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HYDROLOGIC MONITORING OF THE SEATTLE ULTRA-URBAN STORMWATER MANAGEMENT PROJECTS: SUMMARY OF THE 2000-2003 WATER YEARS

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Water Resources Series
Technical Report No.181
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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 over three water years beginning on 1 October 2000 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 mean influent peak flow rate by approximately 60 percent and total runoff volume by more than half, although little or no reduction of either peak flow rate or volume occurs in relatively large storms. Based on estimates for the ditch that preceded the Viewlands Cascade project, the new channel reduces runoff discharged directly 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 99 percent of the wet season runoff. This project's performance has advanced since its installation, to the point that it has not discharged since December of 2002, even during large rainfalls in the autumn of 2003. Maturing vegetation is likely assisting both soil infiltration and evapotranspiration. It was estimated that a street drainage system design according to City of Seattle conventions in the same place would have discharged almost 100 times as much runoff to Pipers Creek as the SEA Streets alternative. Despite serving a catchment less than 10 percent as large as the Viewlands Cascade, the 2nd Avenue NW project retains one-quarter to 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.

The two projects represent two contrasting urban stormwater management options. The SEA Streets project is at the drainage source, where rainfall is converted to surface runoff. It is capable of preventing almost all of the conversion. The large Viewlands Cascade is in an end-of-pipe location and can attenuate large amounts of already flowing runoff, although not at nearly the proportional efficiency of the SEA Streets alternative. Together, using the two strategies where the best opportunities for each exist can significantly advance the ability to manage urban runoff peak flow rates and volumes.

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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:

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

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 thought during project design to be approximately 26 acres (10.5 ha) in area. Subsequently, it was established that an additional area of approximately 46 acres (18.6 ha) discharges via the Dayton Avenue swale to the Cascade during relatively large storms and prolonged events when soils are largely saturated. The full contributing catchment has approximately 29 percent impervious land cover. Slopes range from about 0 to 6 percent, with average slopes north to south being 1.8 percent and east to west 5.7 percent.

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 $225,000.

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), paved 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) paved with asphalt and the remainder in vegetation on the edges. Hard surface was reduced slightly to 0.31 acre (0.13 ha) in the redesign, with the remainder given to ponds. The construction cost was initially bid at $244,000. 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 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. Both inflow and outflow were monitored through April 30, 2002, at which time the outflow station was decommissioned. The inflow station remains in place to support upcoming catchment hydrologic model development. Baseline (pre-construction) monitoring was not possible at this site, because construction began 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 two technical reports in this series (Miller, Burges, and Horner 2001; Horner, Lim, and Burges 2002) document all events in the ultra-urban stormwater management studies through April 2002. 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 summarizes all meteorological data collected from the outset of monitoring in January 2000 through the conclusion of the water year on September 30, 2003. It profiles all flow data collected during the three water years commencing on October 1, 2000 and ending on September 30, 2003. This water-year organization was adopted for this and future reports in the series to offer the clearest portrayal of year-to-year trends in relation to stochastic meteorological cycles. This report therefore reiterates some data presented in previous reports, while also omitting a small amount of data from the early flow monitoring. Miller (2001) and Miller, Burges, and Horner (2001) are sources of these data. This report also covers two large rainfall events that occurred shortly after the close of the most recent water year. This coverage is provided because of the high interest in project performance with much greater daily (or longer duration) rainfall than previously experienced.

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 net 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 were a tipping-bucket recording precipitation gauge and a non-recording collector. These gauges were removed in September 2003, after the relationship between precipitation here and at the nearby Viewlands station was well established. 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. Viewlands Leakage Testing and Calibration

1.3.1. Summary of Early Work

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 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.

Horner, Lim, and Burges (2002) reported on additional water loss tests performed during 2001 in the first cell using an adjacent fire hydrant as a water source. Testing was precluded at the downstream end by the danger of discharging chlorinated water to Pipers Creek. The results proved insufficient for conclusively correcting inflows. Companion upstream weir calibration, also using the fire hydrant for source water, indicated that water losses represent a greater monitoring problem than non-ideal weir conditions at low flow rates. Additional tests were scheduled for the summer of 2002 using refined methods.

1.3.2. 2002 Leakage Test Methods

The purpose of the 2002 tests was to quantify leakage over a range of flow rates in a way considered to be reliable enough to correct influent records by adding estimated losses. Leakage is defined as any water loss through the bed and banks of the first channel cell and around and under the weir and its mounting. These losses register as falling head without any discharge over the weir. The 2002 leakage test series consisted of three procedures appropriate for different phases of operation: (1) filling of the first cell up to the weir notch, (2) flow over the weir at relatively low rates, and (3) flow over the weir at relatively high rates.

The initial step in estimating leakage during filling was an engineering survey of the first cell of the Cascade. The survey permitted computation of volumes contained in the cell in relation to water levels. The cell was filled from the fire hydrant to a series of water levels up to the notch invert. After reaching each filling level, the data logger record was used to determine level fall over measured time intervals (15 minutes in this test). Volumes computed from the survey record for the stages at the beginning and end of the interval were then used to estimate the leakage rate at each filling level:

$$\text{Leakage rate} = (V_0 - V_i)/\Delta t$$

where, V_0 = Cell volume associated with stage at the beginning of the time interval;

V_i = Cell volume associated with stage at the end of the time interval; and
Δt = Length of time interval.

The test for flow over the weir at relatively low rates used two wooden flumes constructed to collect water flowing into and out of the first channel cell and direct it to collection containers. Collection chamber volumes were calibrated by weighing full containers and converting to volume, considering the effect of temperature at the time of calibration on density. See Appendix A for photographs of the test procedure in progress.

The low-flow test procedure was:

1. Tightly attach one flume to the inlet section (Appendix A-1) and the other flume to the outlet side of the weir (Appendix A-2) to catch all flow without loss.

2. Extend a fire hose from the hydrant to deliver flow into the inlet pipe, from where it flows to the inlet flume (see Appendix A-1; hose is held in pipe with a rock).

3. Fill up the cell to the notch invert.

4. Fill the container at the discharge of the inlet flume at a steady set flow rate, while measuring by stopwatch the time required to fill the container (Appendix A-1). Calculate the flow rate before leakage as the known volume divided by the time to fill the container. Replicate this procedure several times.

5. Fill the container at the discharge of the outlet flume at a steady set flow rate, while measuring by stopwatch the time required to fill the container (Appendix A-2). Calculate the flow rate after leakage as the known volume divided by the time to fill the container. Replicate this procedure several times.

6. Calculate the leakage rate as the difference in the two flow rates measured in steps 4 and 5.

7. Repeat the tests at both locations at other set flow rates.

Because the maximum hydrant flow rate was only 0.21 cfs (6 L/s), it was necessary to devise another test for leakage at relatively high flow rates. On a rainy day producing a flow rate well above the hydrant capacity, the water level in the first cell was measured at a time when it was holding steady (i.e., inflow was invariable). Plywood was then attached to cover the entire V-notch, so that weir overflow could not occur. If the level appeared to stay steady, it was measured and the plywood was removed. If the level still appeared to stay steady, it was measured and compared to the initial reading (before installation of the plywood). If the two readings agreed, the existing flow rate was the leakage rate at the steady stage. If the level did not remain steady, and the readings diverged, the test was repeated until a steady period occurred.

1.3.3. 2002 Leakage Test Results and Their Use

Figure 1-1 presents the results of all three test series and the flow rates prevailing during tests both corrected and uncorrected for leakage. The leakage rate tends asymptotically toward zero at a stage of around 1.2 ft (37 cm). Therefore, the leakage rate was taken as zero at this stage and below. While the leakage rate is low at small stages, it is a substantial proportion of the measured flow rate; i.e., the error is relatively large if leakage is ignored. The leakage rate grows as a function of the static head of higher stages, but becomes increasingly less instrumental at the high flow rates associated with these stages. The leakage rate was less than 5 percent of the flow rate in the high flow test and tends asymptotically toward 0.2 cfs (5.7 L/s) in that region. The flow during this test was near the maximum yet measured in the Viewlands Cascade.

