University of Washington
Department of Civil and Environmental Engineering

AN EXAMINATION OF STORMWATER DETENTION AND INFILTRATION AT THE SCALE OF AN INDIVIDUAL RESIDENCE IN THE SAMMAMISH PLATEAU REGION OF KING COUNTY, WASHINGTON

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Water Resources Series
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by

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Project Title:	**Improved Residential Onsite Stormwater Management**
Grant Source:	**State of Washington Department of Ecology Centennial Clean Water Fund Program FY 1993**
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Principal Investigator:	**Stephen J. Burges Professor of Civil Engineering**

Abstract

The conversion of a forested landscape to a residential development may have deleterious hydrologic effects. As forested landscapes are altered and covered with more impervious surfaces, the rate, volume and duration of stormwater runoff increase relative to the normal state. The increased stormwater runoff (stormflow) can increase erosion, sedimentation, and flooding downstream. On-site stormwater detention and infiltration at the scale of an individual dwelling or commercial building are management alternatives for mitigating some of these effects. This report has examined the hypothetical performance of roof runoff detention systems using a simulation model as well as the field performance of two roof runoff infiltration systems. Both types of systems receive only roof runoff which comprises only a small portion of the total runoff generated by residential land areas. Roof runoff demonstrates the small-scale short-term characteristics of storm flow generated by urban areas: increased peak flow rate and rapid recession once storms end.

On-site residential detention and infiltration systems can restore some of the hydrologic characteristics of a forested zero-order basin. Detention systems can store stormwater and release it gradually as would a forest soil. Based on rainfall records for the Water Years 1991 to 1993 from Novelty Hill, WA, a detention system requires 20 mm of storage for a given roof area (or other impervious catchment) with a 5 mm/day (for the same area) release rate to increase the duration of low flow to a level observed for discharge from a natural first-order basin. A minimum of 100 mm of storage (over the roof area) with a release rate of 5 mm/day is needed to control peak rates of flow and their duration to levels more representative of storm flow from an equivalent area of forest, though the release rate still exceeds expected runoff rates for a forested area. having the same general plan form and topography.

On-site residential detention systems can serve multiple purposes by regulating storm flow and supplying water for domestic uses. In the Puget Sound Lowlands of Washington State multipurpose detention systems generally need to be large to provide a supply of water: a storage volume equal to 20 days of the residence's daily water usage may be necessary for a somewhat reliable supply during winter; and, for summer irrigation, 250 mm of storage for the area being irrigated may be needed.

Infiltration systems can be as effective as the largest detention systems considered in this report for reducing the volume and rate of stormwater delivered to receiving waters. The two infiltration systems monitored during the winter of 1994-95 transported all of the roof runoff into the soil where it became subsurface flow.

Detention systems for the region must have low release rates to approximate the rate of storm flow from a forested basin. As a result, these systems must have appropriately large storage capacities on the order of 100 mm over the roof area to avoid spills (i.e., high rate discharges). Infiltration systems can better regulate storm flow with a smaller storage volume because the outflow is distributed to the surrounding soil which delays its movement to a receiving water body. Stormwater control, however, is only maintained as long as the water remains in the ground as subsurface flow and the local groundwater level remains below the infiltration structure.

Both systems offer ancillary benefits. Detention systems can be designed and operated to supply water to a residence. Infiltration systems can increase local aquifer recharge and help to sustain base flow in streams.

Acknowledgments

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The authors are especially grateful to the following individuals whose substantial contributions to this project deserve mention. Kyle Kolsti participated in every part of this work as a member of our project team pursuing improvements in how residential areas function as a part of the environment. Lorin Reinelt and John Koon with King County Surface Water Management were invaluable in the planning and implementation of our field research. Shelly and Steve Grew, and Deborah and Jeff Wade graciously allowed us to install monitoring equipment on their property and to access the equipment periodically. Professor Ronald E. Nece assisted the authors with his insights early on in the project and his astute review of this report.

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Chapter 1: Introduction

Conversion of forests in the Puget Sound, WA lowlands to urban and residential areas has dramatically increased the production of quick response storm flow from the land. Storm flow is the excess rainfall that is transmitted over or through land to receiving water bodies during and after storms. Storm flow is also called runoff or stormwater. The hydrologic changes associated with residential development bring increased storm flow rates, increased duration of peak flow, and increased storm flow volume to urban areas. The increase in storm flow is balanced by reductions in evapotranspiration, infiltration, subsurface flow, and aquifer recharge. In the long-term these hydrologic changes, most notably increased flow rate variability and reduced magnitudes of low flows, destabilize channels, degrade aquatic habitat, create flooding problems, and reduce groundwater resources (Burges et al., 1989).

Public agencies in the Puget Sound region have responded by developing local and regional strategies for managing stormwater. In general, stormwater management strategies have focused either on detaining storm flow, for example in ponds, or on increasing the capacity of drainage networks to convey storm flow, for example by channelizing and armoring streams. Public stormwater management activities include regulating new development, responding to local problems, constructing and maintaining stormwater management facilities, researching stormwater effects associated with differential management techniques, and regional planning.

Stormwater management often takes form as large detention or conveyance structures in residential developments installed to comply with regulations or to mitigate local problems such as flooding or erosion. While these structures may be designed with broad regional goals in mind, they have not necessarily been effective in limiting the deleterious effects of stormwater in the Puget Sound lowlands (Booth and Jackson, 1994). In a broader sense, current stormwater management strategies have not been successful in responding to: (1) the extensive urbanization of the region; (2) the inefficacy of blanket sub-basin control strategies for managing regional storm flows (Hardt and Burges, 1976); and (3) the latent cause of many local stormwater problems which is modification of hydrologic processes by land-use change. The incompleteness of current management strategies may be due, in part, to their focus on channel processes rather than taking a more comprehensive view of basin hydrology. In particular, recognizing the influence of hillslope (i.e., land area that are not channels) processes in generating and controlling runoff production and the associated short-term (storm or several storm) and long-term influences on the water balance needed to be included in stormwater management strategies.

While channel processes have long been the domain of stormwater management (e.g., routing storm flows through conveyance and detention facilities), hillslope hydrology has not received adequate consideration. At best, stormwater management analyses (e.g., to size detention facilities) may apply some form of a runoff coefficient for a land surface to describe the fraction of rainfall that will become runoff during a specific design storm (King Co., 1990; Urbonas and Roesner, 1993). At worst, land is discretely classified as pervious or impervious with no regard for the transient characteristic of its infiltration capacity.

Hillslope hydrology is an appropriate starting point for most stormwater management strategies given the dominant role of hillslopes in runoff production: all runoff is generated by hillslopes, except for rain that falls directly on streams and channels. Since hillslope processes are often overlooked in stormwater analyses and management, opportunities exist to manage stormwater at the scale of single residences in ways that restore some of the hydrologic functions of forested hillslopes while accommodating the constraints imposed by the engineering and societal requirements of residential environments. On-site residential management techniques may better replicate the spatial and temporal distribution of stormwater over a forested hillslope than large, centralized facilities (McCuen, 1979). As such, on-site systems may increase the efficacy of existing regional stormwater management systems and provide benefits that cannot be provided by large, centralized facilities.

This report considers on-site applications of residential stormwater management systems to the Sammamish plateau region that restore some functions of a hillslope's pre-development hydrology. The remainder of this chapter introduces relevant aspects of the hydrology of forested hillslopes in the Puget Sound region, describes the hydrologic effects of residential development particularly on the Sammamish plateau, and suggests goals for on-site residential stormwater management systems.

Two on-site stormwater management alternatives are considered: multipurpose stormwater detention, and stormwater infiltration. In Chapter 2, multipurpose stormwater detention systems are examined. A simulation model and recorded stream discharge from a forested catchment (Novelty Hill) are used to aid in the analysis of on-site detention as a stormwater management strategy. General sizing guidelines are developed based on a three year rainfall record for an urbanized catchment (Klahanie). Both of the catchments are in King Co., WA and are shown along with the infiltration systems in Figure 1.1.

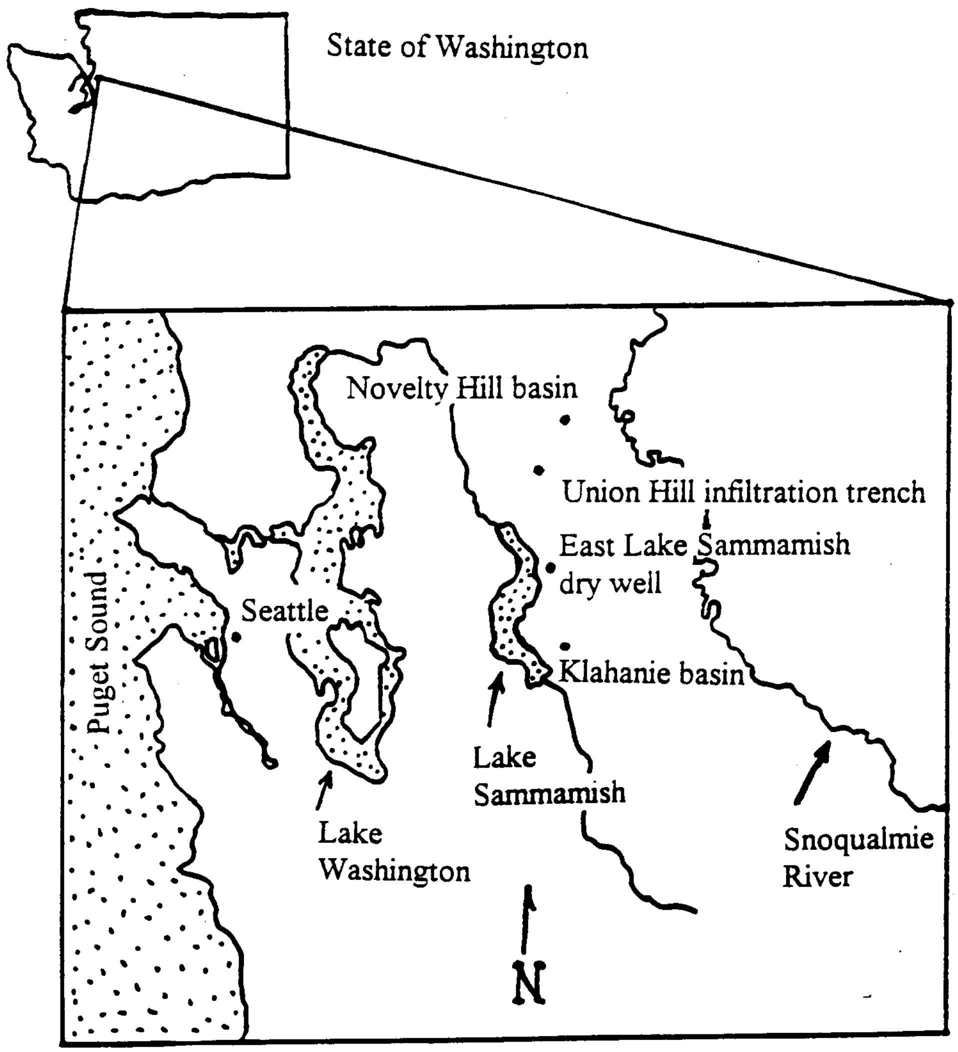

Figure 1.1: Locations of Novelty Hill basin, Union Hill infiltration trench, and East Lake Sammamish dry well

Stormwater infiltration systems are the subject of Chapter 3. Given the difficulties of developing a practicable, general analytic model of on-site residential infiltration systems, field studies of two residential infiltration systems were conducted to explore semi-quantitatively the performance of infiltration systems. These systems are also located in King Co., WA. Chapter 4 presents an integrated approach to residential stormwater management using detention and infiltration systems.

1.1 Hydrology of zero-order basins of the Puget Sound lowlands

A zero-order basin is a small land area that does not generate enough stormwater to have an associated stream channel. The channel head starts at the outlet of a zero-order basin. Two or more zero-order basins may form a first-order basin with a small first order stream draining it or zero-order basins may drain to a higher order stream without changing its order.

Stormwater is generated and conveyed in zero-order basins over and through hillslopes. Natural hillslopes in the Puget Sound region are dominated by conifer forests of Douglas-fir (*Pseuodotsuga menziesii*), Western Hemlock (*Tsuga heterophylla*), and Western Red Cedar (*Thuja plicata*) and mixed conifer and deciduous forests with Red Alder (*Alnus rubra*) and Big Leaf Maple(*Acer macrophyllum*). These forests generate organic material which accumulates as duff on the forest floor and decomposes. When precipitation falls on forested hillslopes, it may be intercepted by vegetation, it may be stored in surface depressions, or it may infiltrate through the organic layers (duff and top soil) and into the mineral soil.

The Puget Sound region is dominated by a maritime climate with an annual rainfall pattern of dry summers and wet winters. Long-duration, low-to-moderate intensity storms with rainfall rates, for example, of 1 - 5 mm of rain per hour occur frequently during the winter months. The periods between such storms typically ranges from 3 hours to 2 weeks (Gan and Burges, 1990).

During frequent, long, and moderately intense storms characteristic of winter in western Washington, shallow subsurface flow moving laterally through the upper soil mantle will be the main storm flow response mechanism where hillslopes are steep and soils are porous and well aggregated (Harr, 1977). The most rapid storm response is likely to occur through subsurface flow in soil macropores (Whipkey, 1965). Saturated overland flow will occur only where and when parts of zero-order basins become saturated. Typically, these variable source areas are in areas of topographic convergence, adjacent to channels and swales or at the foot of hillslopes, especially where soils are thin, and expand upslope when storms are of longer duration, more intense, or more frequent (Dunne et al., 1975).

In the Puget Sound region, precipitation rates are unlikely to exceed the infiltration capacity of forest soils as required for generation of Horton overland flow. The 100 yr. rainfall intensity-duration-frequency curve for Seattle, WA ranges from approximately 100 mm/hr (for less than a 10-min. duration) to 4 mm/hr for a 24 hour duration (Stedinger et al., 1993, Figure 18.8.1). Harr (1977) found that the mean saturated hydraulic conductivity, which is the theoretical limit of a soil's infiltration capacity, for well aggregated gravelly clay loam and blocky clay loams in a Douglas-fir/Western Hemlock forest in the Oregon Cascades ranged from 160 to 4120 mm/hour depending on the depth of the sample. Assuming forests in the Puget Sound region would have soils with a similar structure, which Harr found to be more important than texture for hydrologic properties, precipitation rates are unlikely to exceed infiltration capacities. Field observations support these quantitative measures: Horton (i.e., infiltration-limited rather than saturated) overland flow occurs rarely in natural forests in the region.

The effect of frequent, low intensity rainfall on forested zero-order basins similar to those in the Puget Sound region is likely to be steady subsurface flow, providing a sustained base flow for small streams (Hewlett, 1961) and a source of soil moisture for deep percolation into aquifers. When rainfall intensity and duration increase, more rapid subsurface flow and saturated overland flow contribute to peak stream flows. The Puget Sound's climate and geology limit the applicability of all results presented here to similar regions.

1.2 Physical changes and hydrologic consequences of residential development

The physical changes wrought by residential development in the Puget Sound region have a significant effect on the flow of water over and through the land. The changes and their hydrologic effect are evident especially in areas being converted from forest to residences such as on the Sammamish plateau (east of Lake Sammamish) in King Co., WA. In the typical residential development on the Sammamish plateau, the trees and understory are cleared from the site. Forest duff and the upper layers of soil are scraped off to expose a firm surface for construction. Sites may also be graded (e.g., cut and filled) to achieve a level area for building. Any exposed soil not removed is likely to be compacted in the course of construction.

Many residential structures are constructed on Vashon till, a pervasive geologic unit on the Sammamish plateau, which may be found approximately 1 m below the land surface

(Mullineaux et al., 1965). It is coincident with the Alderwood soil association (Soil Conservation Service, 1972). Vashon till is a silty sedimentary unit that has been compacted by an overriding glacier.

Vashon till transmits little water (vertically or laterally) in comparison to deep, well drained forest soils. Its massive structure and fine texture hold soil moisture readily so that water infiltrates slowly, the soil saturates quickly, and drains marginally. An infiltration test using a 1 m single ring infiltrometer conducted on exposed Vashon till at a construction site on the Sammamish plateau indicated an infiltration rate of 0.1 mm/hr.

While slow, this rate is rapid enough to allow some of a year's rainfall (approximately 1 m) to pass through if the rain was stored and supplied steadily over the course of the year. In a natural forest, a deep soil column acts as a storage reservoir for stormwater, so that as much as 20-30% of incident rainfall can leak through the till. Hydrologic mass balances for Novelty Hill, a 37 ha forested catchment near Redmond, WA made by Wigmosta et al. (1994) indicate modeled recharge through the till layer of less than 300 mm/year.

Once a forest soil is scraped off of the till, frequent or moderately-intense rainfall events can generate infiltration-limited overland flow and increased saturated overland flow relative to the natural state. These runoff mechanisms result in rapid storm flow response and increased stream flow. After rainfall ceases, storm flow recession is rapid where the till lacks deep, well structured soils. The rapid recession results from the till retaining water in its soil pores rather than draining freely as would a forest soil.

Site preparation during residential development alters the paths and pools available for rainwater to flow over and through the land. The storage capacities of vegetation interception, surface depressions, and soil macropores are diminished. Infiltration of rain is reduced when porous, well aggregated forest soils are removed or compacted leaving less permeable surfaces. Without sufficient storage or infiltration capacities to attenuate the runoff response to storm events, hillslopes generate greater storm flow volumes and drain rapidly at the expense of sustained base flow (Harr et al., 1975; Wigmosta et al., 1994).

Evapotranspiration is reduced with the removal of trees which typically have root depths of 1 m. After storms, high levels of soil moisture can persist in cleared areas where forest vegetation would have transpired soil moisture. This leads to higher soil moisture at the start of a storm, which is another factor increasing the quick response of storm flow (Linsley et al., 1982).

Where steep, natural hillslopes are graded (leveled) for development, the velocity of surface storm flow could be reduced. These sites, however, are likely to be more easily saturated, because they have less soil and will collect shallow subsurface flow draining from upslope land. They are also likely to be drained by engineered structures which may transport stormwater rapidly to a receiving water body. The net effect of grading on forested hillslopes, with the exception of steep areas, is likely to be similar to that of site preparation in general: increased saturated overland flow, storm flow velocities, and storm flow quantities.

The hydrologic effects of site preparation may or may not be transient depending on the ultimate disposition of the site. If a site is left alone, forests are likely to regenerate and reinitiate soil development over a period of many decades. If construction continues, the appurtenances (homes, roads, lawns, etc.) of residential development further modify the hydrology of the land. These appurtenances create a new land form for water to flow over and through.

In most cases, roofs, roads, porches, walkways, and lawns exacerbate the hydrologic effects of site preparation: response from storms is more rapid and less water is retained on-site. Large impervious areas (roofs and roads) generate direct overland flow and promote rapid runoff with their relatively smooth surfaces. The dense channel networks (gutters, footing drains, curb drains, storm sewers) imposed by residential development decrease the travel time of storm flow, which in a forest would travel through longer paths through soil, to channels. Straight and smooth culverts and ditches reduce the energy losses that had been provided previously by rough boundary, meandering creeks.

Even lawns, which appear hydrologically benign in comparison to roofs and roads, cannot provide nearly the same storage capacity of a forest's canopy and soil. Soils underlying lawns are generally thinner and have reduced capability to transmit water. As a result, lawns saturate quickly, generate saturated overland flow and shallow subsurface flow, and fail to restore the pre-development hydrology of their area much less the total area affected by development (Wigmosta et al., 1994). In situations where excessive lawn thatch build up occurs, infiltration-limited overland flow can occur.

Cumulative hydrologic effects of site preparation and residential land uses include:

(1) a greater fraction of incident precipitation contributes to quick response storm flow than in a forested landscape;

(2) storm flow volumes and velocities increase with longer duration of high flow levels than for pre-development storm flow;

(3) channel incision and increased sediment transport in streams;

(4) reduced duration of a stream's base flow; and

(5) reduced infiltration, groundwater recharge, and evapotranspiration.

An ASCE Task Committee (1975), Burges et al. (1989), and Hammer (1972) provide descriptions of these hydrologic effects of urbanization. Wigmosta et al. (1994) illustrate the effects in a comparison of the hydrologic responses of a forested basin and a residential development. While these effects can be seen at numerous sites, each residential development will produce unique hydrologic changes depending on the natural hydrology of the area, the proportions and configurations of land-uses, and any drainage system engineered or inadvertently imposed on the land.

The hydrologic effects of urbanization also vary with the intensity, duration, and frequency of storm events. Harr et al. (1975) observed that forest clearing had less effect on storm hydrographs for 50 - 100 yr. storms than it did on more frequent, smaller events. Similarly, Hollis (1975) reviewed studies that, in general, demonstrate a declining impact of urbanization at longer flood recurrence intervals. For example, a basin with 20% paved surfaces may have a post-development peak discharge for a 1 yr. flood equal to 10 times the predevelopment 1 yr. flood peak while the 10 yr. flood peak may be twice the predevelopment peak). Thus, the hydrologic effects of residential development are dictated by site and storm event characteristics.

1.3 Mitigating the deleterious hydrologic effects of development with on-site residential stormwater management

Management of stormwater has a long history. In ancient Greek cities, stormwater management served broad and fundamental social purposes: it limited flooding in areas so they could be occupied or used by people; it provided a supply of domestic water; and it provided a means for wastewater disposal (Crouch, 1993). Perhaps in its most recent incarnation, the aims of stormwater management are to mitigate the deleterious hydrologic consequences of residential, commercial, agricultural, and transportational land uses.

A variety of strategies has been used to manage stormwater and to reduce its deleterious effects on the environment. Recent research and regulatory programs emphasize control of the physical constituents of stormwater (e.g., suspended and dissolved solids, phosphorous, nitrogen, metals, and pathogens) (U.S. EPA, 1983; National Research Council 1993; 33 U.S.C. 1342, 1993). This is difficult as the composition of stormwater is spatially and temporally variable, poorly understood, and difficult to control (U.S. EPA, 1983; Adler et al., 1993). More importantly, this approach does not acknowledge that most altered flow patterns, regardless of the chemical composition of the water, degrade aquatic ecosystems.

By focusing on hillslope hydrologic modifications created by development, on-site stormwater management addresses the mechanisms at the site generating stormwater and its attendant effects. A tenable on-site stormwater management goal is to restore the magnitude and timing of pre-development storm flow. While it is impossible to replicate a forest's hydrology in a residential development, hydrograph restoration can still serve as a goal for on-site management activities.

Too often, however, the totality of hydrologic modification is reduced to one measure of channel flow: peak rate of discharge (Hollis, 1975; Mein, 1980; King Co., 1990; Urbonas and Roesner, 1993). While this is one measure for assessing the effects of changing land uses and the efficacy of management techniques, it may not be adequate for either purpose. It does not necessarily account for increases in the duration or volume of high rate storm flow that results from urbanization. The lag time in stream flow response to runoff generated in different zero-order basins and the increased volume of storm flow and duration of peaks generated by urbanized zero-order basins precludes the development of an effective stormwater management strategy based only on reducing the peak flow generated from zero-order basins (Hardt and Burges, 1976).

Stormwater management strategies can be improved by taking a broader view of basins to incorporate the dynamics of hillslope hydrology. This view includes on-site residential stormwater management systems that aim to restore the predevelopment hydrograph of runoff from a zero-order basin, not just its peak rate of discharge. These systems can be used to reduce quick response storm flow and increase the duration of low flow in stormwater-receiving streams and channels. Ideally, evapotranspiration, infiltration, and groundwater recharge would also be returned to their pre-development levels. In this way, the proximate effects of residential development could be ameliorated. On-site residential stormwater management systems do not stand alone as a stormwater management strategy for large basins, but they are a component of basin-wide stormwater management which has been undervalued.

