Quality Assurance Project Plan

Clark County Clean Water Program Stormwater Flow Reduction Strategy Monitoring

By
Chad Hoxeng

Project Name:	**Vancouver Toyota LID Permeable Pavers Runoff Monitoring**
Project Code:	**TOYLID**
Agency Name:	**Clark County Washington**
Agency Contact Name:	**Chad Hoxeng**
Department:	**Public Works Clean Water Program**
Funding Source:	**Clark County Clean Water Fee**

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July 2009

Approved by:

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Ron Wierenga, Clean Water Program Manager, CWP

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Steve Gustafson, Principal Hydrologist, WCI

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Gregory Winters, Regional Contact, DOE

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Vancouver Toyota Site Manager (to be determined)

Signatures are not available on Internet Version.
CWP — Clean Water Program
WCI — West Consultants, Inc.
DOE — Washington State Department of Ecology

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Statement of Certification

"I certify under penalty of law, that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gathered and evaluated the information submitted. Based on my inquiry of the person or persons who manage the system or those persons directly responsible for gathering information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations."

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Acronyms

BMP	Best Management Practice
CWP	Clark County Clean Water Program
DOE	Washington State Department of Ecology
DQOs	Data Quality Objectives
EPA	Environmental Protection Agency
ICPI	Interlocking Concrete Pavement Institution
LID	Low Impact Development
LIDPS	Low Impact Development Puget Sound
MQOs	Measurement Quality Objective
NIST	National Institute of Standards and Technology
NPDES	National Pollution Discharge and Elimination System
QAPP	Quality Assurance Project Plan
QA/QC	Quality Assurance and Quality Control
QC	Quality Control
RSD	Relative Standard Deviation
SOPs	Standard Operating Procedures
TOYLID	Vancouver Toyota Low Impact Development
WCI	West Consultants, Inc
WSDOT	Washington State Department of Transportation

Abstract

This Quality Assurance Project Plan (QAPP) was prepared by Clark County Public Works Clean Water Program for the Washington Department of Ecology as required by Section S8.F.7 of the National Pollutant Discharge and Elimination System and State Waste Discharge General Permit for Discharges from Large and Medium Municipal Separate Storm Sewer Systems. This document describes Clark County's plan for assessing the effectiveness of permeable pavers as a Low Impact Development (LID) practice. This LID project is currently being constructed at the Vancouver Toyota Dealership, Clark County, Washington.

The Clean Water Program will monitor precipitation volume, stormwater runoff volume, and onsite conditions from this site for a minimum of two years once construction activities are completed. The purpose of this monitoring project is to assess the effectiveness of permeable pavers as a LID practice in order to provide a basis for future reduction of stormwater pollutants discharged to receiving waters. The parameters measured will include continuous precipitation data at the site and continuous flow measurements from the site's storm sewer outfall.

The primary goal of this Plan is to assure the delivery of defensible products and decisions by documenting the quality and integrity of the monitoring efforts, the representativeness of the results, the precision and accuracy of the analyses, and the completeness of the data.

Clark County NPDES Stormwater Hydrologic Management Best Management Practice (BMP) Evaluation Monitoring Quality Assurance Project Plan

Purpose of the Quality Assurance Project Plan

This Quality Assurance Project Plan (QAPP) was prepared by Clark County Public Works Clean Water Program. The QAPP describes the quality assurance and quality control (QA/QC) procedures for field activities associated with stormwater monitoring conducted by Clark County, under Section S8.F.7. of the National Pollutant Discharge and Elimination System and State Waste Discharge General Permit for Discharges from Large and Medium Municipal Separate Storm Sewer Systems. The primary goal of this Plan is to assure the delivery of defensible products and decisions by documenting the quality and integrity of the monitoring efforts, the representativeness of the results, the precision and accuracy of the analyses, and the completeness of the data.

This document was developed with guidance from the Department of Ecology, Guidelines for Preparing Quality Assurance Project Plans for Environmental Studies (Ecology, 2004).

This QAPP is organized into the following sections:

A. Goals and objectives of stormwater monitoring program

B. Type, quality, and quantity of data needed to meet program objectives

C. Measurement procedures needed to acquire data needed

D. Quality assurance and quality control procedures to ensure the Plan is implemented as prescribed

E. Assessment procedures to determine if the data conform to the specified criteria and will satisfy the program objectives, and the analysis and format for presentation of the results

A series of Standard Operating Procedures (SOPs) are being developed to provide guidance to users of this Plan. These SOPs are in draft form and are described in Appendix I. Modifications may be made to these SOPs to incorporate more appropriate procedures for this project once construction activities are completed.

Section A. Goals and Objectives of the Monitoring Program

This section covers basic program management, including history and objectives, delegation of responsibilities, and other details to ensure that the program is well defined and understood by all participants. The following elements are included:

3. Background and Problem Statement
4. Program Description
5. Organization and Schedule

3. Background

Flow reduction strategy monitoring is required under Section S8.F.7 of Clark County's National Pollutant Discharge and Elimination System (NPDES) and State Waste Discharge General Permit for Discharges from Large and Medium Municipal Separate Storm Sewer Systems (Phase I Permit). To address this monitoring requirement, stormwater monitoring will be conducted at a proposed Low Impact Development (LID) site within Clark County.

Low Impact Development practices have been developed as an innovative stormwater management approach to manage runoff at the source using uniformly distributed decentralized micro-scale controls. The goal of LID is to mimic a site's predevelopment hydrology by using design techniques that infiltrate, filter, store, evaporate, and detain runoff close to its source instead of conveying stormwater runoff to “end of the pipe” off-site stormwater facilities.

The use of LID practices, such as pervious pavers, is relatively new and not widespread in Clark County. As the use of LID projects become more common, studies involving long-term data collection and analysis of LID are needed to accurately assess the effectiveness of these practices.

3.1. Permit Overview and Monitoring Requirements

The Washington State Department of Ecology (Ecology or DOE) issued the final NPDES and Phase I Permit on January 17, 2007. The Phase I Permit (Permit) applies to all entities in Washington State that are required to have stormwater permit coverage under current (Phase I) U.S. Environmental Protection Agency (EPA) stormwater regulations. This includes unincorporated portions of counties whose populations exceed 100,000 in the 1990 census, specifically Clark County.

In accordance with the Permit, each permittee is required to develop and implement a comprehensive long-term monitoring program consistent with Special Condition S8 of the Permit. In general, the required monitoring program shall include the following components:

- Stormwater monitoring (S8.D)
- Targeted stormwater management program effectiveness monitoring (S8.E)
- Stormwater treatment and hydrologic management best management practices evaluation monitoring (S8.F).

A separate QAPP will be developed for each component of the monitoring program; this QAPP provides details on the hydrologic best management practices (BMPs) component only (S.8.F.7).

Ecology requires each permittee to conduct a comprehensive long-term stormwater monitoring program in order to provide a basis for future reduction of stormwater

pollutants discharged to receiving waters. The information obtained from this project will contribute to the ongoing efforts in evaluating the effectiveness of stormwater management practices, as well as increase understanding about potential seasonal patterns in stormwater character using flow reduction LID strategies.

3.2. Hydrologic Management Effectiveness Monitoring

Under Section S8.F.7 of the Permit, Clark County is required to monitor the effectiveness of one flow reduction strategy addressing the following criteria:

(1) Monitor the effectiveness of one flow reduction strategy that is in use or planned for installation within the county.
(2) Monitoring of a flow reduction strategy shall include continuous rainfall and surface runoff monitoring.
(3) Flow reduction strategies shall be monitored through either a paired site study or against a predicted outcome.

Clark County will monitor, characterize, and assess the effectiveness of pervious pavers, a LID flow reduction strategy, and compare results against a predicted outcome.

3.3. Study Area

The proposed project addresses the stormwater design for Vancouver Toyota car dealership located in the Northeast corner of section 16, township 2 north, range 2 east of the Willamette Meridian. The site is located north of SR-500, south of NE 53rd Street, east of I-205, and west of 107th NE Ave (Figure 1).

The proposed project is an extension of the existing Vancouver Toyota to additional parcels located to the south and east (Figure 2). The project will include a new building, installation of sidewalk along NE 53rd Avenue, car parking/storage area, and landscaping. The size of the proposed commercial building is approximately 41,985 square feet, and the total size of the parcels is 10.33 acres (Figure 3).

Existing Conditions

The eastern half of the site is currently an unimproved field with sparse grass coverage. The western half of the site has been used as a parking lot and has a layer of gravel over existing soils. There are no existing structures on the site. The historical condition of the site was evaluated using 1974 aerial photography. At that time, the entire site was a field covered with sparse grass without trees (Figure 4).

There are three soil types on site; Tisch Silt Loam (ThA), Lauren Gravelly Loam (LgB), and Wind River Gravelly Loam (WrB). Approximately 90% of the site is covered by ThA, which falls in hydrologic soil group D. Group D soils have a high runoff potential and a very slow infiltration rate when thoroughly wet. The remaining soil types (approximately 10%) fall into hydrologic soil groups A and B which have a high rate (group A) or moderate rate (group B) of infiltration when thoroughly wet. Water movement through these soils is moderately rapid.

Subsurface conditions were explored with fourteen soil test borings and were generally divided into three strata; fill was found at three to seven inches of depth, buried peat and topsoil was found about six to twenty inches below the fill, and native gravelly deposits were found eight to twelve feet below the buried peat and topsoil (Appendix D)

Existing topography slopes slightly from the northeast to the southwest with a small portion of the eastern side of the site sloping towards the east. Under these existing conditions, stormwater runoff will flow out the southwest and southeast corners of the site.

Proposed Developed Condition

Once constructed, the stormwater runoff from the building, parking lots, and the landscaped area will be infiltrated onsite using Eco-Loc Permeable Concrete Pavers. Stormwater will be treated by means of filtration as the stormwater runoff is infiltrated through the base course and underlying sandy soil (Figures 5 and 6).

As described in Appendix C, the base course is designed to store the 100-year storm (24-hr rainfall = 4.30 inches). The designed infiltration rate for the system is 0.5 in/hr and the design void ratio is 0.10. The base course section is designed to be 12 inches deep. The 100-year storm runoff fills the base to an elevation of 0.37 feet from the bottom of the base course section.

Details of the soils that will remain under the pervious pavers, base and bedding materials, and depths are described in Appendices A and C. Infiltration rates of the existing basin and pervious area can be found in Appendix B.

An overflow drainage system will be included in the project. Any runoff that does not infiltrate will be captured and flow downhill to the south and west margins. Curbs along these margins prevent flow from exiting or entering the site. A series of inline field inlets within the south and west perimeter of the site will convey any stormwater runoff that does not infiltrate the site to a single outfall in the southwest corner of the property (Figure 7). Field inlets in this study are concrete structures fitted with a slotted grate to collect stormwater runoff and route it through underground 10 inch non-perforated pipes (Figure 8).

Any runoff from the site collected by the field inlets will discharge to a single outfall which drains into a Washington State Department of Transportation (WSDOT) drainage ditch at the southwest corner of the property as indicated in Figure 8.

3.3.1. Permeable Pavement

The term permeable pavement, or pervious pavement, is used to describe pavements that allow stormwater to infiltrate into a gravel filled reservoir below the pavement surface. This reservoir provides temporary storage of stormwater before it infiltrates into the subgrade or is drained away by perforated pipes. There are three types of permeable pavement designs commonly used to reduce surface runoff volumes; modular interlocking concrete block pavers, plastic lattice of grid systems, and porous asphalt or

pervious concrete. Pervious pavement in this study refers to modular interlocking concrete block pavers.

Modular interlocking concrete block pavers consist of impervious concrete blocks that allow water to infiltrate into the base course through voids within or between the pavers. These voids are typically filled with gravels or soil and grass. The pavement sections for the TOYLID project were designed to recommendations of the Low Impact Development Technical Guidance Manual for Puget Sound (LIDPS) dated January 2005 (Appendix G), and with the Interlocking Concrete Pavement Institution (ICPI) Permeable Interlocking Concrete Pavement Design Manual (Smith, 2006).

3.3.2. Surface Runoff Reduction

Permeable pavement systems are intended to mimic natural hydrologic functions by infiltrating stormwater runoff, promoting groundwater recharge, and maintaining or augmenting baseflows (Appendix H). There have been several field studies that have quantified the runoff reduction benefit of concrete block pavers (Booth and Leavitt, 1999, and Rushton, 1999). These studies demonstrated that 80 – 97% of the surface runoff was infiltrated using pervious pavement making this an effective LID strategy for runoff management. However, little data exists in assessing the effectiveness of previous pavement over time. Recent studies have indicated that infiltration rates can be reduced due to clogging material filling voids within pervious pavement (Dierkes et al., 2002 and Hunt et al., 2002). Clark County will evaluate the effectiveness of pervious pavement over time as a flow reduction BMP for a minimum of two years. The TOYLID site provides the potential to monitor for longer periods.

