a maximum difference in any month of 1.8 inch (46 mm) more; and
Viewlands averaged 0.1 inch (2.5 mm) more than 2nd Avenue NW with a maximum difference in any month of 0.5 inch (13 mm) more.

In theory, the trench gauge should collect more precipitation than the standing device, because of lesser wind effects at the lower elevation. However, during 2000 the reverse was true, with the standing gauge registering 6.5 percent more over the year. Expectations mainly held for the remainder of the record (January 2001-April 2002). The trench gauge collected more rainfall than the standing instrument in 11 of the 16 months, with one even. The trench gauge collection averaged 0.1 inch (2.5 mm) per month higher in the period and, overall, totaled 5.9 percent more than the standing gauge contents. The greatest positive and negative disparities in any month for the trench versus standing gauges were 0.4 inch (9.5 mm) more and 0.2 inch (5.7 mm) less.

3.2. Flow Analysis

3.2.1. Viewlands Cascade Drainage System

Rainfall and Runoff Event Summary

Table 3-2 summarizes Viewlands drainage system rainfall and runoff statistics for 122 events over the period beginning at the onset of the 2001 water year (1 October 2000) and concluding on 30 April 2002. Statistics are tabulated for the 2001 dry season both with and without an unusually large summer storm. Seven precipitation events during April, June, and July 2001 are missing from the Viewlands flow record because of flow instrumentation malfunction. These storms ranged from 0.05 to 1.25 inch (1.3 to 31.8 mm) of rain. This range was covered by the 11 events recorded during the dry season, and it is not likely that overall statistics would be heavily influenced if the missing data were available, although recording of total runoff volumes is incomplete for that season.

All runoff measurements are subject to adjustment once the results of summer 2002 water loss testing are available. Because losses are a major factor only during relatively small flows, it is not expected that adjustment will result in radical changes in overall statistics and the conclusions drawn from them. Two instances of highly negative flow rate decreases were associated with low absolute values and minor differences between upstream and downstream. These cases are probably principally a function of losses.

The rainfall statistics demonstrate the distinctions between the wet and dry seasons (e.g., a mean antecedent dry period more than three times as long in the dry compared to the wet season). They also indicate the different characteristics of the two wet seasons represented. As shown in Table 3-1, the 2001-2002 winter was much wetter overall, with 79 percent more precipitation. However, its mean precipitation intensity was 27 percent less. Rain was spread over an average storm duration 30 percent longer. Mean channel response times (time between start of rain and registration of water in the channel) were 25 percent longer upstream and almost double downstream during the drier 2000-2001 winter. Response times exhibit similar differences between wet and dry seasons (excluding the large August storm).

With no infrequent, large rainfalls in the data record, peak upstream flow rate has not yet approached the maximum 25 cfs (708 L/s) estimated for the 25-year, 24-hour event during project design. The peak seen thus far was during a December 2001 storm (4.06 cfs, 115 L/s), also approached during the large summer 2001 event (3.97 cfs, 113 L/s).

Notwithstanding seasonal and annual distinctions in rainfall and rainfall-runoff relations, channel hydrology and hydraulics did not differ as much from wet to dry seasons and between divergent winters. Maximum discharge rate and flow volume reductions from upstream to downstream were similar in the two winters, about 53 percent for rate and 70 percent for volume. These decreases rose to 65 percent for flow rate and 80 percent

Table 3-2. Viewlands Rainfall and Runoff Event Summary, October 1, 2000-April 30, 2002

Period (No. events)	Statistic	Antecedent Dry Period (Hours)	Rainfall (Inch)	Duration (Hours)	Average Intensity (Inch/Hour)	Upstream Response (Hours)	Downstr. Response (Hours)	Maximum Upstream Flow Rate (cfs)	Maximum Downstr. Flow Rate (cfs)	Flow Rate Decrease (%)	Upstream Flow Volume (ft3)	Downstr. Flow Volume (ft3)	Flow Volume Decrease (%)	Average Velocity (ft/sec)	Minimum Residence Time (Minutes)
10/1/00-3/31/01	Mean	78.2	0.40	13.9	0.033	3.0	2.9	0.62	0.42	52.5	5654	2990	70.2	1.21	4.8
Wet	Std. Dev.	68.5	0.45	11.9	0.019	3.0	5.3	0.66	0.65	37.8	7460	5570	27.3	0.45	5.6
(47)	Maximum	336.3	2.76	61.5	0.094	14.3	31.3	3.88	3.80	100.0	35457	26941	100.0	2.70	41.2
	Minimum	5.8	0.04	1.0	0.009	0.5	0.0	0.00	0.00	0.0	0	0	16.2	0.11	1.7
4/1/01-9/30/01	Mean	223.6	0.48	11.8	0.048	3.2	1.7	1.08	0.74	59.9	6689	3141	76.5	1.50	3.4
Dry	Std. Dev.	240.5	0.56	9.5	0.040	2.1	3.3	1.11	1.17	39.5	11657	6973	23.9	0.56	1.1
(11)	Maximum	826.0	2.15	37.0	0.138	6.3	11.5	3.97	3.66	100.0	41086	23699	100.0	2.72	4.6
	Minimum	8.3	0.15	3.0	0.019	0.8	0.0	0.28	0.00	-16.4	1054	0	38.2	0.97	1.7
4/1/01-9/30/01	Mean	208.8	0.32	9.3	0.047	3.4	0.7	0.79	0.45	65.1	3249	1085	79.9	1.37	3.5
Dry excluding	Std. Dev.	248.2	0.11	4.8	0.042	2.1	0.7	0.59	0.68	37.5	2524	1540	22.1	0.41	0.9
Aug. storm	Maximum	826.0	0.49	19.0	0.138	6.3	2.0	1.78	2.07	100.0	7874	3998	100.0	2.00	4.6
(10)	Minimum	8.3	0.15	3.0	0.019	0.8	0.0	0.28	0.00	-16.4	1054	0	38.2	0.97	2.3
10/1/00-9/30/01	Mean	105.8	0.42	13.5	0.035	3.0	2.6	0.71	0.48	53.9	5850	3018	71.4	1.26	4.6
Water Year	Std. Dev.	131.3	0.47	11.4	0.025	2.8	5.0	0.78	0.77	37.9	8302	5794	26.6	0.48	5.1
(58)	Maximum	826.0	2.76	61.5	0.138	14.3	31.3	3.97	3.80	100.0	41086	26941	100.0	2.72	41.2
	Minimum	5.8	0.04	1.0	0.009	0.5	0.0	0.00	0.00	-16.4	0	0	16.2	0.11	1.7
10/1/01-3/31/02	Mean	59.6	0.48	18.1	0.024	2.4	1.5	0.73	0.53	54.5	15140	10068	68.9	1.35	3.8
Wet	Std. Dev.	69.8	0.66	18.0	0.014	1.7	1.8	0.81	0.71	43.2	27064	20567	29.5	0.50	1.4
(58)	Maximum	341.5	2.95	97.8	0.063	8.5	8.5	4.06	3.11	100.0	108691	82862	100.0	2.75	6.6
	Minimum	5.5	0.01	2.0	0.001	0.5	0.0	0.01	0.00	-121.9	14	0	17.2	0.68	1.6
10/1/00-3/31/01,	Mean	67.9	0.44	16.2	0.028	2.6	2.1	0.68	0.48	53.6	10894	6900	69.5	1.28	4.3
10/1/01-3/31/02	Std. Dev.	69.5	0.57	15.6	0.017	2.4	3.8	0.74	0.68	40.7	21179	16065	28.4	0.48	4.0
2 wet seasons	Maximum	341.5	2.95	97.8	0.094	14.3	31.3	4.06	3.80	100.0	108691	82862	100.0	2.75	41.2
(105)	Minimum	5.5	0.01	1.0	0.001	0.5	0.0	0.00	0.00	-121.9	0	0	16.2	0.11	1.6
10/1/00-4/30/02	Mean	83.9	0.44	15.6	0.030	2.6	2.1	0.72	0.50	53.8	10233	6337	70.3	1.31	4.2
Current study	Std. Dev.	106.7	0.56	15.0	0.020	2.3	3.7	0.78	0.73	40.5	20035	15121	27.7	0.48	3.7
period	Maximum	826.0	2.95	97.8	0.138	14.3	31.3	4.06	3.80	100.0	108691	82862	100.0	2.75	41.2
(122)	Minimum	5.5	0.01	1.0	0.001	0.5	0.0	0.00	0.00	-121.9	0	0	16.2	0.11	1.6