1.4 Alternatives for managing stormwater from residential roofs

Residential stormwater management can begin by focusing on roof runoff. Buildings (i.e., homes and garages) exemplify residential land uses that have extensive hydrologic effects. They cover an appreciable area in residential developments without providing hydrologic functions of the forest they replaced. For example, in an urbanized 13.7 ha catchment (Klahanie) on the Sammamish plateau located 6 km north of Issaquah, WA, 8% of the area is covered by roofs. Based on simulations using a continuous rainfall-runoff model, 13% of the total volume of stream discharge (for a three year period) from the Klahanie development can be attributed to roof runoff (Wigmosta et al., 1994).

Controlling roof runoff is not sufficient to compensate for the total hydrologic impact of residential development, but it is a separable component of residential stormwater management.

Two alternatives for managing roof runoff can contribute to a residential stormwater management strategy that aims to restore the predevelopment hydrograph of a site: multipurpose stormwater detention systems, and stormwater infiltration systems. Multipurpose stormwater detention refers to a system where roof runoff is collected in a reservoir or tank. The water is made available for domestic uses or the release of the water to a drainage system or the ground is delayed. Stormwater detention reduces peak storm flow rates by lengthening the time over which stormwater is discharged. It does not reduce the volume of runoff unless the detained water is used.

Infiltration systems convey roof runoff into the soil, for example, through a trench or dry well. This reduces the volume of surface runoff and increases subsurface flow. Typically, this will result in lower peak storm flow rates. While infiltration systems can be as simple as a pocket of gravel below a roof downspout, their performance is difficult to generalize. Consequently, two different infiltration systems were observed in the field to illustrate the utility of this approach.

Chapter 2: Residential Stormwater Detention Systems

Stormwater detention is a common management strategy used to control runoff from residential developments. Normal professional practice is to convey excess rainfall (i.e., stormwater) from a residential area, which may be many hectares and extend beyond a single development, to a pond with a flow-limited outlet. Stormwater management is not generally practiced at the scale of individual residences. On-site detention at the scale of a single residence, however, has been used around the world since ancient times to supply water to single residences (Hofkes and Huisman, 1981; Crouch, 1993). This chapter considers the application of on-site detention to residential developments in the Puget Sound region to serve multiple purposes: mitigating deleterious hydrologic effects of urbanization, and providing an auxiliary domestic water supply.

On-site multipurpose detention can be incorporated in existing and new residential developments. Since large detention facilities serving existing developments may be undersized or otherwise fail to achieve stormwater management objectives (Booth and Jackson, 1994) on-site detention may be more spatially representative of the soil water storage that would occur in a natural catchment. The release of stormwater from on-site detention systems can be made to approximate more closely "the temporal distribution of storage depletion that existed prior to development than regional facilities" (McCuen, 1979). On-site detention can reduce the size of the stormwater facilities required at new developments and increase the efficacy of the regional stormwater management system. In addition to enhancing stormwater control, on-site detention can provide a source of water for non-potable domestic uses of either existing or new residences.

A mass-balance model has been developed here to simulate the performance of on-site detention systems. Results from the model demonstrate how different system configurations perform in terms of controlling stormwater and providing a domestic water supply.

2.1 System components

An on-site stormwater detention system comprises a catchment, a reservoir, and means to release detained stormwater. Each component of a system can be designed for specific purposes. To maximize yield, a detention system would have the largest possible impervious area feeding a reservoir sized appropriately given the catchment area, rainfall patterns, and the rate that water is used or released. In a residential setting, however, where a primary objective of detention systems is controlling stormwater, it is reasonable to consider that roofs will serve as the catchments and additional impervious surfaces will not be constructed for harvesting rainwater. Other impervious surfaces found in residential areas (e.g., driveways and sidewalks) are not considered as they would require stormwater collection systems and can introduce pollutants to runoff.

While the catchment area for a detention system is limited by the roof area of a house, the volume of the reservoir can be planned specifically for purposes of the detention system and cost constraints. For stormwater control, one might want to store the runoff from a large, single day rainfall event for the Puget Sound lowlands (e.g., the equivalent of on the order of 20 mm over the catchment area). For irrigating landscaped areas during the summer, the reservoir could hold all of the runoff generated in a year: for the Puget Sound lowlands this is approximately the equivalent volume of 1 m over the catchment area). These two examples represent a range of two orders of magnitude in reservoir volume. Thus, it is prudent to identify the purposes for detention systems and to evaluate the performance of systems having different size reservoirs before considering specific application.

The purpose for a stormwater detention system, as well as the available storage volume of the on-site reservoir, will dictate the rate and pattern of release of detained water. For stormwater control, an on-site detention system might release the equivalent of 0.5 mm over the roof area per day. For irrigating landscaped areas, stormwater can be detained until summer though this may result in a full reservoir at some point in the winter. Since a full reservoir cannot control additional stormwater, release rates should be sufficient to ensure storage space is available to accumalate runoff from a follow up storm. Release rates and schedules reflect local storm patterns and active reservoir volume.

Residential stormwater detention systems should also account for site-specific hillslope and basin hydrology. For example, release or overflow from the system should not saturate areas which drain toward any structures or could become unstable and slide when wet (e.g., steep slopes). To be an effective part of a regional stormwater management strategy, releases must be limited so that they do not contribute to de-stabilizing flow rates at downstream channel locations.

Detention systems must be configured appropriately for their purposes. A detention system that both controls stormwater and provides a source of water for domestic uses may be designed to have a slow, continuous release during the winter to maintain free capacity for capturing storm flow and no release during summer to keep water available for use. Alternatively a summer irrigation system with no winter release will require a large reservoir to store sufficient water supply for summer use. A large reservoir is required because the ratio of landscaped areas to roof areas is on the order of 10:1 in some new developments (Wigmosta et al., 1994) and large volumes of water are used to irrigate landscaped areas (e.g., 25 mm/week, for about ten weeks, during dry periods). If such a system provided an additional supply of water for fire fighting or other emergencies, the reservoir would be kept full until October or early November, the usual onset of the wet season.

The performance of detention systems is complex and depends on the system configuration (roof area, reservoir volume, release rate and pattern) and the temporal distribution of rain. Nine different detention system configurations were simulated with a digital computer model written in FORTRAN using a continuous rainfall record. The simulation results are compared with measured discharge for a forested catchment. The simulation model, rainfall record, and stream discharge record are described in Section 2.2. The simulation results and discharge records are compared in Section 2.3. System configurations for specific uses and practical considerations of those systems are discussed in Section 2.4.

2.2 Examination of residential stormwater detention with a simulation model and hydrologic records of rainfall and runoff

A simulation model was developed to examine the performance of different detention system configurations under historical rainfall patterns. The user specifies the reservoir volume, maximum release rate, and pattern of release. The model uses a simple mass balance to calculate: (1) the runoff production from a specified roof area for a given rainfall record, and (2) the storage, release, and overflow volumes from a detention system based on a linear reservoir model where release is equal to the specified rate multiplied by the fraction of the reservoir that is full. The program calculates storage, inflow, release, and overflow at a 15 minute time step assuming an instantaneous response from the roof (i.e., no flow routing) with no evaporation or depression storage losses.

Replication of the rates and pattern of discharge from a forested catchment can serve as an objective for stormwater detention systems used in the Puget Sound region. The detention simulation model allows for comparison of the rainfall-release response of different residential stormwater detention systems with the response of a forested catchment. In particular, the simulated release patterns are compared with measured discharge from a small first-order forested catchment, "Novelty Hill," on the Sammamish Plateau in King Co., WA.

Novelty Hill is a 37 hectare, zero-order basin located 25 km northeast of Seattle, WA. It is at latitude 47° 42' N and longitude 122° 01' W (T26N R6E 34). The catchment is oblong, 1000 m long and an average of 366 m wide. The upper areas of the basin have side slopes 3 to 5% while the lower basin side slopes are between 1 and 2%. A swale runs through the center of the basin for approximately 800 m upstream from its outlet. A second growth forest covers the basin which was logged around the turn of the century. Soils in the basin are classified as Alderwood gravelly sandy loam soils underlain by Vashon till. An observed soil profile consisted of "forest litter and a dark brown organic layer about 0.15 m thick, underlain by brown sand to silty sand with minor amounts of gravel and cobbles. The sandy layer typically extends to a depth of 0.6 to 1 m and overlies the dense Vashon till." (Wigmosta et al., 1994).

Wigmosta et al. (1994) monitored rainfall and runoff for Novelty Hill. The stormwater detention simulations use the Novelty Hill rainfall record for the period from 1 October 1990 to 30 June 1993 (the period of analysis). The record is particularly useful because it demonstrates year-to-year variability in precipitation patterns. In Water Year 1991, a total of 1.33 m of rain fell during 191 rainy days. There were two very large events: one in November 1990 and one in April 1991. Wigmosta et al. (1994) estimated that the 24 hour rainfall on 24 November 1990 had approximately a 50 year return period. In Water Year 1992, a total of 1.0 m of rain fell during 167 rainy days. There was one large event in January 1992. Water Year 1993 (truncated to 30 June 1993) was relatively dry with a total of 0.91 m of rain over 162 wet days.

The recorded discharge from Novelty Hill for the same period of analysis is used to provide a predevelopment scenario of storm flow generation from a forested catchment. The Novelty Hill discharge record has been normalized for a 100 m^2 area of the basin to allow for comparisons with the detention simulations.

The recorded discharge from Novelty Hill is not a perfect benchmark, however, for comparison with the roof catchment used in the residential stormwater detention simulations for two reasons. First, the measured stream discharge from Novelty Hill is from an area 3700 times the area of the roof (100 m^2) used in the detention simulations. Time series of storm flow rates for many 100 m^2 forested areas in the Puget Sound region would be a better benchmarks, no such data are available.

While normalizing the stream discharge for a 100 m^2 forested area addresses part of this problem, delayed runoff response from uplands up to 1 km away from the weir used to measure discharge is reflected in the record. The net effect is uncertain especially when the measured discharge and simulation results are examined at a daily time scale (i.e., discharge volume per day). There is likely to be some variability (higher storm peaks and more rapid recessions) in the hydrographs from the detention simulations that can be

There are also two gaps in the Novelty Hill stream flow record accounting for 64 days (or 6.4% of the period of analysis): 17 January 1993 - 5 February 1993 and 31 March 1993 - 14 May 1993. During these periods, there was 0.25 m of rainfall recorded at Novelty Hill with a maximum daily total of 29.7 mm. This represents 7.6% of the total rainfall for the period of analysis. Discharge for these periods was simulated using the model developed by Wigmosta et al. (1994). The normalized volume of simulated discharge during this period is 8.4 m^3, which is 12% of the total runoff volume for the 2.67 years of record.

2.3 Simulations of residential stormwater detention systems

Nine simulations are presented here to demonstrate mitigative measures and supplemental water supply capabilities of on-site residential stormwater detention systems. The roof area is constant (100 m^2) for all detention simulations. Recorded rainfall depths are multiplied by the roof area to produce a roof runoff volume. The runoff is released using a linear reservoir model (i.e., release is equal to the specified release rate multiplied by the percent of reservoir that is full) without consideration of the hydraulics of the release. If the runoff in any time period exceeds the available storage and release, the excess stormwater overflows. The discharge from the system is the sum of release and overflow. All calculations (rainfall volume, storage volume, release volume, overflow volume, total discharge volume) are made at a 15 minute time increment.

The "no reservoir" simulation provides both the rainfall hyetograph and a baseline of uncontrolled roof runoff for the period of analysis where runoff rate (in liters/hr) is equal to rainfall rate (in mm/hr) multiplied by 100. The other eight simulations have different reservoir volumes and release rates which are summarized in Table 2.1 below.

Table 2.1: Configurations used in simulations of residential stormwater detention systems

Reservoir volume	Release rate	Release Pattern
0 m^3	0 m^3/day	continuous
0.5 m^3	0.5 m^3/day	continuous
0.5 m^3	1 m^3/day	continuous
2 m^3	0.1 m^3/day	continuous
2 m^3	0.5 m^3/day	continuous
2 m^3	1 m^3/day	continuous
10 m^3	0.1 m^3/day	continuous
10 m^3	0.5 m^3/day	continuous
10 m^3	0.5 m^3/day	June - September

The reservoir volumes in the simulations range from a small tank (0.5 m^3) which would be used to control stormwater generated by small, frequent storms to a large tank (10 m^3) that would provide a water supply in addition to stormwater control.

In four of the simulations, the maximum release rate of 0.5 m^3/day was selected to limit peak discharge rates from detention systems. This is equivalent to a specific discharge of 0.0083 cfs/acre which occurs frequently (i.e., many times in most years) for basins in the Puget Sound lowlands. The effects of flows at this rate on a stream will depend on channel bed materials, the duration of flows at this rate, flow patterns at other times, and the stream biota. For the purposes of this report, 0.5 m^3/day is considered to be the upper limit of the lowest flow rate class.

The other simulations have a 0.1 or 1 m^3/day maximum release rate. The lower rate represents a more conservative stormwater control strategy and could only be used when adequate storage is available since this release rate is equal to the runoff generated by 1 mm/day of rain. One simulation is given when there is no release during the winter months; water is supplied from this reservoirs at 0.5 m^3/day for supplemental irrigation from June - September.

2.4 Comparisons of residential stormwater detention systems

Residential development increases the volume of quick flow response from the land surface, increases the frequency and duration of high flow rates experienced by channels, reduces the volume of subsurface flow and groundwater recharge when compared to the predevelopment state. Consequently, the efficacy of residential stormwater detention systems in mitigating the adverse hydrologic effects of urbanization can be characterized by comparing the pre-development and post-development volumetric discharges at different flow rates from a given area. Likewise the duration of discharges at different flow rates can be compared for different systems. For the residential stormwater detention systems simulated here, discharge refers to the combined flow of controlled release and unregulated overflow.

The results of each simulation are provided cumulatively for the period of analysis and at daily and hourly scales. While cumulative results (e.g., volume and duration of discharge from detention systems) can be useful for gross comparisons of detention system configurations, daily information is needed to examine system behavior for large storm events. Hourly discharge rates are necessary to account for short (i.e., less than one day) periods of high-rate discharge.

"Rate class" charts (Figure 2.1 and 2.2) have been provided to summarize the hydrographs of daily discharge for the detention system simulations. In these two figures, the total simulated volume and duration of discharges from detention systems for the period of analysis have been summarized in four ordinal rate classes: 0 - 0.5 m^3/day; 0.5 - 1 m^3/day; 1-2 m^3/day; and >2 m^3/day. Figure 2.1 displays total volume discharged in each rate class from the systems and Figure 2.2 displays total duration of discharges in each rate class from the systems. The numerical values for these figures are presented in Table 2.2.

The purpose of these figures is to provide a means for comparing the duration and volume of discharges from detention systems. Those systems that discharge large volumes of stormwater at high rates are likely to degrade stream channels more than systems that discharge small volumes at high rates. Conversely, those systems that have low discharge rates for long durations are likely to improve base flow conditions in streams.

The rates used to define the rate classes can have a strong effect on the apparent results. If this rate class approach were used to analyze detention systems in an actual basin and stream system, the rate classes could be calibrated to threshold flows for stream channel de-stabilization, flooding, slow subsurface drainage, or any other process of interest. For example, the highest class could be calibrated roughly to indicate discharges from detention that, when scaled up for the basin, are above the critical velocity for bedload transport (McCuen, 1979). Alternatively, the middle classes could be related to flow rates of runoff from frequent storms when the hydrologic effects of urbanization are most pronounced (Hollis, 1975).

Table 2.2: Discharge volume and duration summary for a forested catchment and nine residential stormwater detention systems (rainfall record from Novelty Hill, 1 October 1990 to 30 June 1993)

Reservoi Volume (m^3)	Release Rate (m^3/day)	Volumetric discharge in each rate class (m^3)				Duration of discharge in each rate class (days)				% days in period of analysis with flow
		0-0.5 m^3/day	0.5-1 m^3/day	1-2 m^3/day	$\geq$ 2 m^3/day	0-0.5 m^3/day	0.5-1 m^3/day	1-2 m^3/day	$\geq$ 2 m^3/day	
Novelty Hill[a]		48.19	12.96	9.82	0.00	471	20	12	0	50%
0	0	54.33	81.10	111.99	76.86	303	110	79	28	52%
0.5	0.5	88.98	76.00	93.32	65.89	767	105	66	24	96%
0.5	1	78.09	82.51	97.90	65.77	656	116	70	24	86%
2	0.1	94.26	63.46	94.65	70.07	826	84	68	26	100%
2	0.5	174.75	38.68	68.97	40.82	890	53	47	14	100%
2	1	130.01	105.37	65.60	22.85	790	159	46	6	100%
10	0.1	102.94	60.46	92.43	58.62	836	81	66	21	100%
10	0.5	262.91	16.51	23.98	14.20	962	22	16	4	100%
10	0.5[b]	117.77	59.59	85.91	54.33	581	81	60	19	74%

[a] Novelty Hill stream discharge has been normalized for a 100 m^2 contributing area.

[b] Releases from storage are from 1 June to 30 September

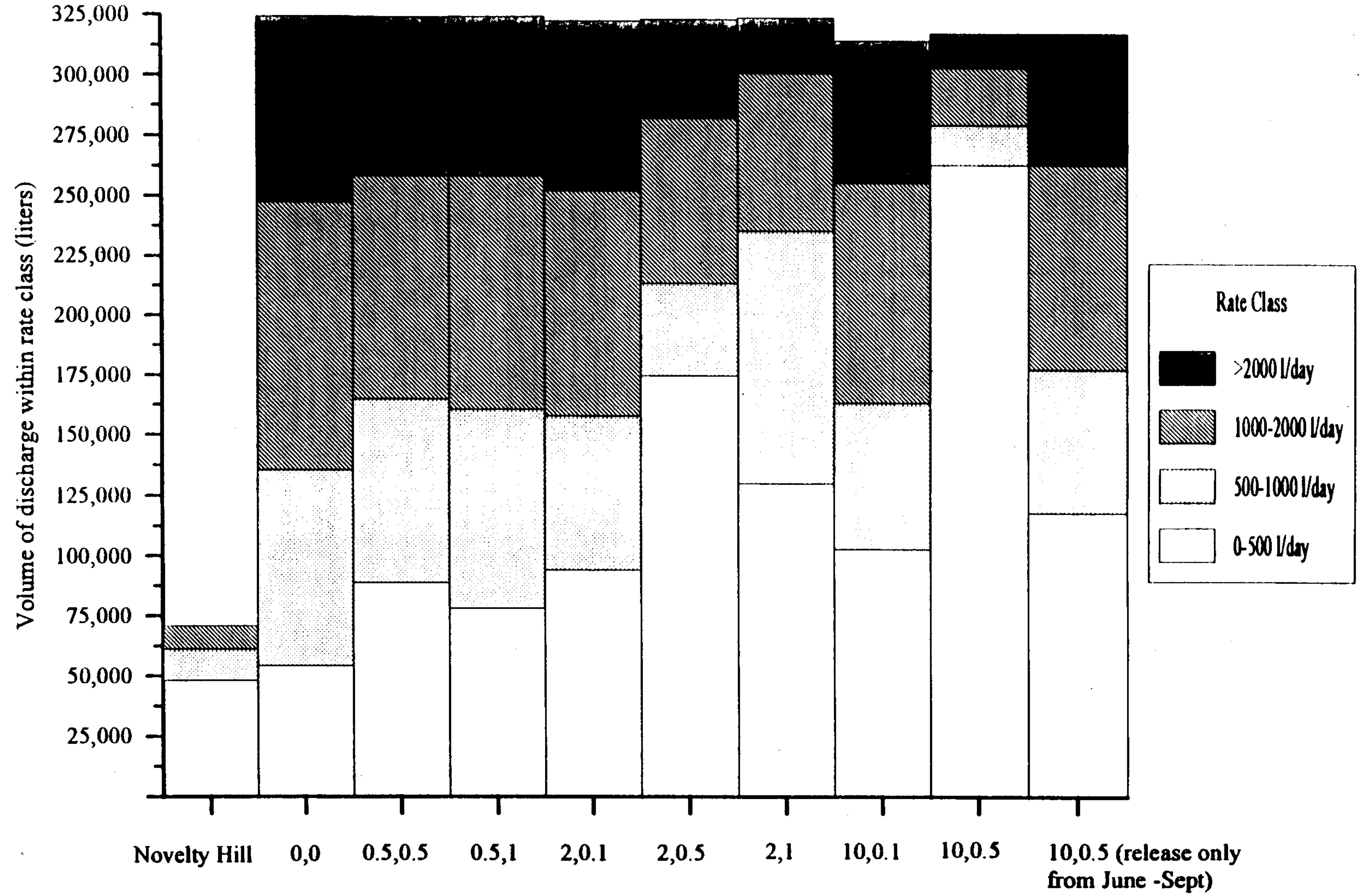

Figure 2.1: Comparison of discharge volume for residential stormwater management systems in four rate classes. Systems are labeled by reservoir volume (in cubic meters) and release rate (cubic meters per day). Rainfall recorded at Novelty Hill, 1 October 1990 to 30 June 1993. Roof area is equal to 100 square meters.

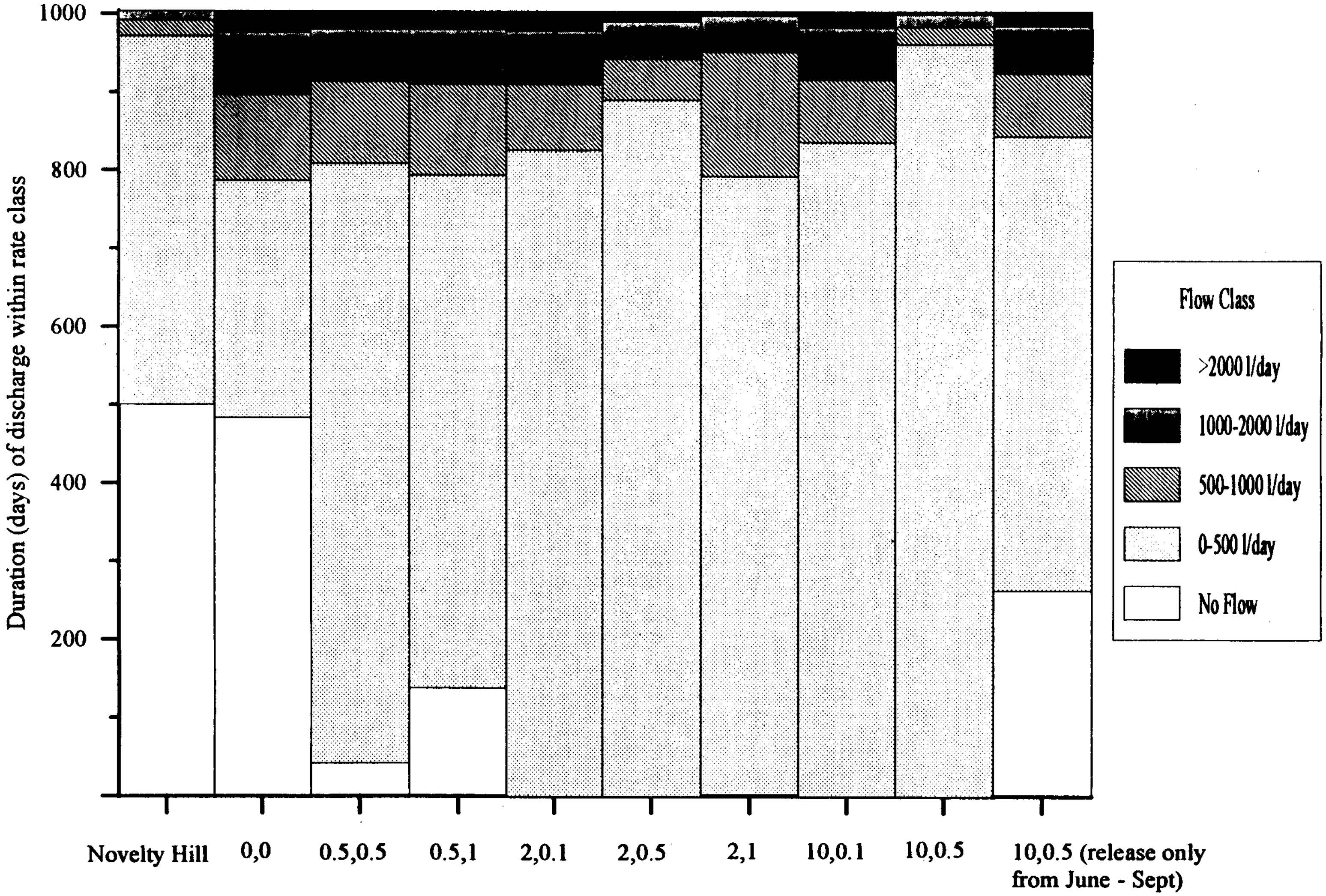

Figure 2.2: Comparison of discharge duration for residential stormwater management systems in four rate classes. Systems are labeled by reservoir volume (in cubic meters) and release rate (cubic meters per day). Rainfall recorded at Novelty Hill, 1 October 1990 to 30 June 1993. Roof area is equal to 100 square meters.