4. Project Description

This section describes the project goals and objectives including the information requirements, specific target characteristics to be monitored, and the data quality required to meet the project objectives. Study area boundaries and practical constraints are also briefly discussed.

4.1. Study Goals and Objectives

The goal of this study is to assess the effectiveness of permeable pavers in reducing surface stormwater runoff and to assure the delivery of defensible products and decisions by documenting the quality and integrity of the monitoring efforts, the representativeness of the results, the precision and accuracy of the analyses, and the completeness of the data.

The objectives of the study are designed to ensure the goals of this study are adequately met and include:

1. Identify one flow reduction strategy to monitor within Clark County that is represented by the NPDES Phase Municipal Stormwater Permit (S8.F.7)
2. Collect continuous flow and precipitation data
3. Quantify the effectiveness of pervious pavers as a flow reduction strategy and compare results against a predicted outcome.
4. Validate and report the monitoring results to Ecology (S8.H.1.c)

4.3. Information Requirements

Information needed to meet the study objectives includes:

- Land use and drainage area of selected site
- Description of soil types, base and bedding materials
- Continuous record of rainfall data
- Continuous record of outfall flow data
- Onsite surface runoff monitoring observations

4.4. Data Collection

The monitoring design contains two primary components that will be conducted at the selected monitoring site for at least two years post construction completion:

- Hydrologic monitoring
- On site observations

Hydrologic Monitoring

Hydrological information is fundamental to the assessment and management of stormwater runoff impacts. Both continuous precipitation and outfall flow data are needed to help understand and manage runoff impacts. The hydrologic data will be utilized for verification of the design criteria regarding the effectiveness of the pervious pavers. Hydrological data will be shared with Ecology.

Hydrologic monitoring will provide the following:

- Precipitation intensity data
- Precipitation depth data
- Outfall flow data

On Site Observations

Onsite observations will be critical in evaluating the success of pervious pavers as a flow reduction strategy. Onsite observations, in combination with hydrologic monitoring, will help assess if the pervious pavers are functioning as designed.

Onsite observations will involve the following measurements:

- Monitoring well depth
- Ponding water
- Seepage from site perimeter
- Site condition
- Tracking of maintenance activities as described in Appendix G (if an agreement is reached between CWP and Vancouver Toyota)

4.6. Study Area Boundary

The study area is an expansion of the existing Vancouver Toyota car dealership (See Figure 3) to be completed in fall 2009. The project will include a new building, installation of sidewalk, car parking/storage area, and landscaping. The total size of the parcels is 10.33 acres and is zoned commercial.

Once constructed, the stormwater runoff from the building, parking lots, and the landscaped area will be infiltrated onsite using Eco-Loc Permeable Concrete Pavers (modular interlocking concrete block pavers), a Low Impact Development (LID) practice. The pavement sections for the TOYLID project were designed by recommendations of Low Impact Development Technical Guidance Manual for Puget Sound (LIDPS) dated January 2005 (Appendix E), and with the Interlocking Concrete Pavement Institution (ICPI) Permeable Interlocking Concrete Pavement Design Manual (Smith, 2006). Low Impact Development practices have been developed as an innovative stormwater management approach to manage stormwater runoff. Stormwater at the this site will be treated by means of filtration as the stormwater runoff is infiltrated through the base course and underlying sandy soil.

Any runoff that does not infiltrate will be captured and flow downhill to the south and west margins. A series of field inlets (Figure 7) line the south and west perimeter of the site and will convey any runoff that does not infiltrate to a single outfall in the southwest corner of the property. Curbs along these margins prevent flow from exiting and entering site (Figure 8). Stormwater monitoring wells will be installed to monitor saturation of the base materials (Figures 9 and 10).

There is no predicted runoff from the outfall for this site for all storm events less than the 100 year event. However, an overflow drainage system will be present for this project. Any runoff will discharge to a single outfall which drains into an existing WSDOT drainage ditch the southwest corner of the property. The outfall (Figures 11 and 12) for

this site will be continuously monitored for any potential runoff volume by Clark County Clean Water Program. Only runoff that enters field inlets will be conveyed to the monitoring vault (i.e. stormwater conveyance pipe is not perforated, therefore water collected/stored in base material cannot enter conveyance system to the outfall). The purpose of this study is to compare real world results with the design criteria for this project

4.7. Practical Constraints

Monitoring may be constrained by specific characteristics of the storm drain system and site hydrology. The choice of flow measurement methods and the accuracy of flow measurement will be limited by pipe diameter and slope, water depth, and backwater conditions.

The limited reliability of precipitation forecasts also creates inherent logistical challenges for onsite monitoring during storm events. Mobilization of field crews for a potential onsite monitoring event will certainly be restricted to no more than a few days prior to the storm, often only a few hours.

4.8. Data Collection

The CWP will collect continuous precipitation and outfall flow data for the duration of the study; the study will be conducted for a minimum of two years. Twelve site visits, one per month and preferably after a rainfall event, will be made each year. In addition to the monthly site visits, additional site visits will be conducted during or immediately after significant rainfall events. Significant rainfall events are defined as any storm event greater than a 2 year (2.5 inches with a 24 hour period) and will be monitored onsite during or immediately after storm event. During all site visits, observations of site conditions will be noted. These observations will include noting monitoring well depths, ponding water, perimeter seepage, contaminants, and any other abnormality that may indicate that pervious pavers are not functioning as designed.

4.9. Decision Making

This study will aid Clark County in deciding if this type of LID practice is an effective flow reduction BMP by evaluating the design criteria of this project under real world conditions of construction and maintenance.

The results of this monitoring program will not be utilized to make specific decisions. Nonetheless, these results will allow Ecology to assess the progress that is being made by stormwater management programs in utilizing LID projects as flow reducing strategies, thereby achieving the larger goal of protecting water quality in receiving waters.

5. Organization and Schedule

This section describes the components of the program team and schedule, including special training that will be required and the process of revising this document.

5.1. Roles and Responsibilities

Table 1 defines the major aspects of the program and the responsible personnel.

Table 1. Project roles and responsibilities for the TOYLID project.

Staff	Title	Responsibilities
Ron Wierenga CWP 360-397-6118 ext. 4264	Program Manager	Approve the final QAPP
Rod Swanson CWP 360-397-6118 ext. 4581	Monitoring Coordinator	Overall management of the County's NPDES Phase I compliance activities. Monitor and assess the quality of work. Comply with corrective action requirements.
Chad Hoxeng CWP 360-397-6118 ext. 4018	Project Manager	Develop, implement, ensure approval of, and maintain the Plan. Verify the Plan is followed and the program is producing data of known and acceptable quality. Ensure adequate training and supervision of all monitoring and data collection activities.
Ian Wigger CWP 360-397-6118 ext. 4282	Monitoring Lead	Manage and oversee monitoring activities, including data management.
Jeff Schnabel CWP 360-397-6118 ext. 4583	Quality Control Coordinator	Review and approve Plan and program deliverables. Validate and verify data collected, and initiate corrective action as appropriate.
Gregory Winters DOE 360-690-7120	Regional Contact	Review the draft QAPP and approves the final QAPP
Vancouver Toyota Site Representative	Site Manager	Provide records describing maintenance actions.
Steve Gustafson WCI 360-571-2290	Principal Hydrologist	Acquire, verify, and transfer data from field and lab.

CWP— Clean Water Program
WCI — West Consultants, Inc.
DOE — Washington State Department of Ecology

5.2. Schedule

The following table indicates the estimated implementation schedule for the project, assuming a 2-year project length after construction activities are completed. Anticipated

schedule may change due to delayed construction activities, inclement weather conditions, or other unforeseen reasons.

Table 2. Anticipated program schedule for the TOYLID project..

Calendar Year 2008				
Activity	**Anticipated Date of Initiation**	**Anticipated Date of Completion**	**Deliverable**	**Deliverable Due Date**
Program startup	10-01-2008	Ongoing	Planning; equipment procurement, installation, and testing; staff training	10-01-2009
Continuous flow and precipitation recording	10-01-2009	Ongoing	Develop baseline rainfall/runoff relationship	10-01-2009
Calendar Years 2009 - 2012				
Complete program startup	Continuing	05-01-2009	Monitoring equipment installation and testing; staff training	To be determined
Precipitation monitoring	05-01-2009 10-01 thereafter 05-01-2011		Stormwater Monitoring Report	Reported to Ecology 3-31 each year starting 2010
Outfall flow monitoring				
Observation well depth				
Seepage from site perimeter				
Maintenance records/actions				
Data validation	As flow is detected	1-30 each year		

5.3. Special Training Needs

Project staff will require the following training:

- All field personnel will receive training in monitoring equipment operation, maintenance, and calibration procedures.
- All field personnel will receive training in identifying, measuring, and recording of onsite observations.

Stormwater monitoring conditions are often wet and cold. Additionally, field crews may be exposed to traffic hazards, disease vectors (e.g. snakes or rats), fall hazards, other hazardous materials, and slippery conditions. In addition to technical training, field personnel will receive guidance that addresses specific monitoring issues that may impact their health and safety.

5.4. Revisions

Ecology must review and approve this Quality Assurance Project Plan (QAPP) for flow reduction strategy monitoring program under Section S8.F.7 of the Phase I Permit (per Permit S8.C.2). Once approved, the QAPP is a living document and will be updated during the course of the study whenever it is appropriate to do so. Justification, summaries, and details of the updates will be documented in a QAPP Addendum and will be distributed to all persons on the distribution list by the Project Manager. QAPP Addendums will be compiled and transmitted no more frequent than quarterly.

Section B. Type, Quality, and Quantity of Data Needed

6. Quality objectives

This section defines the data quality objectives for the hydrologic monitoring program, as well as the measurement quality indicators utilized to meet this study's goals and objectives. These data quality objectives will be achieved through adherence to the procedures presented in this QAPP.

6.1. Data Quality Objectives

Data quality objectives (DQOs) may be either qualitative or quantitative, and describe the type, quality, and quantity of data that are required to fulfill the program objectives. DQOs are defined for this program as follows:

- Precision and accuracy will be known
- Data will be generated from controlled procedures for hydrologic monitoring, onsite observations, and record keeping
- Data collected will be of sufficient quality and quantity to enable calculation of rainfall intensity and depth, runoff volume, and peak discharge

6.2. Measurement Quality Objectives

Measurement quality objectives (MQOs) describe measures of performance and criteria for acceptance that provide the basis for evaluating data quality and usability. They indicate the minimum threshold levels for measures of bias, repeatability, precision, accuracy, and sensitivity that must be associated with the data. These measures are based upon specific types of quality control (QC) measurements that are collected in the field. Additional criteria for completeness and representativeness of the monitoring data collected are also required. Specific MQOs defined for this program for each hydrologic component are presented in Table 3.

Table 3. TOYLID project characteristics, methods, reporting, and accuracy limits.

Characteristic	Method Equipment	Reporting Limit	Accuracy	Reference
		Units	Units or % error	
Precipitation (both rainfall volume and rate) - automated	Tipping Bucket (WaterLOG H-340 gage calibrated to tip for each 0.01 inch collected, Magnetic Reed switch initiates signal to accumulating device, 8.00 inch diameter orifice, temperature range 0 to 60 degree Celsius)	0.01 inches	0.01 in. (@ 4 inches / hr)	WaterLOG Series, Moder H-340 Tipping Bucket Rain Gage Manual
Data Recording	Digital Data Logger, I/O Modules, and Modem (Sutron model 9210-0000-1 XLITE2 data logger, Sutron I/O modules, and Sutron model 8080-0005-1 Voice Modem)	Sensor dependent (e.g. 12.34 ft)	Logger clock 0.01% or 2 seconds per day accuracy at -20 to 60 degrees Celsius	Sutron 9210 XLITE2 Operations and Maintenance Manual
Stage – automated	Stage Sensor – Submersible pressure transducer or bubble gage (Sutron Accububble model 5600-0131 connected to Sutron Accubar pressure transducer)	0.0001 psi (sensor input) or 0.01 feet of water depth (default digital sensor output)	±0.01 feet for water up to 10 feet deep.	Sutron 9210 XLITE2 Operations and Maintenance Manual
Air Temperature	Therm-X Model032/ 44010/1.5/RN/30/ST temperature sensor			
Staff gauge	Type “A”	48 inches	0.02 inches	Stevens Water Monitoring Systems, Inc.
Crest Gauge	Two inch diameter pipe crest-stage gauge			

7. Monitoring Process Design

This section describes the monitoring program, including precipitation and discharge measurements, onsite observations, and data collection to be taken in the field.

7.1. Hydrologic Monitoring

A continuous rainfall and discharge record will be utilized to establish a rainfall/runoff relationship for this site.