Figure 1-2 shows the leakage rate in expanded scale. Between a stage of 1.2 ft (37 cm) and 1.85 ft (56 cm, the height of the V-notch invert), the leakage rate follows a nearly linear relationship with stage. A second-order polynomial regression equation gave a better fit than simple linear regression to compute a leakage correction in this range. At higher stages, up to the maximum tested, the relationship deviates considerably from linear and is better expressed as another function. Table 1-1 summarizes the leakage corrections used in each stage range.

The appropriate correction was applied to each stage reading in the entire record assembled since the beginning of monitoring. Storm peak flow rates, discharge volumes and other variables related to them were then calculated both with and without leakage correction and carried through the analysis presented in this report.

It was only possible to perform tests to quantify leakage at the inlet end of the Viewlands Cascade and not at its outlet because of concern with discharging chlorinated water to Pipers Creek. Correcting the entering but not the exiting flow overestimates the system's performance in reducing discharge to the creek, because the quantity of water remaining at the end of the channel was actually greater than measured. On the other hand not using the correction on the influent underestimates performance overall. The underestimation occurs because the flow at the downstream end was sometimes zero or relatively low, while the uncorrected inflow was actually higher than used in the comparison. Therefore, performing the analysis with and without inflow leakage correction gives a range in performance estimation.

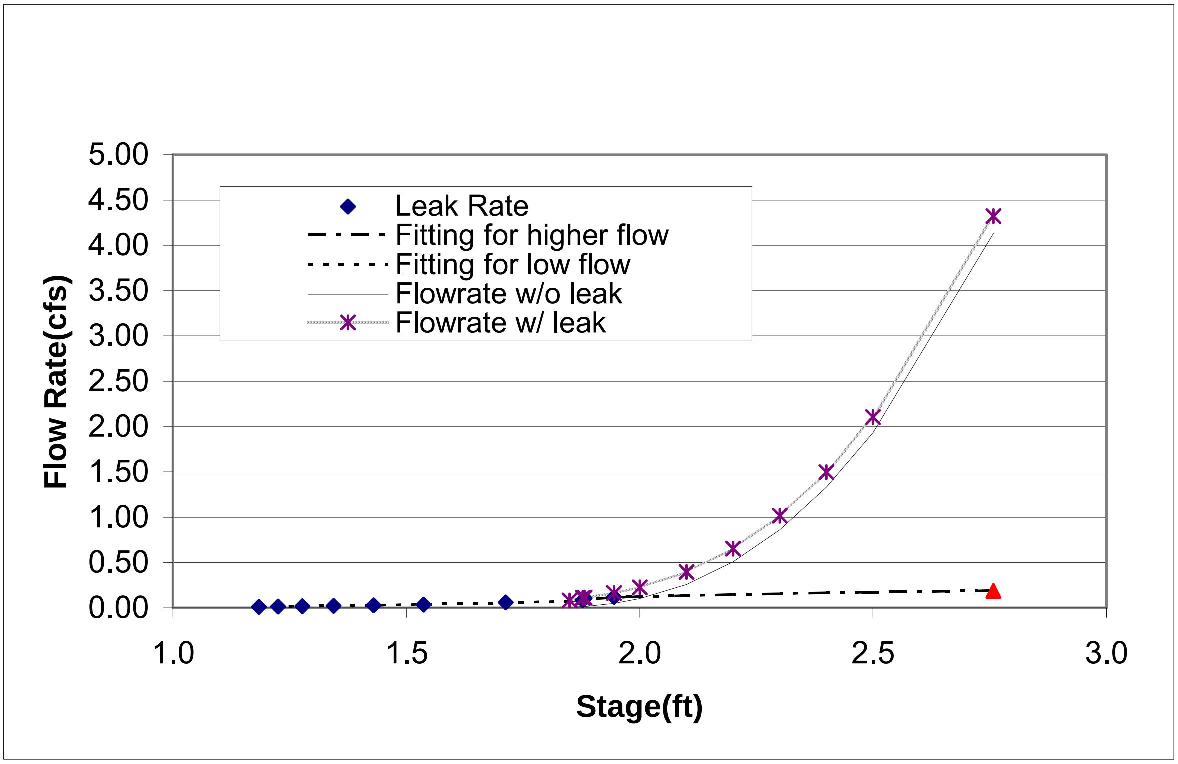

Figure 1-1. Viewlands Cell 1 Leakage Rates and Test Flow Rates Versus Stage

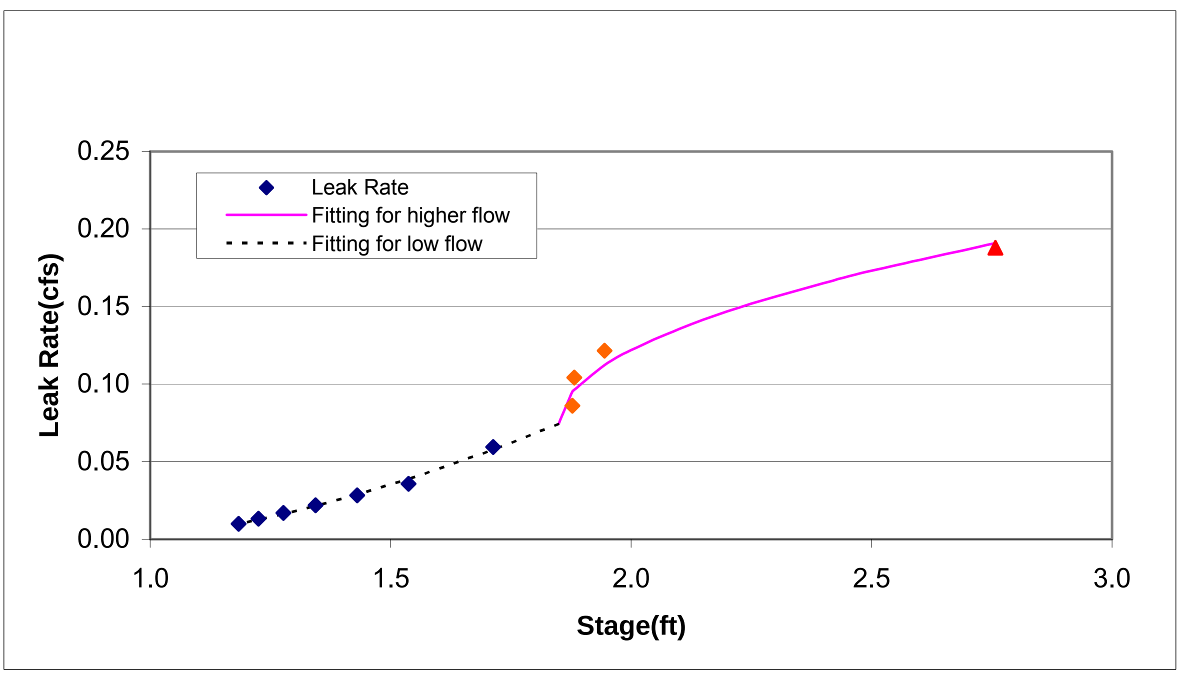

Figure 1-2. Viewlands Cell 1 Leakage Rates Versus Stage

Table 1-1. Leakage Rate Corrections Applied at Different Viewlands Cell 1 Stages

Stage	Leakage Correction (cfs)[a]
≤ 1.2 ft (37 cm)	None
1.2 ft (37 cm) – 1.85 ft (56 cm)	$0.048\ h^2 - 0.049$
> 1.85 ft (56 cm)	$0.12\ (h^2 - 1.85)^{0.5} + 0.075$

[a] h = stage (ft); correction is added to measured flow rate.

CHAPTER 2 – SUMMARY OF METEOROLOGICAL MEASUREMENTS

2.1. Precipitation Summary

Table 2-1 presents monthly and yearly precipitation totals from the onset of monitoring in January 2000 through September 2003 at the project stations and Seattle-Tacoma International Airport (SeaTac), as well as the antecedent 1999 year at the airport. Comparing calendar-year precipitation, 2001 was close to the long-term mean, while 1999 was above average and 2000 and 2002 were 25 and 16 percent below, respectively.