The recorded discharge from Novelty Hill and the residential stormwater detention system results are presented below. Novelty Hill and the "no reservoir" simulation provide benchmarks for evaluating the level of quick response runoff control provided by a residential stormwater detention system. As well as stormwater control, the potential for a system to provide water supply is assessed using the number of days in the period of analysis that the system releases water.

2.4.1 Novelty Hill rainfall

The Novelty Hill hyetograph (rainfall record) is shown in Figure 2.3. The hyetograph (in mm/day) is equivalent to the hydrograph for the "no reservoir" simulation where 1 mm of rain generates 100 liters of runoff for a roof with a plan area of 100 m^2. Using units of 100 l/day (rather than mm/day) , the record serves as the inflow hydrograph to the detention systems for the period of analysis. The general climatic pattern of sequential storm events during the winter and relatively dry summers is evident from the hyetograph. The largest storms during the period of analysis occurred in November, 1990 and April, 1991. These storms produced peak discharges of 6,000 l/day in the simulation. Hydrographs for Novelty Hill and three of the simulations also are provided in Figure 2.3.

The hyetograph indicates the high frequency with which uncontrolled roof runoff would exceed 1000 l/day (i.e., those days with more than 10 mm/day of rain). Much of the runoff (24% or 77,000 l) would exceed a rate of 2,000 l/day. High rate discharges lasted for 2.8% of the time (28 days) during the period of analysis (1000 days). Table 2.2 provides other summary information for the volume and duration of discharges for the "no reservoir" situation.

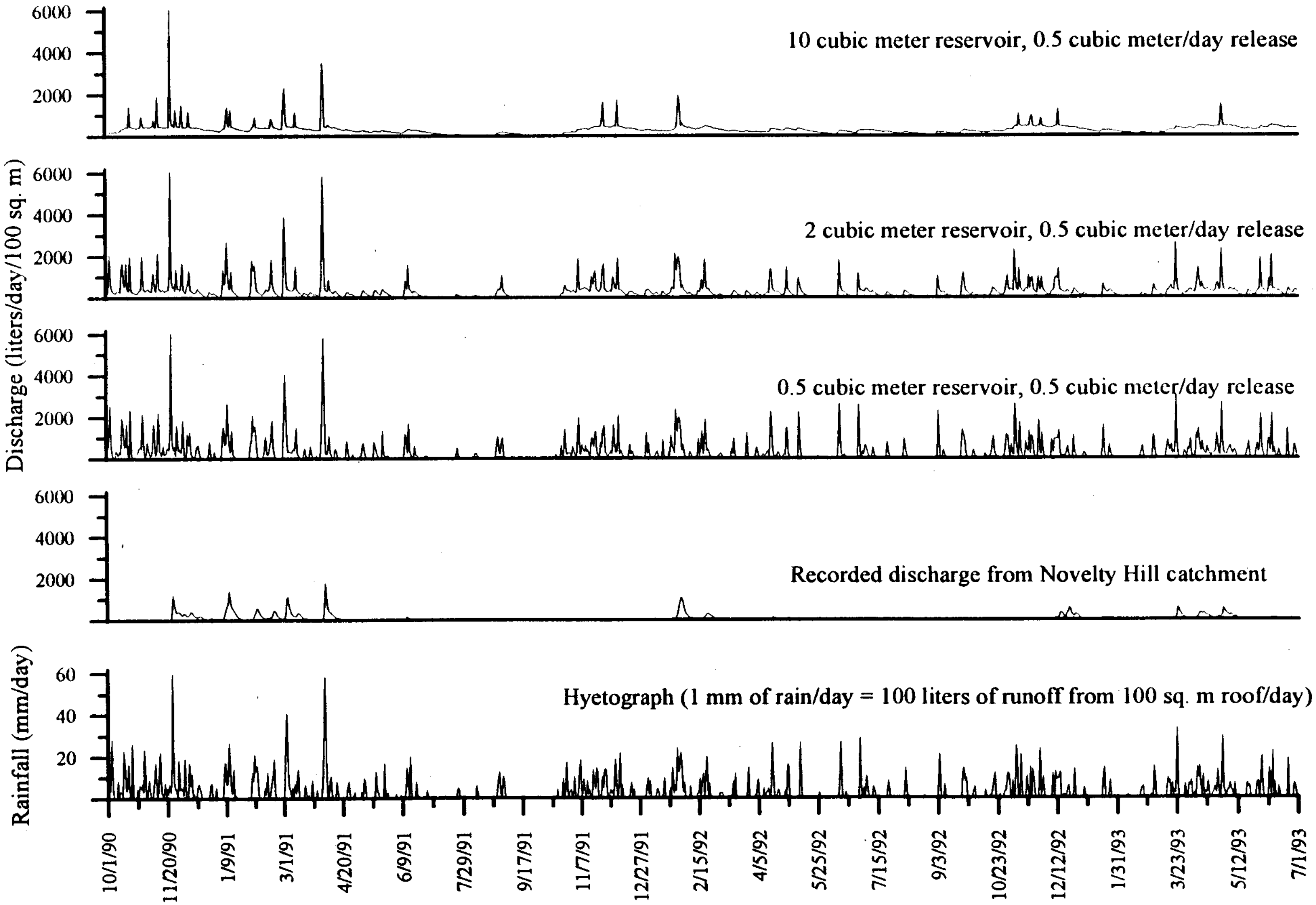

Figure 2.3: Hyetograph for Novelty Hill and hydrographs of daily discharge for Novelty Hill and selected residential stormwater detention simulations for the period 1 October 1990 to 30 June 1993

2.4.2 Novelty Hill stream discharge record

The Novelty Hill stream discharge record illustrates natural storm response from a zero- and or ephemeral first-order basin in the Puget Sound region for the period of analysis. The flow rate from the central swale was recorded by Wigmosta et al. using a weir placed in the till to capture any surface or shallow subsurface flow. The water level (stage) behind the weir was recorded at 15 minute intervals. The rate of discharge was estimated from a stage-discharge curve for the weir to provide a time-series of stream flow rate.

When the Novelty Hill discharge record is normalized for a 100 m^2 area and reduced to measures of cumulative volume discharged and duration of discharge, the results are striking. This forested catchment generated 71 m^3 of discharge per 100 m^2 area over the period of analysis. This represents only 22% of the rain that fell on that area whereas almost 100% of the rain falling on a roof would be expected to become storm flow. The precipitation not accounted for in stream flow was transpired, evaporated, or recharged groundwater beneath the till (Wigmosta et al., 1994).

The hydrograph of normalized daily discharge from Novelty Hill is provided in Figure 2.3. The discharge rate typically was much less than 1000 l/day/100 m^2 with higher rates recorded only during five storm events for the period of analysis. The peak discharge (1780 l/day/100 m^2) was during a large storm in April, 1991. There is no stream flow for long periods during the summer. Stream flow also lags the onset of winter rains.

The rate class summaries of discharge volume and duration for Novelty Hill are compared to the residential stormwater detention simulations in Figure 2.1 and Figure 2.2. These figures show that the total normalized Novelty Hill discharge rate exceeded 1 m^3/day for only 12 days during the period of analysis with only 9.8 m^3 of runoff discharged in this rate class (see Table 2.2). This is much lower than the rain input to the basin which exceeded 1 m^3/day on 107 days for a total volume of 189 m^3 of rain entering the basin at or above this rate. There were only a few days when the runoff rate from the basin approached the rainfall rate multiplied by the basin area. The results indicate the hydrologic effect of a forest cover: a smaller volume of stormwater is generated by a forested catchment (unless the soil column is saturated or close to saturated by a previous storm) and the storm flow rate of runoff is relatively low when compared to stormwater generated in an urban catchment.

The Novelty Hill discharge record and the "no reservoir" simulation results provide benchmarks for evaluating the other residential stormwater detention system simulations which are given below.

2.4.3 Results of residential stormwater detention simulations

All of the detention systems will reduce the volume of runoff generated in the highest rate class (>2 m^3/day) but only the largest one (10 m^3 reservoir with a 0.5 m^3/day release rate) can control high rate flows at a level comparable to a forested catchment. All of the systems increase the duration of flows and the duration of flows in the lowest rate class (0-0.5 m^3/day). A relatively high, continuous release rate is needed to prevent any of the systems from becoming full and overflowing. For example, a 2 m^3 reservoir with a 0.5 m^3/day release rate provides a better reduction in the volume and duration of high rate flows (>2 m^3/day) than a 10 m^3 reservoir with a 0.1 m^3/day release rate.

Small systems can be effective at reducing quick response runoff during frequent, moderate intensity storm events. Longer storms that produce substantial rainfall depths (>20 mm) will fill small reservoirs and cause overflows. The performance of a small reservoir can be enhanced by increasing the release rate: a 2 m^3 reservoir releasing water at 1 m^3/day reduces the volume of high-rate discharges to 23 m^3 and the duration of high-rate discharges to 6 days.

The 10 m^3 reservoir that releases stormwater at a rate of 0.5 m^3/day only during summer for irrigation is only as effective as the smallest continuous release system (0.5 m^3 reservoir) in reducing the volume of stormwater discharged in high rate class. A 50 - 100 m^3 reservoir is needed to provide adequate stormwater control if no water is released during the winter.

2.4.4 Comparison of hydrographs for selected residential stormwater detention systems

Simulated hydrographs have been provided for three selected residential stormwater detention systems (Figure 2.3). These hydrographs show daily discharge for the period of analysis. The increase in the frequency of higher rate discharges as the reservoir volume is reduce is evident. As storm duration, intensity, or frequency increase, all of these detention systems provide less control of peak flows.

Hydrographs for large storm events can also be used to characterize the performance of residential detention systems. Three different storms from the period of analysis had large rainfall depths, high rates of rainfall, and sequential events typical for the Puget Sound region. There storms were:

24 - 28 November 1990 (Fig. 2.4),
3 - 9 April 1991 (Fig. 2.5), and
20 January - 3 February 1992 (Fig. 2.6)

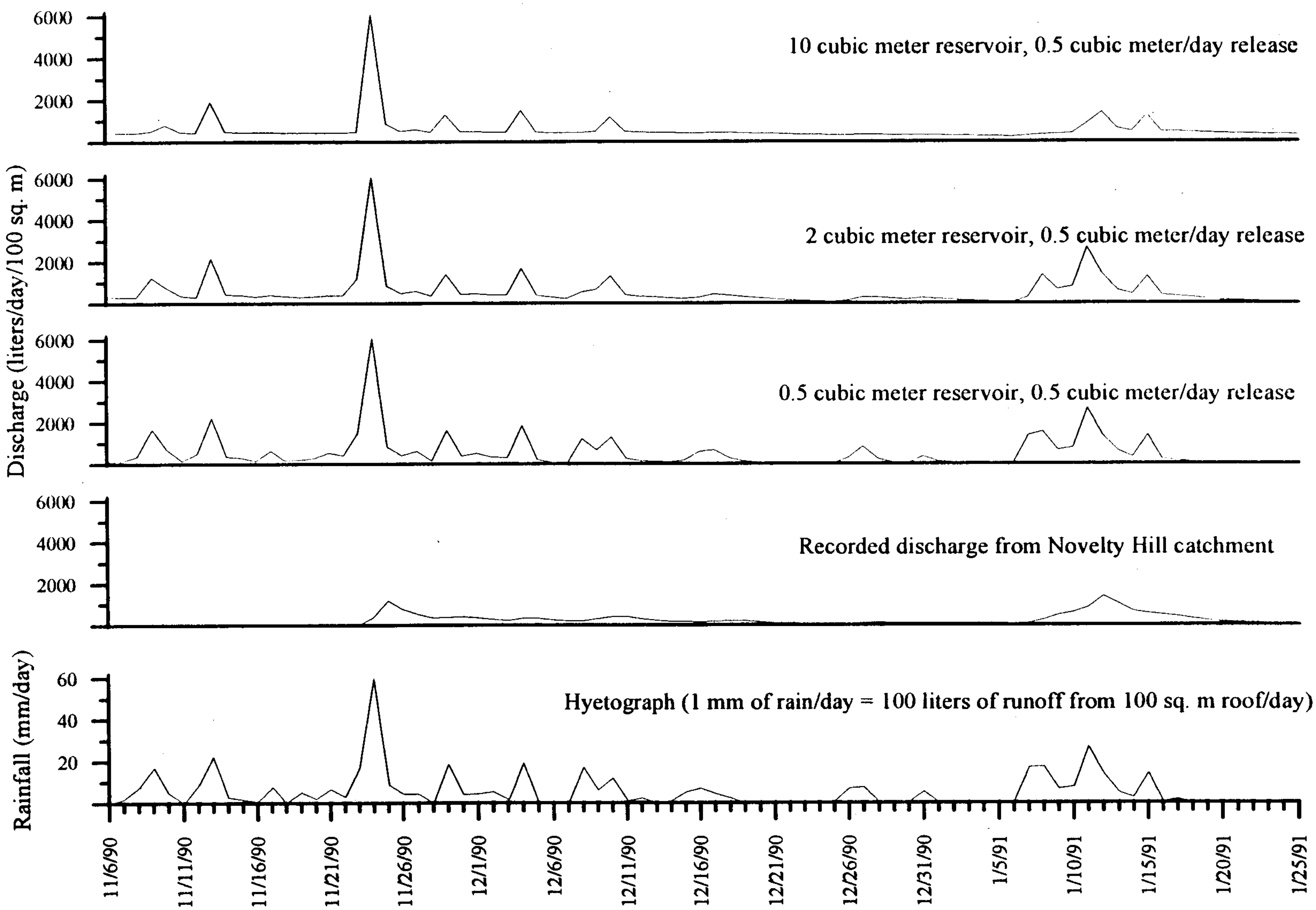

Figure 2.4: Hyetograph for Novelty Hill and hydrographs of daily discharge for Novelty Hill and selected residential stormwater detention simulations for the period 6 November 1990 to 25 January 1991

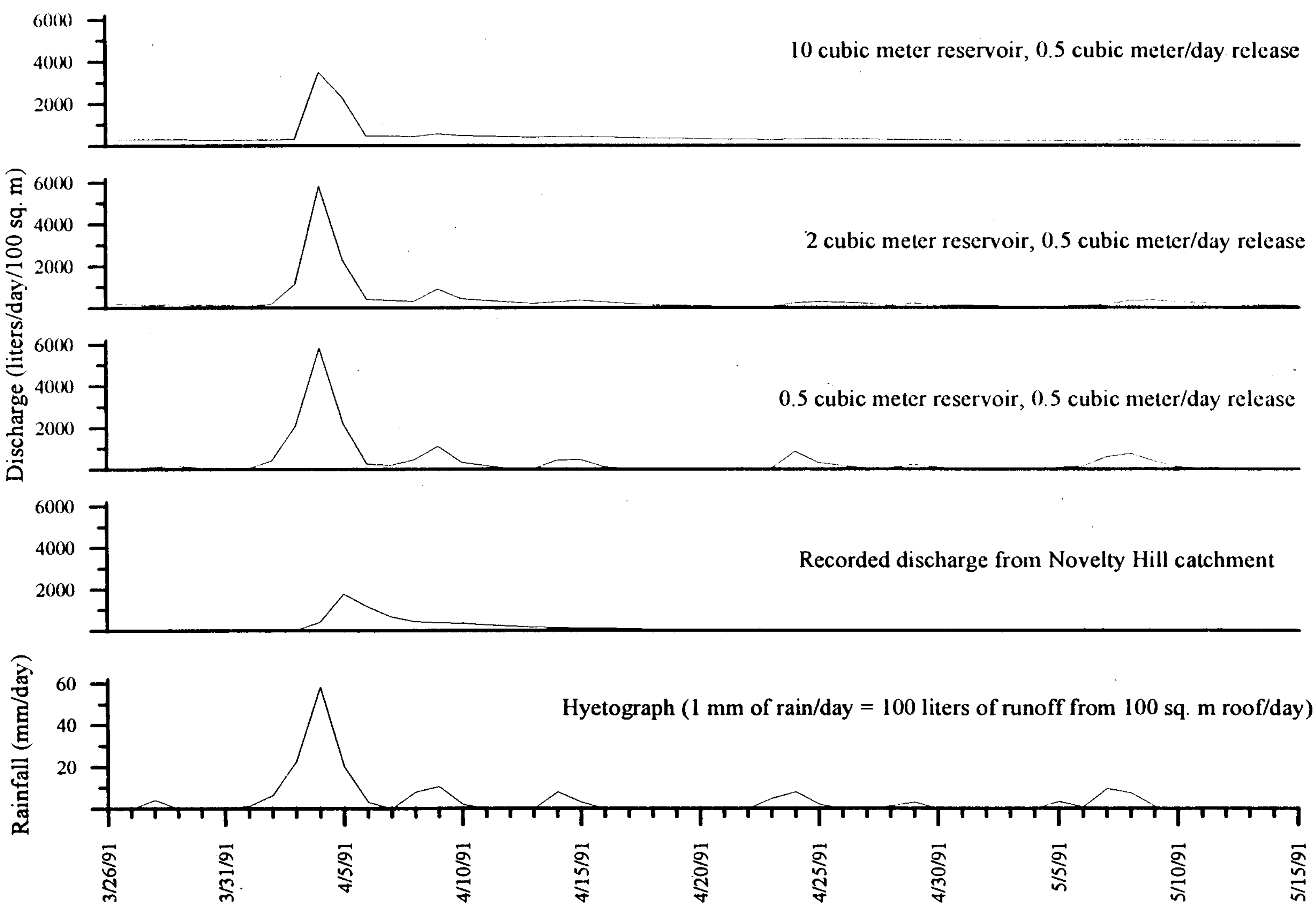

Figure 2.5: Hyetograph for Novelty Hill and hydrographs of daily discharge for Novelty Hill and selected residential stormwater detention simulations for the period 26 March to 15 May 1991

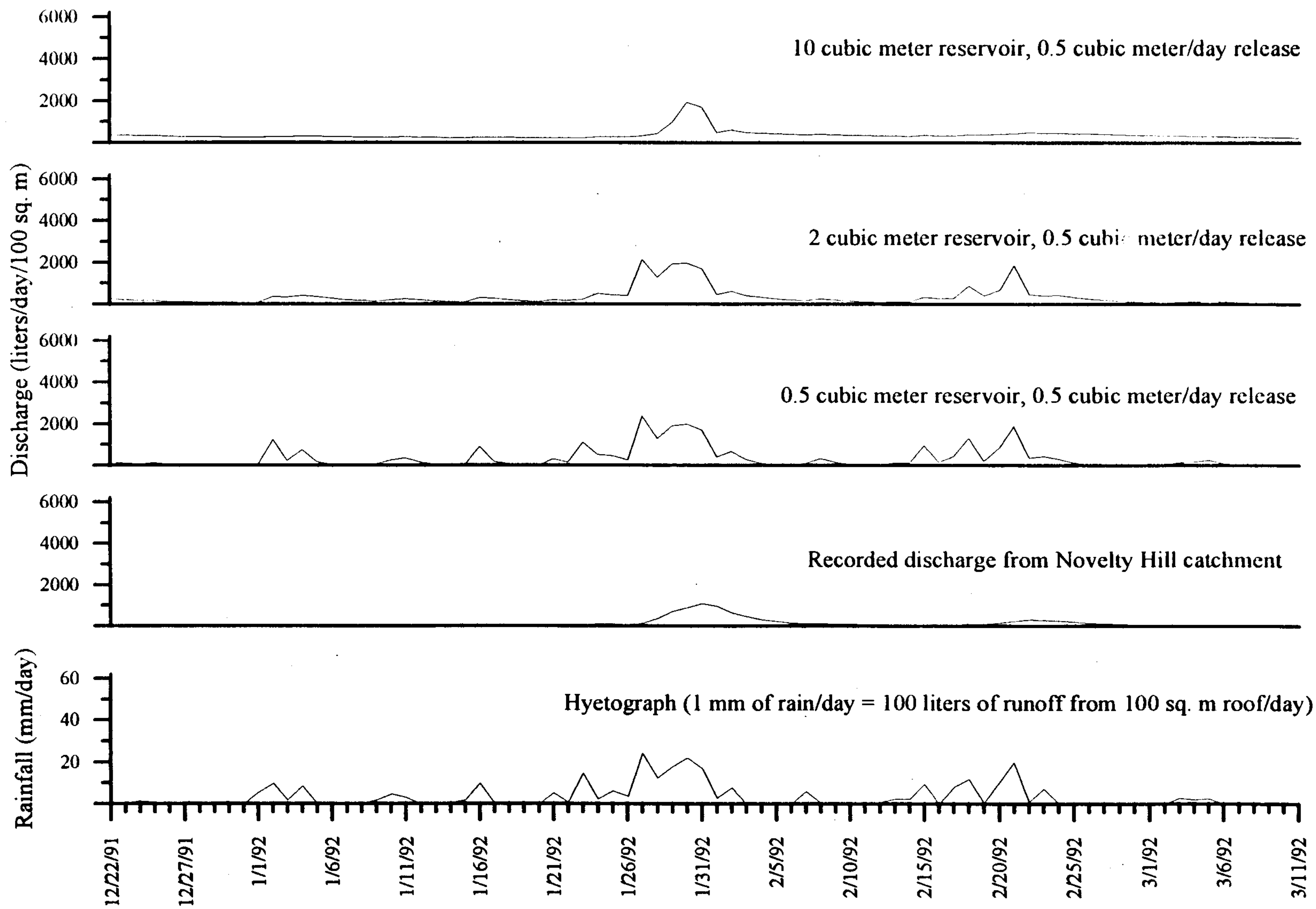

Figure 2.6: Hyetograph for Novelty Hill and hydrographs of daily discharge for Novelty Hill and selected residential stormwater detention simulations for the period 22 December 1991 to 11 March 1992

The November 1990 and April 1991 storm volume were uncommonly large while the January 1992 event illustrates a more common storm. Discharge hydrographs from three of the detention systems and Novelty Hill are included in these figures.