Water stage will be measured inside of the monitoring vault outlet structure (Figures 11 and 12) using a 10 inch Palmer-Bowlus flume, Sutron Accububble, and a self-contained bubble gauge. Rainfall will be measured using a Design Analysis Model H-340 tipping bucket rain gauge. Air temperature will be measured at the rain gauge using a Therm-X model 032/44020/1.5/RN/30/ST temperature sensor. A Sutron model 9210 data logger will record the measured stage, rainfall, and air temperature data. A 12-volt battery will provide the power for all instrumentation. A battery charger will be attached to the battery.

The rain gage will be attached to a 12” x 12” steel plate welded to the top of a 4” steel “I” beam. The “I” beam will be securely bolted to the north side of the concrete outlet structure, extending 10 ft. above the top of the concrete. A NEMA Type 4 steel electrical enclosure (H36” W24” D8”) will be attached to the north side of the “I” beam at eye level. The data logger, self-contained bubbler, and battery will be installed inside of the locked enclosure.

Type “A” staff gages on a 2” x 6” board and a 2” diameter pipe crest-stage gage will be attached with bolts to the inside wall of the outlet structure next to the stage sensor.

All of the equipment described above is routinely used by the USGS in Washington and Oregon (Table 4).

Table 4. Additional information about the equipment being used in this project can be found using the following links.

Data sheet	Manual
http://www.sutron.com/products/Accububblegauge.htm	http://www.sutron.com/downloads/DownloadsUpdates/AccububbleManual.pdf
http://www.sutron.com/products/XLite9210.htm	http://www.sutron.com/downloads/DownloadsUpdates/9210B%20User%20Manual.pdf
http://waterlog.com/downloads/Brochures/H340Brochure.pdf	http://waterlog.com/downloads/manuals/Entire%20H-340%20Manual.pdf
http://www.therm-x.com (special order - no data sheet)	(special order - no manual)
http://www.plasti-fab.com/wastewater-products/palmer-bowlus-flumes	(special order - no manual)

7.2. Onsite Observations and Data Collection

Onsite visits will be conducted at least monthly for the duration of the study. Onsite visits will include downloading data loggers and noting monitoring well depths, ponding water, perimeter seepage, contaminants, and any other abnormality that may indicate that pervious pavers are not functioning as designed. Additionally, site visits will occur during or immediately after significant rainfall events.

Bias

Bias represents a difference from the “true” value and the population mean. Potential sources of bias include faulty calibration of the measurement process. Errors of bias are minimized through use of standardized calibration and maintenance procedures conducted by properly trained staff.

Precision

Precision is a measure of the repeatability of a set of replicated results, and is considered to represent random error in the measurement process. Poor precision is due to difficulties in obtaining measurements under identical conditions (e.g. equipment malfunction or variability of field conditions during the time the replicate measurements

are collected). Replicate measurements will be conducted in the field if flow from outfall is present. The two values (measurement and replicate) will be compared to provide a measure of the relative percent difference between them (RPD). In addition, peak flow recorded will be compared to readings from a crest gauge to compare stage peaks.

$$[\text{RPD} = \frac{R_1 - R_2}{[R_1 + R_2]/2} \; x100\,].$$

Accuracy

Accuracy is an estimate of the closeness of a measurement result to the true value. Accuracy of discharge measurements is related to limitations of the equipment, specifically to the limits of discharge that the sensors are calibrated to operate within and deviation from ideal conditions, such as backwater, clogging or debris in outlet, etc. Maximizing accuracy of discharge is achieved by appropriate selection of measurement technology for the discharge conditions that will occur.

Completeness

Completeness is defined as the proportion of measurements collected relative to the total number planned to be collected. Completeness represents an assessment of how field problems affected the success of the data collection effort.

Data that are qualified but still usable according to quality control criteria that have not been met will be counted as valid data for assessing completeness, although data that are rejected for use will not be considered.

Missing or gaps in continuous flow data may be substituted with onsite visual observation data and crest gauge data if deemed appropriate by the project manager and QC coordinator. Missing or gaps in continuous precipitation data may be substituted with the use of other rain gauges in the area.

During the data validation process, an assessment will be made whether enough valid data exist to meet the requirements of the Permit. If not enough valid data are obtained, corrective actions will be initiated by the principle investigator or a designee.

Representativeness

TOYLID data are intended to be representative of conditions at the outfall monitoring station. The Clean Water Program utilizes standard monitoring procedures which are designed to facilitate the collection of representative hydrological data. Hydrological measurements and data acquisition are performed according to standard procedures developed by the United States Geologic Survey (United States Department of Interior, 1982).

The frequency of automated measurements is designed to capture all important variations in flow and precipitation. Automated stage recordings will be taken at minute intervals to compute flow variations. Precipitation recordings are triggered by 0.01 inches of rain to ensure precise rainfall measurements over specific timeframes.

Section C. Measurement procedures

8. Monitoring procedures

Field operations will follow specific SOPs developed by CWP, West Consultants, Inc. (WCI), and collaborating permittees. This section provides an overview of the field procedures. Specific details are provided in the SOPs (Appendix I).

Field activities are documented in detail in field sheets (Appendix J) that are assembled into a loose leaf notebook. Entries should be made in permanent waterproof ink, initialed and dated. Corrections are made by drawing a single line through the error so it remains legible, writing the corrections adjacent to the errors, and initialing the correction. Notes on the collection of data should be sufficiently detailed to allow a reviewer's understanding and evaluation of the process.

Required field log entries include the following—

- Name of program, and location of field work
- Date
- Identity of field crew
- Site and climatic conditions
- Instrument calibration procedures, if any
- Field measurement results
- Description of QC measurements collected, if appropriate
- Photo location (GPS point) and direction if photos are taken
- Unusual circumstances that may affect interpretation of data, if appropriate

8.1. Precipitation and Flow Monitoring

Data from the rain gage and flow meter will be downloaded once every month while onsite observation monitoring is being conducted. The rain gage and flow meter will be inspected during these site visits and serviced as needed; calibration will be conducted according to the recommended method and frequency determined by the manufacturer.

West Consultants Inc. is under contract for installation and maintenance of the flow monitoring and precipitation equipment for this monitoring station. Clark County staff will provide operation and maintenance.

Checklists and logs are kept for all field activities. Appendix J: TOYLID Discharge Measurements Checklist lists detailed information for typical field equipment utilized in this project. Logs may consist of standardized field sheets (Appendix J) as well as bound log books containing ancillary data and observations. Logs are waterproof and entries are made with pencil or indelible ink. Corrections are made by drawing a single line through the error such that it remains legible, writing the correction adjacent to the error, and

initialing the correction. Records are cross-checked for consistency between data sheets, field logs, and other relevant data. Log books are archived in Clean Water Program files.

Section D. Quality Assurance and Quality Control Procedures

9. Quality Control

This section describes the quality control (QC) requirements for all field activities conducted by this program. Data quality will be evaluated according to the measurement quality objectives (MQOs) described in Section 6.

9.1. Field Quality Control

Field QC requirements include procedures for field measurement and documentation, data collection, field QC measurements, and corrective action for identified issues for field activities.

9.1.1. Field Quality Control Procedures

Standard quality control procedures are used for field discharge and precipitation measurements. This includes keeping all components of the TOYLID station in proper working order.

Installation and calibration of all automated stage and precipitation gage recording stations are generally performed according to accepted U.S.G.S. standard operating procedures (U.S.D.I., 1982, Vol. 1). Automated stage and precipitation sensors are certified to be calibrated and validated with instruments traceable to the National Institute of Standards and Technology (NIST).

Inspection and maintenance of all precipitation and field discharge measurement equipment will be done during field visits. Precipitation monitoring stations are

inspected and cleaned, as needed, during field visits. Flow meters are visually inspected and cleaned, if needed. These activities are used to help ensure that field instruments are attaining stated accuracy and resolution specifications.

Stage measurements are recorded to the nearest 0.01 feet and discharge calculations are reported to the nearest 0.1 cubic feet per second. Factors that may detrimentally affect field discharge or precipitation measurements are noted on field sheets to help interpret calculated estimates.

To allow comparisons and to maintain reliability, the TOYLID site will be equipped with a 10 inch Palmer-Bowlus flume, Accububble (self contained bubbler system), staff gauge, and crest-stage gauge. The gauge at the TOYLID monitoring site will be programmed to record the entire range of expected stages. During field visits, the automated stage recorder's latest stage reading is compared to the current water surface level at the gauge to ensure all instrumentation is working properly. Downloaded data are organized and reviewed for completeness and reasonableness in a timely manner. This will help identify anomalous readings indicating problems with gauging station equipment or other issues affecting results. Gauges are periodically checked by running levels to their reference marks. The TOYLID site will have a crest-stage gauge to mark high-water levels for information backup in case of equipment failure or to mark stage peaks between automated data recordings.

9.1.2. Corrective Actions

Data quality problems encountered during the measurement and calculation of discharge and precipitation are addressed and corrected as needed. Generally, this could involve analysis of QC measurements, re-calibration of equipment, modifications to the field procedures, increased staff training, or by qualifying results appropriately. Corrective actions may also be done during field work.

10. Data Management Methods

Water discharge through the outlet pipe will be computed by developing a stage using a flume with known characteristics. The stage will be measured inside the concrete storm outlet structure. Discharge measurements will be obtained at various flows to develop the stage-discharge rating. These measurements will be made at or near the end of the outlet pipe using one or several of the following methods; current meter, volumetric, or portable Parshall flume, or by use of a flow meter in the pump outlet hose.

Stage will be measured every 1 minute however the data will only be logged when the stage is high enough for water to flow through the outlet pipe. Rainfall data will be measured and logged every 5 minutes. Air temperature will be measured and logged every 15 minutes. Battery voltage will be logged every 60 minutes.

The stage sensor will be set to read the same as the staff gage. The staff gage will also be used to periodically check live readings from the stage sensor. Peak stages recorded by the crest-stage gage will be used to verify peak stages recorded by the data logger.

Data will be transferred from the data logger to a field computer every month and more often during times if flow is anticipated. Each field site visit will include inspecting the crest-stage gage, checking the battery, and if possible reading the staff gage. The rain gage will be kept clean and level.

All recorded data will be input into Kisters, WISKI (Hydrological Data Management System), Biber (Discharge Measurement and Evaluation), Sked (Rating Curve Editor) software. Plots and tables of all recorded data will be reviewed with corrections applied as needed. Water discharge will be computed for every valid stage reading. Precipitation rates and totals will be computed and compared to rainfall data collected at the nearby CCPW Whatley Pit (ORCHDS) rain gage. Tables and graphs of the data will be provided as requested.

TOYLID data and field notes are recorded or retrieved, stored, and managed in both hardcopy and digital form at CWP. Applicable data are entered into spreadsheets for summary statistic calculations including total instantaneous discharge. West Consultants Inc. and the QA coordinator / project manager are responsible for validating and cross-checking data entry and explaining any necessary data qualifiers. Summary statistics are stored digitally for long-term storage, retrieval, and analysis. Alternatively, automated outfall stage and precipitation measurements are digitally recorded on data loggers and then downloaded in the field from the outfall site onto computers for data storage. Outfall discharge is later computed in the office for every five minute stage value.

Section E. Assessment Procedures

11. Audits and Reports

This section describes the processes that will ensure that the quality assurance procedures specified by this QAPP are being implemented correctly, that the quality of the data is acceptable, and that corrective actions are conducted in a timely manner.

11.1. Audits

Audits are an important tool to verify that the quality assurance procedures described in this Plan are being adequately implemented. During an audit, the reviewer will check for the following—

- Sufficient documentation of all required activity
- Compliance with the QAPP
- Identification and justification for any activity that is not in the QAPP
- Correction of any problems that have been identified

Audits may be scheduled by request from the West Consultants Inc., monitoring lead, project manager, or Ecology. The project manager will be responsible for initiating audits, selecting the team of reviewers, and overseeing the implementation of the audit.

Any nonconformance to established protocols will result in appropriate corrective action. The results of the audit and oversight activities will be reported to the project manager, who has ultimate responsibility for ensuring that the corrective action response is appropriate, complete, and documented.

11.2. Reports

Reports that will be generated for this program include status reports and the Annual Stormwater Monitoring Report.

11.2.1. Status Report

Annual status reports compiled by CWP address project methods, summarize data accuracy and completeness, describe any significant data quality problems, and suggest modifications for future monitoring. Reports are peer reviewed by CWP staff. TOYLID project summaries are incorporated as attachments to the county's annual NPDES permit compliance report to Ecology. Executive summaries, and full reports as warranted, are placed on the county's website to facilitate dissemination of information to the public. The hydrological data will be utilized for effectiveness of pervious pavement as a LID practice. For example, the data may be used to help develop regional design equations for LID projects.

11.2.2. Annual Stormwater Monitoring Report

The Stormwater Monitoring Report is required to be submitted with the NPDES permit annual report every year (S8.H). Each report will contain all monitoring data collected during the previous year (January 1 through December 31). As appropriate, the reports will integrate data from earlier years into the analysis of results.