Comparing wet season (October-March) precipitation based on the airport station, totals were:

> 53-year mean—28.9 inches (734 mm);
> 1999-2000—17.4 inches (442 mm), 60 percent of 53-year mean;
> 2000-2001—16.3 inches (414 mm), 56 percent of 53-year mean;
> 2001-2002—31.3 inches (794 mm), 108 percent of 53-year mean; and
> 2002-2003—26.7 inches (678 mm), 92 percent of 53-year mean.

Initial monitoring occurred during relatively dry winters. The two most recent winters overall approximate typical conditions in the region, and thus provide 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 (1.2 inch, 30 mm) and one that exceeded 24 hours duration and the precipitation total associated with the 1-year, 24-hour event (1.9 inch, 48 mm). 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. The October 2002 to March 2003 wet season exhibited one event with rainfall total slightly over the 6-month, 24-hour amount, two approximating the 1-year, 24-hour quantity, and one event measuring somewhat more, at 2.29 inches (58 mm) of precipitation. All of these events considerably exceeded 24 hours duration, however.

While the maximum rainfall in any event during the water years summarized in this report was 2.95 inches (75 mm), two much larger episodes occurred just after the beginning of the subsequent water year. The Viewlands station recorded 4.22 inches (107 mm) of rain from late on October 19, 2003 to the morning of October 21 (a period of 32.5 hours). The SeaTac Airport gauge registered its highest ever 24-hour rainfall total during this event. A quantity of 3.86 inches (98 mm) fell at Viewlands over a 51.25-hour period from November 17 to 19, 2003. Average rainfall intensities during these two storms were 0.13 and 0.075 inches/hour (3.3 and 1.9 mm/h), respectively.

Table 2-1. Precipitation Summary for Full Monitoring Period Through September 30, 2003

Location	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
						Millimeters							
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]	153	105	72	66	33	27	18	1	7	12	63	139	**697**
Viewlands 2003[a]	174	40	135	67	35	13	0	7	26				
2nd Ave. NW 2000[b]			77	33	54	27	11						
2nd Ave. NW 2001	86	52	66	54	28	81	22	53	10	87	231	141	**911**
2nd Ave. NW 2002	156	101	78	64	40	26	18	1	9	15	71	160	**738**
2nd Ave. NW 2003[c]	183	40	137	66	30	11	0	8					
SeaTac 1999	174	177	93	35	54	47	30	24	4	57	244	129	**1067**
SeaTac 2000	96	133	72	39	83	40	6	8	26	74	83	64	**723**
SeaTac 2001	69	53	69	80	35	77	26	59	21	79	235	150	**954**
SeaTac 2002	165	106	72	109	28	44	16	1	11	17	94	152	**815**
SeaTac 2003	202	46	165	70	29	13	2	8	23				
SeaTac 53-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[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]	6.0	4.1	2.8	2.6	1.3	1.1	0.7	0.1	0.3	0.5	2.5	5.5	**27.4**
Viewlands 2003[a]	6.9	1.6	5.3	2.6	1.4	0.5	0.0	0.3	1.0				
2nd Ave. NW 2000[b]			3.0	1.3	2.1	1.1	0.4						
2nd Ave. NW 2001	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	6.1	4.0	3.1	2.5	1.6	1.0	0.7	0.0	0.3	0.6	2.8	6.3	**29.1**
2nd Ave. NW 2003[c]	7.2	1.6	5.4	2.6	1.2	0.4	0.0	0.3					
SeaTac 1999	6.8	7.0	3.7	1.4	2.1	1.9	1.2	0.9	0.2	2.3	9.6	5.1	**42.0**
SeaTac 2000	3.8	5.3	2.8	1.5	3.3	1.6	0.2	0.3	1.0	2.9	3.3	2.5	**28.5**
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	**37.5**
SeaTac 2002	6.5	4.2	2.8	4.3	1.1	1.7	0.6	0.0	0.4	0.7	3.7	6.0	**31.4**
SeaTac 2003	8.0	1.8	6.5	2.7	1.2	0.5	0.1	0.3	0.9				
SeaTac 53-yr mean	5.7	4.1	3.8	2.6	1.7	1.5	0.8	1.1	1.7	3.4	6.0	5.9	**38.1**

[a] All Viewlands readings are from the trench recording gauge, except for February 2001, when the standing recording gauge reading was used because of snow melt that produced an inaccurate trench gauge total.
[b] Monitoring performed only from March through July.
[c] Gauges removed in September 2003.

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

SeaTac averaged 0.2 inch (5.0 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 the same as 2nd Avenue NW, located 13 city blocks away, with a maximum difference in any month of 0.5 inch (14 mm) more.

The trench gauge should collect more precipitation than the standing device, because of lesser wind effects at the lower elevation. These expectations held overall. Although the trench gauge collection exceeded the quantity in the standing gauge during only 19 of the 36 months between October 1, 2000 and September 30, 2003, trench gauge deficits relative to the standing gauge were generally small. Excesses, however, were frequently substantially larger. The trench gauge collection averaged 0.2 inch (5.0 mm) per month higher in the three-year period and totaled 16.3 percent more than the standing gauge contents during this time. 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.

2.2. Evaporation Summary

Table 2-2 gives monthly evaporation totals from the beginning of monitoring in July 2000 through September 2003. Directly measured pan evaporation averaged 0.30 inch (8 mm) higher than potential evaporation, calculated using the Penman-Monteith method (Mansell 2004) and data from instruments at the Viewlands station, in the 34 months for which data comparison was possible. However, deviation was substantial in some months, with a maximum of 2.10 inches (53 mm) in July 2003. The pan measurement exceeded the potential estimate in 20 of the 34 months. In the single year thus far having sufficient measurements of all quantities to make a comparison (2002), the total annual evaporation by pan measurement, 29.84 inches (758 mm) was 5.31 inches (135 mm) higher than the potential. Shallow lake evaporation, a surrogate for potential evaporation, is approximately 70 percent of pan evaporation. On that basis, the pan calculated potential evaporation is 0.7 x 29.84 = 20.9 inches (531 mm).

Table 2-3 presents monthly pan and potential evaporation averaged over the available months in the record, along with reported averages from Washington State University's Puyallup evaporation station. Averaging by month reveals the clear tendency of pan readings to exceed potential evaporation estimates in the warmer, drier months starting in April, with deviations of about 1.25 inch (32 mm) in each of the three summer months. On the other hand, potential evaporation averaged higher in the months November to March, but by much smaller margins. Annually, about 4 inches (102 mm) more pan evaporation was measured than would be expected through the potential estimate.

Table 2-2. Monthly Evaporation at the Viewlands Meteorological Station

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Millimeters[a]													
Pan evaporation 2000	NA	NA	NA	NA	NA	NA	126	107	68	32	8	1	
Pan evaporation 2001	5	10	40	77	97	NA	NA	NA	64	33	13	2	
Pan evaporation 2002	6	19	22	60	85	132	146	146	86	34	8	12	**758**
Pan evaporation 2003	5	5	28	61	107	142	183	140	77				
Potential evaporation 2000[b]	NA	NA	NA	NA	NA	NA	NA	NA	61	33	12	9	
Potential evaporation 2001[b]	11	23	38	62	91	94	116	100	60	31	14	11	**651**
Potential evaporation 2002[b]	9	22	32	55	74	100	110	100	65	30	15	10	**623**
Potential evaporation 2003[b]	12	19	37	57	87	116	130	102	66				
Puyallup pan evaporation[c]	NM	18	40	62	101	118	142	126	74	33	15	NM	**730**
Inches[a]													
Pan evaporation 2000	NA	NA	NA	NA	NA	NA	4.96	4.21	2.67	1.28	0.33	0.05	
Pan evaporation 2001	0.19	0.41	1.58	3.04	3.83	NA	NA	NA	2.53	1.28	0.52	0.09	
Pan evaporation 2002	0.25	0.77	0.87	2.38	3.33	5.21	5.74	5.77	3.39	1.33	0.33	0.48	**29.84**
Pan evaporation 2003	0.18	0.19	1.09	2.40	4.20	5.58	7.20	5.52	3.02				
Potential evaporation 2000[b]	NA	NA	NA	NA	NA	NA	NA	NA	2.42	1.28	0.48	0.35	
Potential evaporation 2001[b]	0.45	0.92	1.50	2.44	3.59	3.69	4.56	3.92	2.38	1.21	0.53	0.45	**25.64**
Potential evaporation 2002[b]	0.34	0.87	1.27	2.16	2.93	3.93	4.34	3.95	2.54	1.20	0.59	0.41	**24.53**
Potential evaporation 2003[b]	0.46	0.73	1.47	2.24	3.42	4.56	5.10	4.01	2.58				
Puyallup pan evaporation[c]	NM	0.71	1.58	2.46	3.97	4.63	5.61	4.97	2.92	1.28	0.61	NM	**28.74**

[a] NA indicates data are not available.