for volume in the dry season (excluding the large August storm). Average velocity was only slightly higher and minimum hydraulic residence time just marginally shorter in the wetter 2001-2002 winter compared to the preceding year. Dry and wet season average minimum residence time and average velocity differed little.

During the initial study period 14 of 36 events (39 percent) produced no downstream discharge (Miller 2001; Miller, Burges, and Horner 2001). Instances of complete attenuation fell slightly to 30 percent since 10/1/00, which encompassed the wetter 2001-2002 winter. The highest reductions in flow volume were coupled with the greatest decreases in peak flow rate from channel entrance to exit, consistent with previous observations. In the earlier period up to approximately 1000 ft^3 (28.3 m^3) of influent was fully attenuated (Miller 2001; Miller, Burges, and Horner 2001). The larger data set now available almost always confirms that aspect of performance. The initial analysis found that an average precipitation depth of 0.13 inch (3.3 mm) could be fully attenuated during wet conditions (Miller 2001; Miller, Burges, and Horner 2001). This conclusion was also confirmed with more data.

Total Discharge Summary

The 2001-2002 wet season registered 79 percent more precipitation than the preceding winter and more than three times as much total inflow to the Viewlands Cascade (878103 ft^3, or 24884 m^3, in 2001-2002 versus 265743 ft^3, or 7531 m^3, in 2000-2001). On average, antecedent dry periods were shorter, storm durations were longer, and rainfall quantities were larger in the wetter winter. Even though the latter winter had slightly lower precipitation intensity, the combination of other factors accentuated the effect of overall rainfall on runoff hydrology.

As averages from all events, the high mean flow volume decreases shown in Table 3-2 are misleading. Most events have relatively small rainfall quantities, and infiltration is generally more complete with small volumes. Averaged in this way, therefore, the relatively more numerous small events influence the statistics more than the fewer large rainfalls. More indicative of overall recharge from the drainage channel are the total seasonal and annual flow volume decreases:

10/1/00-3/31/01 (wet season)—	47.1%
4/1/01-9/30/01 (dry season)—	53.0%
4/1/01-9/30/01 (dry season excluding August storm)—	66.6%
10/1/00-9/30/01 (water year)—	48.4%
10/1/01-3/31/02 (wet season)—	33.5%
10/1/00-3/31/01, 10/1/01-3/31/02 (two wet seasons)—	36.7%
10/1/00-4/30/02 (current study period)—	38.1%

Flow volume decreases for the most recent wet season and the full current study period are very consistent with those registered in the initial study period, which were 34 percent for the wet season and 38 percent overall (Miller 2001; Miller, Burges, and Horner 2001).

Attenuation was greater during the relatively dry 2000-2001 winter. Lower dry season reduction occurred in the 2001 summer, 67 percent as compared to 78 percent during the preceding summer, even omitting the exceptionally large August 2001 storm. This difference is likely the result of the wetter antecedent conditions following the above-average winter rainfall.

The new Viewlands drainage system prevented direct release to Pipers Creek of 294176 ft^3 (8337 m^3) of runoff in the 2001-2002 wet season and 485379 ft^3 (13755 m^3) through the current study period. While the proportion of inflow attenuated fell from the preceding, drier winter, the volume retained was 2.3 times as large.

The Viewlands catchment exhibited runoff coefficients differing greatly between seasons and years. Based on a catchment area of 26 acres (10.5 ha), 17.7 inches (450 mm) of precipitation during the 2000-2001 wet season, and 31.1 inches (790 mm) in the following winter, the runoff coefficient (inflow/rainfall volume) was 16 percent in the first case and 30 percent in the second. Cumulative dry period runoff coefficient was only 8 percent. These results demonstrate the large effect of specific conditions on runoff coefficients and the unreliability of characterizing hydrology with their use.

Hydrograph Analysis

Figures 3-1 through 3-6 show hyetographs and hydrographs for selected events at Viewlands. Comparisons of the upstream and downstream hydrographs illustrate the roles of antecedent moisture conditions and, especially, precipitation intensity in determining flow attenuation by the Viewlands channel. Events from winter, spring, and fall graphed in Figures 3-1, 2, and 4, respectively, exhibit substantial peak rate reductions. The first two of these events had antecedent dry periods of one day or less, while there was no rain for more than 5 days prior to the third. This relatively long antecedent dry period ameliorated a somewhat elevated intensity.

In contrast, the other three hydrographs, again representing three different seasons, show little attenuation, even increased downstream rate for a time in Figure 3-5. All of these cases had intense bursts of precipitation, and the April 2001 event (Figure 3-5) had the highest average intensity in the current study period. Again, their antecedent dry periods varied, from less than 2 days for the large February event (Figure 3-6) to more than 15 days for the unusually large summer storm (Figure 3-3). Regardless of the low soil moisture, little flow rate attenuation occurred during the most intense interval of the latter event, when rain fell at approximately 0.5 inch/hour (13 mm/h). Maximum downstream discharge rate was close to the highest of any storm during the present study period. It is clear that good performance in flow attenuation strongly depends on having moderate intensities, both during intra-storm intervals and over the full storm. Of course, this pattern generally prevails in Seattle, to the benefit of performance in drainage courses like the Viewlands Cascade.