The detention systems show little differentiation in the November 1990 storm (Figure 2.4). The large volume of stormwater generated in this storm would have filled any of these reservoirs causing all of them to spill at rates equivalent to uncontrolled roof runoff. A "10 m^3 reservoir, 1 m^3/day release rate" system (not shown) limits the rate of runoff during this event to 1 m^3/day unlike the "10 m^3 reservoir, 0.5 m^3/day release rate" system. While this demonstrates the effect of a high release rate on maintaining storage capacity, a low release rate will better mitigate the deleterious hydrologic effects of urbanization in most storms.

All of these detention systems discharged rates an order of magnitude higher than the peak rate of discharge from the Novelty Hill catchment. The significance of antecedent conditions are evident in a comparison of the Novelty Hill runoff response between November and January. In January, a less intense storm produced more runoff from the catchment than did the November storm probably because of the greater extent of saturated soils in the basin during January.

The runoff response of the four residential stormwater detention systems is more varied during the April 1991 storm (Figure 2.5). Each system's capacity to control discharge rates is determined by its reservoir volume. Again, the Novelty Hill discharge is much lower and lags behind the discharge from the detention systems.

The January 1992 storm demonstrates the influence of storm patterns on system performance for moderate rainfall depths (Figure 2.6). Storm patterns can be characterized as periods of more intense rainfall rates (>2.5 mm/hr) separated by 12 to 36 hours. Totals for distinct storm events were calculated from the rain record and are included here.

All of the reservoirs completely store the 6 mm storm event on 21 January. The next storm event (22 mm) on 23 January causes the 0.5 m^3 reservoir to spill. By the time the next small event (9 mm) arrives, this system has drained and can contain the roof runoff. The large event (87 mm) during 27-29 January causes all systems to spill. The rainfall on 30-31 January arrives to soon after the 27-29 January event to be contained by any of the systems. This example illustrates the influence of rainfall depth and the time between storms on residential detention system performance. It also demonstrates the utility of continuous time series/multiple-storm analyses.

2.4.5 Domestic supply

Detention systems can supply water to residences in addition to regulating storm flow patterns. Systems can either supply water steadily during the year or store water for use in dry summer periods. To supply a residence with water for regular uses, a multipurpose detention system must have a relatively large reservoir and a low release rate. For the analysis of supply, the detention system model was programmed to release water at a constant rate rather than in proportion to water available in the reservoir (i.e., the linear reservoir model used above). Neither a "10 m^3 reservoir, 0.5 m^3/day release" configuration nor a "25 m^3 reservoir, 1 m^3/day release" configuration will provide a dependable supply in the Puget Sound region.

A system with the low release rate (0.5 m^3/day or 132 gallons/day) could provide all of the water to meet an average residence's physical water demand for only part of the year and only if sufficient storage is available. For example, a 10 m^3 reservoir would provide 0.5 m^3/day for 577 days (57% of the time) over the period of analysis. The system supplied water most days during winter but became more unreliable during the spring, summer, and autumn. A larger reservoir will not increase the reliability of this system as a supply of water because this system (10 m^3 reservoir, 0.5 m^3/day release) stores and releases essentially all (96% for the period of analysis) of the roof runoff. A larger collection area is needed to supply a suitably sized reservoir to provide for summer use.

The reliability of the water supply from a multipurpose detention system can be increased by reducing the rate of water use. The maximum sustainable supply rate over the period of analysis is 0.32 m^3/day and requires a 56 m^3 reservoir which represents 175 days of supply. This flow rate may be less than daily water use in many residences. It appears that, at best, a detention system for a 100 m^2 roof could supplement residential water supply by meeting all demand for part of the year or part of demand all of the year.

Residential water use in the Puget Sound region increases by as much as 50 % during summer months (Seattle Water Department, 1992). Much of the increase is due to lawn watering and other irrigation of landscaped areas. To deliver water for summer irrigation, a detention system must have a large volume and begin the summer full of water.

For the two "summer irrigation" systems, a simple appropriation rule (supply water only June through September) has been simulated though it would be possible to release water during winter for stormwater control and store it in the spring for irrigation supply. A summer irrigation rate of 0.5 m^3/day is used in both simulations. At this rate one could irrigate the equivalent of a 140 m^2 (1500 ft^2 or 0.035 acre) lawn with no natural storms from the period of June through September assuming an irrigation rate of 25 mm/week (Seattle Water Department, 1993) applied to lawn areas.

Winter rains on a 100 m^2 roof will almost certainly fill a 25 m^3 reservoir. Releasing water at 0.5 m^3/day beginning in June, the detention system would provide water until July 20. A 50 m^3 reservoir with the same release rate would provide a supply into September, though summer storms may replenish the supply so that water is available through October.

Neither of these system would be adequate for irrigating the expansive lawn area of many new homes. For example, the average lawn area per lot at a residential development on the Sammamish Plateau in King Co., WA is approximately 0.1 ha (1000 m^2) or 10 times the area covered by roofs (Wigmosta et al., 1994). If there were multipurpose detention systems providing 50 m^3 of stormwater storage for every 100 m^2 of roof area (garages and homes) in the development, the maximum lawn area that could be dependably irrigated during the summer is 6.3 ha (or 53% of the lawn area in the development).

2.5 Summary of residential stormwater detention systems

The results of these simulations suggest that detention systems must be planned carefully to be multipurpose. A system with a large volume and slow release rate controls the duration and volume of high rates of storm flow generated by a roof, though no simple detention system can reduce the total volume of runoff to the level observed at Novelty Hill where only 22% of the rainfall volume was discharged as stream flow. Large volume, slow release systems can also supply water for domestic uses during much of the year, though the level of stormwater control provided by these systems drops when stormwater is stored in them rather than released.

Other system configurations provide limited benefits but fail to achieve other purposes. For example, a small reservoir with a low release rate can prolong storm flow recession as would be expected from a forested catchment but it will not control peak discharge rates (in terms of volume of high rate discharge or duration of high rate discharge). The minimum reservoir size and maximum release rates for four purposes described above can be divided by the roof area to obtain storage and release quantities in terms of the depth of equivalent for any catchment area:

1. Increase duration of low flow - 20 mm of storage, 5 mm/day release rate;
2. Control peak flow duration and volume - 100 mm of storage, 5 mm/day release rate;
3. Supplement regular domestic uses - 20 days of storage (at the residence's daily use) for supply during winter (the example above used 100 mm of storage over the roof area and a supply rate of 5 mm/day); and
4. Supply irrigation water during the dry season - 250 mm of storage for the area being irrigated.

All of the above analyses have been done for a relatively short (less than 3 years) record of measured rainfall and stream flow for the Sammamish plateau region in King Co., WA. Comparable analyses can be made at any location provided continuous precipitation data are available. Few stream flow records exist for zero-and first-order basins to permit comparison between pre- and post-development conditions.

Chapter 3: Residential stormwater infiltration systems

Infiltration of stormwater into the ground at a location other than which it was produced is another management strategy used to control stormwater generated in residential areas. In the lowland around Puget Sound, WA, infiltration systems in use range from large ponds serving residential developments to gravel pockets located at the outlet of roof downspouts (Hilding, 1993). In this chapter, on-site infiltration systems are considered as alternatives for managing the roof runoff from a single residence. An overview of infiltration systems is given. This is followed by analytical approaches to assessing the flow of water through infiltration systems. Finally, field investigations of two residential infiltration systems are described.

3.1 System function and components

Infiltration at the land surface is "the downward entry of water into soil" (Richards et al., 1952). As such, infiltration systems are designed to collect stormwater and promote its entry into the soil. By diverting stormwater from a central conveyance network and providing an opportunity for it to enter the soil, infiltration can mitigate some of the deleterious hydrologic effects of urbanization. Stormwater infiltration systems can reduce the volume and peak rates of quick-response storm flow by increasing subsurface flow in the unsaturated zone. In some areas, this may recharge groundwater and increase base flow in streams. If infiltration systems are used extensively, they could cause developed land areas to generate runoff in quantities and at rates that are similar to those occurring in the predevelopment state (Harrington, 1988).

On-site residential infiltration systems at the scale of a single residence are used to manage roof runoff and water collected by house foundation foot drains. These infiltration systems are often limited to managing runoff from these sources because runoff from other areas (e.g., roads, land surfaces) can transport sediments, which clog infiltration structures, and pollutants which can contaminate groundwater.

Large infiltration systems are used to manage runoff from roads or exposed land surface. Such infiltration systems are employed to remove pollutants from runoff rather than for managing the hydrologic effects of urbanization (Urbonas and Roesner, 1993). Infiltration systems receiving runoff with pollutants and sediments must be designed to remove pollutants and need to be cleaned periodically (Bouwer, 1989; Harrington, 1988). This may make them impractical for use at individual residential lots. The scope of on-site residential infiltration is restricted here to systems that collect stormwater from roofs and footing drains.

On-site residential infiltration systems convey water from roofs and drains into structures where the water infiltrates into the soil. "Infiltration structure" describes any part of a system storing water which drains freely to the soil. Residential infiltration structures include ponds, permeable pavement, trenches, dry wells, and gravel pockets. Surface infiltration structures (ponds and furrowed fields) are often used for managing large volumes of water (U.S. EPA, 1984). Subsurface structures such as wells and trenches are more common for residential applications with the possible exception of a downspout allowed to drain freely onto the ground surface (King Co., 1990).

3.2 Continuity analysis of infiltration systems

A general approach for analyzing an infiltration system is described here. The approach is fundamental to any evaluation of the performance of an infiltration system in achieving stormwater management objectives or in the development of general design criteria for infiltration systems. The shortfalls of purely analytical approaches are emphasized and point to the need for the field studies described in later sections.

The function of any infiltration system can be described in terms of the conservation of water mass. For water of constant density, the volumetric rate of inflow to the system minus the volumetric rate of outflow (leakage to soil) from the system in a given time increment equals the volumetric change of water stored in the system. The mass balance can be used to describe an infiltration system at an instant in time when inflow, outflow, and change in storage must be expressed as rates, or over a longer specified period of time (hour, day, week, etc.) when the terms are measured in terms of volume or mass. The inflow can be estimated using information about rainfall and the catchment. Changes in storage can be monitored if there is sufficient information about the infiltration structure and the quantity of water it holds over time. Outflow rate is the most difficult quantity to measure or estimate. Typically, it is estimated by using information about the infiltration structure and the soil. Estimation of volumetric inflow and outflow rates and storage change are discussed below.

3.2.1 Estimating inflow to an infiltration system as a function of measured rainfall rate

Rainfall over the catchment area determines the inflow to a system. By measuring rainfall rate and catchment area, the volumetric inflow rate can be calculated. As rainfall varies in both space and time, the inflow rate to a system is likely to be variable from minute to minute, hour to hour, and day to day. This variability has been neglected in many design approaches which may use only the total volume of rain from a design storm with a specific recurrence interval (King Co., 1990; State of Maryland, 1987; Schueler, 1987; Stahre and Urbonas, 1988). A continuous time series of rainfall is needed to evaluate how an infiltration system performs.

3.2.2 Calculating changes in storage

Infiltration systems must have sufficient capacity to store water when inflow volume (rainfall over the catchment area) is greater than outflow volume due to infiltration. Infiltration systems store water in structures such as pipes and the voids around fill material. The volume of water stored in a system can be calculated using measurements of water level in the infiltration structure assuming a known distribution of void space at all elevations. This assumption is more accurate for dry wells that have confining (no flow) boundaries and contain only water and air than for trenches or other structures that have irregular boundaries or are filled with heterogeneous material such as soil or rocks. Thus, calculations of storage and any changes in storage will have varying degrees of uncertainty depending on the type of system.

3.2.3 Outflow rate as a function of the infiltration structure geometry and construction

The infiltration flux, or outflow from a system, is controlled by the infiltration structure and soil characteristics which are discussed in the next section. The infiltration structure establishes the hydraulic grade necessary to move water through the system and into the soil. Hydraulic grade is the difference in mechanical energy of water flowing between two parts of the system. It has components of pressure and elevation gradients. (In a strict sense, the mechanical energy grade should be used. For water infiltrating soil, the flow velocities are so small that almost all of the mechanical energy in the fluid flow is associated with pressure and elevation. The hydraulic gradient is almost identical to the energy gradient so it is used in lieu of the energy gradient).The dominant factors affecting the hydraulic grade include: the difference in elevation between the free surface of ponded water and its contact with the soil (i.e., the piezometric head); head loss from flow through pipes; and the number and geometry of the outlets.

The piezometric head of stormwater stored in an infiltration structure indicates the potential energy of the water to flow. A given volume of stormwater stored in narrow (i.e., having a small horizontal cross-sectional area) structures will have greater piezometric head than the water in a wide structures. The role of piezometric head gradients in subsurface flow is discussed in Section 3.3.

The infiltration structure also influences the area through which infiltration may occur. Design standards typically use the bottom area of an infiltration structure, side areas, or the sum of these areas in calculations of infiltration fluxes (King Co., 1990; Harrington, 1988). Some standards allow only the side areas to be considered in infiltration calculations because of possible clogging of the bottom area. Bottom area clogging has been postulated based on observations of shallow, standing water in infiltration systems (Stahre and Urbonas, 1988).

The postulate that no water flows through the base of a trench or other structure and the solution using a constant rate of outflow through the sides of the infiltration structure, neglecting infiltration through the bottom area, do not reflect the physics of seepage from a well or trench. A serious omission in this approach is that the area through which water infiltrates changes with head. (An alternative physical explanation of observed standing water is provided in Section 3.3.)

3.2.4 Outflow rate as a function of soil-water movement

If a detention system has been constructed with ample capacity to store the runoff generated by a roof and then deliver it to the soil, the system's outflow rate will be limited only by the area of soil through which water infiltrates and the soil's infiltration capacity. Infiltration has been studied as a natural hydrologic process occurring at the soil surface. Outflow from a stormwater infiltration structure, however, often occurs below the soil surface. As a result, outflow from infiltration systems is governed by the characteristics of subsurface soil-water movement and the geometry of the outflow surface.

A hypothetical infiltration trench can be used to demonstrate some of the principles of soil-water movement affecting the outflow rate from infiltration systems. A cross-section of the trench is provided in Figure 3.1. Roof runoff enters the pipe in the trench and discharges through orifices. The water drains through the large spaces between rocks in the trench until it encounters the native soil. In this figure, the volumetric rate of inflow exceeds the volumetric rate of outflow, so standing water is building up in the bottom of the trench. The influence of soil permeability, saturated and unsaturated soil hydraulic conductivity, the hydraulic gradient, and water temperature on subsurface flow are discussed below.

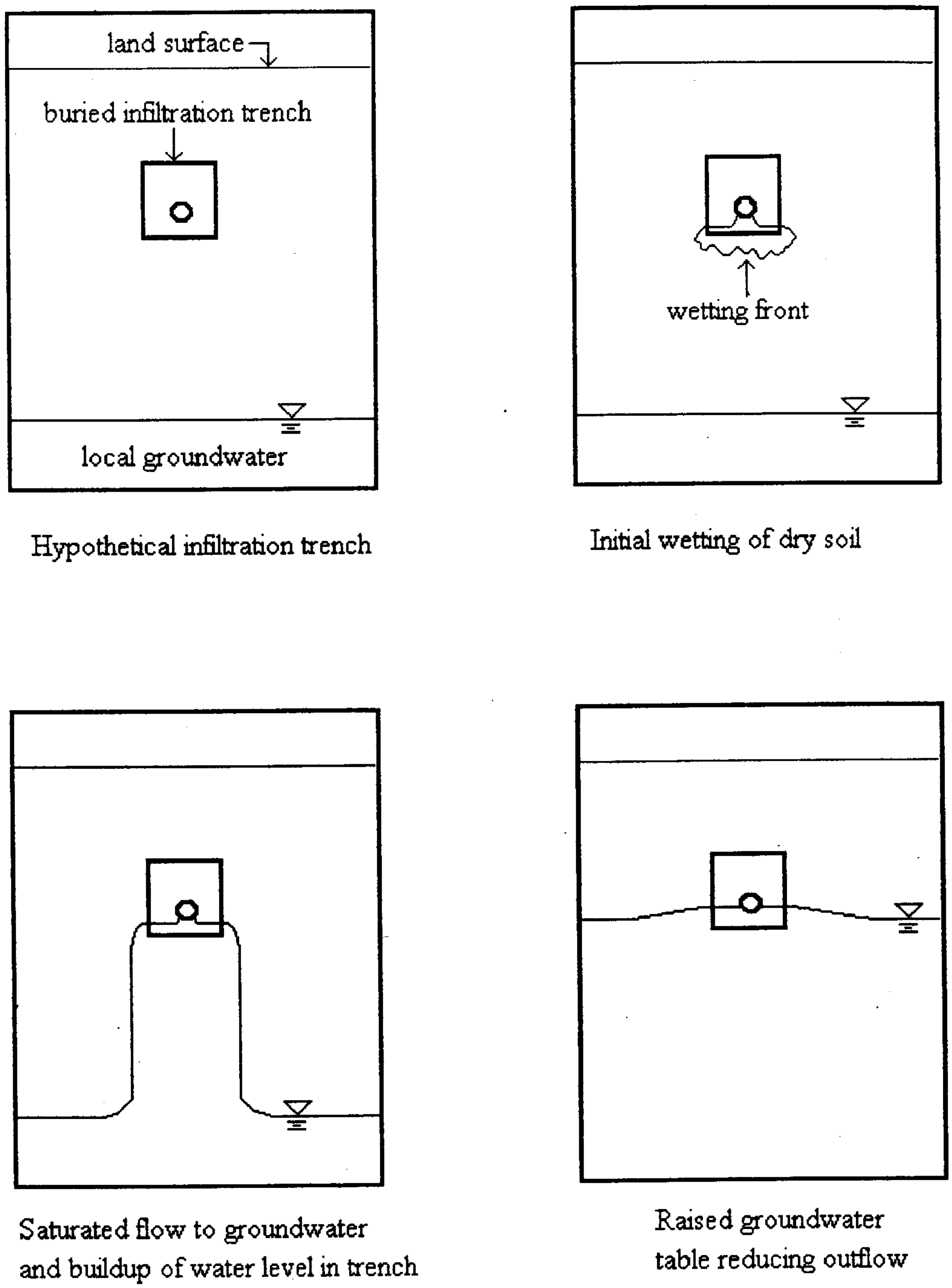

Figure 3.1: Schematic cross-sections of hypothetical infiltration trenches under a variety of conditions

Water enters the soil through the bottom and sides of the trench. Water moves through soil pores displacing air at the "wetting front". The air pressure can build up in constricted pores and restrict water movement in these pores. The diameter and connectivity of soil pores are intrinsic features which limit the movement of fluids and, thus, determine the soil's permeability. While these features may be relatively constant year to year, they are heterogeneous in space especially where roots, cobbles, soil layering, and macropores (e.g., worm holes, cracks, eroded pipes, decomposed root canals) disrupt a more uniform distribution of soil particles (Beven and Germann, 1982). The movement of water will not be uniform through a natural soil because of heterogenities (Youngs, 1988).

Soil-water movement also depends on the moisture content of a soil. Moisture content indicates the extent to which soil pores are filled with water. At low levels of soil moisture, water flows only in the smaller pores that are filled (Bear, 1972) so the flow through an unsaturated soil has a lower rate than flow through a saturated one for the same hydraulic gradient. Whenever there is standing water in the infiltration trench, the soil for a finite distance around it is saturated.

An infiltration system may saturate a soil column but the system will continue to discharge water only as long as water in this saturated column is flowing. This condition is met only in the unsaturated zone and capillary fringe above a free water surface where the soil water pressure is less than hydrostatic pressure. If the local groundwater table rises to the water level in the trench, then there would not be a vertical hydraulic gradient and outflow from the trench would cease.

Temperature has a strong effect on the viscosity of fluids and, consequently, their movement through soil. Surface infiltration rates of sand and loam soils depend strongly on temperature with a 67 - 77% increase in saturated infiltration rates when the temperature is increased from 5° C to 25° C (Constantz and Murphy, 1991). While temperature of runoff will vary depending on the time of year and whether the storm is a cold front or a warm front, the temperature of water in the trench will be relatively constant year round.

3.3 Saturated outflow from an infiltration system

Soil-water movement is often described in terms of a soil's hydraulic conductivity (K) which is a function of soil structure (permeability), the degree of soil saturation, and water temperature. A soil's hydraulic conductivity, the hydraulic gradient, and the geometry of the infiltration structure dictate the subsurface flow field of outflow from an infiltration system. Outflow rates can be analyzed on the basis of assumed steady-state, saturated porous media flow.

Henry Darcy observed in 1856 that the discharge rate of water moving through a saturated porous medium is proportional to the gradient in piezometric head in the direction of flow. Darcy's law states:

$$q = -K_{sat} (\Delta h/\Delta z)$$

where q is specific discharge (in units of length/time) equal to the volumetric discharge of water divided by the plane geometric cross-sectional area normal to the direction of flow; K_{sat} is the saturated hydraulic conductivity of the medium (in units of length/time); and $(\Delta h/\Delta z)$ is the gradient in piezometric head in the z direction (Freeze and Cherry, 1979).

Fluid properties are related to saturated hydraulic conductivity as:

$$K_{sat} = -k\ g\ \rho\ /\ \mu$$

where k is the permeability of the medium (a fixed property of the soil pore structure with units of length^2), g is gravitational acceleration, ρ is density, and μ is the dynamic viscosity of the fluid. For stormwater analyses, the fluid is water.

Darcy's law is applicable to a soil with a well defined matrix and is valid for a region of space as small as approximately 30 average pore diameters. It can be used to calculate the outflow from infiltration systems if the hydraulic gradient, flow field, and saturated hydraulic conductivity are known. For most infiltration systems, these are variable parameters (in space and/or in time).

Natural soils have heterogeneities that strongly affect the flow of water. Macropores will allow rapid subsurface flow through small areas of the soil. Vertical flow can be inhibited by very thin layers of silt so that water will flow laterally.

Tortuous soil pores and diffusion will cause multidirectional flow of water into unsaturated areas especially in tight, unconductive soils. The location of "no flow" boundaries in most soils is not known and change depending on the trench's water level.

The inflow rate to any infiltration system is variable in time and, for our example, the water level in the trench will rise and fall. Higher water levels may increase the outflow rate because of a larger area available in the trench walls to transport water and, possibly, because of increased piezometric head at the bottom of the trench.

If the infiltration system is well-above the local groundwater table, outflow from the system will reach a steady-state only after a prolonged period of steady input establishes a fully saturated soil column. In a transient state, there is air in the soil pores ahead of the infiltrating water resisting water movement with its pressure (Youngs, 1988). Since soil water pressure has a hysteretic relationship to soil water content, this pressure (and hydraulic conductivity) will vary depending on whether the soil is wetting or drying even if its water content is constant.