[b] Potential evaporation was calculated by the Penman-Monteith method (Mansell 2004) and readings from other Viewlands instruments.

[c] Puyallup pan evaporation quantities are averages from 1931 through 1995. NM indicates no measurements in the month.

Table 2-3. Average Monthly Evaporation at the Viewlands and Puyallup Stations

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Millimeters													
Pan evaporation	5	12	30	66	96	137	152	131	74	33	10	5	**751**
Potential evaporation[a]	11	21	36	58	84	103	119	101	63	31	13	10	**650**
Puyallup pan evaporation[b]	NM	18	40	62	101	118	142	126	74	33	15	NM	**730**
Inches													
Pan evaporation	0.21	0.46	1.18	2.60	3.79	5.40	5.97	5.17	2.90	1.30	0.39	0.21	**29.58**
Potential evaporation[a]	0.42	0.84	1.41	2.28	3.31	4.06	4.67	3.96	2.48	1.23	0.53	0.40	**25.59**
Puyallup pan evaporation[b]	NM	0.71	1.58	2.46	3.97	4.63	5.61	4.97	2.92	1.28	0.61	NM	**28.74**

[a] Potential evaporation was calculated by the Penman-Monteith method (Mansell 2004) and readings from Viewlands instruments.

[b] Puyallup pan evaporation quantities are averages from 1931 through 1995. NM indicates no measurements in the month.

The Viewlands pan figures exceeded the long-term Puyallup averages in half of the months (Puyallup does not measure in December and January). The greatest differences were in the three summer months, when Viewlands evaporation exhibited higher average evaporation in each month. The few years of record at Viewlands are probably not typical of the many years in the Puyallup database, with the 2003 summer being exceptionally clear, warm, and dry. The Puyallup average annual evaporation is 0.84 inch (21 mm) less than the Viewlands average, half of which is accounted for by the practice of not measuring at Puyallup in December and January.

2.3. Summary of Other Meteorological Data

Tables 2-4 to 2-7 provide statistics on temperature, relative humidity, wind speed, and net radiation, respectively, for the full record available at Viewlands. The tables give credence to the observation above about the effect of the 2003 summer on evaporation, showing higher temperatures, lower relative humidities, and generally higher net radiation than in earlier years.

Table 2-8 presents monthly averages over the available years of record for each meteorological variable. Temperature, relative humidity, and net radiation exhibit the expected seasonal variations. Average wind speed varies relatively little month to month, with September and October being the calmest months and March and April the windiest.

Table 2-4. Monthly and Annual Temperature (^{0}C) Statistics

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Annual**
2000[a]--Mean	NA	NA	NA	NA	NA	NA	NA	16.0	14.6	10.8	5.8	4.9	
Maximum	NA	NA	NA	NA	NA	NA	NA	27.7	27.0	25.5	14.8	13.3	
Minimum	NA	NA	NA	NA	NA	NA	NA	8.9	4.5	3.9	-2.3	-3.0	
Standard deviation	NA	NA	NA	NA	NA	NA	NA	3.5	3.4	2.9	3.1	2.7	
2001--Mean	5.7	4.6	7.3	8.7	11.9	13.3	15.6	16.9	14.2	9.9	8.2	5.0	10.1
Maximum	13.7	11.6	19.5	21.8	25.8	23.3	24.7	29.7	22.7	20.2	16.2	12.1	29.7
Minimum	-1.4	-3.7	0.0	1.8	3.5	6.1	9.0	10.0	6.2	1.7	1.5	-1.4	-3.7
Standard deviation	2.7	2.7	2.6	3.7	4.0	3.3	3.4	3.5	3.0	2.8	2.8	2.3	5.1
2002--Mean	5.3	5.7	5.1	8.9	10.8	15.3	16.7	16.8	14.6	9.8	8.2	6.3	10.3
Maximum	14.1	14.4	13.6	18.3	25.4	29.9	29.4	29.8	25.1	17.7	16.4	12.6	29.9
Minimum	-1.5	-1.8	-2.4	1.7	1.2	7.0	8.6	9.7	6.2	-1.6	-2.2	-1.0	-2.4
Standard deviation	3.1	2.7	3.2	2.9	3.8	4.0	3.8	4.0	3.4	3.2	3.3	2.3	5.5
2003--Mean	7.5	5.3	8.1	9.1	11.8	15.9	18.2	17.3	15.5				
Maximum	14.0	12.0	18.7	17.3	22.0	29.3	31.5	27.2	28.0				
Minimum	0.8	-2.5	0.9	1.8	4.2	7.7	8.9	8.1	7.5				
Standard deviation	2.6	2.7	3.0	3.0	3.8	4.4	4.3	3.6	3.7				

[a] NA indicates data are not available.

Table 2-5. Monthly and Annual Relative Humidity (%) Statistics

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Annual**
2000[a]--Mean	NA	NA	NA	NA	NA	NA	NA	74.7	76.8	82.4	81.1	83.4	
Standard deviation	NA	NA	NA	NA	NA	NA	NA	15.2	16.9	14.5	13.0	12.2	
2001--Mean	83.5	79.5	79.3	74.8	70.4	74.0	73.8	74.7	79.5	83.0	84.9	82.7	**78.3**
Standard deviation	13.9	14.0	11.7	15.4	15.2	13.2	14.5	16.0	13.4	13.6	9.4	11.6	**14.3**
2002--Mean	86.5	77.4	77.9	75.3	74.2	71.0	71.3	70.1	74.0	81.4	83.7	83.5	**77.2**
Standard deviation	7.6	14.9	13.9	14.7	15.1	14.8	15.2	16.6	13.6	15.9	13.0	12.8	**15.1**
2003--Mean	85.0	82.0	79.4	74.8	72.0	65.2	63.9	67.2	72.6				
Standard deviation	12.3	14.8	12.4	14.9	14.6	17.2	16.3	16.4	15.9				

[a] NA indicates data are not available.

Table 2-6. Monthly and Annual Wind Speed [at 2.7 m Height] (m/s) Statistics

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Annual**
2000[a]--Mean	NA	1.27	1.36	1.44	1.39	1.35	1.36	1.20	1.15	1.11	1.06	1.17	
Maximum	NA	3.89	4.25	3.97	3.62	3.56	3.55	3.26	3.37	3.38	3.49	4.26	
Standard deviation	NA	0.66	0.73	0.74	0.62	0.65	0.67	0.64	0.69	0.61	0.55	0.64	
2001--Mean	1.14	1.32	1.50	1.50	1.30	1.26	1.21	1.23	1.06	1.27	1.31	1.44	**1.30**
Maximum	4.25	5.00	4.06	4.21	3.58	3.13	3.52	3.56	3.00	4.89	4.60	4.91	**5.00**
Standard deviation	0.64	0.79	0.72	0.74	0.68	0.60	0.63	0.67	0.66	0.74	0.69	0.89	**0.72**
2002--Mean	1.41	1.45	1.65	1.54	1.34	1.33	1.27	1.13	1.13	0.92	1.13	1.36	**1.30**
Maximum	4.02	4.17	4.75	4.09	3.30	3.93	4.02	3.54	3.51	3.16	3.97	5.70	**5.70**
Standard deviation	0.79	0.79	0.90	0.82	0.68	0.70	0.71	0.73	0.70	0.58	0.74	0.90	**0.78**
2003[a]--Mean	1.18	1.15	1.60	1.47	1.29	1.47	NA	NA	NA				
Maximum	4.13	3.74	4.21	3.84	3.85	3.34	NA	NA	NA				
Standard deviation	0.81	0.64	0.79	0.70	0.69	0.67	NA	NA	NA				

[a] NA indicates data are not available; data are again available from October 2003 forward.