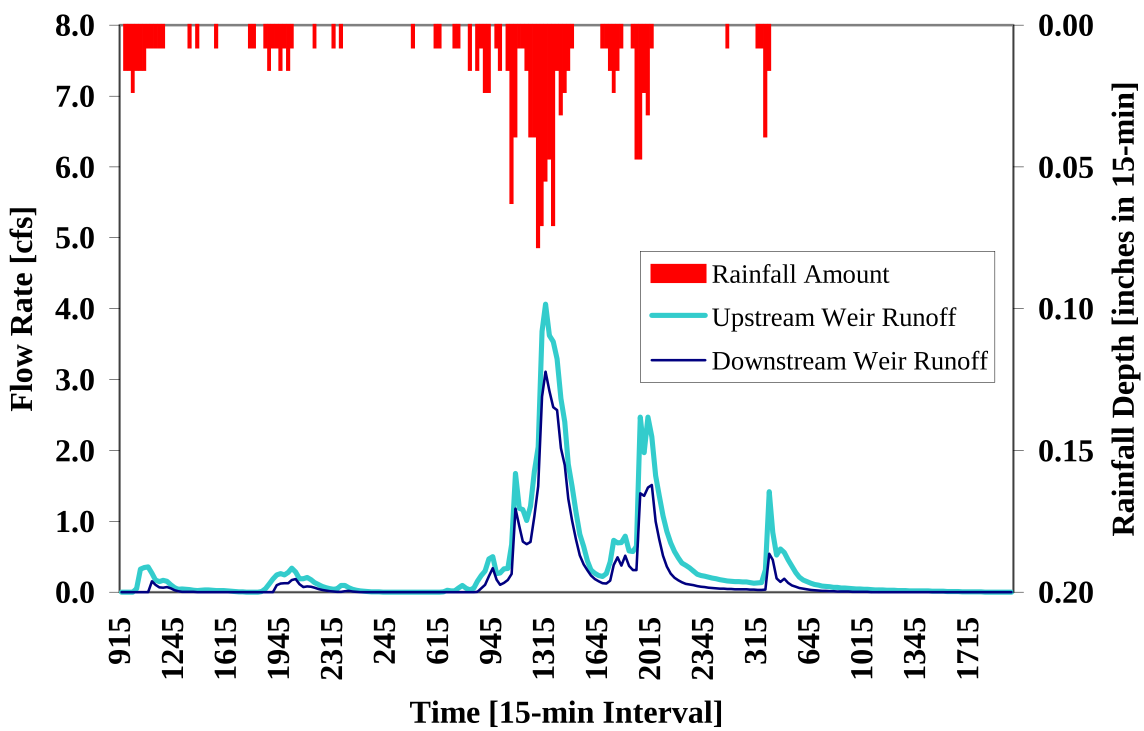


Figure 3-1. Viewlands Rainfall Hyetograph and Runoff Hydrograph, December 12 (9:15 AM) - December 14 (7:30 PM), 2001

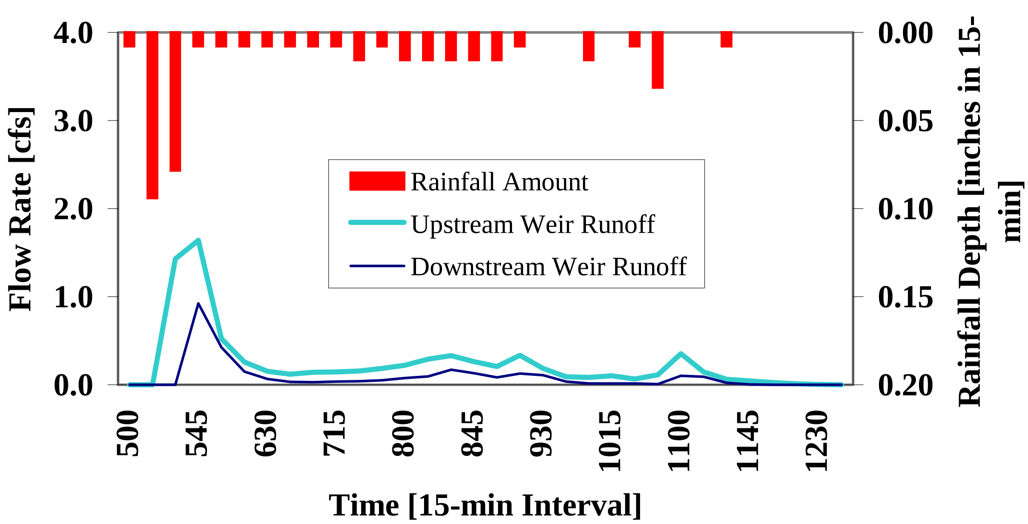
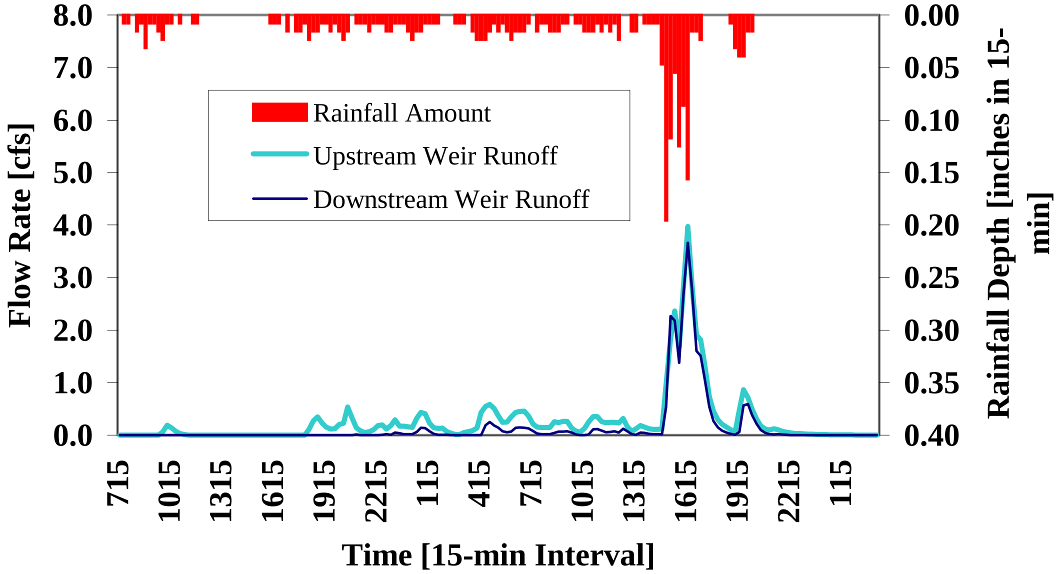