3.3.2 Approaches for calculating outflow from infiltration systems

Typically, practical estimates of infiltration capacity of a soil are obtained by digging a hole to the appropriate depth in the ground, filling it with water, and timing the drop in water level (i.e., a "percolation" test). Infiltration capacity is expressed as the drop in water level divided by the time. Such measurements, however, provide a very limited spatial sample of the relevant soil structure (Youngs, 1991) and are influenced by the transient conditions such as initial soil moisture, temperature, and piezometric head. Water infiltrates from a hole at variable rates across the side and bottom surfaces in a way unrepresentative of outflow from a residential stormwater infiltration structure. Soil characteristics (e.g. soil texture) have been used to estimate hydraulic properties of soils, but this approach has not provided accurate estimates of hydraulic conductivity (Springer and Cundy, 1987).

The designs of most infiltration systems for residential application use infiltration rates measured during percolation tests combined with a few simple calculations of roof runoff to determine the necessary storage volume (Evans and Associates, 1988; Harrington, 1988; King Co., 1990; Mikklesen and Jacobsen, 1993). For an infiltration trench, these calculations might include:

Storm volume = roof area depth of rainfall for design storm;
Trench volume = storm volume ÷ porosity of fill; and

Trench bottom area (or side area) = storm volume ÷ (infiltration capacity of soil time period between storms).

These calculations may be adequate for constructing a trench which will work. They do not describe how the trench functions: the area infiltrating water and the infiltration rate vary with time, water level in the trench, initial soil moisture, depth to groundwater, etc. A single value for infiltration rate based on a point measure does not capture the temporal or spatial variability of a soil's hydraulic properties; it is bound to be an inaccurate value much of the time. This approach does not take into account the flow paths followed by the water after it leaves the trench.

Alternatively, the outflow from an infiltration system can compared to the problem of seepage from an open channel. Saturated seepage from an open channel has been analyzed for a vertical cross-section (Polubarinova-Kochina, 1962, Harr, 1962; Hunt, 1972). The analysis, formulated by Vedernikov, assumes the outflow field from an open channel is fully saturated and bounded by free surfaces (atmospheric pressure). Figure 3.2 shows a general example. At some distance below the trench, any lateral macroscopic flow can be neglected as subsurface flow is essentially vertical (Wooding, 1968). The distance will be smaller for highly permeable soils and larger for relatively impermeable soils.

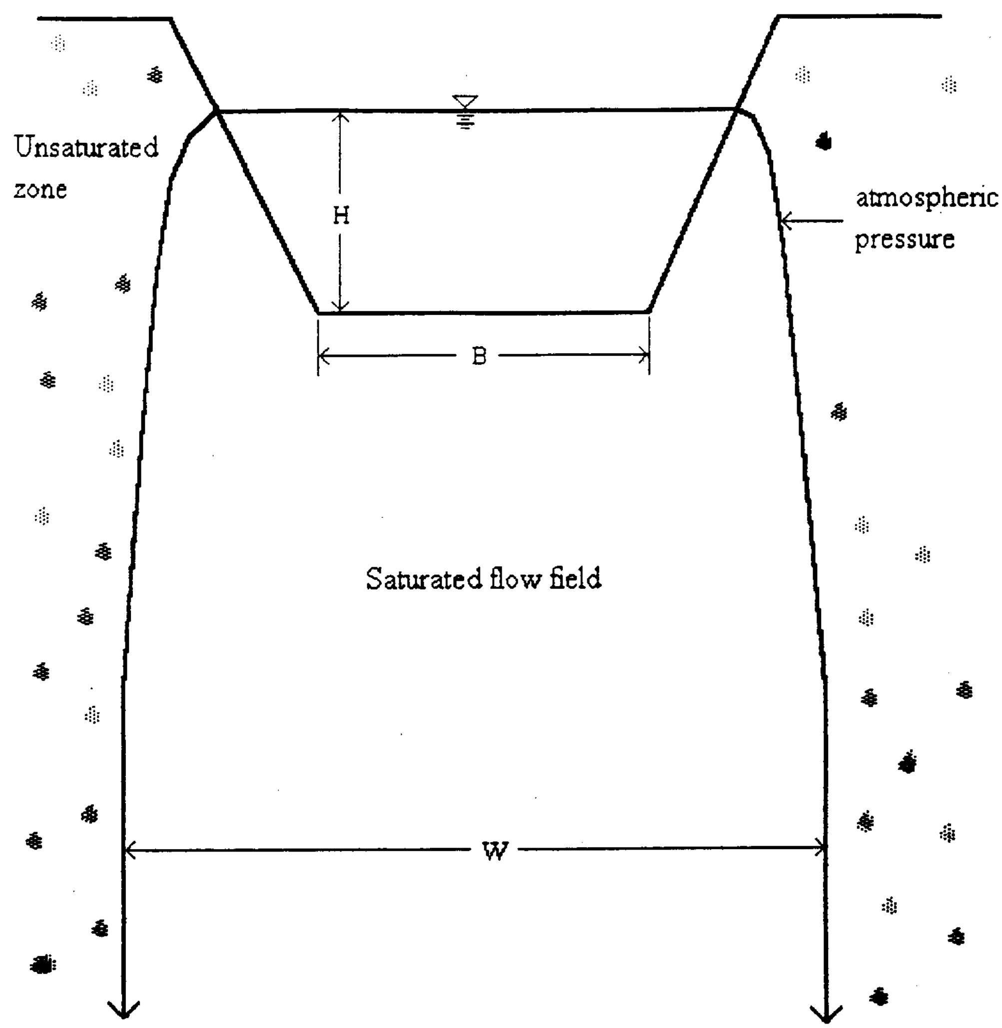

Figure 3.2: Vertical seepage from an open channel located a semi-infinite distance above the regional groundwater table

The problem is to determine the width of the flow field below the trench where the flow is essentially vertical. Assuming the soil is isotropic, the flow field width depends on the geometry of the channel and the elevation (H) of the free surface in the channel. The flow field width (W) is described by the equation: $W = B + C^{*} H$ where B is the width of the base of the channel and C is a coefficient that depends on channel geometry (Harr, 1962). Where flow is vertical, the magnitude of the hydraulic gradient in the vertical direction is unity. Hence, the outflow rate (Q) from the channel (or other structure) per unit length of channel is given by: $Q = K_{sat}{*}W$.

The approach is limited, however, in its applicability to infiltration systems. The value of the parameter C depends on the ratio of water depth to channel width. The steady state solution indicates the maximum rate at which water will move vertically downward toward the local groundwater table.

Few if any infiltration systems in the Puget Sound lowlands lend themselves to complete analytical treatments. A schemes for sizing of infiltration systems is provided in Appendix 1 and relies on a simple analytical approach. The approach, however, does not describe accurately, infiltration from a system under natural rainfall patterns. The depth of soil through which outflow will occur is too shallow relative to regional or perched groundwater tables and the flow is too variable to permit application of the analogy of steady-state saturated seepage from a trench. This necessitates empirical examination of particular systems representative of the conditions encountered in practice.

Two on-site residential infiltration systems were monitored during the winter of 1994-5. The first of these systems, an infiltration trench, is described in Section 3.4. The second system, a dry well, is described in Section 3.5.

3.4 *Field investigation of the Union Hill infiltration trench*

The first study site is located on the Sammamish plateau. An aerial photograph of the study site is shown in Figure 3.3. The Sammamish plateau is located between Lake Sammamish and the Snoqualmie River valley at an elevation of 150 - 175 m. It is characterized by an undulating topography dotted with wetlands and intermittent streams. The plateau has experienced considerable residential development in recent years which will continue for the foreseeable future. The concentration of new development on the plateau, its soils, and its hydrogeology make the plateau an ideal location for infiltration systems.

A 17 hectare area of Union Hill on the Sammamish plateau east of Redmond, WA was recently converted from second growth forest (mixed coniferous and broadleaf trees) to a residential development. The soils in the area before development comprised 0.5 - 1.5 meters of loamy sand on top of hardpan (Evans and Associates, 1989). The hardpan, or Vashon till, was deposited and compacted by advancing glaciers during the last glaciation. It consists of cobbles, sand, and silt particles. The silt renders the till relatively impermeable. The area's soils are generally classified as Alderwood (moderately well drained gravely sandy loam with consolidated substratum) with some Everett series soils (excessively drained gravelly sandy loam).

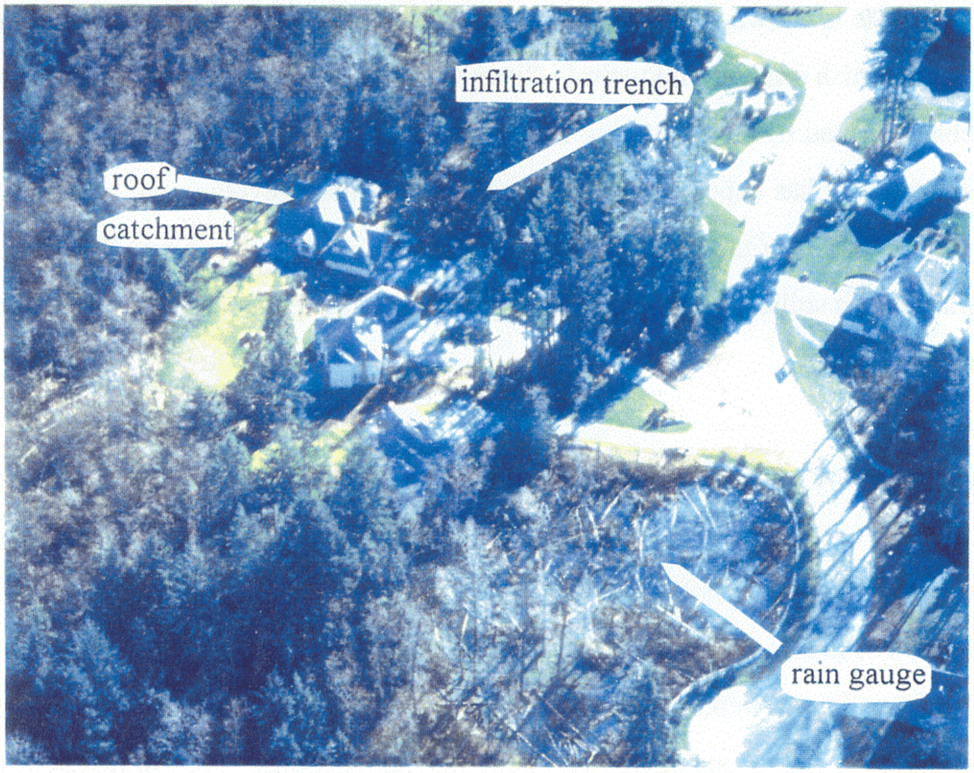

Figure 3.3: Union Hill stormwater infiltration trench study site

King County ordinances required the developer to manage excess stormwater generated by the imperious surfaces added during development (King Co., 1988). This was accomplished through means of a central storm drain which discharges to a detention pond and individual lot infiltration systems. Infiltration systems were constructed for nine of the 42 lots. Lots are approximately 3250 m^2. All of the infiltration systems rely on trenches to distribute roof runoff and water from footing drains away from the dwellings.

After reviewing the design and location of the systems, some of which were still under construction, and inquiring of residents about their willingness to permit monitoring on their properties, one system (referred to as the Union Hill infiltration trench) was selected for monitoring. The Union Hill infiltration trench has been in use since 1993.

3.4.1 Union Hill infiltration trench construction

The Union Hill infiltration trench is approximately 23 m long and 0.6 m wide (Figure 3.4). The trench receives roof runoff from the house (and garage) which have a combined plan area of approximately 230 m^2. There is a 10 cm diameter perforated pipe running along the bottom of the trench approximately 1.8 m below the ground surface. The bottom of the trench is located in glacial till which underlies approximately 1.2 m of loamy sand. The trench is filled with 5 cm diameter rounded rocks to a depth of 0.3 m which have been covered with a dense polyethylene plastic sheet.

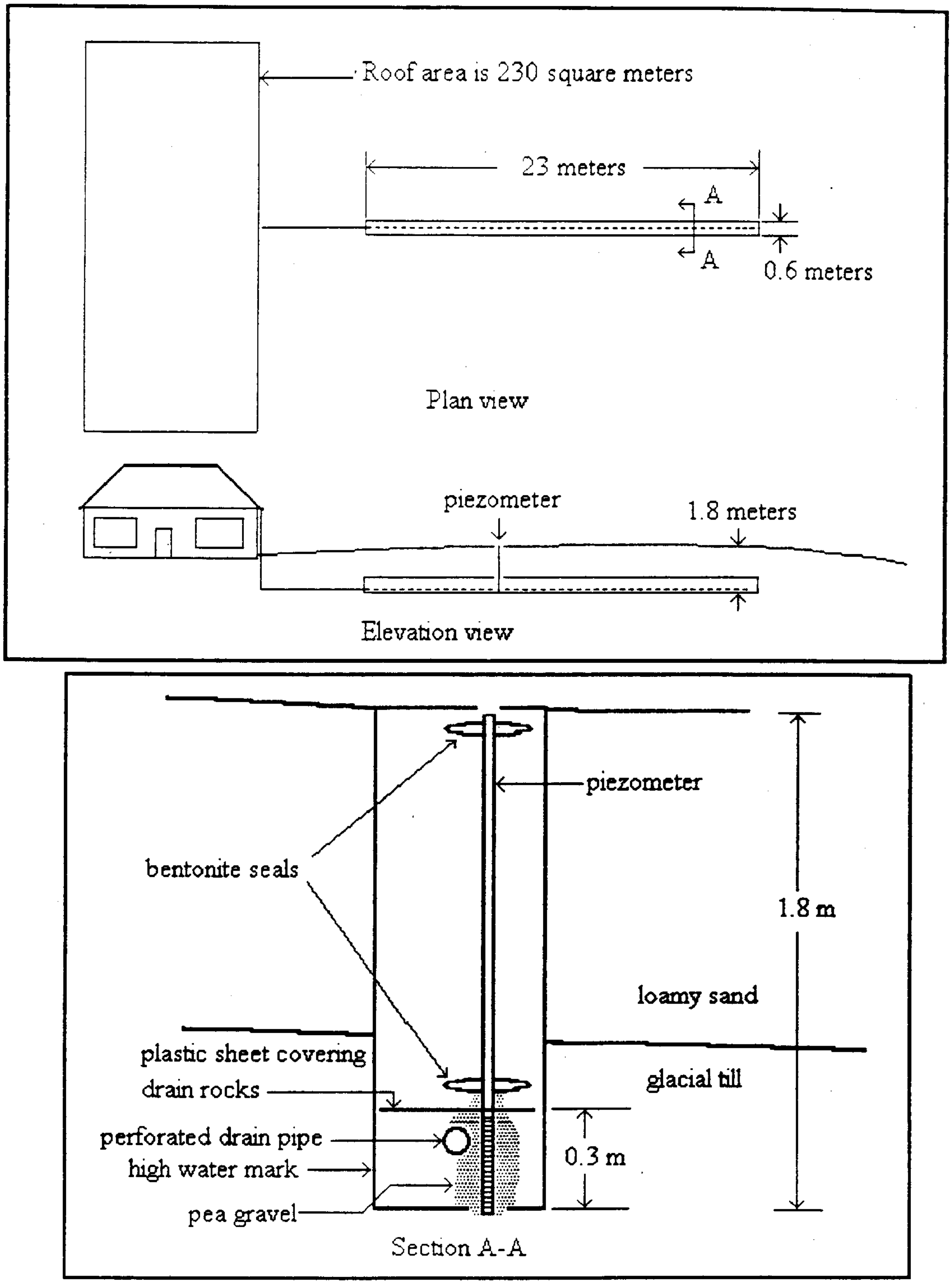

Figure 3.4: Schematic diagram of Union Hill infiltration trench

Figure 3.5 shows the trench at this stage in construction. The rest of the trench was backfilled with disturbed, natural soil to the ground surface. Soil particles have since migrated into the voids around the drain rocks as observed during piezometer installation. Lawn covers the trench area. There is an intermittent stream, with a vegetated buffer, which runs parallel to and 20 m from the trench. The stream bed material is glacial till and the elevation of the bed is approximately 1.2 meters below the bottom of the trench.

3.4.2 Field methods for monitoring the Union Hill infiltration trench

A piezometer was installed in the trench to monitor changes in water levels during and after storms. The piezometer consists of a 5 cm diameter, 1.8 m long PVC pipe with an open bottom and slots in the bottom 0.5 m. The base of the pipe was placed at the bottom of the trench, 10 cm below the inflow pipe. Pea gravel was packed around the pipe and then backfilled with soil. Bentonite clay was used to seal around the piezometer 1 m from the bottom of the trench and just below the soil surface.

A 1 m long Unidata capacitive probe was used to measure the water level in the piezometer to the nearest 1 mm. The measurements were recorded every 15 minutes with an automatic data logger. A 152 mm diameter tipping bucket rain gauge was installed nearby in an opening in the forest canopy. Rainfall was measured in 0.25 mm increments and recorded every 15 minutes. Rainfall and water level measurements began on 25 October 1994.

Figure 3.5: Union Hill infiltration trench during construction (above) and installation of piezometer (below)

3.4.3 Results of field monitoring of the Union Hill infiltration trench

The measured data for the Union Hill infiltration trench are displayed in Figure 3.6. The figure shows records of water level in the trench, the rainfall rate, and cumulative rainfall for the period from 25 October 1994 - 23 March 1995. The water level in the trench increases during storm events though there is not always a response from light rain (<0. 6 mm/15 min). The water in the trench reached a maximum level of 0.1 m above the bottom of the trench. At this level, the water was 0.05 m below the discharge pipe and is contained well within the till layer which extends up 0.6 m from the bottom of the trench (Figure 3.3). Generally, the trench drains within 2 hours of the end of rainfall with a maximum drop in water level of 0.03 m/15 min and 0.08 m/hour.

The response of water level in the trench to storms is displayed in Figures 3.7a and 3.7b for the periods 30 January - 1 February 1995, and 18 - 20 February 1995, respectively, when there were successive, moderately intense storms. Rainfall totals for these periods were 60, and 70 mm respectively. The longest duration of standing water was on 20 February 1995 when water remained in the trench for 8 hours after rainfall stopped. The maximum change in water level during these two periods is 0.05 m/hr (for 30 January - 1 February 1995) and 0.07 m/hr (for 18 - 20 February 1995).

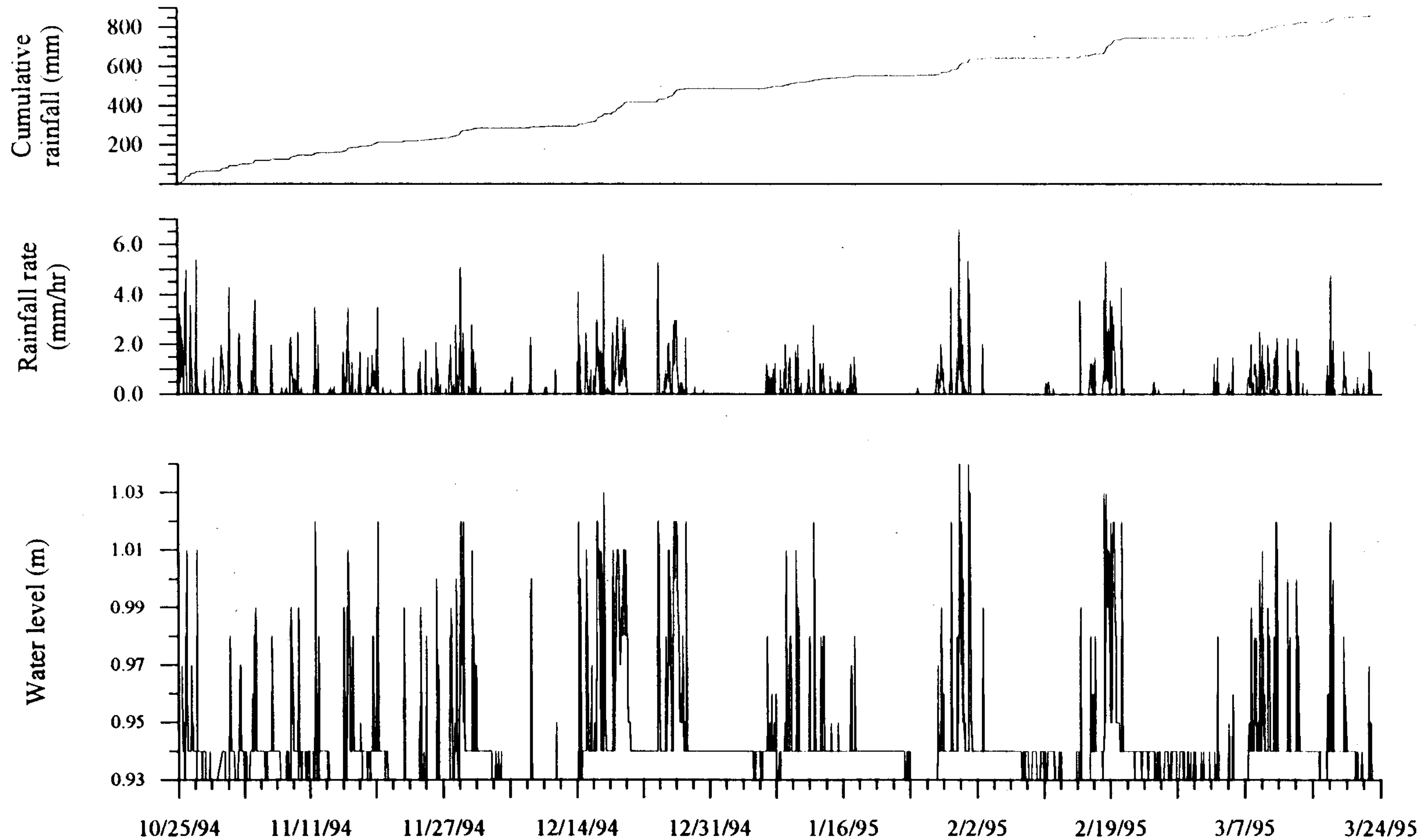

Figure 3.6: Cumulative rainfall and hourly time series of rainfall and water level in Union Hill infiltration trench for the period 25 October 1994 to 23 March 1995