Table 2-7. Monthly and Annual Net Radiation (W/m^2) Statistics

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Annual**
2000[b]--Mean	NA	NA	NA	NA	NA	NA	NA	NA	80.8	42.5	10.1	4.2	
Maximum	NA	NA	NA	NA	NA	NA	NA	NA	645.5	489.1	339.8	325.1	
Standard deviation	NA	NA	NA	NA	NA	NA	NA	NA	161.0	108.6	55.4	40.1	
2001--Mean	8.8	36.9	57.6	96.3	129.3	138.7	158.5	127.9	84.3	39.3	12.3	6.4	**74.9**
Maximum	255.9	456.2	652.8	655.8	670.8	751.0	706.0	666.1	632.3	448.4	311	233.6	**751.0**
Standard deviation	44.6	104.9	126.9	172.0	198.3	204.5	221.9	195.6	155.0	99.9	48.2	36.7	**157.9**
2002--Mean	4.9	30.6	46.7	85.7	108.2	136.9	144.3	125.5	84.5	41.9	15.8	3.0	**69.2**
Maximum	205.3	474.4	566.5	619.2	688.0	716.0	745.0	605.3	525.8	443.1	349	168.8	**745.0**
Standard deviation	28.4	100.2	116.8	169.1	186.1	209.4	218.9	199.6	165.5	106.7	64.7	34.9	**156.5**
2003--Mean	9.1	29.4	51.3	87.2	126.4	150.4	155.6	122.4	84.3				
Maximum	252.3	389.5	602.2	669.8	726.0	691.7	684.7	619.1	606.9				
Standard deviation	46.7	87.8	117.0	160.4	193.4	214.5	223.1	201.8	165.3				

[a] The maximum value per month is based on the largest 15-minute recorded value. The means and standard deviations are calculated from the 15-minute data.

[b] NA indicates data are not available.

Table 2-8. Mean Monthly and Annual Temperature, Relative Humidity, Wind Speed, and Net Radiation at the Viewlands Meteorological Station

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	**Annual**
Temperature (^{0}C)	6.2	5.2	6.8	8.9	16.5	14.8	16.8	16.8	14.7	10.2	7.4	5.4	**10.8**
Relative humidity (%)	85.0	79.6	78.9	75.0	72.2	70.1	69.7	71.7	75.7	82.3	83.2	83.2	**77.2**
Wind speed (m/s)	1.24	1.30	1.53	1.49	1.33	1.35	1.28	1.19	1.11	1.10	1.17	1.32	**1.28**
Net radiation (W/m^2)	7.6	32.3	51.9	89.7	121.3	142.0	152.8	125.3	83.5	41.2	12.7	4.5	**72.1**

CHAPTER 3 – RAINFALL-RUNOFF ANALYSIS

3.1. Viewlands Cascade Drainage System

3.1.1. Rainfall and Runoff Event Summary

Table 3-1 summarizes Viewlands drainage system rainfall and runoff statistics for 210 events for the three water years beginning on 1 October 2000 and concluding on 30 September 2003. Downstream monitoring stopped at the end of April 2002. Flow rate and volume statistics for the upstream station, and in comparison with the downstream monitoring point, are given both with and without the correction for leakage at the upstream weir. It was not possible to determine a correction at the downstream station, although leakage was evident there too. The correction has a proportionately greater effect on relatively small compared to larger values. Over the three years the mean corrected upstream flow rate estimate was 21 percent larger than the uncorrected measure, whereas the maximum corrected rate was only 5 percent larger.

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 (1.4 to 35.4 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.

The rainfall statistics demonstrate the distinctions between the wet and dry seasons (e.g., a mean antecedent dry period three times as long in the dry compared to the wet season). They also indicate the different characteristics of the three wet seasons represented. As discussed earlier, the 2001-2002 and 2002-2003 winters were wetter overall than the first two years of the program. However, their mean precipitation intensities were less, and rain was spread over a longer average storm duration.

The Viewlands Cascade did not experience a large rain event of infrequent occurrence until October and again in November 2003, just after the start of the present water year. The 19-21 October 2003 storm produced the highest maximum inflow rate in the record thus far, 4.51 cfs (128 L/s). The inflow rate during the 17-19 November 2003 event was 3.86 cfs (109 L/s). Interestingly, much smaller rainfall totals in earlier years produced comparable peak rates, 4.26 cfs (121 L/s) in December 2001 and 3.97 cfs (112 L/s) in August 2001. The explanation is probably that the Fall 2003 storms followed an especially dry summer and early autumn. Higher peaks could be expected should such precipitation quantities fall on an already relatively saturated catchment. However, measurements thus far give no indication that, without an exceptional event of extreme infrequency, the inflow rate would ever approach the 25 cfs (708 L/s) estimated for the 25-year, 24-hour rainfall event and used to design the project.

Table 3-1. Viewlands Rainfall and Runoff Event Summary, 1 October 2000-30 September 2003

Period (No. events)	**Statistic**	Antecedent Dry Period (Hours)	Rainfall (Inch)	Duration (Hours)	Average Intensity (Inch/Hour)	Storm Response Time (Hours)	Maximum Upstream Flow Rate[a] (cfs)	Maximum Downstr. Flow Rate[b] (cfs)	Flow Rate Decrease[a,b] (%)	Upstream Flow Volume[a] (ft3)	Downstr. Flow Volume[b] (ft3)	Flow Volume Decrease[a,b] (%)	Average Velocity[a,b] (ft/sec)	Minimum Residence Time[a,b] (Minutes)
10/1/00-3/31/01	Mean	78.2	0.40	13.9	0.033	3.0	0.76/0.62	0.42	62.8/52.5	11078/5654	2990	84.7/70.2	1.3	3.7
Wet	Std. Dev.	68.5	0.45	11.9	0.019	3.0	0.68/0.66	0.65	31.1/37.8	11015/7460	5570	16.8/27.3	0.4	1.2
(47)	Maximum	336.3	2.76	61.5	0.094	14.3	4.07/3.88	3.80	100/100	48304/35457	26941	100/100	2.7	7.9
	Minimum	5.8	0.04	1.0	0.009	0.5	0.08/0.00	0.00	6.6/0	1021/0	0	44.2/16.2	0.6	1.6
4/1/01-9/30/01	Mean	223.6	0.48	11.8	0.048	3.2	1.24/1.08	0.74	64.9/59.9	11060/6689	3141	85.1/76.5	1.6	3.0
Dry	Std. Dev.	240.5	0.56	9.5	0.040	2.1	1.13/1.11	1.17	36.9/39.5	16835/11657	6973	16.5/23.9	0.5	0.8
(11)	Maximum	826.0	2.15	37.0	0.138	6.3	4.16/3.97	3.66	100/100	60999/41086	23699	100/100	2.8	3.9
	Minimum	8.3	0.15	3.0	0.019	0.8	0.43/0.28	0.00	-6.0/-16.4	2195/1054	0	59.2/38.2	1.1	1.6
10/1/00-9/30/01	Mean	105.8	0.42	13.5	0.035	3.0	0.86/0.71	0.48	63.2/53.9	11075/5850	3018	84.8/71.4	1.4	3.5
Water Year	Std. Dev.	131.3	0.47	11.4	0.025	2.8	0.79/0.78	0.77	31.9/37.9	12151/8302	5794	16.6/26.6	0.4	1.1
(58)	Maximum	826.0	2.76	61.5	0.138	14.3	4.16/3.97	3.80	100/100	60999/41086	26941	100/100	2.8	7.9
	Minimum	5.8	0.04	1.0	0.009	0.5	0.08/0.00	0.00	-6.02/-16.4	1021/0	0	44.2/16.2	0.6	1.6
10/1/01-3/31/02	Mean	59.6	0.48	18.1	0.024	2.4	0.88/0.73	0.53	62.1/54.5	23628/15140	10068	82.0/68.9	1.4	3.7
Wet	Std. Dev.	69.8	0.66	18.0	0.014	1.7	0.82/0.81	0.71	36.7/43.2	36260/27064	20567	19.9/29.5	0.5	1.3
(58)	Maximum	341.5	2.95	97.8	0.063	8.5	4.26/4.06	3.11	100/100	142035/108691	82862	100/100	2.8	6.8
	Minimum	5.5	0.01	2.0	0.001	0.5	0.11/0.01	0.00	-71.7/-121.9	991/14	0	39.1/17.2	0.7	1.6
4/1/02-9/30/02	Mean	220.5	0.27	9.9	0.031	2.4	0.53/0.41			5593/2618			1.1	5.4
Dry	Std. Dev.	436.3	0.22	7.0	0.016	1.9	0.49/0.45			6891/4452			0.5	3.3
(19)	Maximum	1986.8	0.90	27.8	0.071	8.0	1.87/1.70			27145/18519			2.0	14.1
	Minimum	10.3	0.05	0.8	0.013	1.0	0.02/0.00			199/0			0.3	2.2
10/1/01-9/30/02	Mean	99.3	0.43	16.1	0.026	2.4	0.79/0.65			19178/12050			1.3	4.1
Water Year	Std. Dev.	231.5	0.59	16.3	0.014	1.8	0.77/0.74			32536/24157			0.5	2.1
(77)	Maximum	1986.8	2.95	97.8	0.071	8.5	4.26/4.06			142035/108691			2.8	14.1
	Minimum	5.5	0.01	0.8	0.001	0.5	0.02/0.00			199/0			0.3	1.6