Figure 3-2. Viewlands Rainfall Hyetograph and Runoff Hydrograph,



April 30 (5:00 AM) - April 30 (12:45 PM), 2001

Figure 3-3. Viewlands Rainfall Hyetograph and Runoff Hydrograph, August 21 (7:15 AM) - August 23 (3:00 PM), 2001

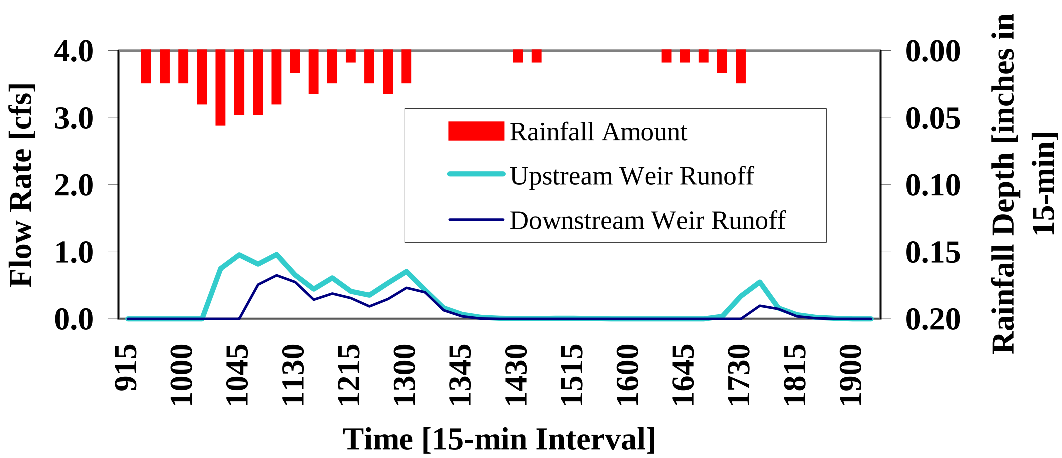


Figure 3-4. Viewlands Rainfall Hyetograph and Runoff Hydrograph, October 16 (9:15 AM) - October 16 (7:15 PM), 2001

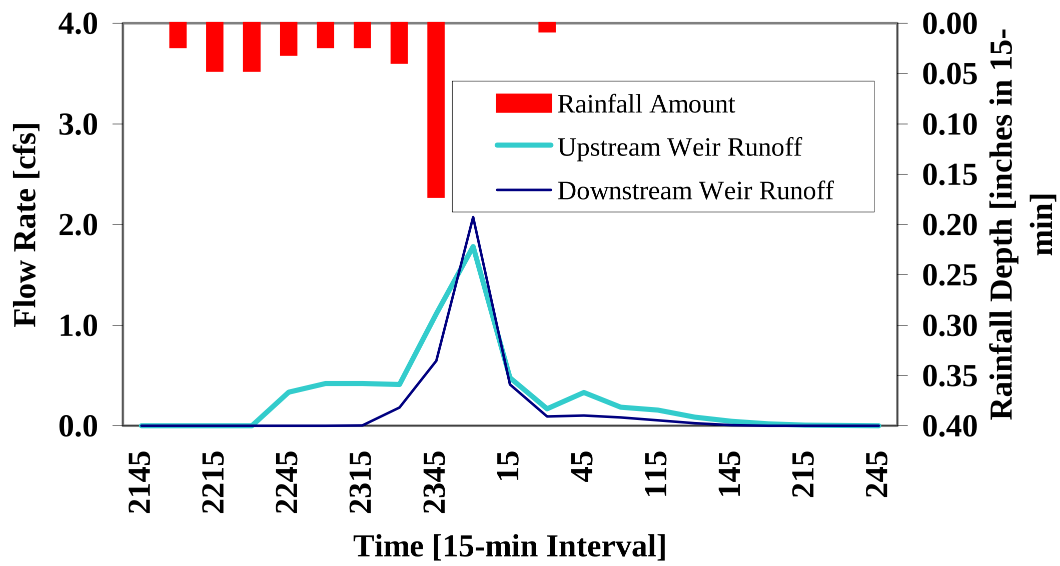


Figure 3-5. Viewlands Rainfall Hyetograph and Runoff Hydrograph, April 16 (9:45 PM) - April 17 (2:45 AM), 2001

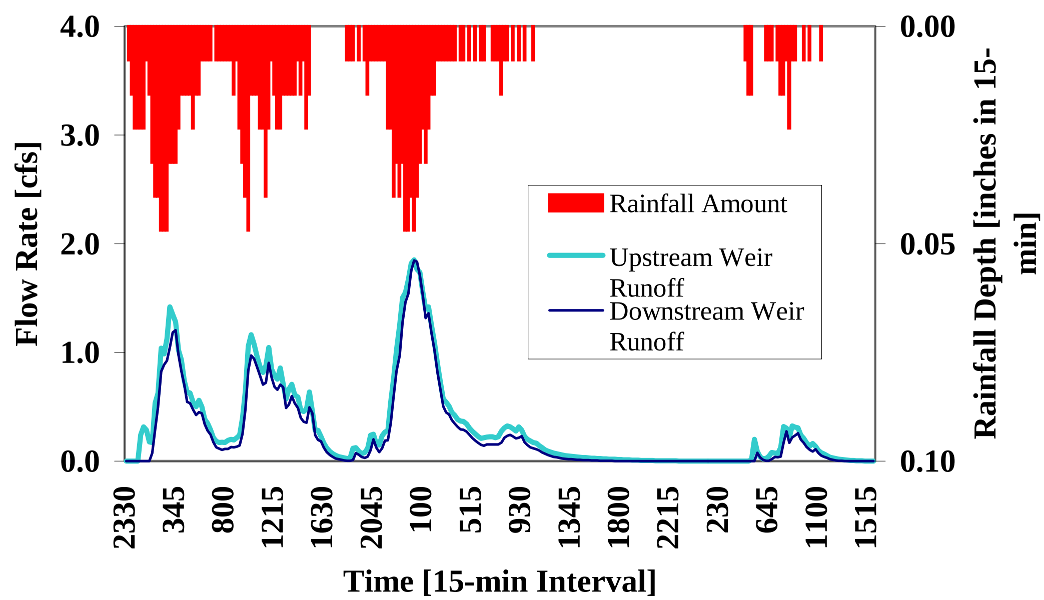


Figure 3-6. Viewlands Rainfall Hyetograph and Runoff Hydrograph, February 20 (11:30 PM) - February 23 (3:30 PM), 2002

Comparison with Preceding Ditch

Table 3-3 compares key performance aspects of the Viewlands Cascade Drainage System with the ditch that preceded it based on the estimation procedure for the ditch described earlier. With equivalent meteorology, the ditch is estimated to attenuate through infiltration only about one-third as much flow volume, during both dry and wet seasons, under average and maximum conditions, and in total. This uniformity in prediction is an artifact of the simple model used to estimate infiltration from the old ditch but is generally indicative of the different potential recharge in the two cases. The preceding drainage conduit would have released to Pipers Creel approximately 191000 ft^3 (5413 m^3) of runoff that was retained by its successor during the 2001-2002 wet season and 319000 ft^3 (9040 m^3) over the course of the current study period.