Reports will be submitted in both paper and electronic form and will include:

- A summary description of each monitoring site, describing location, land use, drainage area, and hydrology
- A comprehensive data report, including QA/QC, with an explanation and discussion of the results
- The annual and seasonal (wet and dry) hydrologic flow data

The stormwater monitoring summary description will include any new basin information that is not presented in this Plan, such as basin changes that may affect the hydrology. The data report will explain the program results, present the data, document the overall quality of the data (i.e. completeness, representativeness, and usability), and discuss any data anomalies. The QA/QC section will include a summary and discussion of the field QC procedures and results, and whether the measurement quality objectives were met. This section will include any planned changes to the current Plan that may be appropriate to address QA issues that have been observed.

12. Data Verification and Validation

This section describes the data review, verification and validation procedures that determine whether the data conform to the criteria required by the program objectives.

12.1. Summary of Procedures

During each monitoring trip, field crews review field logs to confirm that all necessary field measurements have been collected. Field results are reviewed, verified, and documented by field staff in data reports to the CWP. Upon receipt, contracted hydrological data are reviewed for errors, omissions, and data qualifiers prior to data entry.

Data review is the process of examining the data for errors or omissions. Data verification is based on the QC results, and determines whether the data meet acceptance criteria. Data validation includes the complete monitoring process to assess whether the appropriate procedures were following in collection of data.

All data generated by this program will be review and verified for conformance to the requirements of the program. Data will then be validated according to the data quality objectives described in Section 6. Once data are found to be supported by acceptable QC data and to meet the specified measurement quality objectives, they will be considered acceptable and usable for the program.

Procedures for verification and validation will be conducted according to the guidance provided by EPA, 2002 (Guidance on environmental data verification and data validation, EPA QA/G-8). The QC coordinator, in collaboration with the monitoring lead, is responsible for ensuring that field data are reviewed and verified. After each successful download event, the QC coordinator or designee will review rainfall and flow data for gross error (e.g. outliers or data gaps) to verify the completeness of the data. The monitoring lead will also check to see that flow measurements were collected in accordance with required criteria.

12.2. Methods of Verification and Validation

This section presents a brief overview of the methods that may be used for verifying and validating data, including the input that will be necessary, the specific methods to be used, and the output from the verification process.

12.2.1. Data Input

A variety of records will be necessary for electronic data downloads, data input verification, and data validation. These could include, but are not limited to, the following—

- Field logs
- Electronic data transfer
- QC results

12.2.2. Data Verification Methods

Data verification methods will be documented throughout the course of the process, and may be revised as appropriate to the situation. Data verification involves examination of results collected during the project to provide an indication of whether precision, bias, and accuracy MQOs have been met. To evaluate whether precision targets have been met, pairs of duplicate discharge measurement QC results are compared by estimating their relative standard deviation (RSD). Duplicate discharge measurements will consist of staff gauge observations and crest gauge observations compared to flow meter data during discharge events. The RSD is calculated by dividing the duplicates' standard deviation by their respective mean. Converted to a percent of the mean, it is used to judge whether the %RSD precision target has been met. Generally, for TOYLID discharge estimates this target is 5%. Bias (systematic error) will be kept to a minimum by careful adherence to standard operating procedures.

Data validation consists of a detailed examination of the complete data package using professional judgment to assess whether the procedures in the Standard Procedures and QAPP have been followed. Data validation is performed by West Consultant Inc. and the project manager / QA coordinator during the preparation of annual reports.

These methods will include—

- Identification of data gaps or missing data

- Identification of data anomalies or outside the limits of reality
- Cross checks with field sheets and calibration records
- Evaluation of expected patterns versus observed
- Evaluation of QC results (within acceptable criteria?)
- Comparison of TOYLID rain gauge with other nearby rain gauges

13. Data Quality Assessment

Once data have been verified and validated, the final data quality assessment is conducted. The Data Quality Objectives defined in this Plan (Section 6) must be satisfied in order for the data to be considered usable for meeting program objectives. The main goals of this assessment are to determine if the data meet the quantity of measurements required and is representative of stormwater runoff conditions of the site.

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Figures

Figure 1 Vancouver Toyota Vicinity Map

Figure 2 Vancouver Toyota Existing Basin

Figure 3 Vancouver Toyota Proposed Project

Figure 4 1974 Vancouver Toyota Site Condition

Figure 5 Permeable Concrete Pavement Section

Figure 6 Permeable Concrete Pavement Section

Figure 7 Stormwater Infrastructure Location

Figure 8 Field Inlet Description

Figure 9 Monitoring Well Description

Figure 10 Monitoring Well Locations

Figure 11 Monitoring Vault And Outfall Location

Figure 12 Monitoring Vault Structure

Figure 1. Vancouver Toyota Site vicinity map.

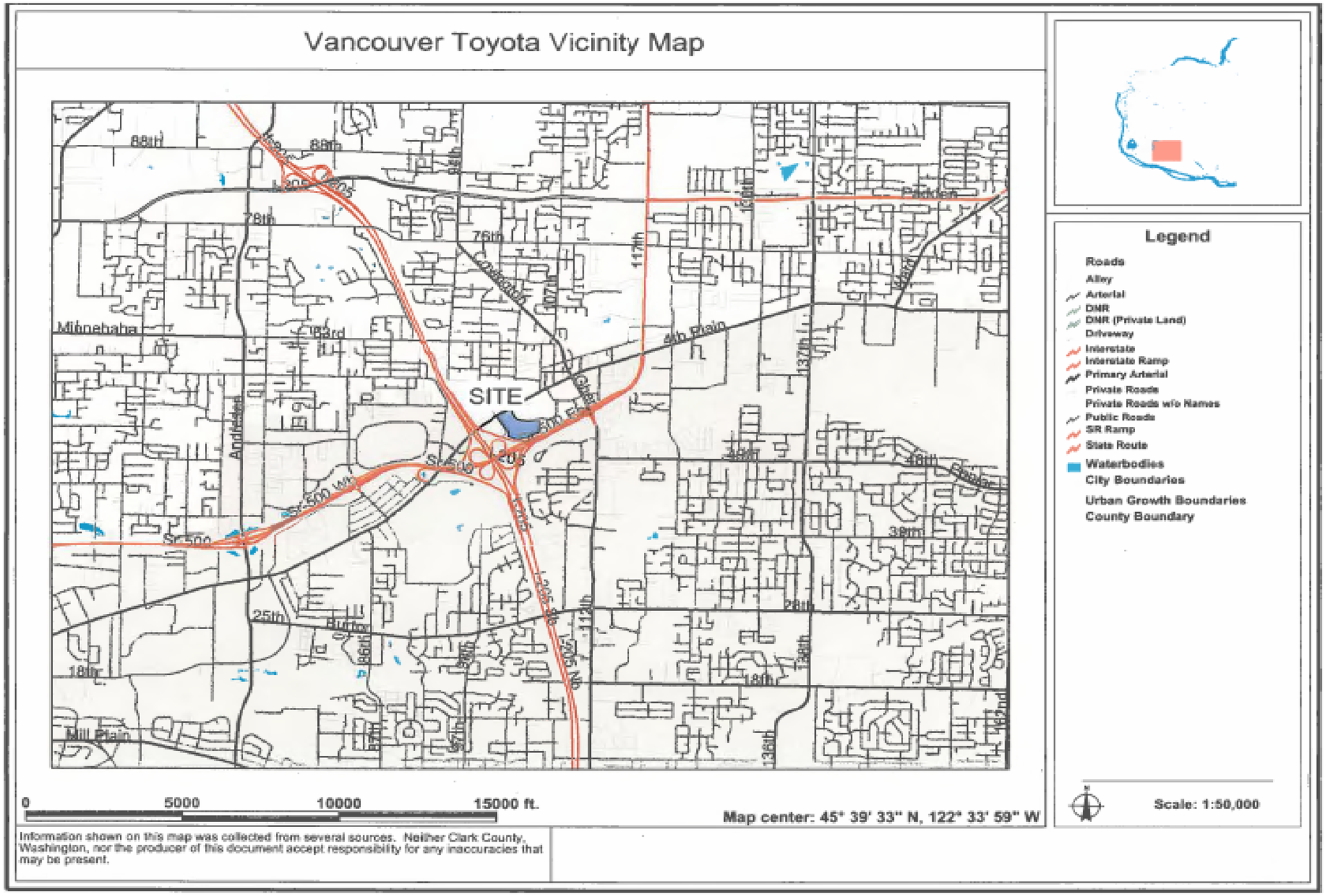


Figure 2. Existing Vancouver Car dealership and planned expansion to additional parcels.

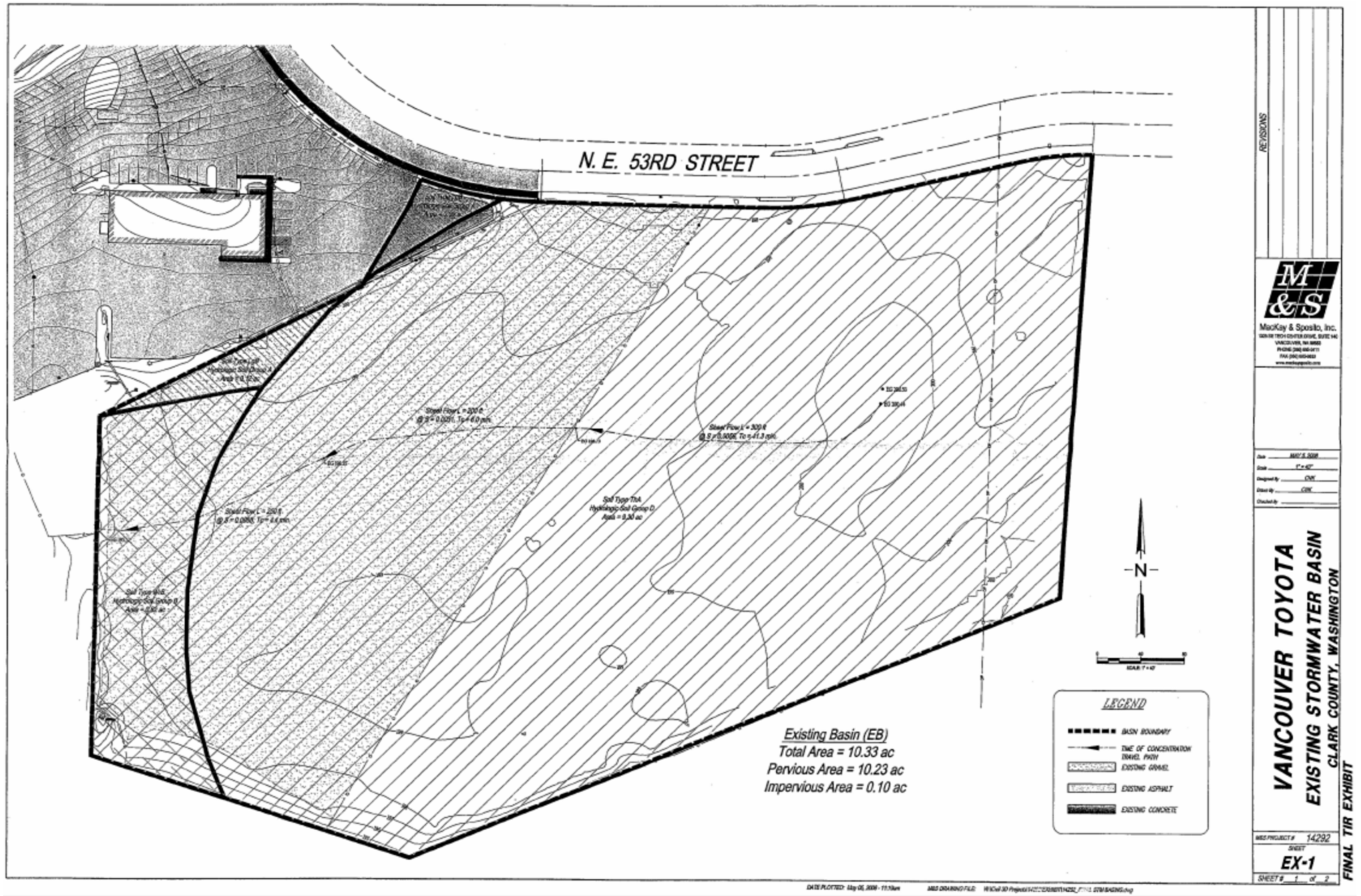


Figure 3. Existing Vancouver Car dealership and planned expansion with new building to additional parcels.