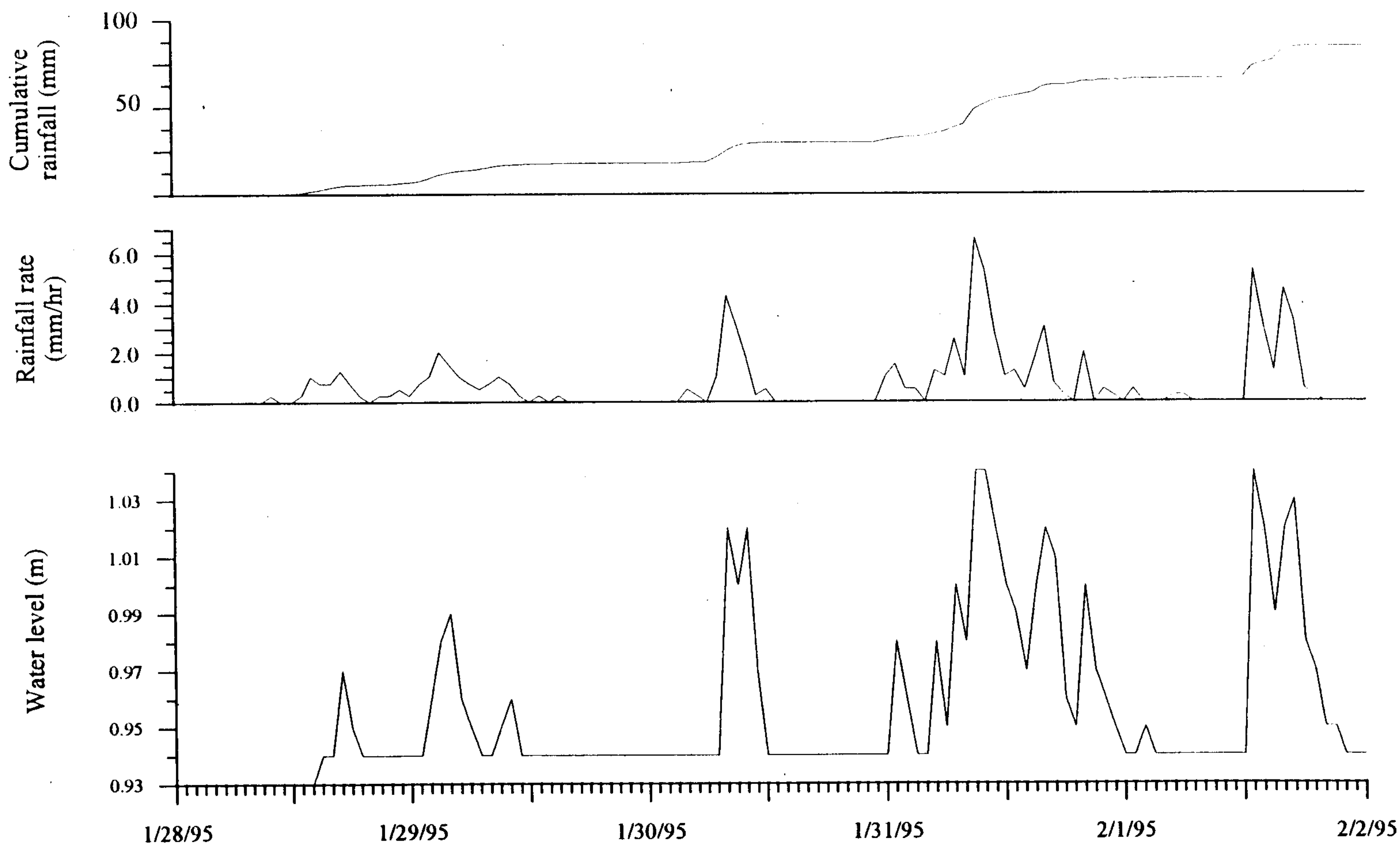

Figure 3.7a: Cumulative rainfall and hourly time series of rainfall and water level in Union Hill infiltration trench for the period 28 January 1995 to 1 February 1995

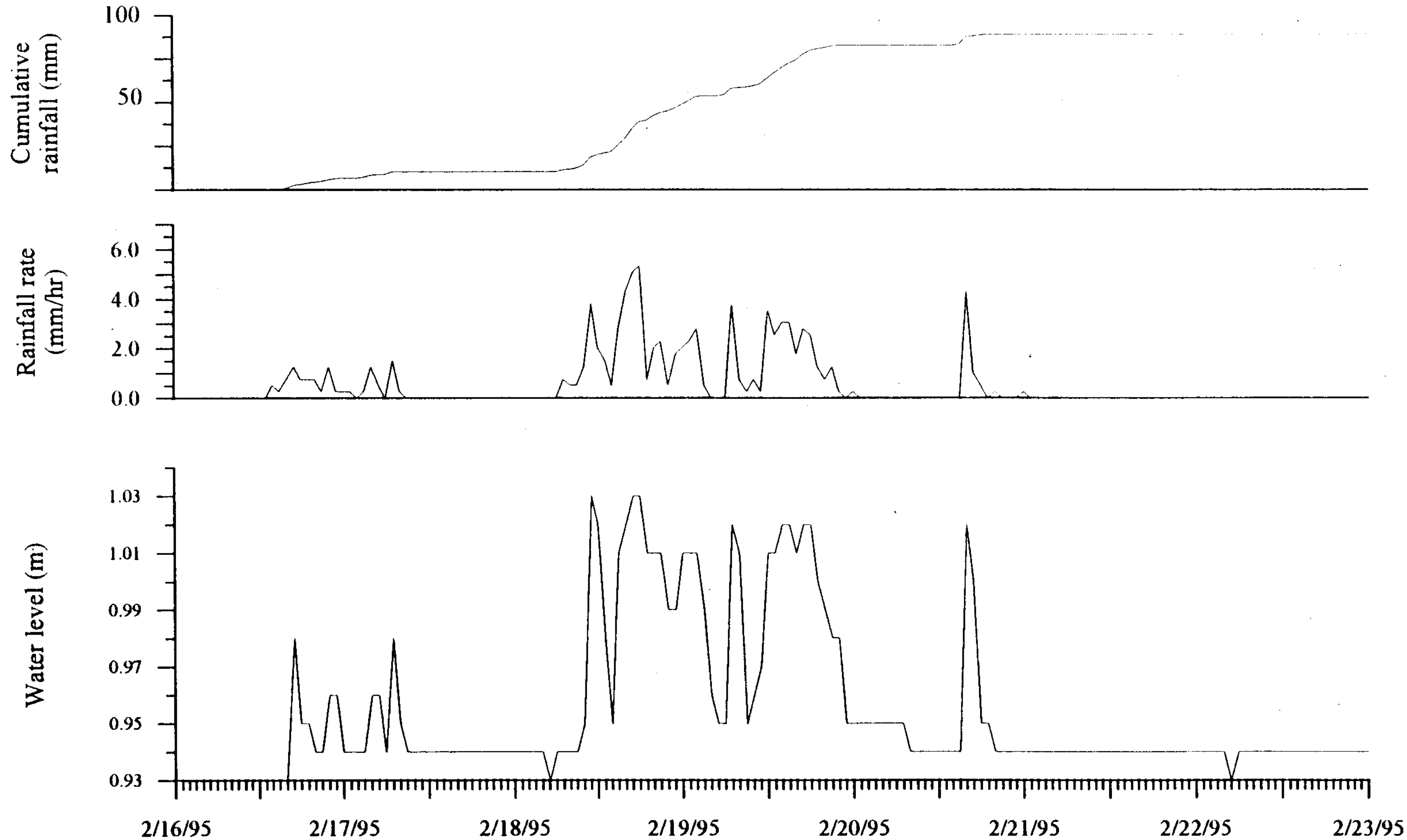

Figure 3.7b: Rainfall and water level in Union Hill infiltration trench
for the period 16 February 1995 to 22 February 1995

3.4.4 Analysis

The volume of water stored in the trench per meter of water depth can be approximated as follows. The plan area of the trench bottom is approximately 14 m^2. Using the porosity of a rocky, unconsolidated material of 35% (Freeze and Cherry, 1979), the volume of water stored in the trench per unit depth of standing water is equal to 5 m^3/m (i.e., 0.35*14) with a total capacity of 1.5 m^3 (or 5 m^3/m * 0.3 m) around the drain rocks filling the trench.

Inflow to the Union Hill infiltration trench is equal to a rain depth multiplied by the roof area. The roof area served by the trench is approximately 230 m^2. A rain depth of 10 mm should produce 2.3 m^3 of stormwater in the absence of rain retention on, or evaporation from, the roof.

The outflow rate from the trench can be calculated by multiplying the change in water level by the 5 m^3 trench storage volume per unit depth of water for the periods of time when there was no inflow (Table 3.1). For an hour time period, the maximum observed water level drop of 0.08 m corresponds to an outflow rate of 0.4 m^3/hr. This rate represents an effective specific discharge (volumetric outflow rate/trench bottom area, i.e., 0.4 $m^3/hr/14\ m^2$) for the soil beneath the trench of 0.03 m/hr. For a 15 minute time period, a maximum water level drop of 0.03 m was observed. Employing the same methodology, this corresponds to an outflow rate of 0.6 m^3/hr and an effective specific discharge for the soil of 0.04 m/hr.

In a second approach (Table 3.1), the cumulative inflow volume is calculated for a time period when the water level in the trench rises and falls back to its initial level (i.e., there is no net change in water level). Since the net change in storage during this time period is zero, the outflow volume from the trench must equal the inflow volume. Time-averaged outflow rates for four time periods were calculated using this approach (Table 3.1). They ranged from 0.46 - 0.63 m^3/hr which agree with the previous outflow calculations.

Whereas the first approach assumed a relationship between water level and trench storage, the second approach assumes that the recorded rainfall depth multiplied by roof area provides an accurate estimate of inflow volume. Given the independence of these assumptions, the two approaches should not be biased toward supporting each other's results.

The maximum rate of discharge for the winter of 1994-95 can be used to assess the response of the water level in the trench to intense rainfall rates. The maximum rate of rainfall recorded from 1 October 1990 - 30 June 1993 at Novelty Hill, WA was 6 mm/hr (Chapter 2). For a one hour period, this rain rate would generate 1.4 m^3 of runoff from the Union Hill house's roof.

If this water entered the trench instantaneously, the water level in the trench would be expected to rise 0.27 m. At this level, water would fill all of the drain rock in the trench and begin saturating the soil used to backfill the trench. Assuming an outflow rate from the trench of 0.4 m^3/hr, the trench would take 3.5 hours to drain. While there appears to be adequate soil storage for most storms that could be expected for the area, the outflow rate for the trench would be reduced if the local groundwater table rose. Thus, the trench's performance during long periods of steady or intense rain is uncertain.

Table 3.1: Comparisions of two methods for calculating outflow from the Union Hill infiltration trench for for data recorded in 1995

Method 1:
(Observed drop in water level)*(volume of water stored in infiltration structure)/(depth of standing water)

	Units		
time scale		15 min	1 hour
		18-Feb	19-Feb
max. observed drop in water level	m	0.03	0.08
storage volume/depth of water	m^3/m	5	5
estimated outflow rate	m^3/hr	0.60	0.40

Method 2:
(Cumulative inflow)/(time for water level to return to pre-event level)

	Units	30-Jan	1-Feb	18-Feb	19-Feb
cumulative rain	mm	10	18	22	23
roof area	m^2	230	230	230	230
net inflow	m^3	2.3	4.14	5.06	5.29
time for water level to return to pre-event level	hours	5	9	8	11
estimated outflow rate	m^3/hr	0.46	0.46	0.63	0.48

3.5 Field investigation of the East Lake Sammamish Dry Well

The second infiltration system investigated was a dry well located in a hillslope above the east shore of Lake Sammamish shown in Figure 3.8. The site is located midway up a 500 m long, steep, forested hillslope that stretches from the Sammamish Plateau down to Lake Sammamish. The slope averages 30% between the plateau and lake.

The site was completely cleared and graded for construction of six houses. A dry well infiltration system serving one of the houses was selected for monitoring during the winter of 1994-95.

The soils are classified predominately as Alderwood with small pockets of the "more excessively drained" Alderwood-Kitsap association (Soil Conservation Service, 1972). Soils at the site are brown loamy sand to 0.6 m and sandy loam with cobbles to at least 1 m. Silt particles were observed at the bottom of the excavation for the dry well at a depth of 2 m.

3.5.1 East Lake Sammamish dry well construction

The dry well consists of a cylindrical concrete catch basin. Its internal diameter is 1.5 m and it is 1.8 m tall with one closed end and one open end. A 2 m deep hole was excavated for the catch basin and 0.6 m of washed rock were placed in it. The catch basin was placed on top of the washed rock with its open end facing down. The top of the catch basin is directly under the driveway serving the residence. All of the roof downspouts and footing drains from the residence are conveyed into the catch basin through a pipe located 1.7 m from the bottom of the excavation (i.e., 1.1 m from the bottom of the catch basin). The top of catch basin can be entered through an access cover.

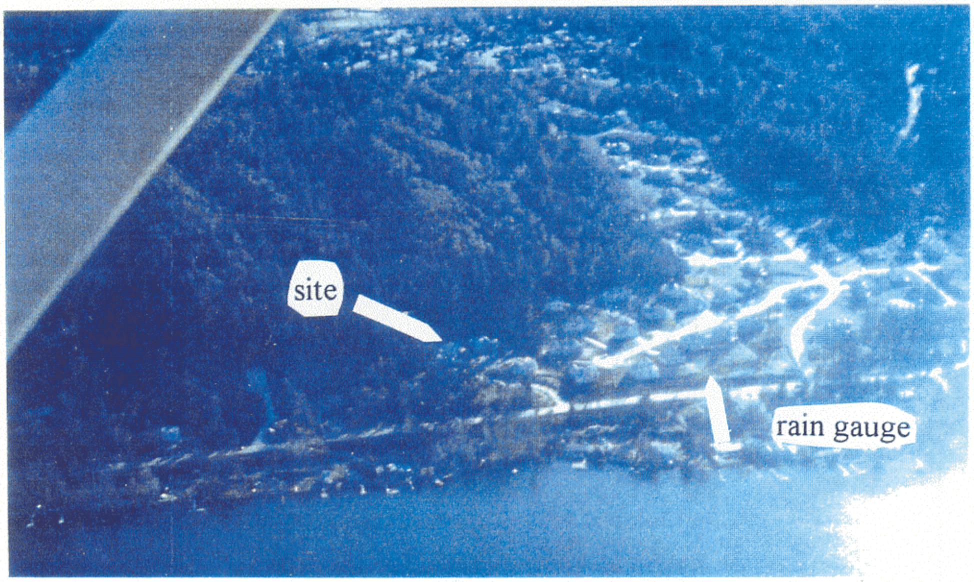

Figure 3.8: Aerial photographs of hillslope from Sammamish plateau to Lake Sammamish (above) with East Lake Sammamish dry well study site (below)

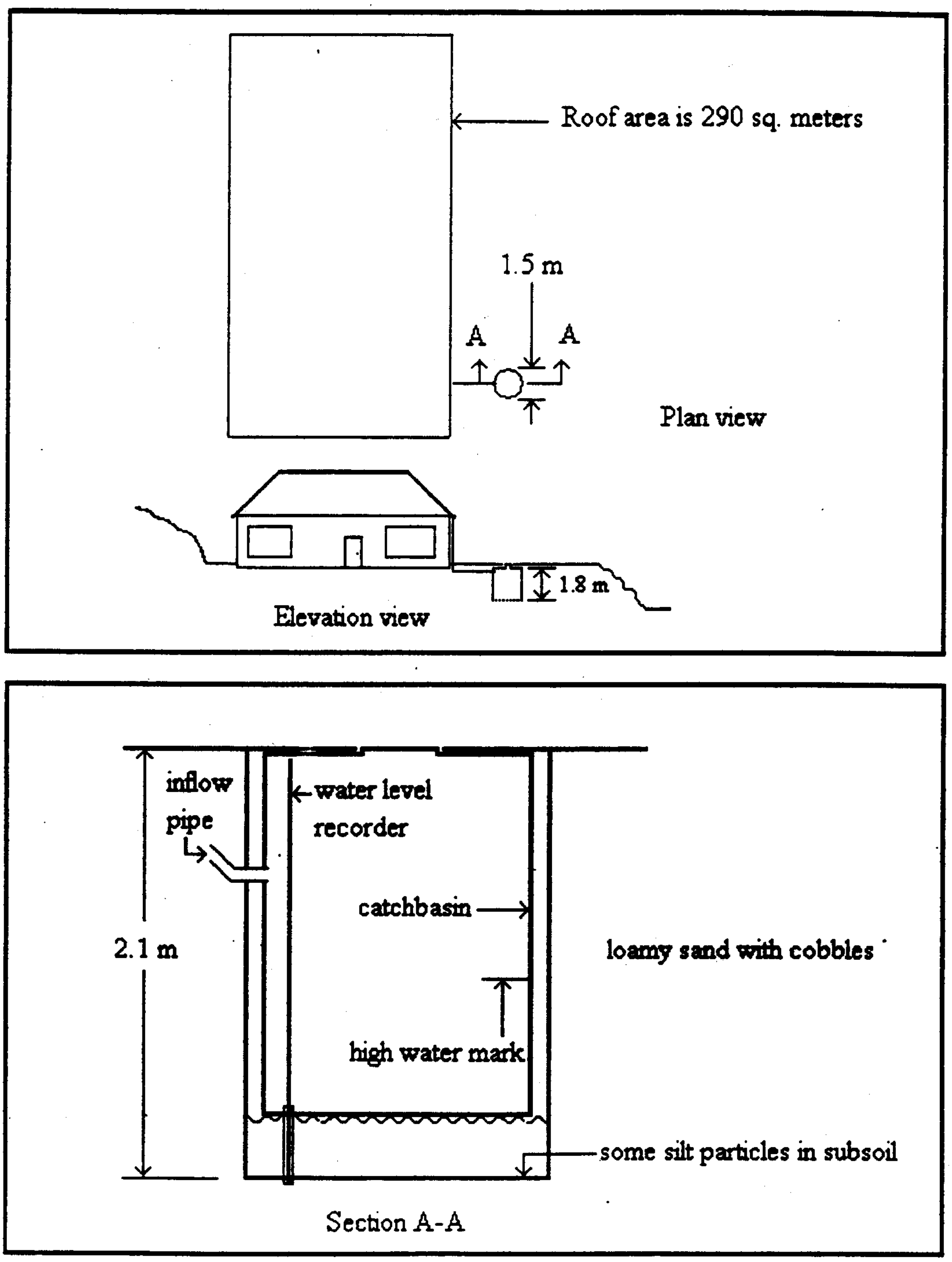

Figure 3.9: Schematic diagram of East Lake Sammamish dry well

3.5.2 Monitoring the East Lake Sammamish dry well

A water level recorder was installed in the dry well to monitor the water level in it. A slotted, 5 cm diameter PVC sleeve (open at the bottom) was placed at the base of the dry well on top of intact soil. A 2 m long capacitive probe was used to measure the water level to the nearest 2 mm. The measurements were recorded every 15 minutes. A rain record was obtained from a 152 mm diameter tipping bucket rain gauge at a nearby (less than 300 m) King County stormwater detention facility. Rainfall was measured in 0.25 mm increments and recorded every 15 minutes. The record of rainfall and water level began on 19 November 1994.

3.5.3 Results of field investigations for the East Lake Sammamish dry well

The measured data for the East Lake Sammamish dry well are displayed in Figure 3.10. The figure shows a time series of water level in the trench, the rainfall rate, and cumulative rainfall for the period from 19 November 1994 - 23 March 1995. The water level in the dry well rises in close correspondence to the patterns of rainfall. The maximum observed drops in water level were -0.18 m/15 min and -0.46 m/hr.

Figures 3.11a and 3.11b display two periods (30 January - 1 February 1995 and 18 February - 20 February 1995) during the winter when there were successive, moderately intense storms. During these storms, the water level in the dry well rose as much as 0.6 m above the level of pre-storm standing water. The water level reached a maximum of 1.0 m on February 1 and 18, 1995. Standing water persisted after storm events and rose slowly during and after successive storms.

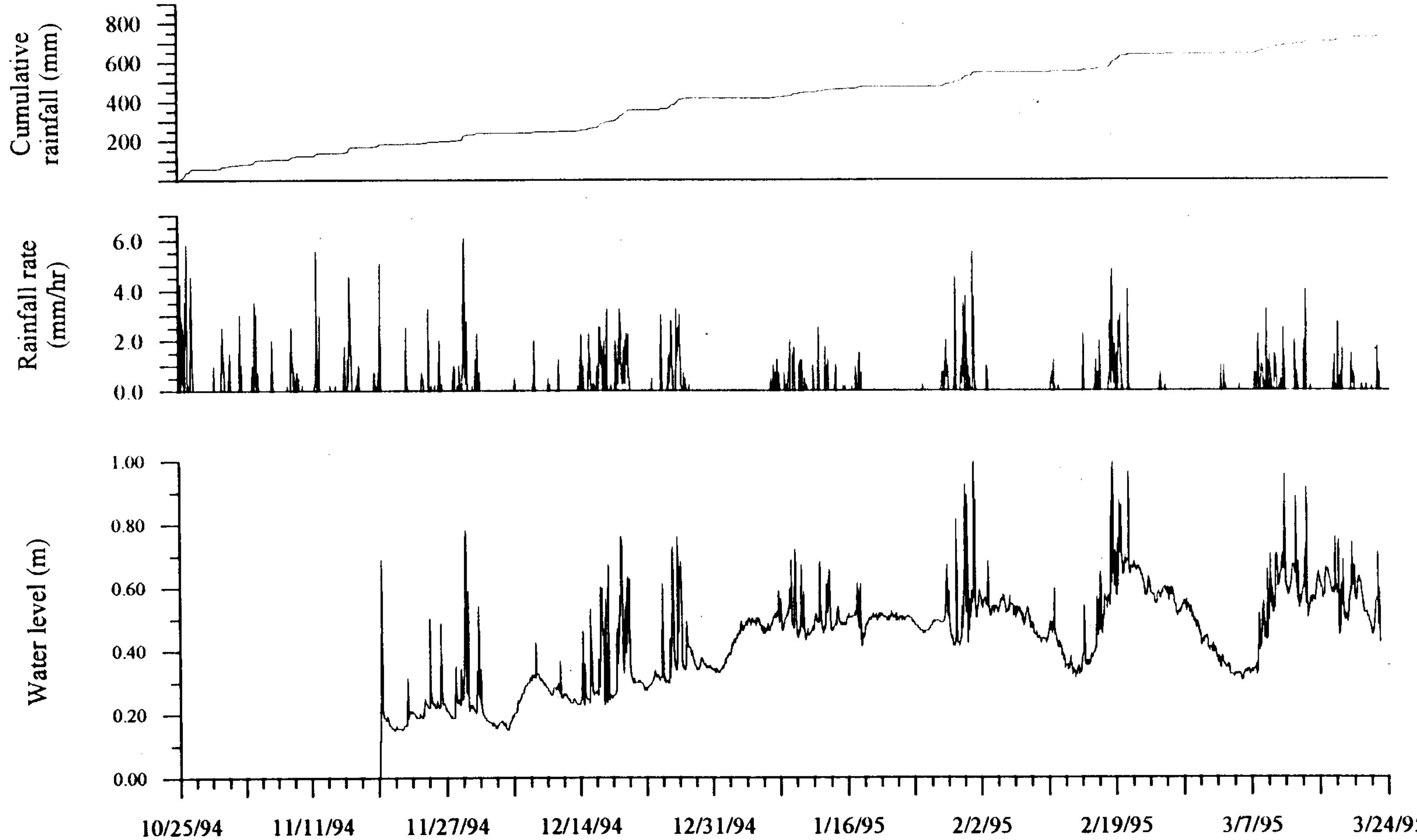

Figure 3.10: Cumulative rainfall and hourly time series of rainfall and water level in East Lake Sammamish Dry Well for the period 11 November 1994 to 23 March 1995 (the starting date for cumulative rainfall is 25 October 1994)