Average velocities are estimated to be approximately 20 percent higher in the old ditch under the full range of conditions. One of the main reasons for rebuilding in the cascade configuration was to reduce the observed high velocities in the ditch, which resulted in frequent bypass of the downstream drain inlet and erosion of the slope beyond it. The lack of relatively large storms has not provided a real test of velocity reduction yet. Minimum hydraulic residence times are estimated to be about a factor of two longer in the new system compared to the old ditch under most circumstances observed to date, although closer in the two channels in the smallest storms.

3.2.2. 2nd Avenue NW SEA Streets

Rainfall and Runoff Event Summary

Table 3-4 summarizes rainfall and 2nd Avenue NW runoff statistics for 96 events over the period beginning just after completion of construction (20 January 2001) and concluding on 30 April 2002. As to be expected because of the proximity of the two sites, the rainfall statistics demonstrate the same tendencies as described for Viewlands.

Undetected shaft encoder slippage produced unreliable readings from 10 March 2001 until 7 April 2002. Reliable manual stage measurements were available to determine discharge from 1 May to 20 October 2001 and 3 January to 30 March 2002. For the remaining intervals the antecedent dry periods, rainfall totals, storm durations, average intensities, and estimated precipitation volumes were examined in relation to the same measurements for events when discharge was measured. There was never any measured discharge when the estimated precipitation volume was less than 2300 ft^3 (65.2 m^3), representing substantial ranges of the meteorological variables. Thus, it was safe to assume that there was no discharge associated with any unmeasured events below that rainfall volume total. This volume is associated with a rainfall total of about 0.75 inch (19 mm), the runoff from which can apparently be completely attenuated by the storage ponds.

Table 3-3. Comparison of Viewlands Cascade Drainage System Performance with Estimates for Preceding Ditch

		Viewlands Cascade			Preceding Ditch					
Period (No. events)	Statistic	Flow Volume Decrease (ft^3)	Average Velocity (ft/sec)	Minimum Residence Time (Minutes)	Flow Volume Decrease ft^3)	Ratio Ditch/ Cascade	Average Velocity (ft/sec)	Ratio Ditch/ Cascade	Minimum Residence Time (Minutes)	Ratio Ditch/ Cascade
10/1/00-3/31/01 Wet (47)	Mean	2664	1.21	4.8	881	0.33	1.50	1.24	2.1	0.44
	Maximum	12326	2.70	41.2	3778	0.31	3.12	1.16	28.9	0.70
4/1/01-9/30/01 Dry (11)	Mean	3548	1.50	3.4	1291	0.36	1.85	1.23	1.3	0.38
	Maximum	17387	2.72	4.6	7047	0.41	3.14	1.15	1.7	0.37
4/1/01-9/30/01 Dry excluding Aug. storm (10)	Mean	2164	1.37	3.5	716	0.33	1.72	1.26	1.3	0.37
	Maximum	4552	2.00	4.6	1618	0.36	2.42	1.21	1.7	0.37
10/1/00-9/30/01 Water Year (58)	Mean	2832	1.26	4.6	959	0.34	1.57	1.25	2.0	0.43
	Maximum	17387	2.72	41.2	7047	0.41	3.14	1.15	28.9	0.70
10/1/01-3/31/02 Wet (58)	Mean	5072	1.35	3.8	1784	0.35	1.61	1.19	1.7	0.45
	Maximum	31901	2.75	6.6	12979	0.41	3.16	1.15	7.4	1.12
10/1/00-3/31/01, 10/1/01-3/31/02 2 wet seasons (105)	Mean	3994	1.28	4.3	1380	0.35	1.56	1.22	1.9	0.44
	Maximum	31901	2.75	41.2	12979	0.41	3.16	1.15	28.9	0.70
10/1/00-4/30/02 Current study period (122)	Mean	3896	1.31	4.2	1350	0.35	1.59	1.21	1.8	0.43
	Maximum	31901	2.75	41.2	12979	0.41	3.16	1.15	28.9	0.70

Table 3-4. 2nd Avenue NW Rainfall and Runoff Event Summary, January 20, 2001-April 30, 2002

Period (No. events) [No. discharging]	Statistic	Antecedent Dry Period (Hours)	Rainfall (Inch)	Duration (Hours)	Average Intensity (Inch/Hour)	Precipitation Volume (ft3)	Flow Volume (ft3)	Flow Volume Decrease (%)
1/20/01-3/31/01	Mean	79.6	0.32	12.5	0.034	925	8	99.6
Partial wet	Std. Dev.	59.6	0.20	10.2	0.025	594	36	1.6
(19)	Maximum	182.3	0.79	37.3	0.118	2301	157	100
[1]	Minimum	13.3	0.04	1.3	0.011	112	0	93.2
4/1/01-9/30/01	Mean	216.8	0.43	17.1	0.026	1244	0	100
Dry	Std. Dev.	255.3	0.45	13.0	0.015	1318	0	0
(18)	Maximum	815.8	1.86	50.5	0.060	5428	0	100
[0]	Minimum	10.0	0.05	5.0	0.003	134	0	100
4/1/01-9/30/01	Mean	195.9	0.34	15.8	0.024	997	0	100
Dry excluding	Std. Dev.	246.8	0.28	12.2	0.014	829	0	0
Aug. storm	Maximum	815.8	1.25	50.5	0.060	3641	0	100
(17)	Minimum	10.0	0.05	5.0	0.003	134	0	100
[0]								
1/20/01-9/30/01	Mean	146.3	0.37	14.7	0.030	1080	4	99.8
Partial	Std. Dev.	193.4	0.35	11.7	0.021	1012	26	1.1
Water Year	Maximum	815.8	1.86	50.5	0.118	5428	157	100
(37)	Minimum	10.0	0.04	1.3	0.003	112	0	93.2
[1]								
10/1/01-3/31/02	Mean	99.7	0.61	21.2	0.027	1777	42	99.2
Wet	Std. Dev.	170.9	0.73	19.1	0.015	2121	106	1.8
(53)	Maximum	815.8	3.05	99.5	0.068	8891	445	100
[9]	Minimum	7.3	0.05	2.3	0.006	134	0	95.0
1/20/01-3/31/01,	Mean	94.4	0.53	18.9	0.029	1552	33	99.3
10/1/01-3/31/02	Std. Dev.	149.6	0.64	17.5	0.018	1878	94	1.8
1 + partial	Maximum	815.8	3.05	99.5	0.118	8891	445	100
wet season	Minimum	7.3	0.04	1.3	0.006	112	0	93.2
(72)								
[10]								
1/20/01-4/30/02	Mean	96.5	0.47	17.6	0.027	1375	25	99.5
current study	Std. Dev.	136.8	0.58	16.1	0.017	1681	82	1.6
period	Maximum	815.8	3.05	99.5	0.118	8891	445	100
(96)	Minimum	7.3	0.04	1.3	0.003	112	0	93.2
[10]								