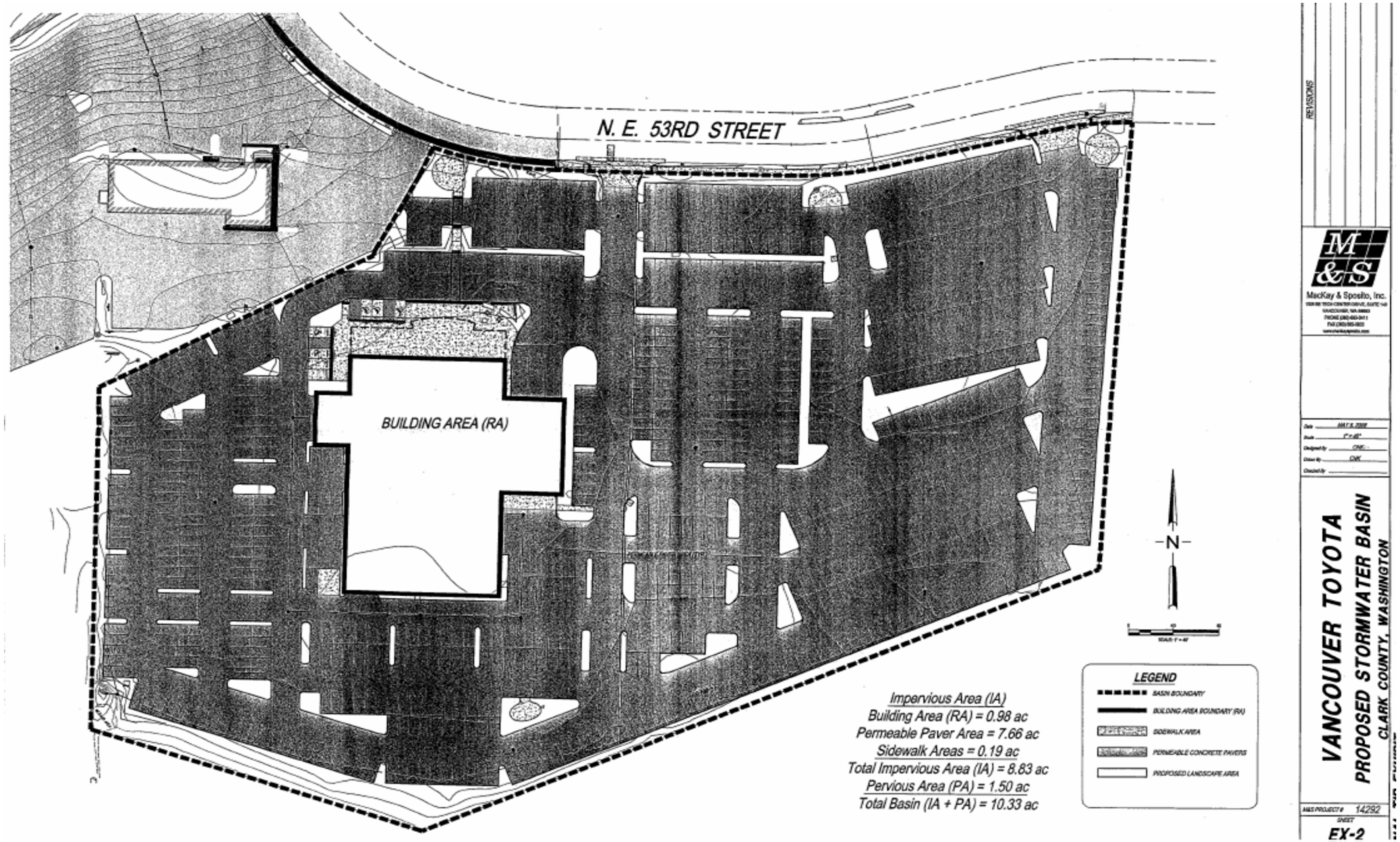


Figure 4. 1974 site conditions consisted of field covered areas.

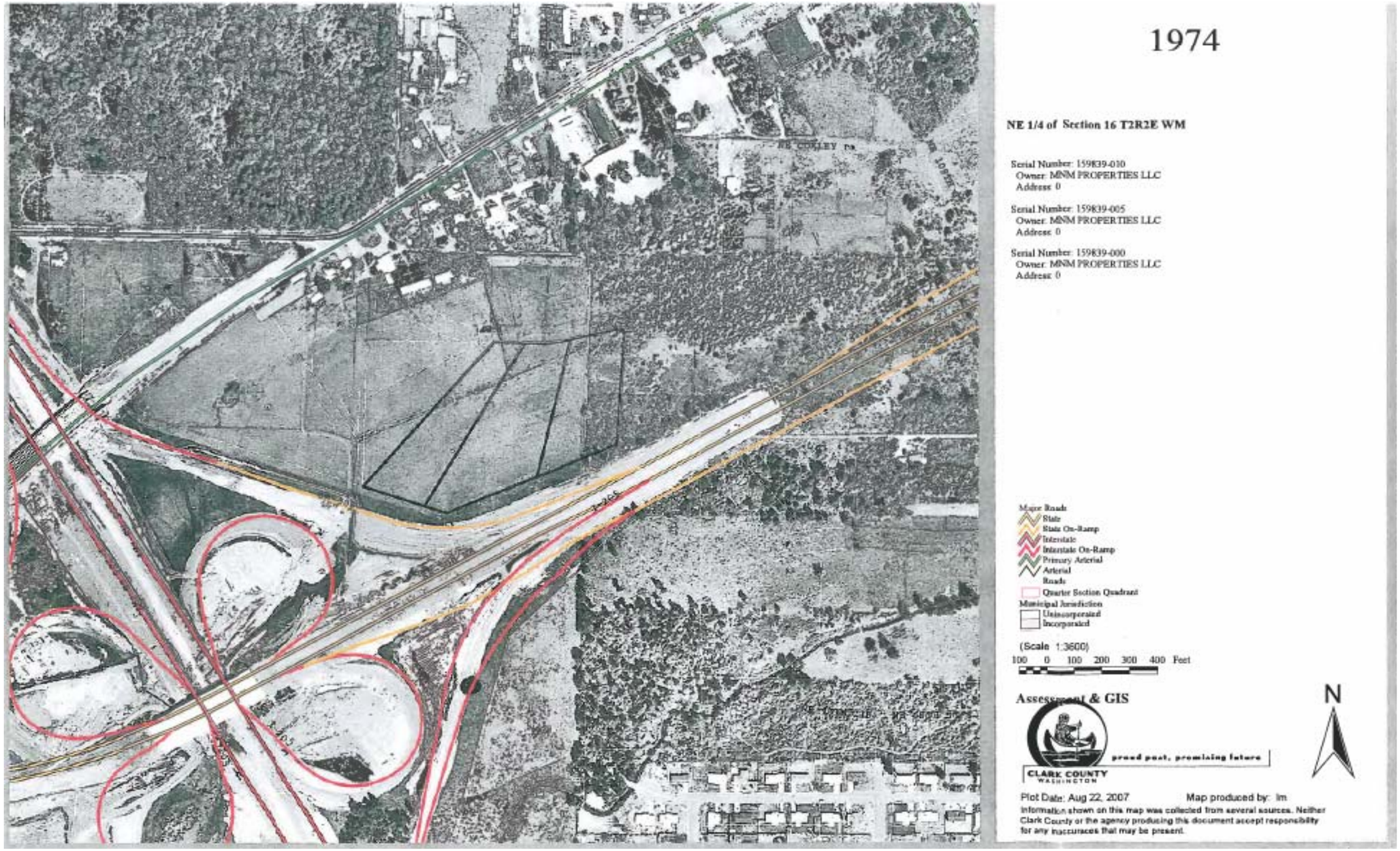
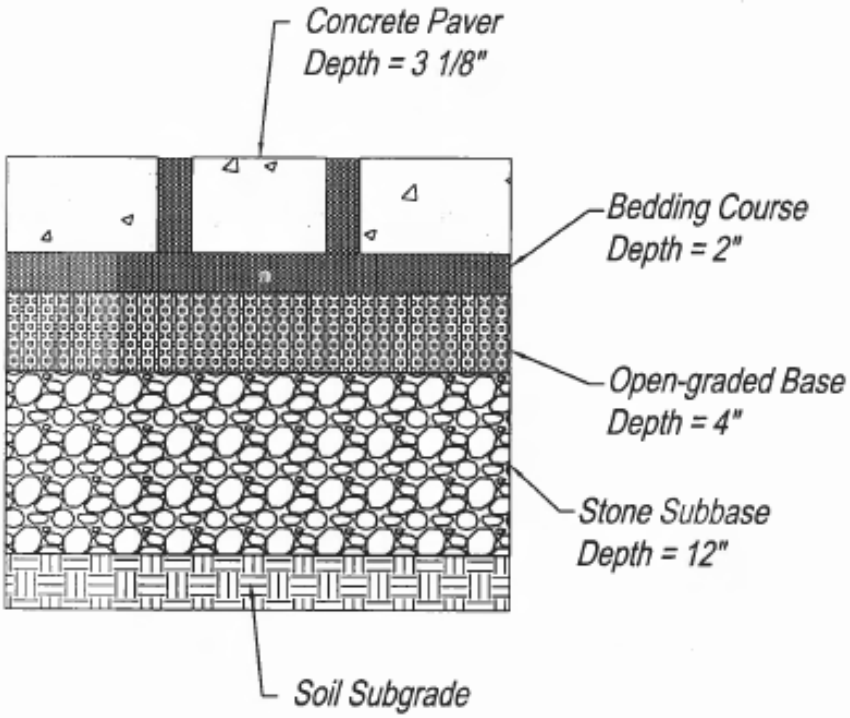


Figure 5. Cross section view of Eco-Loc Permeable Concrete Pavers and underlying base materials.

Figure 6. Cross section view of Eco-Loc Permeable Concrete Pavers and underlying base materials from approved plans.