10/1/02-3/31/03	Mean	68.2	0.38	14.5	0.027	3.0	0.81/0.68			13298/8228				1.3	4.6
Wet	Std. Dev.	118.8	0.48	13.8	0.018	2.4	0.91/0.88			21836/5900				0.6	2.8
(53)	Maximum	770.0	2.29	66.8	0.086	12.0	3.66/3.47			99807/71125				2.6	15.4
	Minimum	5.5	0.01	0.5	0.001	0.5	0.01/0.00			73/0				0.3	1.6
						Storm	Maximum	Maximum	Flow	Upstream	Downstr.	Flow			Minimum
		Antecedent			Average	Response	Upstream	Downstr.	Rate	Flow	Flow	Volume	Average		Residence
Period		Dry Period	Rainfall	Duration	Intensity	Time	Flow Rate[a]	Flow Rate[b]	Decrease[a,b]	Volume[a]	Volume[b]	Decrease[a,b]	Velocity[a,b]		Time[a,b]
(No. events)	**Statistic**	(Hours)	(Inch)	(Hours)	(Inch/Hour)	(Hours)	(cfs)	(cfs)	(%)	(ft3)	(ft3)	(%)	(ft/sec)		(Minutes)
4/1/03-9/30/03	Mean	178.8	0.22	9.4	0.039	6.2	0.29/0.19			2321/732				0.9	6.7
Dry	Std. Dev.	282.8	0.18	8.7	0.040	6.8	0.26/0.22			2311/927				0.4	3.5
(22)	Maximum	1141.3	0.67	31.3	0.161	24.8	0.80/0.65			7732/2602				1.5	14.7
	Minimum	6.8	0.04	0.8	0.010	0.5	0.02/0.00			96/0				0.3	3.1
10/1/02-9/30/03	Mean	100.6	0.33	13.0	0.030	3.8	0.66/0.53			10078/6029				1.2	5.2
Water Year	Std. Dev.	187.6	0.42	12.7	0.027	4.1	0.81/0.78			19024/13773				0.6	3.2
(75)	Maximum	1141.3	2.29	66.8	0.161	24.8	3.66/3.47			99807/71125				2.6	15.4
	Minimum	6.5	0.03	0.5	0.006	0.5	0.01/0.00			73/0				0.3	1.7
Oct.-Mar.	Mean	68.0	0.42	15.6	0.028	2.7	0.82/0.68			16430/10000				1.3	4.0
2000-2003	Std. Dev.	88.7	0.54	15.0	0.017	2.4	0.81/0.79			26493/19556				0.5	2.0
3 Wet Seasons	Maximum	770.0	2.95	97.8	0.094	14.3	4.26/4.06			142035/108691				2.8	15.4
(158)	Minimum	5.5	0.01	0.5	0.001	0.5	0.01/0.00			73/0				0.3	1.6
Apr-Sep.	Mean	203.5	0.29	10.1	0.038	4.1	0.58/0.46			5365/2681				1.1	5.4
2000-2003	Std. Dev.	334.5	0.32	8.2	0.033	4.7	0.70/0.67			9249/6253				0.5	3.3
3 Dry Seasons	Maximum	1986.8	2.15	37.0	0.161	24.8	4.16/3.97			60999/41086				2.8	14.7
(52)	Minimum	6.8	0.04	0.8	0.010	0.5	0.02/0.00			96/0				0.3	1.6
10/1/00-9/30/03	Mean	101.6	0.39	14.3	0.030	3.0	0.76/0.63			13690/8187				1.3	4.3
3 Water Years	Std. Dev.	191.5	0.50	13.8	0.022	3.0	0.79/0.77			23896/17517				0.5	2.4
(210)	Maximum	1986.8	2.95	97.8	0.161	24.8	4.26/4.06			142035/108691				2.8	15.4
	Minimum	5.5	0.01	0.5	0.001	0.5	0.01/0.00			73/0				0.3	1.6

The paired upstream and downstream stations operated through all of two wet seasons and one dry season, permitting comparison in flow rates and volumes at the two points. Notwithstanding seasonal and annual distinctions in rainfall and rainfall-runoff relations, channel hydrology and hydraulics did not differ much from wet to dry seasons and between divergent winters.

The mean peak flow rate decrease from upstream to downstream was approximately 62-65 percent in all periods of observation, based on upstream readings corrected for leakage, or 53-60 percent based on uncorrected readings. As pointed out earlier the higher number is probably an overestimate, while the lower one is likely to be an underestimate. Therefore, it can be concluded that the Viewlands Cascade is capable of reliably reducing peak flow rate by an average of about 60 percent over a period of a number of storms. However, the reduction for any individual storm is highly dependent on the event's characteristics and was seen to be nil in the larger events observed.

The mean event total discharge volume decreased from upstream to downstream by 82-85 percent based on corrected readings, or 69-77 percent based on uncorrected values. Taking the first as an overestimate and the second as an underestimate, it can be concluded that the channel can reliably decrease individual event runoff volume by an average of about 75-80 percent over a period of time. As with peak flow rate, there was little volume reduction in the largest storms.

One of the key objectives of the Viewlands project was to reduce formerly high velocity flows that bypassed the downstream drain inlet and eroded the steep adjacent slope. The estimated mean velocity was generally in the range 1.0-1.5 ft/s (0.3 – 0.46 m/s) regardless of season or year. It typically rose to about 2.8 ft/s (0.85 m/s) during the largest storms. The channel was estimated to provide a mean hydraulic residence time of around 4-5 minutes most of the time, although the average increased in relatively dry conditions and dropped in wetter ones. These residence times apply to the occasions when flow would reach all the way to the end of the channel. With full attenuation, of course, the surface flow residence time is infinite.

Of the 128 events with both upstream and downstream measurements, 34 (27 percent) had no surface discharge to Pipers Creek. 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. Earlier analysis concluded that up to approximately 1000 ft^3 (28.3 m^3) of influent could be fully attenuated through the channel, based on data uncorrected for leakage (Miller 2001; Miller, Burges, and Horner 2001; Horner, Lim, and Burges 2002). Examination of the data set for all 128 events confirms that judgment. Applying the correction would increase the estimate to approximately 2500 ft^3 (71 m^3). Again on the philosophy that under- and overestimates bracket the likely true value, it is reasonable to conclude that the Viewlands channel can prevent surface runoff of up to about 1750 ft^3 (50 m^3) of incoming water. The initial analysis also 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 the larger data set now available.

3.1.2. Total Discharge Summary

Inflow volume varied noticeably in the different winters. The 2001-2002 wet season registered 79 percent more precipitation than the preceding winter and, more than 2.6 times as much total inflow to the Viewlands Cascade (based on leakage-corrected values). 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. The third winter in the period was intermediate in terms of influent quantity.