Three storms with measured discharges were available to make estimates for the seven unmeasured events having larger estimated volumes. Discharge during these three events ranged from 3.1 to 6.8 percent of the rainfall volume, averaging 4.9 percent. The seven missing discharges were accordingly estimated as 5 percent of the respective precipitation volumes. This factor may overestimate some cases and underestimate others, depending on meteorological and soil moisture conditions. The misestimate is probably no more than about 2 percent, with approximate compensation of low and high estimates.

After the SEA Streets project was in place, discharge was measured or estimated for only 10 of the 96 events (10.4 percent). In strong contrast, flow over the weir occurred during all 35 events measured before project construction, even though most were in the drier months. With the new street design there was no dry-season release, even during the large August storm, an event when the shaft encoder was functioning well, allowing direct discharge measurement.
Even with so few events yielding any discharge, attenuation was so close to complete that the mean flow volume decreases shown in Table 3-4 are quite indicative of recharge over the full seasonal and annual periods:

1/20/01-3/31/01 (partial wet season)—	99.1%
4/1/01-9/30/01 (dry season)—	100%
4/1/01-9/30/01 (dry season excluding August storm)—	100%
1/20/01-9/30/01 (partial water year)—	99.6%
10/1/01-3/31/02 (wet season)—	97.6%
1/20/01-3/31/01, 10/1/01-3/31/02 (1 + partial wet season)—	97.8%
1/20/01-4/30/02 (current study period)—	98.2%

Hydrograph Analysis

Figures 3-7 and 3-8 show hyetographs and hydrographs for two somewhat contrasting events at 2nd Avenue NW. The early January 2002 storm (Figure 3-7) followed an antecedent dry period of only 10.3 hours, had average intensity of 0.048 inch/hour (1.2 mm/h), and produced an estimated total precipitation volume of 6635 ft^3 (188 m^3). The later January event (Figure 3-8) had a much longer antecedent dry period of 94.5 hours and about half the average intensity (0.026 inch/hour, 0.7 mm/h), although it had a brief burst of relatively intense rainfall late in the storm. The total volume estimate was 3418 ft^3 (97 m^3). Durations of the two storms were similar (47.8 and 44.8 hours, respectively).

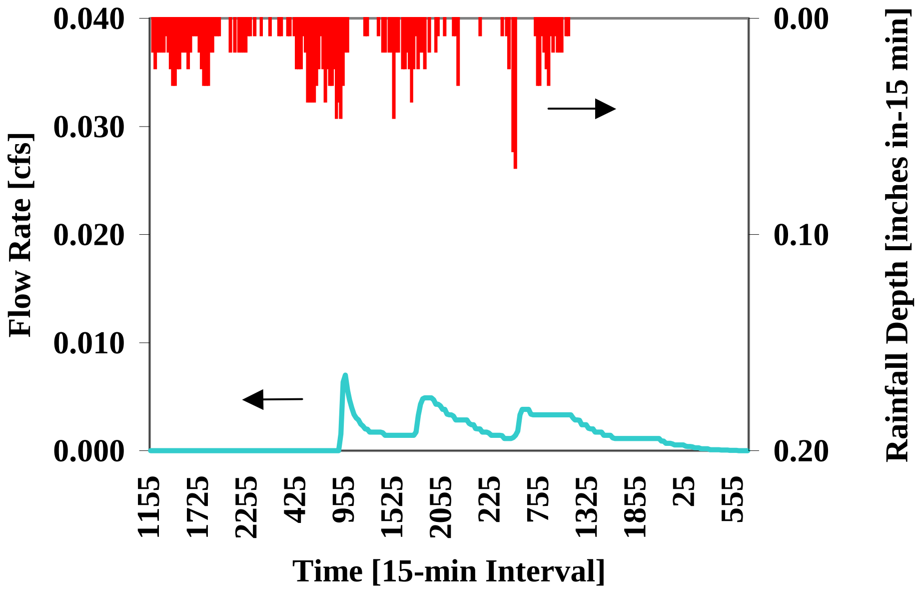


Figure 3-7. 2nd Avenue NW Rainfall Hyetograph and Runoff Hydrograph, January 6 (11:55 AM) - January 9, (7:25 AM), 2002

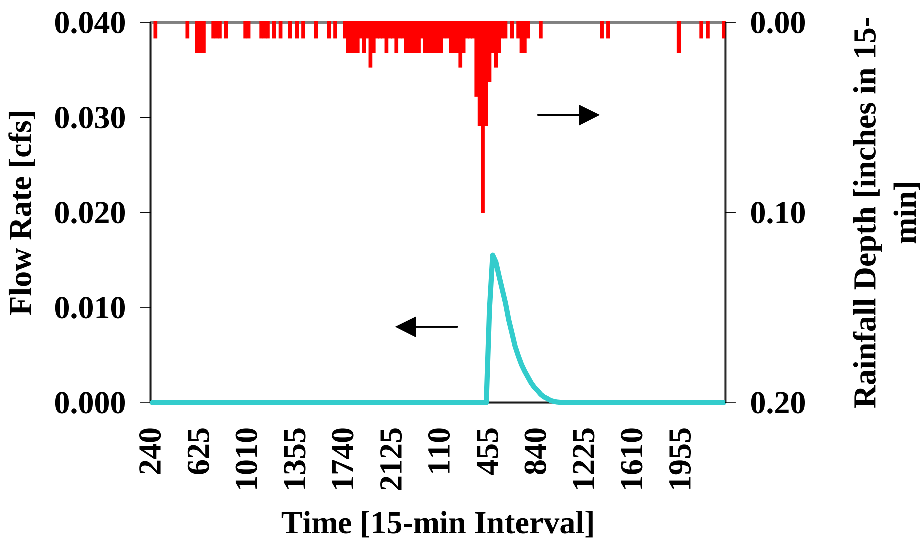


Figure 3-8. 2nd Avenue NW Rainfall Hyetograph and Runoff Hydrograph, January 24 (2:40 AM) - January 25 (10:10 PM), 2002

Discharge occurred much earlier and lasted longer in the first case, with higher soil moisture and overall more intense rain. The extended, light, steady rain through most of the second event produced no discharge, which only occurred suddenly with the precipitation burst. Although the latter storm had less quantity and intensity of rain and lower soil moisture at the beginning, its maximum discharge rate was more than twice the heavier, earlier event (it should be noted, though, that both rates were very low).