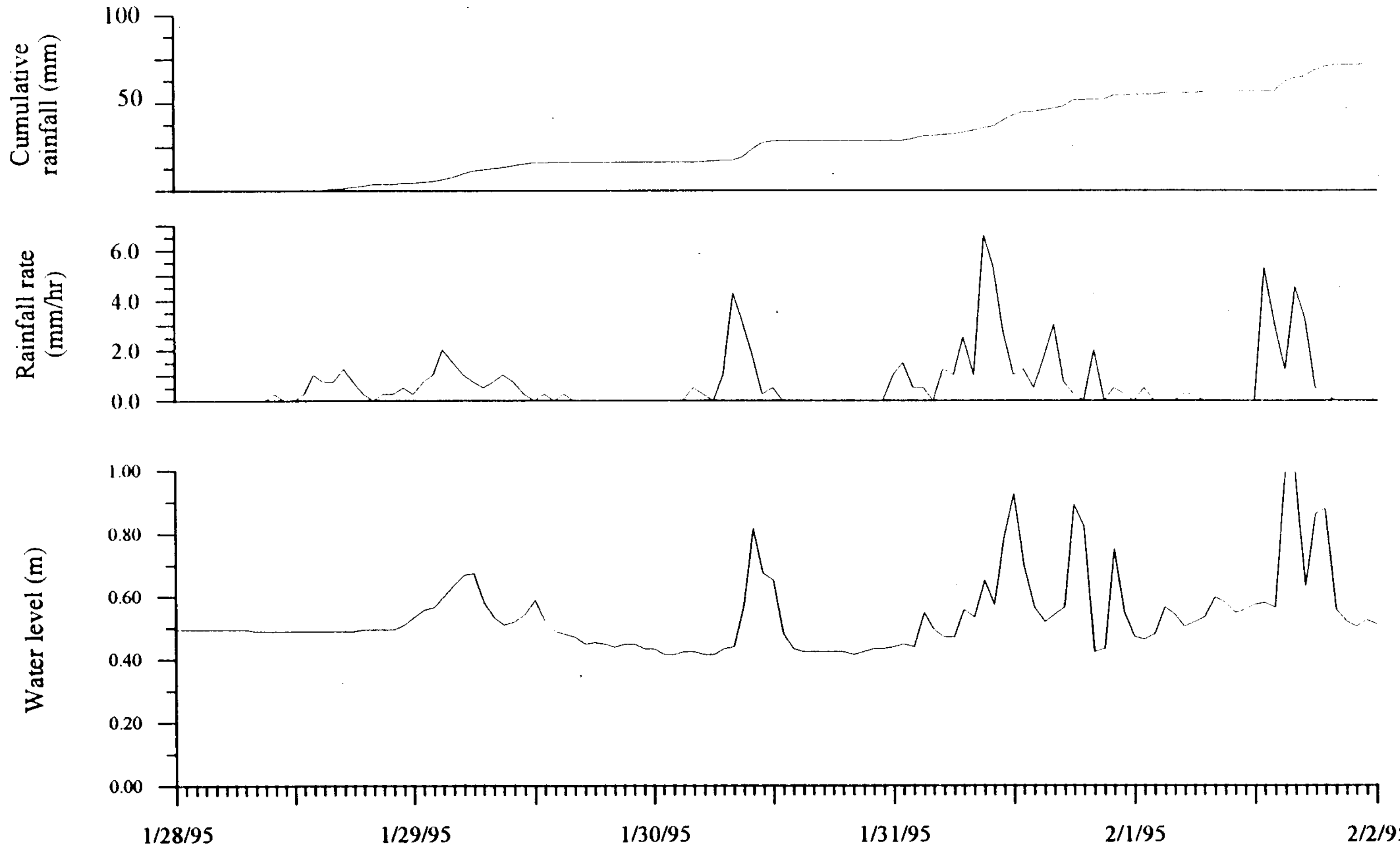

Figure 3.11a: Cumulative rainfall and hourly time series of rainfall and water level in East Lake Sammamish Dry Well for the period 28 January to 1 February 1995

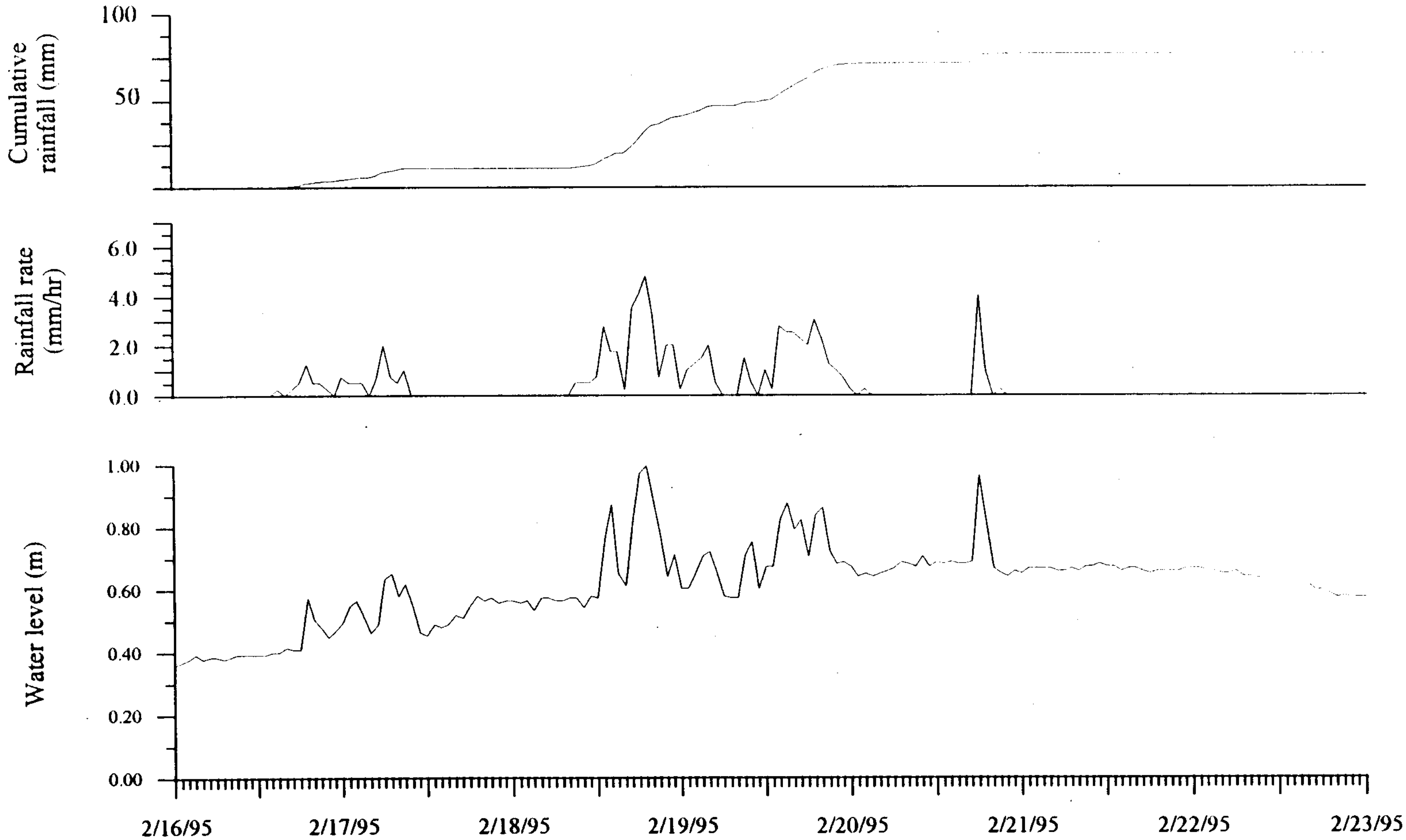

Figure 3.11b: Cumulative rainfall and hourly time series of rainfall and water level in East Lake Sammamish Dry Well for the period 16 February 1995 to 22 February 1995

Generally, the dry well drains rapidly (0.3 to 0.4 m/hr) for up to 2 hours after rainfall ceases at which point the rate of outflow slows down. The maximum drop in water level observed during the period from 30 January to 1 February was 0.18 m/15 min and 0.43 m/hr. For the period from 18 February to 20 February, the maximum drop was 0.15 m/15 min and 0.43 m/hr.

During the interludes between storms, the base water level in the dry well can rise as it did, for example, on 20 February 1995. This may be due to subsurface drainage from the long, forested hillslope above the site or local groundwater mounding created by the infiltration system and locally infiltrating rainfall.

3.5.4 Analysis

The volume of water stored in the dry well per meter of water depth can be approximated as follows. The plan area of the dry well is approximately 1.8 m^2, so the volume of stored water in the dry well equals 1.8 m^3/m depth of standing water.

The inflow rate to the East Lake Sammamish dry well is equal to a rain intensity multiplied by the roof area. The roof area served by the dry well is approximately 290 m^2. A rain depth of 10 mm should produce 2.9 m^3 of stormwater in the absence of retention on, or evaporation from, the roof.

The outflow rate from the dry well can be calculated by multiplying the change in water level by the 1.8 m^3 dry well storage volume per depth. The maximum observed water level drop over one hour was 0.46 m which corresponds to an outflow rate of 0.83 m^3/hr. The maximum drop over a 15 minute time period was 0.18 m which corresponds to an outflow rate of 1.3 m^3/hr. The measured water level drops (0.46 - 0.72 m/hr) can be used as estimates of the effective specific discharge (volumetric outflow rate/dry well bottom area) for the soil beneath the dry well.

Another approach was used to verify the outflow rate calculations (Table 3.2). For this approach, the cumulative inflow volume is calculated for a time period when the water level in the dry well rises and falls back to its initial level (i.e., there is no net change in water level). Since the change in storage during this time period is zero, the outflow volume from the dry well must equal the inflow volume. Outflow rates for four time periods were calculated using this approach (Table 3.2). They ranged from 0.64 - 0.99 m^3/hr which overlaps but is slightly lower than the previous outflow calculations.

Where as the first approach assumed a relationship between water level and trench storage, the second approach assumes that the recorded rainfall depth multiplied by roof area provides an accurate estimate of inflow volume. Given the independence of these assumptions, the two approaches should not be biased toward supporting each other's results.

The maximum rate of discharge observed during the winter of 1994-95 can be used to assess the response of the water level in the dry well to intense rainfall rates. The maximum rate of rainfall recorded from 1 October 1990 - 30 June 1993 at Novelty Hill, WA was 6 mm/hr (Chapter 2). For a one hour period, this rain rate would generate 1.7 m^3 of runoff from the East Lake Sammamish house's roof.

Table 3.2: Comparisions of two methods for calculating outflow from the East Lake Sammamish dry well for data record in 1995

Method 1:
(Observed drop in water level)*(volume of water stored in infiltration structure)/(depth of standing water)

	Units		
time scale		15 min	1 hour
		18-Feb	19-Feb
max. observed drop in water level	m	0.18	0.46
storage volume/depth of water	m^3/m	1.8	1.8
estimated outflow rate	m^3/hr	1.30	0.83

Method 2:
(Cumulative inflow)/(time for water level to return to pre-event level)

	Units	30-Jan	1-Feb	18-Feb	19-Feb
cumulative rain	mm	11	14	17	20
roof area	m^2	290	290	290	290
net inflow	m^3	3.19	4.06	4.93	5.8
time for water level to return to pre-event level	hours	5	5	5	9
estimated outflow rate	m^3/hr	0.64	0.81	0.99	0.64

If this water entered the dry well instantaneously, the water level in the dry well would be expected to rise 1 m. Assuming a maximum outflow rate from the dry well of 0.8 m^3/hr, it would take 2.2 hours to drain. While there appears to be adequate soil storage for most storms that could be expected for the area, the outflow rate for the dry well would be reduced if the local groundwater table rises. Less storage capacity would be available if standing water is present at the onset of a storm. Thus, the dry well's performance during steady and intense rain is uncertain.

3.6 Summary of field results for the Union Hill infiltration trench and East Lake Sammamish dry well

The two infiltration systems studied during the winter of 1994 - 95 infiltrated all of the stormwater that they received. The winter can be characterized by moderate, sequential storms. There were eight periods with sequential storms separated by one to four weeks. Within each of these periods, peak rainfall rates of up to 5 mm/hr were recorded six to twelve hours apart.

Both systems discharged stormwater at a moderate depth in the soil column (1.5 to 2 meters below ground surface). At Union Hill, the effect of the infiltration trench is to increase the soil moisture at a deeper level in the soil column than would stormwater infiltrating at the surface. At East Lake Sammamish, the more rapid delivery of stormwater to an apparently saturated zone may increase the local groundwater elevation and subsurface flow.

Both infiltration systems had storage capacity to spare during all storms. On February 18, 1995 both systems reached their maximum recorded levels: in the Union Hill infiltration trench, there was 0.1 m of standing water, 0.05 m below the base of the horizontal discharge pipe; in the East Lake Sammamish dry well, standing water reached 1.0 m, 1.4 m below the inflow pipe. Standing water was observed in the East Lake Sammamish dry well during interludes between storms. This may be a local effect of the dry well or a regional effect of hillslope drainage.

These residential infiltration systems are likely to be able to manage the roof runoff generated in short, intense periods of rain, for example, of 10 mm of rain over an hour. The behavior of these systems during longer, more intense, or frequent storms is difficult to predict especially if local groundwater rises close to the bottom of the infiltration structures.

Infiltration rates, at times, were equal to or more rapid than might be expected based on general region-wide properties of the sites' soils. At Union Hill, the specific discharge of 0.03 m/hr inferred from the measured data is less than the values used to design the trench (0.2 m/hr) but approximately equal to 0.025 m/hr which is the Soil Conservation Service (renamed the Natural Resource Conservation Service) estimate of infiltration rate for a sandy loam (e.g., Alderwood soil) (King Co., 1990). At East Lake Sammamish, the specific discharge of 0.4 m/hr is greater than the estimate of infiltration rate for loamy sand soil present at the site (0.1m/hr, King Co., 1990).

Chapter 4: Conclusions

On-site stormwater detention and infiltration at the scale of an individual dwelling or commercial building are management alternatives for mitigating some of the deleterious hydrological effects of residential development. This report has examined hypothetical detention systems using a simulation model and two infiltration systems with field studies. Both types of systems receive only roof runoff which comprises only a small portion of the total runoff generated by residential land areas. Roof runoff does, however, demonstrate the small-scale short-term characteristics of storm flow generated by urban areas: increased peak rate variability and rapid recession once storms end.

On-site residential detention and infiltration systems can restore some of the hydrologic characteristics of a forested zero-order basin. Detention systems can store stormwater and release it gradually as would a forest soil. Based on rainfall records for the Water Years 1991 to 1993 from Novelty Hill, WA, a detention system requires 20 mm of storage for a given roof area (or other impervious catchment) with a 5 mm/day (for the same area) release rate to increase the duration of low flow to a level observed for discharge from a natural first-order basin. A minimum of 100 mm of storage (over the roof area) with a release rate of 5 mm/day is needed to control peak rates of flow and their duration to levels more representative of storm flow from an equivalent area of forest, though the release rate still exceeds expected runoff rates for a forested area. having the same general plan form and topography.

On-site residential detention systems can serve multiple purposes by regulating storm flow and supplying water for domestic uses. Multipurpose detention systems generally need to be large to provide a supply of water: a storage volume equal to 20 days of the residence's daily water usage may be necessary for a somewhat reliable supply during winter; and, for summer irrigation, 250 mm of storage for the area being irrigated may be needed.

Infiltration systems can be as effective as the largest detention systems considered in this report for reducing the volume and rate of stormwater delivered to receiving waters. The two infiltration systems monitored during the winter of 1994-95 transported all of the roof runoff into the soil where it became subsurface flow. Seepage was not observed downslope from the trench indicating that the subsurface flow was not contributing to overland or channel flow in the immediate area.

Detention and infiltration systems can be compared with each other in terms of two fundamental physical characteristics: storage volume, and outflow rate. A summary of these characteristics for detention and infiltration systems is provided in Table 4.1:

Table 4.1: Summary of storage volume and outflow rate for detention and infiltration systems

	Detention	Infiltration
Storage volume	0.5-10 m^3	1.5*-3.2 m^3
Outflow rate	0.1-1 m^3/day	0.4 - 1 m^3/hr

* 1.5 m^3 is the estimated storage volume of the Union Hill infiltration trench when there is a 0.3 m depth of standing water in the trench.

Detention systems must have low release rates to approximate the rate of storm flow from a forested basin. As a result, these systems must have appropriately large storage capacities on the order of 100 mm over the roof area to avoid spills (i.e., high rate discharges). Infiltration systems can better regulate storm flow with a smaller storage volume because the outflow is distributed to the surrounding soil which delays its movement to a receiving water body. Stormwater control, however, is only maintained as long as the water remains in the ground as subsurface flow and the local groundwater level remains below the infiltration structure.

Both systems offer ancillary benefits. Detention systems can be designed and operated to supply water to a residence. Infiltration systems can increase local aquifer recharge and help to sustain base flow in streams. The value of these benefits depends on the abundance of local water supply and the general ecological health of associated streams.

On-site residential stormwater management represents a largely untapped strategy for mitigating some of the deleterious hydrologic effects of urbanization. On-site detention and infiltration provide alternatives to larger-scale stormwater management activities. These alternatives can better replicate some hydrologic characteristics of pre-development zero-order basins by increasing the spatial distribution of stormwater storage and promoting the entry of stormwater into the soil.

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Appendix A: Infiltration system dimensions

This appendix has been develped for use by individual home owners as well as land develpment and landscaping professionals. It is intended for use in the Puget Sound lowlands of Washington State.

An infiltration system should infiltrate roof runoff from most storms without overflowing. A relatively simple mass balance approach is used for determining the appropriate dimensions of an infiltration system for a given roof area and soil type. The most important part of sizing an infiltration system is to provide an adequate overflow path for any storm. Once an overflow path is planned, the dimensions of an infiltration system needed to control roof runoff generated by a particular rainfall pattern (e.g., the record from a nearby rain gauge of a large storm, the 10-year, 24-hour design storm, or the total depth of large storm), can be estimated.

This appendix provides a simple design standard for calculating the dimensions of an infiltration system. The standard is that the total volume of runoff generated by a roof during a design storm must be stored or infiltrated by the system during the storm. The infiltration system is assumed to be empty at the beginning of the storm and full at the end of the storm. The rainfall and infiltration rates are assumed to be constant for the duration of the storm. These assumptions are not conservative, where the infiltration rates used should include a factor of safety.

The design standard has been used to develop Tables A.1 through A.5 which provide dimensions for infiltration systems based on the total rain depth for the 2-year, 24-hour design storm developed from rainfall records at Seattle-Tacoma International Airport (Sea-Tac). The rainfall depth at Sea-Tac for this design storm (50 mm or 2 in) will be equaled or exceeded during a 24 hour period, on average, every other year. Storms of this magnitude are much more frequent in the southern and eastern regions of Puget Sound especially at higher elevations.

Infiltration systems constructed to manage runoff from the 2 yr, 24 hr design storm for Sea-Tac will control all roof runoff during most storms in the Puget Sound lowlands and will infiltrate most of the total volume of roof runoff produced during a year. As a result, infiltration systems designed for this storm provide most of the potential benefits of on-site stormwater management without being prohibitively expensive or large. This design storm, however, is only one standard that can be used to design infiltration trenches.

Other standards will need to be considered in three situations:

1. where local governments have established another standard;
2. in areas receiving more rain than Sea-Tac; or
3. when a higher level of stormwater control is desired (i.e., to reduce the frequency that the system overflows).

Some local governments require use of actual rainfall records or larger design storms (e.g., the 10 yr, 24 hr storm) to estimate the runoff from a new development and the necessary level of control. In some cases, there are uniform standards such as 10 m (30 feet) of infiltration trench for every 100 m^2 (1000 ft^2) of roof area. In these cases, the applicable requirements must be followed.

To design an infiltration system in areas receiving more rain than Sea-Tac or for a higher level of stormwater control, a larger rainfall depth can be used to calculate inflow to the system using the mass balance equations described in Section A.3. Alternatively, the dimensions given in Tables A.1 - A.5 can be scaled up using a factor for a particular site or storm. These modifications are described in more detail below.

In the end, infiltration system dimensions must comply with any applicable local design requirements. In other cases, an infiltration system of any size is likely to provide some stormwater management benefits. The guidelines listed below are only suggestions. Larger systems will be able to manage roof runoff during larger storms. Larger systems also provide a margin of extra capacity which may extend the system's useful life. If a site cannot accommodate an infiltration system with the suggested dimensions, and there are no local drainage requirements, a smaller system can be used provided there is an adequate overflow path.

A.1 Infiltration system dimension tables

Tables A.1 through A.5 show values for a single design variable (e.g., trench length) for each system assuming all other dimensions are constant. The values in the tables are arrayed according to the infiltration capacity of the soil and roof area. The following assumptions were made for each type of system though different designs can be used if they provide equivalent storage volume and infiltration (i.e., plan view) area.

Infiltration trenches are 0.6 m (2 ft) wide and 0.5 m (1.6 ft) deep and filled with washed rock having a porosity of 30%. The washed rock is placed at a depth of approximately 1 - 2 m (3 - 7 ft) below the final grade of the land surface. The primary design variable is the length of trench. For wider, shallower trenches, the dimensions of a dispersion trench can be used.

Dry wells are cylindrical containers with an open bottom and no fill material inside. They are generally 1 to 2 m (3 to 7 ft) below the ground surface. The primary design variable is the diameter of the dry well and can range from less than 1 m (3 ft) to 2 or more meters (7 or more feet).

Tanks are rectangular with a 1.5 m (5 ft) width and 1.5 m (5 ft) depth and no fill material inside. The primary design variable is the length of the tank.

Dispersion trenches are 1 m (3 ft) wide and 0.3 m (1 ft) deep and filled with washed rock having a porosity of 30%. The primary design variable is trench length. For a narrower, deeper trench, the dimensions of an infiltration trench can be used.

Surface dispersion systems can be tailored to any site. For this appendix, surface dispersion is assumed to have a 0.5 m (1.6 ft) wide flow path and does not allow more than 1 cm (0.4 in) of water to pond on the ground. The primary design variable is the vegetated flow path length.

A.2 Use of tables

The first column in each table is roof area. At the top of the remaining columns, there are representative infiltration capacities for soil. Below each of these infiltration capacities, there are values for the design variable (e.g., trench length, dry well depth, dispersion area) based on the roof area contributing to the system. The infiltration capacity (at the top of the columns) that is closest to the infiltration capacity of the soil at the site should be used. Dimensions for the primary design variable are in the column under the infiltration capacity. The dimensions are in rows corresponding to the roof area that will be served by the system.