As averages from all events, the high mean flow volume decreases shown in Table 3-1 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. Furthermore, whether or not the leakage correction is applied has a major bearing on the flow volume decrease from upstream to downstream averaged over all events. Leakage is a much greater factor in relatively small compared to large events, again giving undue influence to the small storms in the Table 3.1 statistics. More indicative of overall recharge from the drainage channel are the total seasonal and annual flow volume decreases (given corrected and uncorrected for leakage):

> 10/1/00-3/31/01 (wet season)—73.2 % corrected, 47.1% uncorrected;
> 4/1/01-9/30/01 (dry season)—71.6 % corrected, 53.0% uncorrected;
> 10/1/00-9/30/01 (water year)—72.9 % corrected, 48.4% uncorrected; and
> 10/1/01-3/31/02 (wet season)—57.4 % corrected, 33.5% uncorrected.

It thus appears that the Viewlands Cascade can prevent surface discharge to Pipers Creek of about 50-70 percent of the inflow in relatively dry or moderately wet conditions and about 35-55 percent in relatively wet periods. The first number in each case is probably an underestimate and the second likely an overestimate. What is safe to say is that it can prevent surface discharge of over half of the entering flow, unless conditions are quite wet, in which case attenuation slips a bit under half. The total inflow in the three water years was estimated at approximately 2,870,000 ft^3 (81,300 m^3). Assuming roughly half of that quantity did not exit the channel, its presence kept roughly 1.5 million ft^3 (43,000 m^3) from directly discharging to Pipers Creek.

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. Dry period runoff coefficient in the first two years of monitoring 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.

3.1.3. Comparison with Preceding Ditch

Table 3-2 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 and evapotranspiration 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. If the Cascade prevented the release of roughly half (1.5 million ft^3, 43000 m^3) of the influent surface runoff to Pipers Creek over three water years, then, the preceding ditch would have held back only about 0.5 million ft^3 (14000 m^3) if it were still in place, or approximately one-sixth of the total produced by the catchment.

Average velocities were estimated to be approximately 20 percent higher in the old ditch. Minimum hydraulic residence times were computed to be about a factor of three longer in the new system compared to the old ditch.

Table 3-2. Comparison of Viewlands Cascade Drainage System Performance with Estimates for Preceding Ditch

		Viewlands Cascade[a]			Preceding Ditch[a]					
Period	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	Mean	8104/ 2664	1.3	3.7	2694/ 882	0.33/ 0.33	1.7	1.31	1.4	0.38
	Maximum	35246/ 12326	2.7	7.9	11082/ 3779	0.31/ 0.31	3.2	1.19	2.8	0.35
4/1/01-9/30/01 Dry	Mean	7920/ 3548	1.6	3.0	2896/ 1291	0.37/ 0.37	2.0	1.25	1.1	0.37
	Maximum	37300/ 17387	2.8	3.9	15238/ 7047	0.41/ 0.41	3.2	1.14	1.5	0.38
10/1/00-9/30/01 Water Year	Mean	8069/ 2832	1.4	3.5	2733/ 959	0.34/ 0.34	1.7	1.21	1.3	0.37
	Maximum	37300/ 17387	2.8	7.9	15238/ 7047	0.41/ 0.41	3.2	1.14	2.8	0.35
10/1/01-3/31/02	Mean	13560/ 5072	1.4	3.7	4766/ 1784	0.35/ 0.35	1.7	1.21	1.4	0.38
Wet	Maximum	64943/ 31901	2.8	6.8	24515/ 12979	0.38/ 0.38	3.2	1.14	2.4	0.35

[a] Given with the leakage correction applied to the upstream measurement; except where there are two values, the first number is with the correction and the second is uncorrected.

3.2. 2nd Avenue NW SEA Streets

3.2.1. Rainfall and Runoff Event Summary

Table 3-3 summarizes rainfall and 2nd Avenue NW runoff statistics for 183 events over the period beginning just after completion of construction (20 January 2001) and concluding on 30 September 2003. 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.

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 11 of the 183 events (6 percent). Seven of these occurrences were during the period of shaft encoder slippage, when it was necessary to estimate. Therefore, it is possible that outflow did not actually occur in all of these cases. 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 2001 storm, an event when the shaft encoder was functioning well, allowing direct discharge measurement. Moreover, the project did not discharge at any time following the period fully documented in this report up the the present time, even during or after the large October and November 2003 storms. In fact, the last recorded outflow was on 14 December 2002. On the few discharge occasions there was a lag of more than 15 hours on average before the onset of flow.

Table 3-3. 2nd Avenue NW Rainfall and Runoff Event Summary, 20 January 2001-30 September 2003

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
1/20/01-9/30/01	Mean	146.3	0.37	14.7	0.030	1080	4	99.8
Partial Water Year	Std. Dev.	193.4	0.35	11.7	0.021	1012	26	1.1
(37)	Maximum	815.8	1.86	50.5	0.118	5428	157	100
[1]	Minimum	10.0	0.04	1.3	0.003	112	0	93.2
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
4/1/02-9/30/02	Mean	219.3	0.29	12.7	0.024	831	0	100
Dry	Std. Dev.	436.9	0.24	10.5	0.010	703	0	0
(19)	Maximum	1987.5	0.97	40.8	0.047	2837	0	100
[0]	Minimum	10.8	0.02	2.0	0.008	70	0	100
10/1/01-9/30/02	Mean	103.4	0.48	17.7	0.026	1401	25	99.5
Water Year	Std. Dev.	237.7	0.63	17.1	0.014	1831	85	1.5
(72)	Maximum	1987.5	3.05	99.5	0.068	8891	445	100
[9]	Minimum	7.3	0.02	2.0	0.006	70	0	95.0
10/1/02-3/31/03	Mean	68.0	0.44	14.8	0.029	1275	0	100
Wet	Std. Dev.	119.3	0.52	12.8	0.017	1520	2	0
(53)	Maximum	770.5	2.45	53.5	0.095	7154	15	100
[1]	Minimum	4.8	0.03	1.0	0.007	93	0	99.8
4/1/03-9/30/03	Mean	186.3	0.23	10.8	0.040	667	0	100
Dry	Std. Dev.	298.1	0.17	9.4	0.051	488	0	0
(21)	Maximum	1141.3	0.67	33.8	0.223	1941	0	100
[0]	Minimum	6.5	0.03	0.8	0.008	93	0	100
10/1/02-9/30/03	Mean	101.6	0.38	13.7	0.032	1102	0	100
Water Year	Std. Dev.	193.3	0.46	12.0	0.031	1337	2	0
(74)	Maximum	1141.3	2.45	53.5	0.223	7154	15	100
[1]	Minimum	4.8	0.03	0.8	0.007	93	0	99.8

Table 3-3 continued

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 (%)
Oct.-Mar. 2001-03	Mean	67.1	0.47	16.4	0.029	1362	15	99.7
2^+ Wet Seasons	Std. Dev.	91.4	0.58	15.4	0.018	1690	66	1.2
(125)	Maximum	770.5	3.05	99.5	0.118	8891	445	100
[11]	Minimum	4.8	0.03	1.0	0.006	93	0	95.0
Apr.-Sep. 2001-03	Mean	206.6	0.31	13.4	0.030	900	0	100
3 Dry Seasons	Std. Dev.	333.4	0.31	11.1	0.033	904	0	0
(58)	Maximum	1987.5	1.86	50.5	0.223	5428	0	100
[0]	Minimum	6.5	0.02	0.8	0.003	70	0	100
1/20/01-9/30/03	Mean	111.3	0.42	15.5	0.029	1215	10	99.8
2+ Water Years	Std. Dev.	211.5	0.51	14.2	0.023	1500	55	1.0
(183)	Maximum	1987.5	3.05	99.5	0.223	8891	445	100
[11]	Minimum	4.8	0.02	0.75	0.003	70	0	95.0

The SEA Streets project has never attenuated less than 93.2 percent of the potential event runoff volume nor released more than 157 ft^3 (4.4 m^3), measured, or 445 ft^3 (12.6 m^3), estimated during the period of equipment malfunction. With so few events yielding any discharge, attenuation was so close to complete that the mean flow volume decreases shown in Table 3-3 are quite indicative of recharge over the full seasonal, water-year, and multi-year periods. Over the wet seasons and water years of record, the project reduced the total potential surface runoff by 99 percent.