Even with the differences in meteorological and soil moisture conditions in the two cases, the SEA Streets runoff mitigation features performed similarly. Runoff quantities were only 4.9 and 3.2 percent of the precipitation volumes falling on the catchment in the respective events. The project has the ability to attenuate all or almost all runoff over a fairly wide range of conditions.

Comparison with Preceding Street and Conventional Street Design

The SEA Streets design thoroughly out-stripped the prediction made during the initial study period that it would reduce total discharge from the pre-existing street for equivalent conditions by only 42 percent (Miller 2001; Miller, Burges, and Horner 2001). Precipitation volume retained by a conventional street design is expected to be about 20 percent as great as with the SEA Streets design, and total discharges from the latter configuration are small percentages of those estimated from a conventional streetscape:

1/20/01-3/31/01 (partial wet season)—	1.1%
4/1/01-9/30/01 (dry season)—	0%
4/1/01-9/30/01 (dry season excluding August storm)—	0%
1/20/01-9/30/01 (partial water year)—	0.5%
10/1/01-3/31/02 (wet season)—	3.0%
1/20/01-3/31/01, 10/1/01-3/31/02 (1 + partial wet season)—	2.7%
1/20/01-4/30/02 (current study period)—	2.3%

3.3. Design Comparison

This section compares the relative amounts of flow volume reduction achieved with the various drainage system designs covered in this report, including: (1) the Viewlands Cascade Drainage System versus the ditch that preceded it, (2) the 2nd Avenue NW SEA 1Streets project versus the original street drainage system, (3) the 2nd Avenue NW SEA Streets project versus a conventional street drainage system design, and (4) the 2nd Avenue NW SEA Streets project versus the Viewlands Cascade Drainage System. The designs are compared as ratios for dry and wet seasons and overall by normalizing in terms of the runoff volume retained per month. In addition, the Viewlands Cascade and SEA Streets projects are compared in relation to: (1) the runoff volume retained per unit area of contributing catchment, and (2) the runoff volume retained per month and per dollar of unit area construction cost. A month is the normalization basis because runoff was sometimes measured or estimated for the various designs during different periods, not always a full season in length. The exercise uses all data available from the beginning of monitoring, both those detailed in this report and in the preceding thesis by Miller (2001) and report by Miller, Burges, and Horner (2001). Runoff not monitored at Viewlands during the 2001 dry season was estimated upstream and downstream from the measurements available for that season in proportion to the rainfall during the periods with and without flow measurements. This procedure implicitly assumes constant runoff coefficients in the contributing catchment and the channel for the full dry season.

Unit catchment area comparisons involving the 2nd Avenue NW SEA Streets project are expressed with respect to both the street right-of-way area (0.91 acre, 0.37 ha) and the total catchment area (2.3 acres, 0.93 ha). The rationale for using the right of way is the observation that it produces almost all of the runoff, with the nearly flat, extensively pervious properties on the east side contributing little. On the other hand, comparisons of the Viewlands and SEA Streets projects should use the full contributing areas of both, since these projects represent the effort and capital expenditure made by the City of Seattle to control runoff from the complete catchments, not just the areas within them that generate flow. Tables 3-5 and 3-6 present the comparisons.

The benefit ratios for Viewlands Cascade/preceding ditch, SEA Streets/original street, and SEA Streets/conventional street in Table 3-5 reiterate the points made in Sections 3.2.1 and 3.2.2: the improved drainage systems retain several times as much runoff volume as their respective predecessors or, in the case of SEA Streets, the alternative of designing according to the City of Seattle's current convention. Runoff retention is principally a wet season benefit, when it acts to reduce channel-disturbing peak flow rates and total discharge volumes in Pipers Creek. For the wetter months the project benefit ratios range from about 3 to almost 5. The SEA Streets/original street comparison should be regarded as only indicative and not conclusive, since the baseline monitoring period was abbreviated and skewed toward the drier months. This monitoring pattern accounts for the benefit ratio being larger overall than in either seasonal period, a mathematical impossibility if there had been equivalent monitoring attention to the two cases being compared.

The SEA Streets/Viewlands Cascade ratios in Table 3-5 show that the 2nd Avenue NW project attenuates more than one-third as much runoff as the new Viewlands channel, even though the SEA Streets project serves less than one-tenth as much contributing catchment area. When placed on an areal basis (Table 3-6, second column), that advantage multiplies greatly. However, calculating according to unit area cost (Table 3-6, third column) puts a different light on the comparison. Costing roughly the same as Viewlands but serving a much smaller catchment, the 2nd Avenue NW project has a fractional cost-benefit compared to Viewlands. These financial comparisons take no account of potential savings that might be realized with experience and economies of scale in future construction of both project types.

Table 3-5. Monthly Benefit Comparisons of Ultra-urban Drainage System Designs

Comparison	Period	Retained Volume/Month[a]
Viewlands Cascade/Preceding ditch	Drier months[b]	8682/2098 = 4.1
	Wetter months[c]	34950/12076 = 2.9
	Overall	24764/7541 = 3.3
SEA Streets/Original street	Drier months[b]	4187/1824 = 2.3
	Wetter months[c]	13094/4532 = 2.9
	Overall	9032/2424 = 3.7
SEA Streets/Conventional street	Drier months[b]	4187/1092 = 3.8
	Wetter months[c]	13094/2776 = 4.7
	Overall	9032/1911 = 4.7
SEA Streets/Viewlands Cascade	Drier months[b]	4187/8682 = 0.48
	Wetter months[c]	13094/34950 = 0.37
	Overall	9032/24764 = 0.36

[a] Expressed as the ratio of SEA Streets/Viewlands Cascade, both in ft^3/month (divide by 35.3 for m^3/month)
[b] April-September
[c] October-March