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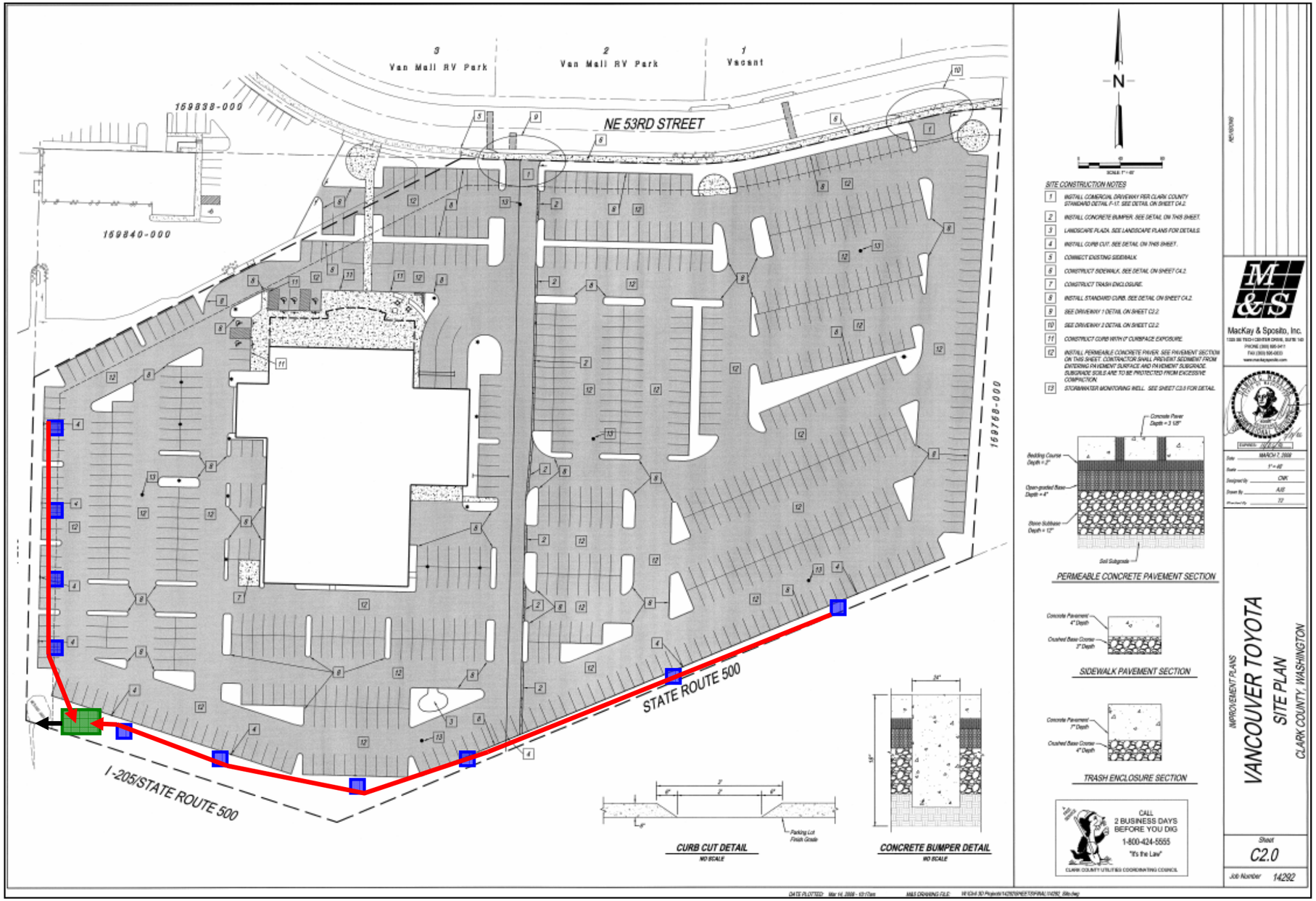
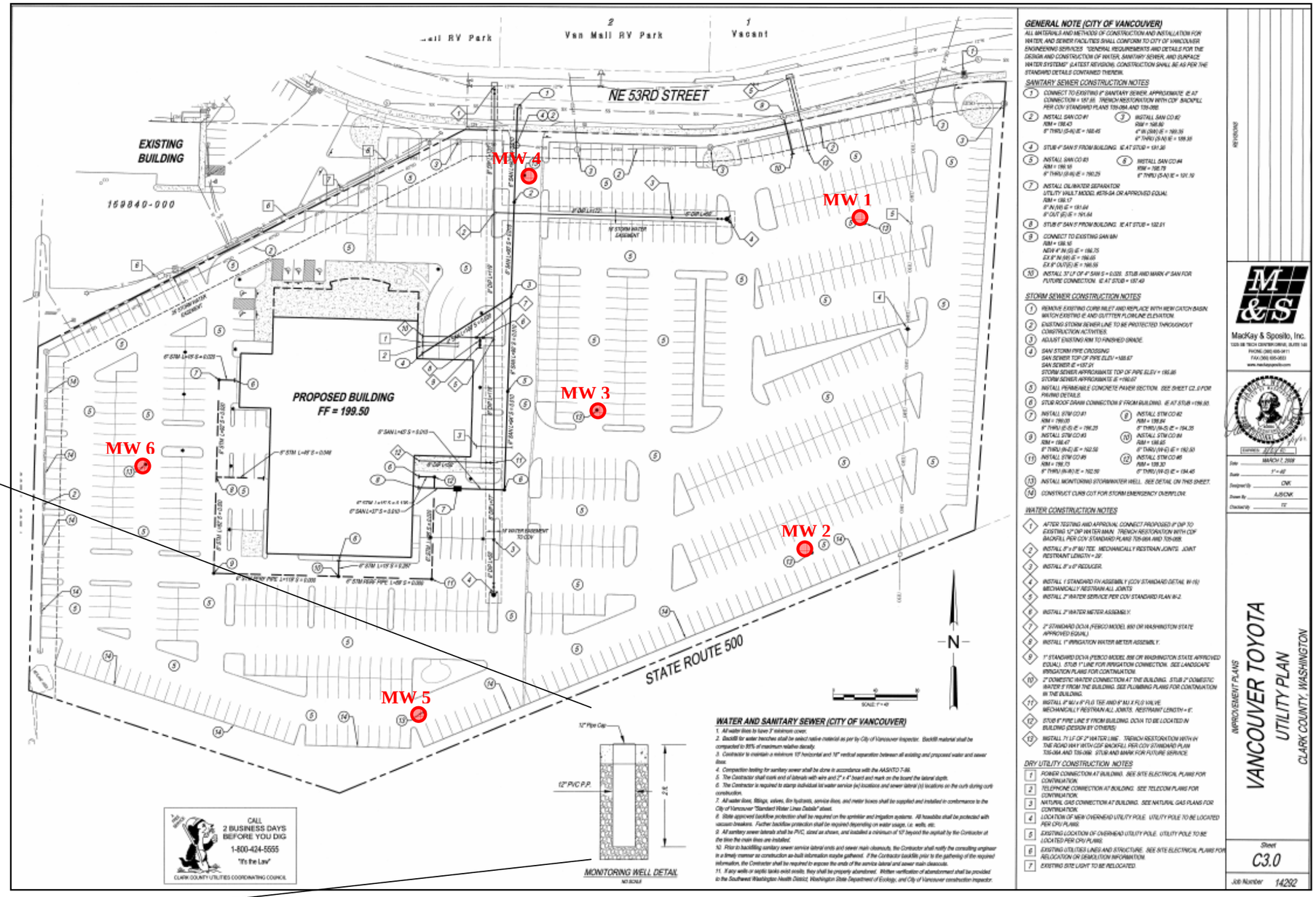


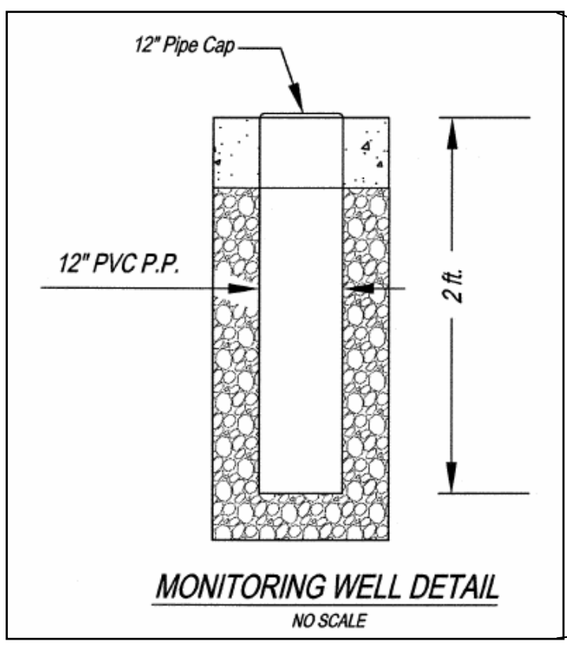
Figure 7. Site plan for the Vancouver Toyota site. Red arrows represent stormwater infrastructure (10 inch non-perforated pipe) that will direct any overflow collected from field inlets (represented by blue squares) to a monitoring vault (4x5x4 foot concrete vault represented by green rectangle). Black arrow at monitoring location represents a 10 inch pipe outfall from monitoring vault to existing WSDOT drainage ditch.

Figure 8. Field inlets in this study are a concrete structure fitted with a slotted grate. The field inlet is designed to collect stormwater runoff the does not infiltrate site and route it through underground 10 inch pipes as seen in photos A and B. Field inlets at this site are located within south and west concrete curb perimeter of the site as seen in photos B and C. Field inlets will convey any stormwater runoff that does not infiltrate the site to a single outfall in the southwest corner of the property (see Figure 7).

Figure 9. Monitoring wells in this study are a 10 inch perforated pipe of various depths that penetrates base material as seen in photo A. Monitoring wells are enclosed in a concrete structure and have a solid lid as seen In photos B and C. Monitoring wells are an accurate way of determining the depth of water stored in the base material.

Figure 10. Site will contain six observation monitoring wells (outlined in red). Monitoring wells (MW 1 - MW 6) are designed to penetrate sub base materials into existing soils. Photo of monitoring well can be seen in Figure 9.





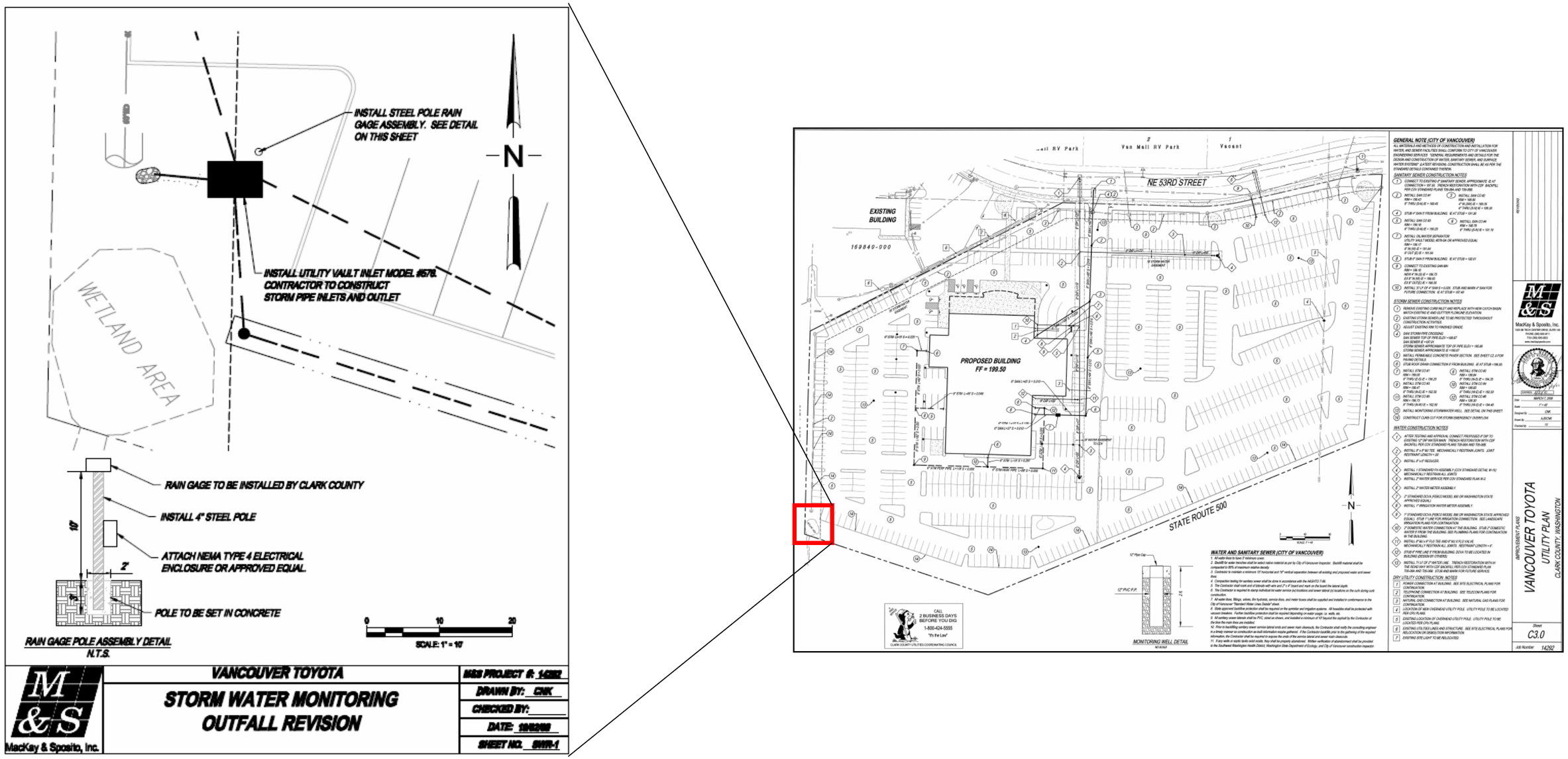


Figure 11. Proposed monitoring location for the Vancouver Toyota Site outfall and rain gauge.

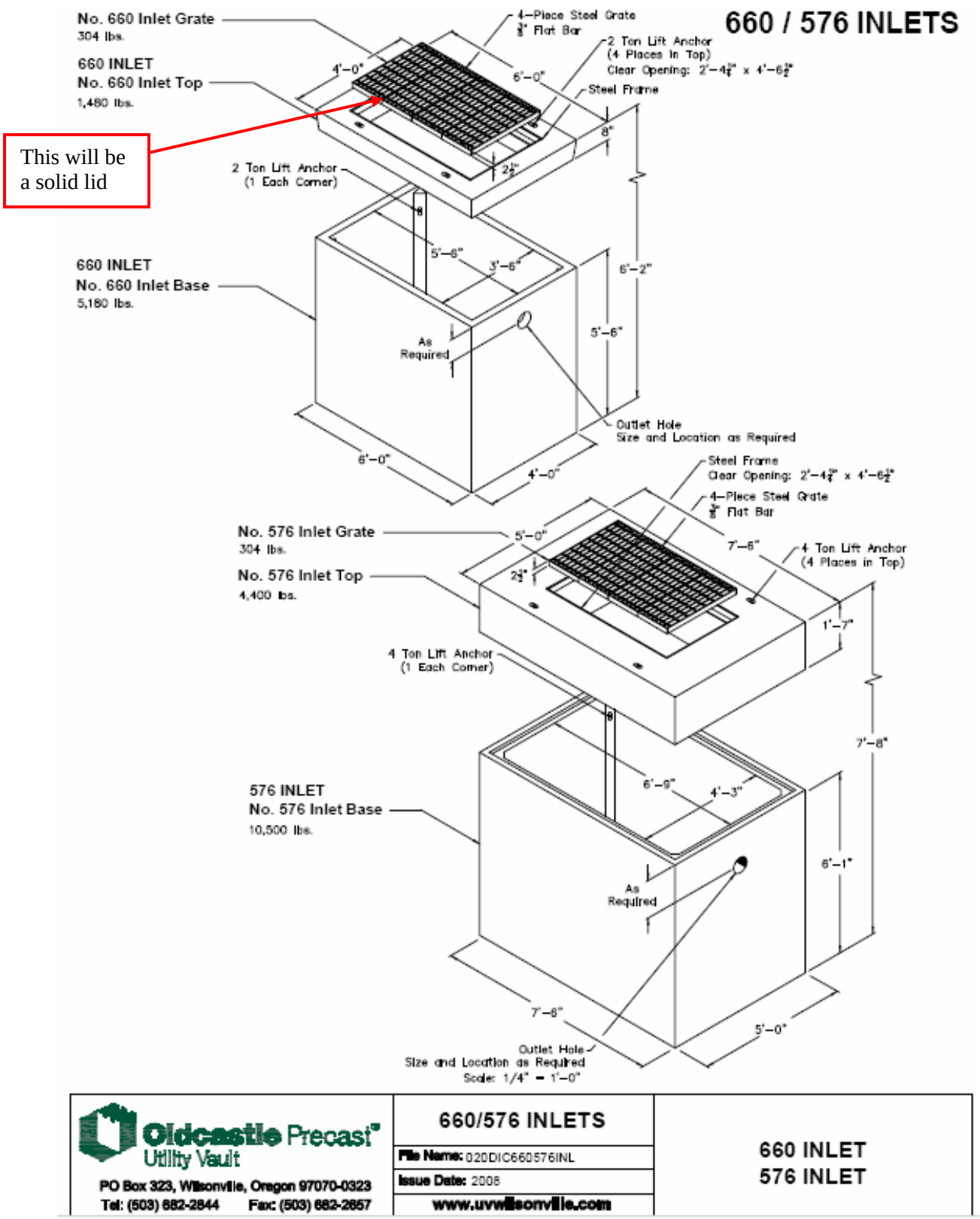


Figure 12. Proposed access vault and monitoring station for the Vancouver Toyota site.