The 2^{nd} Avenue NW SEA Streets site is demonstrating a clear tendency to store and prevent surface runoff from even more rainfall than during its early years. The previous estimate that the complex can fully attenuate up to 0.75 inch (19 mm) of precipitation no longer applies, and the quantity now seems to be much larger. The reason for this development can only be speculation, but it is likely that the maturing vegetation both takes up more water and assists its passage into the soil. What fractions leave the system through infiltration versus evapotranspiration is unknown. Analyzing the water's fate is a subject being taken up in connection with hydrologic modeling being performed in ongoing work.

Even though the project has now experienced some large, low-frequency storms, monitoring continues because these events occurred after one of the driest summers on record. The ultimate test will come when it receives a similar quantity of rainfall following wet antecedent conditions.

3.2.2. 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 42 percent and from a conventional Seattle street with curb and gutter drainage by 66 percent (Miller 2001; Miller, Burges, and Horner 2001). The model used in these earlier estimates predicts that, if a conventional street had been the place of the SEA Streets project during the almost three water years reported on here, it would have released 75 times as much surface runoff during the wet seasons and 98 times as much overall.

CHAPTER 4 – DESIGN COMPARISON

This chapter 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 Streets project versus a conventional street drainage system design, and (3) the 2nd Avenue NW SEA Streets project versus the Viewlands Cascade Drainage System. Horner, Lim, and Burges (2002) compared the 2nd Avenue NW SEA 1Streets project versus the original street drainage system as well. That comparison is omitted now because of the great disparity in data availability from SEA Streets versus the brief baseline monitoring opportunity. The designs are compared as ratios for dry and wet seasons 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 the data from the three water years for Viewlands and the almost three water years for SEA Streets.

Comparisons of the Viewlands and SEA Streets projects used the full 2.3-acre (0.93 ha) contributing catchment area for SEA Streets and the 26-acre (10.5 ha) subcatchment that generally contributes runoff to Viewlands, omitting for this purpose the additional area that sometimes drains to the Cascade. The rationale for this choice is that these areas were the basis for the designs and capital expenditures made by the City of Seattle. Tables 4-1 and 4-2 present the comparisons.

The benefit ratios for Viewlands Cascade/preceding ditch and SEA Streets/conventional street in Table 4-1 reiterate the points made in Sections 3.1 and 3.2: the improved drainage systems retain several times as much runoff volume as the predecessor ditch at Viewlands or, in the case of SEA Streets, the alternative of designing according to the City of Seattle's current convention. The project benefit ratios range from about 3 to 5.

The tables show ratios both with and without weir measurement leakage correction applied to the Viewlands influent. As discussed earlier correcting is thought to overestimate the Cascade's benefit and not correcting to underestimate it. Therefore, mean values of the ratios are also reported in the tables and are probably better indicators than either of the ratios with and without correction.

On this basis the SEA Streets/Viewlands Cascade ratios in Table 4-1 show that the 2nd Avenue NW project is estimated to attenuate about one-quarter to 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 4-2, second column), that advantage multiplies. However, calculating according to unit area cost (Table 4-2, 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 4-1. Monthly Benefit Comparisons of Ultra-urban Drainage System Designs

Comparison	Period	Retained Volume/Month[a]
Viewlands Cascade/Preceding ditch	Drier months[b]	2.7
	Wetter months[c]	2.9
SEA Streets/Conventional street	Drier months[b]	5.1
	Wetter months[c]	5.1
SEA Streets/Viewlands Cascade	Drier months[b]	0.20 (0.45) [0.33][d]
	Wetter months[c]	0.12 (0.33) [0.23][d]

[a] Expressed as the ratio of the volume retained per month by the first site divided by the volume retained per month by the second site in the comparison; in ft^3/month (divide by 35.3 for m^3/month)
[b] April-September
[c] October-March
[d] The first number is with the leakage correction applied to the upstream Viewlands measurements; the second number in parentheses is without the correction; the third number in brackets is the mean value.

Table 4-2. 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]	2.3 (5.1) [3.7][d]	0.016 (0.037) [0.27][d]
Wetter months[e]	1.4 (3.7) [2.6][d]	0.010 (0.027) [0.19][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 costs of construction alone of \$225,000 and \$244,000 for Viewlands and 2nd Avenue NW, respectively
[c] April-September
[d] The first number is with the leakage correction applied to the upstream Viewlands measurements; the second number in parentheses is without the correction the third number in brackets is the mean value.
[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 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 5 – SUMMARY AND CONCLUSIONS

1. Flow has been monitored at the Viewlands Cascade Drainage System over three full water years between 1 October 2000 and 30 September 2003, representing 210 precipitation events. Both upstream and downstream monitoring occurred for the first 128 of these events, to quantify flow attenuation in the channel. The 2nd Avenue NW SEA Streets project has received flow monitoring for nearly as long, since it went into service in late January 2001, over 183 events. The wet seasons represented have differed in meteorological characteristics. The 1999-2000 and 2000-2001 winters had little more than half of the long-term average rainfall, while the following two wet seasons were above or close to average. There were no especially large storms in those winters, but the early weeks of the present water year had the largest 24-hour rainfall in the region's history and another event almost as large. However, this precipitation occurred after an exceptionally dry summer and early fall. Therefore, the projects have still not experienced the most challenging conditions.

2. The Viewlands channel has had considerable attention to try to prevent water losses in the cells where monitoring occurs, which are not recorded as flow. When losses could not be prevented fully, attention turned to quantifying them at the upstream station using metered city water flows. The same technique could not be used at the downstream station, which could discharge chlorinated city water to Pipers Creek. Correction factors were derived for the inflow station. Using a correction upstream and not downstream tends to overestimate the flow reduction in the channel. On the other hand, not using the correction tends to underestimate the decrease. Therefore, using both corrected and uncorrected upstream values is thought to bracket estimates of flow decrease.

3. The Viewlands meteorological station has assembled several years of record on temperature, relative humidity, wind speed, net radiation, and pan evaporation data, which will be invaluable in developing a hydrologic model for the Pipers Creek watershed. From the record to date, pan evaporation totals about 30 inches (76 cm) per annum, about 4 inches (10 cm) higher than potential evaporation from a standard model.

4. According to the best estimates, the Viewlands Cascade can cut the average peak flow rate of entering runoff by about 60 percent and the total influent volume over a period of time by over half. However, little or no reduction of either peak flow rate or volume occurs during relatively large storms. There was no discharge from the end of the channel in 27 percent of the events monitored. It can completely infiltrate the catchment response to about 0.13 inch (3.3 mm) of precipitation and 1750 ft^3 (50 m^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.

6. During the three water years the new Viewlands channel retained roughly 1.5 million ft^3 (43000 m^3) of runoff that entered it, preventing its direct release to Pipers Creek and the elevation of erosive flows there. This quantity is about 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 99 percent of the wet season and overall runoff. Whereas all events in the baseline monitoring period, which occurred mostly in the dry season, created a discharge, only 6 percent have since the project's construction.

8. It was estimated that over the nearly three water years of record a street drainage system design according to City of Seattle conventions would have discharged almost 100 times as much runoff to Pipers Creek.

9. The 2nd Avenue NW SEA Streets drainage system has been withholding more water from discharge as time goes on. Apparently, vegetation is both intercepting and taking up more water for evapotranspiration and assisting infiltration.

10. Despite serving a catchment less than 10 percent as large as the Viewlands Cascade, the 2nd Avenue NW project retains one-quarter to one-third as much runoff volume in as Viewlands. On the basis of unit runoff contributing area, the SEA Streets project is around three times as effective as Viewlands. 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. The Viewlands and 2nd Avenue NW projects represent different strategies for controlling the quantity of urban runoff. The latter is a source control strategy that can manage a large proportion of the precipitation falling on its catchment. The former is an "end-of-pipe" solution that can attenuate a large quantity of runoff, although not nearly as a high a percentage as the source control option. Used in concert as opportunities arise, the two alternatives can work together to bring significant change to the effectiveness of urban stormwater management.

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APPENDIX A

LEAKAGE TEST PHOTOGRAPHS

A-1. Flume at Viewlands Cascade Inlet

A-2. Flume at Viewlands Cascade Cell 1 Outlet