Table 3-6. Benefit and Cost-Benefit Comparisons of 2nd Avenue NW SEA Streets and Viewlands Cascade Projects

Period	Retained Volume/(Month-Unit Contributing Area)[a]	Retained Volume/(Month-Unit Area Cost)[b]
Drier months[c]	4601/334 = 14, 1820/334 = 5.4[d]	0.016/1.00 = 0.016, 0.040/1.00 = 0.040[d]
Wetter months[e]	14389/1344 = 11, 5693/1344 = 4.2[d]	0.049/4.04 = 0.012, 0.123/4.04 = 0.030[d]
Overall	9925/952 = 10, 3927/952 = 4.1[d]	0.034/2.86 = 0.011, 0.085/2.86 = 0.030[d]

[a] Expressed as the ratio of SEA Streets/Viewlands Cascade, both in ft^3/month-acre (divide by 14.3 for m^3/month-ha)
[b] Expressed as the ratio of SEA Streets/Viewlands Cascade, both in ft^3/(month-$/acre) (multiply by 0.011 for m^3/month-$/ha), using construction costs of $225000 and $244000 for Viewlands and 2nd Avenue NW, respectively
[c] April-September
[d] The first number in the series is based on the right-of-way area (0.91 acre, 0.37 ha); the second is based on the total contributing catchment area (2.3 acres, 0.93 ha).
[e] October-March

With its position at the discharge of its subbasin, the Viewlands Cascade might be termed a “downstream” solution. Managing runoff at or near its source, the 2nd Avenue NW project site is an “upstream” solution. Its relatively greater effectiveness on an areal basis is a demonstration of the common observation in stormwater management that acting closer to the source on smaller quantities of water yields better results than downstream intervention. In this case, the unit cost of the upstream project was much higher because of its nature, not its catchment position. Thus, lower cost effectiveness is not a general drawback of upstream projects.

There is another factor not represented in these numbers that should be considered in interpreting and applying them in project planning. The Viewlands Cascade’s downstream position makes it the last opportunity to attenuate runoff before discharge to Pipers Creek. The 2nd Avenue NW project site is slightly more distant from the stream, both geographically and hydrologically. There would be a subsequent chance for attenuation, for example by channeling drainage from this subbasin and others into a cascade-type channel. The strategy in any situation should be guided by the opportunities and constraints posed by the case, the benefits that can accrue from different options, and the cost of achieving them.

CHAPTER 4 – SUMMARY AND CONCLUSIONS

1. Flow has been monitored at the Viewlands Cascade Drainage System over two full wet seasons, one complete dry season, a portion of a second one. The 2nd Avenue NW SEA Streets project has received flow monitoring for one full wet season and part of a second one, plus a complete dry season; and monitoring continues there. The wet seasons represented have differed in meteorological characteristics. The 2000-2001 winter had only 57 percent of the long-term average rainfall. The following winter was slightly above average in total precipitation but had generally low-intensity storms. Neither winter had any very large storms.

2. The Viewlands channel has had considerable attention to quantify the water losses in the cells where monitoring occurs, which are not recorded as flow, and the performance of the very large weirs, which have certain non-ideal characteristics. Attempts through the summer of 2001 did not produce a suitable quantification of losses, but a different procedure was attempted this summer. Low-flow readings are subject to correction when the data are analyzed. It was determined in 2001 that water losses are a greater monitoring problem at low flows than non-ideal weir conditions.

3. At both Viewlands and 2nd Avenue NW flow attenuation by the drainage projects is especially strongly influenced by rainfall intensity, and also by antecedent dry period length. Flow reduction, primarily by infiltration, is markedly greater in low-intensity storms compared to high-intensity events, a pattern that is accentuated with relatively low soil moisture attending a preceding dry period of at least a number of days.

4. Over all of the monitoring performed, the Viewlands Cascade has quite consistently reduced the influent runoff volume by slightly more than one-third during the wetter months and overall for the year (the majority of relatively small dry-season flows are attenuated). Also, about one-third of events exhibit no discharge from the end of the channel. It can completely infiltrate about 0.13 inch (3.3 mm) of precipitation and 1000 ft^3 ($28m^3$) of influent regardless of the season or conditions.

5. Based on estimates for the ditch that preceded the Viewlands Cascade project, the new channel reduces runoff discharged to Pipers Creek in the wet months by a factor of three relative to the old ditch and cuts flow velocities by approximately 20 percent, both under identical conditions. Reducing velocities and associated erosiveness was a major goal of the project, but the lack of large storms has not provided a good test of its effectiveness in this regard.

6. During the 2001-2002 wet season the new Viewlands channel retained almost 300000 ft^3 (8500 m^3) of runoff that entered it, preventing its direct release to Pipers Creek and the elevation of erosive flows there. This quantity is nearly three times the amount of retention estimated were the preceding narrow, partially concreted ditch still been in place.

7. During monitoring thus far the 2nd Avenue SEA Streets project has prevented the discharge of all dry season flow and 98 percent of the wet season runoff. Whereas all events in the baseline monitoring period, which occurred mostly in the dry season, created a discharge, only about 10 percent have since the project's construction.

8. The SEA Streets design can fully attenuate 2300 ft^3 (65.2 m^3) of runoff, which represents the volume produced by approximately 0.75 inch (19 mm) of rain on its catchment. For context, the mean storm quantity at Seattle-Tacoma International Airport is 0.48 inch (12 mm).

9. Based on estimates for a street drainage system design according to City of Seattle conventions, the SEA Streets alternative reduces runoff discharged to Pipers Creek in the wet months by a factor of 4.7 relative to the conventional street.

10. Despite serving a catchment less than 10 percent as large as the Viewlands Cascade, the 2nd Avenue NW project retains more than one-third as much runoff volume in the wet season as Viewlands. On the basis of unit runoff contributing area, the SEA Streets project is at least four times as effective as Viewlands, depending on how the benefit is computed. However, when normalized in terms of the cost per unit catchment area served, the 2nd Avenue NW reconstruction is much less cost-effective than the Viewlands Cascade.

11. Sufficient data are available now for estimating potential hydrologic benefits of future projects of the Viewlands Cascade and SEA Streets types in similar catchments.

REFERENCES

Miller, A.V. 2001. Hydrologic Monitoring of the Seattle Ultra-urban Stormwater Management Projects. M.S.C.E. thesis, Department of Civil and Environmental Engineering, University of Washington, Seattle, WA.

Miller, A.V., S.J. Burges, and R.R. Horner 2001. Hydrologic Monitoring of the Seattle Ultra-urban Stormwater Management Projects. Water Resources Series Technical Report No. 166, Department of Civil and Environmental Engineering, University of Washington, Seattle, WA.