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Appendices

Appendix A: Vancouver Toyota Final Stormwater Report with Appendix A maps of site.

Appendix B: Hydrologic Calculations of Vancouver Toyota Final Stormwater Report

Appendix C: Hydraulic Calculations of Vancouver Toyota Final Stormwater Report

Appendix D: Geotechnical Report of Vancouver Toyota Final Stormwater Report

Appendix E: Low Impact Development Technical Guidance Manual for Puget Sound from Vancouver Toyota Final Stormwater Report

Appendix F: WSDOT Highway Runoff Manual M31-16 from the Vancouver Toyota Final Stormwater Report

Appendix G: ICPI Permeable Interlocking Concrete Pavements Design Manual from the Vancouver Toyota Final Stormwater Report

Appendix H: Performance Evaluation of a Permeable Pavement and a Bioretention Swale from the Vancouver Toyota Final Stormwater Report

Appendix I: Standard Operating Procedures Under Development

Appendix J: TOYLID Field Data Sheets /Observation Logs

Appendix A: Vancouver Toyota Final Stormwater Report with Appendix A maps of site

Vancouver Toyota

Final
Stormwater Report

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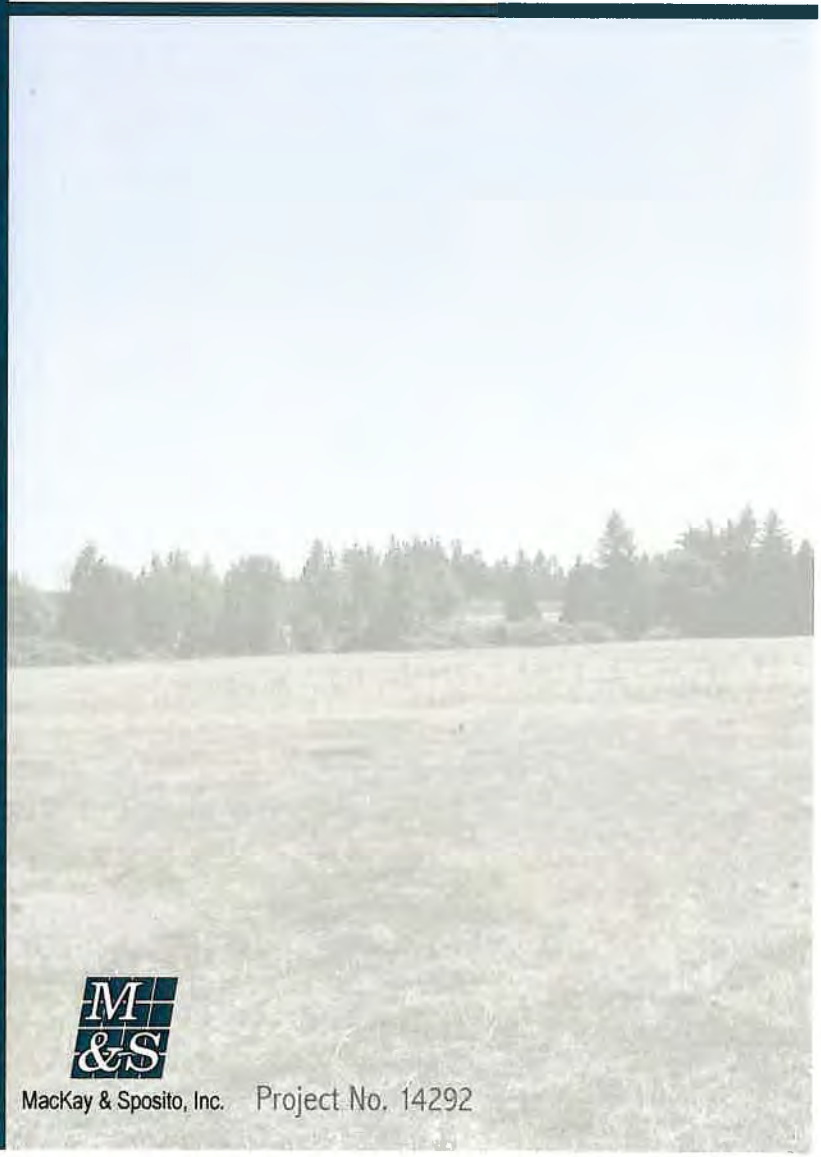

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Vancouver Toyota

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Appendix H: Performance Evaluation of a Permeable Pavement and a Bioretention Swale

Stormwater Report Narratives

A. Project Overview

The following report addresses the stormwater design for Vancouver Toyota located in the Northeast quarter of section 16, township 2 north, range 2 east of the Willamette Meridian. The site is located north of SR-500, south of NE 53rd Street, east of I-205, and west of 107th NE Ave. Please see Appendix A for vicinity map.

The proposed project is an expansion of the existing Vancouver Toyota to additional parcels located to the south and east. The project includes a new building, installation of sidewalk along NE 53rd Avenue, car parking/storage area, and landscaping. The size of the proposed commercial building is approximately 41,985 square feet, and the total size of the parcel is 10.2 acres.

Existing topography suggests a slight slope from the northeast to the southwest; however, a small portion of the eastern side of the site slopes towards the east. Under these existing conditions, storm water runoff will flow out the southwest and southeast corners of the site.

The eastern half of the site is currently an unimproved field with sparse grass across the site. The western half of the site has been used as a parking lot and has a layer of gravel over the top. There are no existing structures on the site. The historical condition of the site was evaluated by using a 30 year old aerial photograph. At that time, the entire field was in the condition similar to the eastern half. A site map and an aerial photo from 1978 have been included in Appendix A.

The runoff from the building and parking lots and the landscaped area will be infiltrated on-site through Eco-Loc Permeable Concrete Pavers. Stormwater runoff will be treated by means of filtration as the stormwater runoff is infiltrated through the base course. There is no planned runoff from this site. As the permeable pavers are considered an experimental BMP for Clark County, stormwater monitoring wells will be installed to monitor the stormwater effluent quality.

B. Hydrologic Analysis

(a) Design Criteria

The hydrological analysis for this site follows the methods and guidelines outline in Chapter III of the Puget Sound Manual and in accordance with the Clark County Stormwater Ordinance, Chapter 40.380.

(b) Assumptions

There are no notable hydrologic assumptions.

(c) Detailed Hydrologic Analysis

The Peak flows and volumes have been calculated using the Santa Barbara Urban Hydrograph (SBUH) method in HydroCAD hydrology modeling software by HydroCAD Software Solutions LLC. The stormwater from this site will be infiltrated through the permeable concrete pavers. The stormwater runoff will be treated by means of filtration as the stormwater runoff is infiltrated through the base course.

C. Quantity Control System Design

(a) Conceptual Design and Revisions

The only impervious area on site will be the proposed building and sidewalks; there will be no planned discharge from this site. All stormwater runoff will be infiltrated onsite through the permeable concrete pavers. The roof drainage will drain to a separate infiltration system. The permeable paver base course section has been designed to contain all stormwater runoff from all the required design storms. The 100-year storm fills the sub-grade 0.37 feet from the bottom elevation. The emergency overflow system will consist of curb cuts along the outer curb of the site that flow into a WSDOT drainage ditch to the south and surface flow through the proposed driveways to NE 53rd Street in the north.

The only change to the conceptual design is the addition of a separate infiltration system for roof drainage.

(b) Geotechnical Information

There are three different soil types on site: Tisch Silt Loam (ThA), Lauren Gravelly Loam (LgB), and Wind River Gravelly Loam (WrB). Approximately 90% of the site is covered by ThA, which falls in hydrologic soil group D. The other soil types all fall in the hydrologic soil groups A and B. A map with the approximate locations of each soil type is located in Appendix A on the Existing Conditions Map.

(c) Design Criteria

The hydrological analysis for this site follows the methods and guidelines outlined in Chapter III of the Puget Sound Manual and in accordance with the Clark County Stormwater Ordinance, Chapter 40.380.

The pavement section has been designed according to the recommendations of Low Impact Development Technical Guidance Manual for Puget Sound (LIDPS) dated January 2005, chapter 6 (see Appendix E), Washington State Department of Transportation (WSDOT) Highway Runoff Manual M31-16 dated May 2006 chapter 5 (see Appendix F), and with the Interlocking Concrete Pavement Institute (ICPI) Permeable Interlocking Concrete Pavements Design Manual (see Appendix G). The design recommendations for the given references are summarized in the Table 1.

Table 1 - Recommended Design Parameters

Reference	Void Ratio	Min. Infiltration Rate (in/hr)	Min. Base Depth
LIDPS	NR	0.5	6"
WSDOT	0.2	0.1	6"
ICPI	0.32	0.25	8"

NR = No Recommendation

(d) Initial Conditions

Existing topography suggests a slight slope from the northeast to the southwest; however a small portion of the eastern side of the site slopes towards the east. Under these existing conditions, storm water runoff will flow out the southwest and southeast corners of the site. The eastern half of the site is currently an unimproved field with sparse grass across the site. The western half of the site has been used as a parking lot and has a layer of gravel over the top. There are no existing structures on the site. The historical condition of the site was evaluated by using a 30 year old aerial photograph. At that time the entire field was covered with sparse grass without trees.

(e) Assumptions

The permeable paver subgrade section was assumed to have a voids ratio of 0.1. This is a very conservative design assumption and will increase the design life of the permeable pavers. ICPI recommends for pavement design a base course voids ratio of 0.32. WSDOT recommends a void ratio of 0.20 for a conservative design.

The infiltration rate for the site was assumed to be 0.5 in/hr. The lowest tested infiltration rate on site was 1 in/hr while the largest tested infiltration rate was 360 in/hr. In accordance with Clark County Code, the infiltration rate was reduced by a factor of safety of two. The site was designed assuming that the entire site infiltration rate was 1 in/hr and the then reduced by the factor of safety of two to determine the 0.5 in/hr infiltration rate. The minimum recommended infiltration rate for permeable pavers by WSDOT is 0.1 in/hr, ICPI recommends a minimum infiltration rate of 0.25 in/hr, and LIDPS recommends a minimum infiltration rate of 0.5 in/hr.

(f) Analysis of Stormwater Facilities

The stormwater runoff will be infiltrated through the permeable concrete pavers. The design infiltration rate is controlled by the lowest infiltration of all the components of the pavement section. The individual component infiltration rates are:

Table 2 Pavement Section Infiltration Rates

Component	Infiltration Rate (in/hr)
Concrete Paver	7.8
Base Course	500 -2000
Soil	0.5

The base course has been designed to store the 100-year storm. Please see Appendix C for pavement section.

(g) Reference Calculations and Design Aides

Please refer to Appendix C for hydraulic calculations.

(h) Summary of Quantity Control System Design

The stormwater runoff for this site will be infiltrated through permeable pavers with no other quantity control system. The design infiltration rate for the system is 0.5 in/hr and the design void ratio of 0.10. The base course section is design to be 12 inches deep. The 100-year storm runoff fills the base to an elevation of 0.37 feet from the bottom of the base course section.

D. Conveyance Systems Analysis and Design

(a) Conveyance System

All stormwater will be infiltrated through the permeable concrete pavers.

The roof drainage will be piped to an infiltration trench south of the proposed building. The pipes were sized to convey the 10-year, 24-hour storm. The minimum required pipe for roof drainage conveyance is a 6-inch storm pipe at a slope of 0.020 ft/ft. Please refer to Appendix C for a detailed analysis of all pipe flows and capacities.