Table A.1 (English)

Infiltration Trench Dimensions

	Width (ft)	Depth (ft)	Porosity
	2	1.6	0.3
Design Storm:	Depth (in)	Hours	
	2	24	

Length (ft) of trench for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (in/hr)				
(ft^2)	7.8	2.4	1	0.5	0.05
100	3.00	3.00	3.35	5.76	14.68
200	3.00	3.11	6.70	11.52	29.35
300	3.00	4.66	10.05	17.28	44.03
400	3.00	6.21	13.40	23.05	58.71
500	3.00	7.77	16.75	28.81	73.38
600	3.03	9.32	20.10	34.57	88.06
700	3.54	10.87	23.45	40.33	102.73
800	4.04	12.42	26.80	46.09	117.41
900	4.55	13.98	30.15	51.85	132.09
1000	5.05	15.53	33.50	57.62	146.76
1100	5.56	17.08	36.85	63.38	161.44
1200	6.07	18.64	40.20	69.14	176.12
1300	6.57	20.19	43.55	74.90	190.79
1400	7.08	21.74	46.90	80.66	205.47
1500	7.58	23.30	50.25	86.42	220.15
1600	8.09	24.85	53.59	92.19	234.82
1700	8.59	26.40	56.94	97.95	249.50
1800	9.10	27.95	60.29	103.71	264.18
1900	9.60	29.51	63.64	109.47	278.85
2000	10.11	31.06	66.99	115.23	293.53
2100	10.61	32.61	70.34	120.99	308.20
2200	11.12	34.17	73.69	126.76	322.88
2300	11.62	35.72	77.04	132.52	337.56
2400	12.13	37.27	80.39	138.28	352.23
2500	12.64	38.83	83.74	144.04	366.91
2600	13.14	40.38	87.09	149.80	381.59
2700	13.65	41.93	90.44	155.56	396.26
2800	14.15	43.48	93.79	161.33	410.94
2900	14.66	45.04	97.14	167.09	425.62
3000	15.16	46.59	100.49	172.85	440.29

Table A.2 (English)

Dry Well Dimensions

	Depth (ft)	
	5	
Design Storm:	Depth (in)	Hours
	2	24

Diameter (ft) of dry well for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (in/hr)				
(ft^2)	7.8	2.4	1	0.5	0.05
100	2.00	2.00	2.00	2.00	2.00
200	2.00	2.00	2.00	2.00	2.00
300	2.00	2.00	2.00	2.00	2.00
400	2.00	2.00	2.00	2.00	2.03
500	2.00	2.00	2.00	2.10	2.27
600	2.00	2.00	2.12	2.30	2.48
700	2.00	2.00	2.29	2.48	2.68
800	2.00	2.06	2.45	2.65	2.87
900	2.00	2.19	2.60	2.81	3.04
1000	2.00	2.31	2.74	2.96	3.21
1100	2.00	2.42	2.87	3.11	3.36
1200	2.00	2.53	3.00	3.25	3.51
1300	2.00	2.63	3.12	3.38	3.66
1400	2.00	2.73	3.24	3.51	3.79
1500	2.00	2.83	3.35	3.63	3.93
1600	2.01	2.92	3.46	3.75	4.06
1700	2.07	3.01	3.57	3.86	4.18
1800	2.13	3.10	3.67	3.98	4.30
1900	2.19	3.18	3.77	4.09	4.42
2000	2.24	3.26	3.87	4.19	4.53
2100	2.30	3.34	3.97	4.29	4.65
2200	2.35	3.42	4.06	4.40	4.76
2300	2.41	3.50	4.15	4.49	4.86
2400	2.46	3.58	4.24	4.59	4.97
2500	2.51	3.65	4.33	4.69	5.07
2600	2.56	3.72	4.41	4.78	5.17
2700	2.61	3.79	4.50	4.87	5.27
2800	2.65	3.86	4.58	4.96	5.36
2900	2.70	3.93	4.66	5.05	5.46
3000	2.75	4.00	4.74	5.13	5.55

Table A.3 (English)

Infiltration Tank Dimensions

	Width (ft)	Depth (ft)
	5	5
Design Storm:	Depth (in)	Hours
	2	24

Length (ft) of tank for design storm

	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
Roof Area	Infiltration Rates (in/hr)				
(ft^2)	7.8	2.4	1	0.5	0.05
100	1.50	1.50	1.50	1.50	1.50
200	1.50	1.50	1.50	1.50	1.50
300	1.50	1.65	1.82	1.90	1.96
400	1.61	2.20	2.43	2.53	2.62
500	2.01	2.75	3.04	3.16	3.27
600	2.42	3.30	3.65	3.79	3.92
700	2.82	3.85	4.26	4.43	4.58
800	3.22	4.40	4.87	5.06	5.23
900	3.62	4.95	5.47	5.69	5.89
1000	4.03	5.50	6.08	6.32	6.54
1100	4.43	6.05	6.69	6.95	7.19
1200	4.83	6.60	7.30	7.59	7.85
1300	5.23	7.15	7.91	8.22	8.50
1400	5.64	7.70	8.51	8.85	9.16
1500	6.04	8.26	9.12	9.48	9.81
1600	6.44	8.81	9.73	10.11	10.46
1700	6.84	9.36	10.34	10.75	11.12
1800	7.25	9.91	10.95	11.38	11.77
1900	7.65	10.46	11.55	12.01	12.42
2000	8.05	11.01	12.16	12.64	13.08
2100	8.45	11.56	12.77	13.28	13.73
2200	8.86	12.11	13.38	13.91	14.39
2300	9.26	12.66	13.99	14.54	15.04
2400	9.66	13.21	14.60	15.17	15.69
2500	10.06	13.76	15.20	15.80	16.35
2600	10.47	14.31	15.81	16.44	17.00
2700	10.87	14.86	16.42	17.07	17.66
2800	11.27	15.41	17.03	17.70	18.31
2900	11.67	15.96	17.64	18.33	18.96
3000	12.08	16.51	18.24	18.96	19.62

Table A.4 (English)

Dispersion Trench Dimemsions

	Width (ft)	Depth (ft)	Porosity
	3	1.6	0.3
Design Storm:	Depth (in)	Hours	
	2	24	

Length (ft) of trench for design storm

Roof Area (ft^2)	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (in/hr)				
	7.8	2.4	1	0.5	0.05
100	3.00	3.00	3.00	3.84	9.78
200	3.00	3.00	4.47	7.68	19.57
300	3.00	3.11	6.70	11.52	29.35
400	3.00	4.14	8.93	15.36	39.14
500	3.00	5.18	11.17	19.21	48.92
600	3.00	6.21	13.40	23.05	58.71
700	3.00	7.25	15.63	26.89	68.49
800	3.00	8.28	17.86	30.73	78.27
900	3.03	9.32	20.10	34.57	88.06
1000	3.37	10.35	22.33	38.41	97.84
1100	3.71	11.39	24.56	42.25	107.63
1200	4.04	12.42	26.80	46.09	117.41
1300	4.38	13.46	29.03	49.93	127.20
1400	4.72	14.49	31.26	53.78	136.98
1500	5.05	15.53	33.50	57.62	146.76
1600	5.39	16.57	35.73	61.46	156.55
1700	5.73	17.60	37.96	65.30	166.33
1800	6.07	18.64	40.20	69.14	176.12
1900	6.40	19.67	42.43	72.98	185.90
2000	6.74	20.71	44.66	76.82	195.69
2100	7.08	21.74	46.90	80.66	205.47
2200	7.41	22.78	49.13	84.50	215.25
2300	7.75	23.81	51.36	88.35	225.04
2400	8.09	24.85	53.59	92.19	234.82
2500	8.42	25.88	55.83	96.03	244.61
2600	8.76	26.92	58.06	99.87	254.39
2700	9.10	27.95	60.29	103.71	264.18
2800	9.43	28.99	62.53	107.55	273.96
2900	9.77	30.03	64.76	111.39	283.74
3000	10.11	31.06	66.99	115.23	293.53

Table A.5 (English)

Surface Dispersion Dimensions

	Width (ft)	Depth (ft)	Porosity
	0.5	0.01	0.5
Design Storm:	Depth (in)	Hours	
	2	24	

Length (ft) of dispersion area for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (in/hr)				
(ft^2)	7.8	2.4	1	0.5	0.05
100	10.00	10.00	16.62	34.59	391.78
200	10.00	13.65	33.25	69.18	783.56
300	10.00	20.48	49.87	103.78	1175.35
400	10.00	27.30	66.50	138.37	1567.13
500	10.41	34.13	83.12	172.96	1958.91
600	12.50	40.96	99.75	207.55	2350.69
700	14.58	47.78	116.37	242.14	2742.47
800	16.66	54.61	133.00	276.73	3134.26
900	18.74	61.44	149.62	311.33	3526.04
1000	20.83	68.26	166.24	345.92	3917.82
1100	22.91	75.09	182.87	380.51	4309.60
1200	24.99	81.91	199.49	415.10	4701.39
1300	27.07	88.74	216.12	449.69	5093.17
1400	29.16	95.57	232.74	484.29	5484.95
1500	31.24	102.39	249.37	518.88	5876.73
1600	33.32	109.22	265.99	553.47	6268.51
1700	35.41	116.05	282.62	588.06	6660.30
1800	37.49	122.87	299.24	622.65	7052.08
1900	39.57	129.70	315.86	657.25	7443.86
2000	41.65	136.52	332.49	691.84	7835.64
2100	43.74	143.35	349.11	726.43	8227.42
2200	45.82	150.18	365.74	761.02	8619.21
2300	47.90	157.00	382.36	795.61	9010.99
2400	49.98	163.83	398.99	830.20	9402.77
2500	52.07	170.66	415.61	864.80	9794.55
2600	54.15	177.48	432.24	899.39	10186.34
2700	56.23	184.31	448.86	933.98	10578.12
2800	58.31	191.13	465.48	968.57	10969.90
2900	60.40	197.96	482.11	1003.16	11361.68
3000	62.48	204.79	498.73	1037.76	11753.46

Table A.1 (Metric)

Infiltration Trench Dimensions

	Width (m)	Depth (m)	Porosity
	0.6	0.5	0.3
Design Storm:	Depth (mm)	Hours	
	50	24	

Length (m) of trench for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (mm/hr)				
(m^2)	200	60	25	12	1
10	1.00	1.00	1.11	1.90	4.79
20	1.00	1.05	2.22	3.81	9.58
30	1.00	1.57	3.33	5.71	14.37
40	1.00	2.10	4.44	7.61	19.16
50	1.00	2.62	5.56	9.51	23.95
60	1.01	3.14	6.67	11.42	28.74
70	1.18	3.67	7.78	13.32	33.52
80	1.35	4.19	8.89	15.22	38.31
90	1.52	4.72	10.00	17.12	43.10
100	1.68	5.24	11.11	19.03	47.89
110	1.85	5.77	12.22	20.93	52.68
120	2.02	6.29	13.33	22.83	57.47
130	2.19	6.81	14.44	24.73	62.26
140	2.36	7.34	15.56	26.64	67.05
150	2.53	7.86	16.67	28.54	71.84
160	2.69	8.39	17.78	30.44	76.63
170	2.86	8.91	18.89	32.34	81.42
180	3.03	9.43	20.00	34.25	86.21
190	3.20	9.96	21.11	36.15	91.00
200	3.37	10.48	22.22	38.05	95.79
210	3.54	11.01	23.33	39.95	100.57
220	3.70	11.53	24.44	41.86	105.36
230	3.87	12.05	25.56	43.76	110.15
240	4.04	12.58	26.67	45.66	114.94
250	4.21	13.10	27.78	47.56	119.73
260	4.38	13.63	28.89	49.47	124.52
270	4.55	14.15	30.00	51.37	129.31
280	4.71	14.68	31.11	53.27	134.10
290	4.88	15.20	32.22	55.18	138.89
300	5.05	15.72	33.33	57.08	143.68

Table A.2 (Metric)

Dry Well Dimensions

	Depth (m)	
	1.5	
Design Storm:	Depth (mm)	Hours
	50	24

Diameter (m) of dry well for design storm

Roof Area (m²)	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (mm/hr)				
	200	60	25	12	1
10	0.50	0.50	0.50	0.50	0.50
20	0.50	0.50	0.50	0.50	0.50
30	0.50	0.50	0.50	0.52	0.56
40	0.50	0.50	0.55	0.60	0.65
50	0.50	0.52	0.62	0.67	0.72
60	0.50	0.57	0.67	0.73	0.79
70	0.50	0.62	0.73	0.79	0.86
80	0.50	0.66	0.78	0.84	0.91
90	0.50	0.70	0.83	0.90	0.97
100	0.50	0.74	0.87	0.94	1.02
110	0.53	0.77	0.91	0.99	1.07
120	0.55	0.81	0.95	1.03	1.12
130	0.57	0.84	0.99	1.08	1.17
140	0.59	0.87	1.03	1.12	1.21
150	0.62	0.90	1.07	1.16	1.25
160	0.64	0.93	1.10	1.19	1.29
170	0.66	0.96	1.14	1.23	1.33
180	0.67	0.99	1.17	1.27	1.37
190	0.69	1.01	1.20	1.30	1.41
200	0.71	1.04	1.23	1.33	1.45
210	0.73	1.07	1.26	1.37	1.48
220	0.75	1.09	1.29	1.40	1.52
230	0.76	1.12	1.32	1.43	1.55
240	0.78	1.14	1.35	1.46	1.58
250	0.79	1.16	1.38	1.49	1.62
260	0.81	1.19	1.40	1.52	1.65
270	0.83	1.21	1.43	1.55	1.68
280	0.84	1.23	1.46	1.58	1.71
290	0.86	1.25	1.48	1.61	1.74
300	0.87	1.27	1.51	1.63	1.77

Table A.3 (Metric)

Infiltration Tank Dimensions

	Width (m)	Depth (m)
	1.5	1.5
Design Storm:	Depth (mm)	Hours
	50	24

Length (m) of tank for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (mm/hr)				
(m^2)	200	60	25	12	1
10	0.50	0.50	0.50	0.50	0.50
20	0.50	0.50	0.50	0.50	0.50
30	0.50	0.50	0.53	0.59	0.66
40	0.50	0.54	0.70	0.79	0.88
50	0.50	0.68	0.88	0.99	1.10
60	0.50	0.81	1.05	1.18	1.32
70	0.50	0.95	1.23	1.38	1.54
80	0.57	1.08	1.40	1.58	1.76
90	0.64	1.22	1.58	1.77	1.98
100	0.71	1.36	1.75	1.97	2.20
110	0.78	1.49	1.93	2.17	2.42
120	0.85	1.63	2.11	2.36	2.64
130	0.92	1.76	2.28	2.56	2.86
140	0.99	1.90	2.46	2.76	3.08
150	1.06	2.03	2.63	2.96	3.30
160	1.13	2.17	2.81	3.15	3.52
170	1.21	2.30	2.98	3.35	3.74
180	1.28	2.44	3.16	3.55	3.96
190	1.35	2.57	3.33	3.74	4.18
200	1.42	2.71	3.51	3.94	4.40
210	1.49	2.85	3.68	4.14	4.62
220	1.56	2.98	3.86	4.33	4.84
230	1.63	3.12	4.04	4.53	5.06
240	1.70	3.25	4.21	4.73	5.28
250	1.77	3.39	4.39	4.93	5.50
260	1.84	3.52	4.56	5.12	5.72
270	1.91	3.66	4.74	5.32	5.94
280	1.99	3.79	4.91	5.52	6.16
290	2.06	3.93	5.09	5.71	6.38
300	2.13	4.07	5.26	5.91	6.60

Table A.4 (Metric)

Dispersion Trench Dimemsions

	Width (m)	Depth (m)	Porosity
	1	0.5	0.3
Design Storm:	Depth (mm)	Hours	
	50	24	

Length (m) of trench for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (mm/hr)				
(m^2)	**200**	**60**	**25**	**12**	**1**
10	1.00	1.00	1.00	1.14	2.87
20	1.00	1.00	1.33	2.28	5.75
30	1.00	1.00	2.00	3.42	8.62
40	1.00	1.26	2.67	4.57	11.49
50	1.00	1.57	3.33	5.71	14.37
60	1.00	1.89	4.00	6.85	17.24
70	1.00	2.20	4.67	7.99	20.11
80	1.00	2.52	5.33	9.13	22.99
90	1.00	2.83	6.00	10.27	25.86
100	1.01	3.14	6.67	11.42	28.74
110	1.11	3.46	7.33	12.56	31.61
120	1.21	3.77	8.00	13.70	34.48
130	1.31	4.09	8.67	14.84	37.36
140	1.41	4.40	9.33	15.98	40.23
150	1.52	4.72	10.00	17.12	43.10
160	1.62	5.03	10.67	18.26	45.98
170	1.72	5.35	11.33	19.41	48.85
180	1.82	5.66	12.00	20.55	51.72
190	1.92	5.97	12.67	21.69	54.60
200	2.02	6.29	13.33	22.83	57.47
210	2.12	6.60	14.00	23.97	60.34
220	2.22	6.92	14.67	25.11	63.22
230	2.32	7.23	15.33	26.26	66.09
240	2.42	7.55	16.00	27.40	68.97
250	2.53	7.86	16.67	28.54	71.84
260	2.63	8.18	17.33	29.68	74.71
270	2.73	8.49	18.00	30.82	77.59
280	2.83	8.81	18.67	31.96	80.46
290	2.93	9.12	19.33	33.11	83.33
300	3.03	9.43	20.00	34.25	86.21

Table A.5 (Metric)

Surface Dispersion Dimensions

	Width (m)	Depth (m)	Porosity
	0.5	0.01	0.5
Design Storm:	Depth (mm)	Hours	
	50	24	

Length (m) of dispersion area for design storm

Roof Area	Medium Sand	Fine, Loamy Sand	Sandy Loam	Loam	Clay Loam
	Infiltration Rates (mm/hr)				
(m^2)	200	60	25	12	1
10	3.00	3.00	3.00	3.41	34.48
20	3.00	3.00	3.31	6.83	68.97
30	3.00	3.00	4.96	10.24	103.45
40	3.00	3.00	6.61	13.65	137.93
50	3.00	3.46	8.26	17.06	172.41
60	3.00	4.15	9.92	20.48	206.90
70	3.00	4.84	11.57	23.89	241.38
80	3.00	5.54	13.22	27.30	275.86
90	3.00	6.23	14.88	30.72	310.34
100	3.00	6.92	16.53	34.13	344.83
110	3.00	7.61	18.18	37.54	379.31
120	3.00	8.30	19.83	40.96	413.79
130	3.00	9.00	21.49	44.37	448.28
140	3.00	9.69	23.14	47.78	482.76
150	3.12	10.38	24.79	51.19	517.24
160	3.33	11.07	26.45	54.61	551.72
170	3.54	11.76	28.10	58.02	586.21
180	3.75	12.46	29.75	61.43	620.69
190	3.95	13.15	31.40	64.85	655.17
200	4.16	13.84	33.06	68.26	689.66
210	4.37	14.53	34.71	71.67	724.14
220	4.58	15.22	36.36	75.09	758.62
230	4.79	15.92	38.02	78.50	793.10
240	4.99	16.61	39.67	81.91	827.59
250	5.20	17.30	41.32	85.32	862.07
260	5.41	17.99	42.98	88.74	896.55
270	5.62	18.69	44.63	92.15	931.03
280	5.83	19.38	46.28	95.56	965.52
290	6.04	20.07	47.93	98.98	1000.00
300	6.24	20.76	49.59	102.39	1034.48

The dimensions provided in this table are based on control of 50 mm (2 in) of rain in 24 hours assuming the infiltration system is initially empty. Around Sea-Tac airport and in other areas receiving similar amounts of rain, such a system will overflow, on average, every other year and several times in some years. The dimensions of an infiltration system can be scaled up in areas in the Puget Sound lowlands receiving more rain or for a system that will overflow less frequently. The primary design variable (e.g., trench length, dry well diameter) can be multiplied by the following factors to provide a higher level of stormwater control.

Table A.6: Scaling factors for design of infiltration system at sites around Puget Sound for design storms of various recurrence intervals

Recurrence Interval of Design Storm Depth for Seattle-Tacoma Airport Location	2 yr, 24 hr 50 mm/2 in	10 yr, 24 hr 75 mm/3 in	50 yr, 24 hr 90 mm/3.5 in
Seattle, Tacoma, Puyallup, Bellingham, Bellevue, Port Angeles, Redmond, Monroe, Kent, Auburn, Mt. Vernon, Sedro Woolley	1	1.5	1.75
Everett, Port Townsend, Friday Harbor, Oak Harbor, Anacortes	0.75	1.25	1.5
Issaquah, Enumclaw, Bremerton, Olympia	1.25	1.75	2.25
Shelton, Index, Darrington, Concrete	1.5	2	2.75

As an example, consider an infiltration trench in a sandy loam soil designed a 100 m^2 (1075 ft^2) roof. Table A.1 should be used for this example. If the system is located in Puyallup, Washington, and designed for the 2 yr, 24 hr storm, the scaling factor is 1. The required trench length is 11 m (34 ft). The preliminary design is a trench 0.6 m (2 ft) wide, 0.5 m (1.5 ft) deep with a length of 11 m (34 ft). Filter fabric is placed in the trench with washed rock on top of it and a perforated pipe in the top 0.1 m (4 in) of the washed rock. Filter fabric covers the layer of washed rock and the trench is backfilled with soil. If a 11 m (34 ft) long trench cannot be located at a site given set-backs from structures, septic systems, steep slopes, property lines, etc., two 6 m (20 ft) trenches or a 1 m wide, 7 m long trench could be used as either of these configurations provide as much infiltration area and storage volume as the preliminary design (11 m * 0.6 m or 34 ft * 2 ft).

A scaling factor can be applied easily to the values in the tables when designing an infiltration system for larger storms and for other areas in the Puget Sound lowlands. The factor corresponding to the closest city to the site and the desired design storm should be used. For an infiltration trench in the Olympia, Washington, area that can control the 2 yr, 24 hr storm, the trench length should be multiplied by 1.25 so a 14 m (46 ft) trench would be required for the 2 yr, 24 hr storm. To control the 10 yr, 24 hr storm in Shelton, Washington, the trench length should be multiplied by 2 for a total of 22 m (68 ft).

A.3 Mass balance calculations

This section describes the calculations used to determine the dimensions in tables A.1 - A.2. The approach can be used to calculate the dimensions of systems not included in the tables. Such systems may be configured differently (e.g., wider than the suggested design) or are sized for large storms using the scaling factors provided in Table A.6.

Infiltration systems are designed to allow water to infiltrate into the soil and to store a limited volume of water when the roof runoff rate exceeds the capacity of the system to infiltrate water. The performance of an infiltration system over a specified period of time can be described with a mass balance:

Inflow Volume - Outflow Volume - Overflow Volume =
Change in Storage Volume (1)

where Inflow = runoff from the roof,
Outflow = water infiltrating into the soil,
Overflow = water that cannot be infiltrated or stored in the system, and
Storage = water being held in the system until it can infiltrate.

The primary objective when designing an infiltration system is that the combined capacity of storage, outflow, and overflow must be greater than or equal to the inflow for any storm or series of storms. This objective can be satisfied in all cases by having an adequate overflow path to convey stormwater away from the system during long duration, large rainfall depth storms. In most cases, overflow should be routed to the existing or planned central stormwater drainage system (e.g., a regional storm sewer, street drains, or ditches) for the area.

The secondary design objective for an infiltration system is that change in storage plus the outflow capacity of the system will be greater than the inflow for all storms up to those of a given magnitude (i.e., the design storm). In this case, the overflow term is dropped from the mass balance:

Design Inflow Volume - Design Outflow Volume = Change in Storage (2)

This equation represents the design standard used to determine the dimensions an infiltration system. An infiltration system's dimensions must be sufficient to store or infiltrate all roof runoff for the duration of the design storm. Storage in the system is assumed to be zero at the beginning of the storm and full at the end of the storm. To use this design approach, three quantities must be calculated.

Design Inflow Volume =
Design Storm Depth * Roof Area (3)

Design Outflow Volume =
Infiltration Capacity * Duration of Storm * Infiltration Area (4)

Change in Storage =
Volume of Infiltration Structure - Volume of any fill material (5)

The design inflow volume depends on the selected design storm and the roof area served by the system. The design storm depth is equal to 50 mm (2 in) multiplied by an appropriate scaling factor for the location of the site and the desired level of stormwater control (refer to Table A.6). The design outflow volume is controlled by the plan view area over which water is infiltrated and the infiltration capacity of the soil which is a rate measured in units of length divided by time. The storage capacity is controlled by the dimensions of the infiltration structure and the presence of any fill material inside of the structure. These equations can be solved to determine if a specific system will be adequate for a given storm at a given site (i.e., is Storage Capacity > Change in Storage).

Alternatively, the equations can be solved to determine the appropriate dimensions for an infiltration system. In this case, the following equations can be used.

Infiltration Area = (Design Storm Depth * Roof Area) / (Infiltration Capacity of Soil * Duration of Storm + Depth of Infiltration Structure * Porosity of Fill) (6)

Infiltration System Length * Infiltration System Width = Infiltration Area (7)

A typical value for the porosity of washed rock is 0.3 and other values are determined as described for Equations 3 - 5. In the end, any combination of infiltration structure length and width that produces the design infiltration area will be appropriate.