E. Water Quality Design

(a) Water Quality Design

The treatment of the stormwater will occur by filtration and by the naturally occurring microbial action as the stormwater infiltrates through the concrete pavers and sub-grade. All storm events on this site will be treated. There are no other forms of stormwater quality treatment planned for this site.

(b) Identify Best Management Practices

The BMP for this site is permeable concrete pavers. The pavement section has been designed by the recommendations of Low Impact Development Technical Guidance Manual for Puget Sound (LIDPS) dated January 2005, chapter 6 (see Appendix E) and with the Interlocking Concrete Pavement Institute (ICPI) Permeable Interlocking Concrete Pavements Design Manual (see Appendix G). Please see Appendix E for the attached documentation.

(c) Initial Site Conditions

There are no existing stormwater quality facilities onsite.

(d) Assumptions

There are no notable assumptions.

(e) Water Quality System Analysis

There was no water quality analysis performed on this site as all storm events will be treated as the stormwater infiltrates through the concrete pavers and the sub-grade. Please see appendix H for a study from the Toronto and Region Conservation Authority comparing the Performance of Permeable Pavement and a Bioretention Swale.

(f) Summary of Water Quality System Design

The treatment of the stormwater will occur by filtration and by the naturally occurring microbial action as the stormwater infiltrates through the concrete pavers and the sub-grade. All storm events on this site will be treated. There are no other forms of stormwater quality treatment planned for this site.

F. Soils Evaluation

(a) Discuss Site Soils

An onsite geotechnical evaluation was performed on October 27th, 2005 by Professional Service Industries, Inc. (PSI). The soil profile consists of 2 feet to 4 feet of fill underlain by an upper silt strata with a gravel strata at 2.5 feet to 6 feet below grade. There are three different soil types on site: Tisch Silt Loam (ThA), Wind River Gravelly Loam (WrB) and Lauren Gravelly Loam (LgB). Approximately 90% of the site is covered by ThA which falls in hydrologic soil group D. The full geotechnical report is located in Appendix D.

(b) High Water Table

Ground water was encountered during the geotechnical site exploration at approximately 8.5 feet below existing grade. Variations in groundwater levels should be expected due to the season. Please refer to Appendix D for full geotechnical report.

(c) Site Soil Design Parameters

Please refer Appendix D for the site geotechnical report.

(d) Infiltration BMP's

Please see Appendix D for full Geotechnical Report.

G. Special Reports and Studies

There are no special reports or studies included in this report for this site.

H. Other Permits

An NPDES is to be obtained before construction begins.

I. Groundwater Monitoring Program

There is no groundwater monitoring proposed for this site.

J. Operations and Maintenance Manual

The onsite stormwater sewer system will be owned and maintained privately per the manufacturer's recommendations. Please see Appendix G for the manufacturer's recommend maintenance. A covenant shall be provided to Clark County for the purpose of inspecting the privately owned facilities.

References

United States Department of Agriculture, Soil Conservation Service. "Soil Survey of Clark County Washington," Washington, D.C., 1972.

United States Department of Agriculture, Soil Conservation Service, Engineering Division, "Technical Release 55: Urban Hydrology for Small Watersheds, 2nd Ed.," Washington, D.C., 1986.

United States Department of Transportation, Federal Highway Administration, "Hydraulic Engineering Circular No. 12: Drainage of Highway Pavements," Springfield, VA, 1984.

United States Department of Transportation, Federal Highway Administration, "Hydraulic Engineering Circular No. 15: Design of Roadside Channels with Flexible Linings," Springfield, VA, 1984.

Washington State Department of Ecology, "Stormwater Management Manual for the Puget Sound Basin," Olympia, WA, 1992.

Washington State Department of Transportation, "Hydraulic Manual," Olympia, WA, 1989.

Clark County Code, "Chapter 40.380 (Stormwater and Erosion Control Ordinance)," Vancouver, WA, January, 2004.

Clark County Department of Assessment and GIS, "2006 Clark County Road Atlas," Vancouver, WA, 2006.

APPENDIX | A

Maps

Vicinity Map - A1

Flood Plain Map - A2

Groundwater Protection Area Map - A3

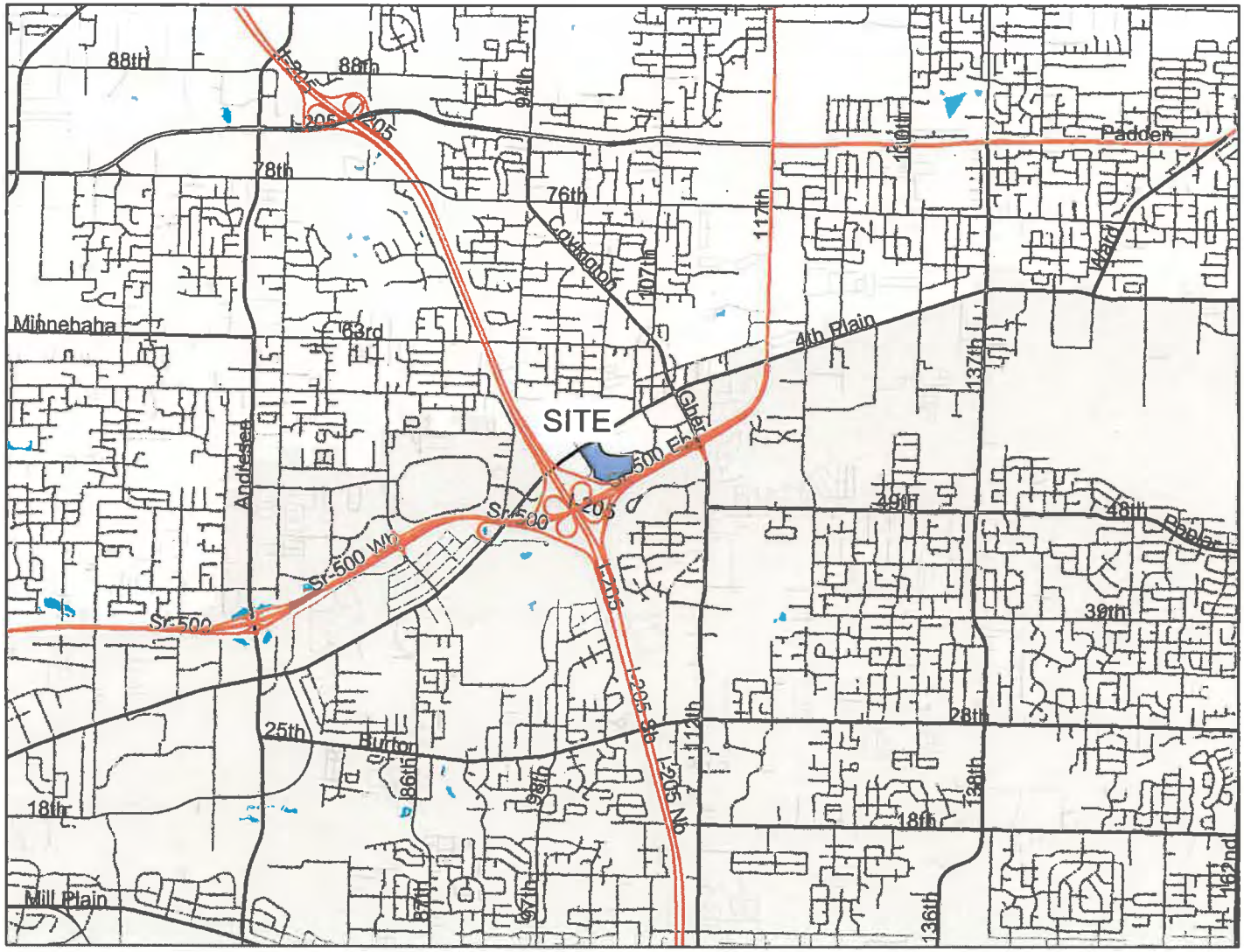
Current Aerial Photo - A4

1974 Aerial Photo - A5

Existing Conditions and Soils Map - EX-1

Proposed Conditions and Soils Map - EX-2

Vancouver Toyota Vicinity Map



Legend

- Roads
- Alley
- Arterial
- DNR
- DNR (Private Land)
- Driveway
- Interstate
- Interstate Ramp
- Primary Arterial
- Private Roads
- Private Roads w/o Names
- Public Roads
- SR Ramp
- State Route
- Waterbodies
- City Boundaries
- Urban Growth Boundaries
- County Boundary

0 5000 10000 15000 ft.

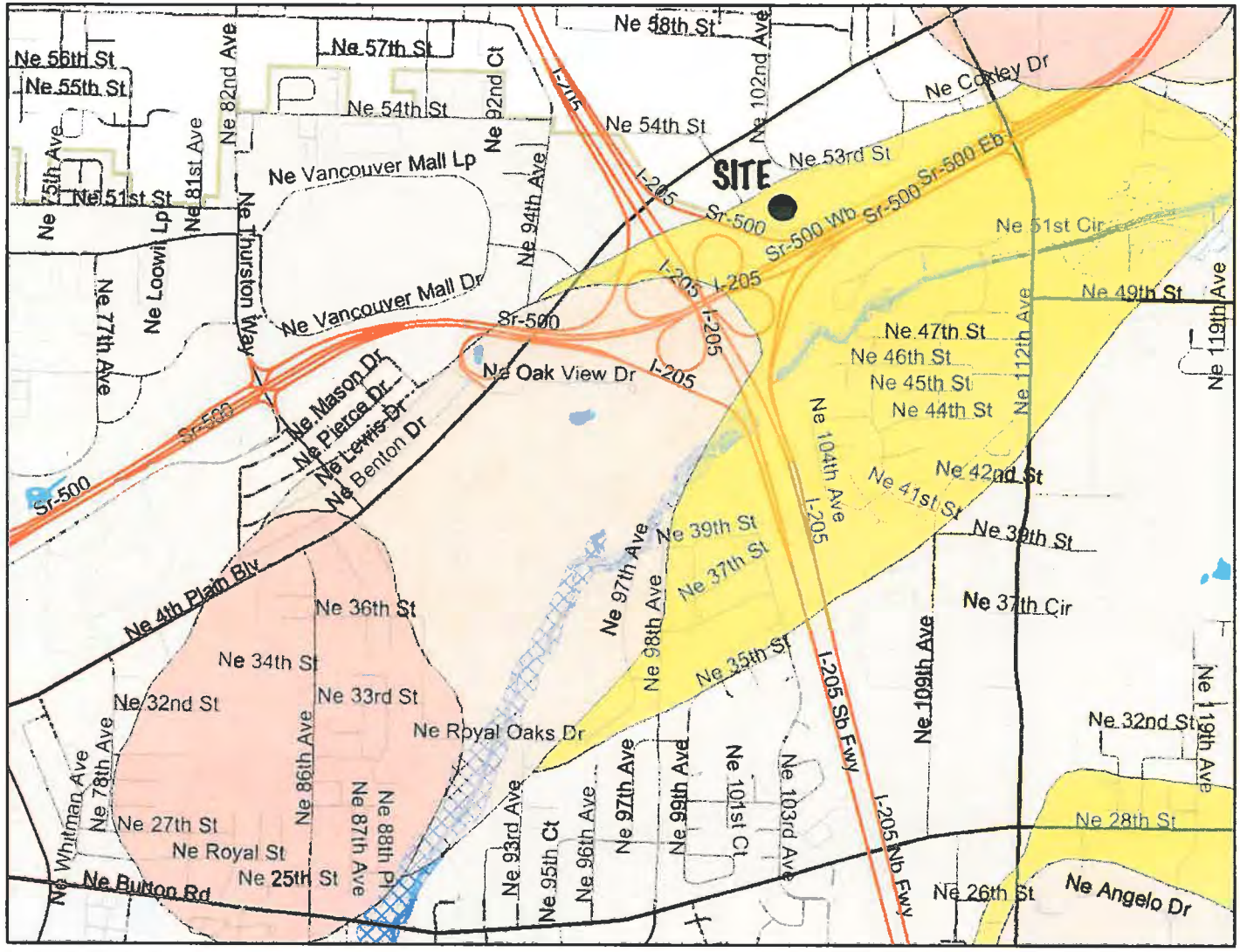
Map center: 45° 39' 33" N, 122° 33' 59" W

N

Scale: 1:50,000

Information shown on this map was collected from several sources. Neither Clark County, Washington, nor the producer of this document accept responsibility for any inaccuracies that may be present.

FLOOD PLAIN MAP



Legend

Public Wells

- 1 Year
- 5 Year
- 10 Year

Roads

- Alley
- Arterial
- DNR
- DNR (Private Land)
- Driveway
- Interstate
- Interstate Ramp
- Primary Arterial
- Private Roads
- Private Roads w/o Names
- Public Roads
- SR Ramp
- State Route

Proposed Floodplains

- Floodway Fringe
- Floodway

Floodplains

- Floodway Fringe
- Floodway

Waterbodies

City Boundaries

Urban Growth Boundaries

County Boundary

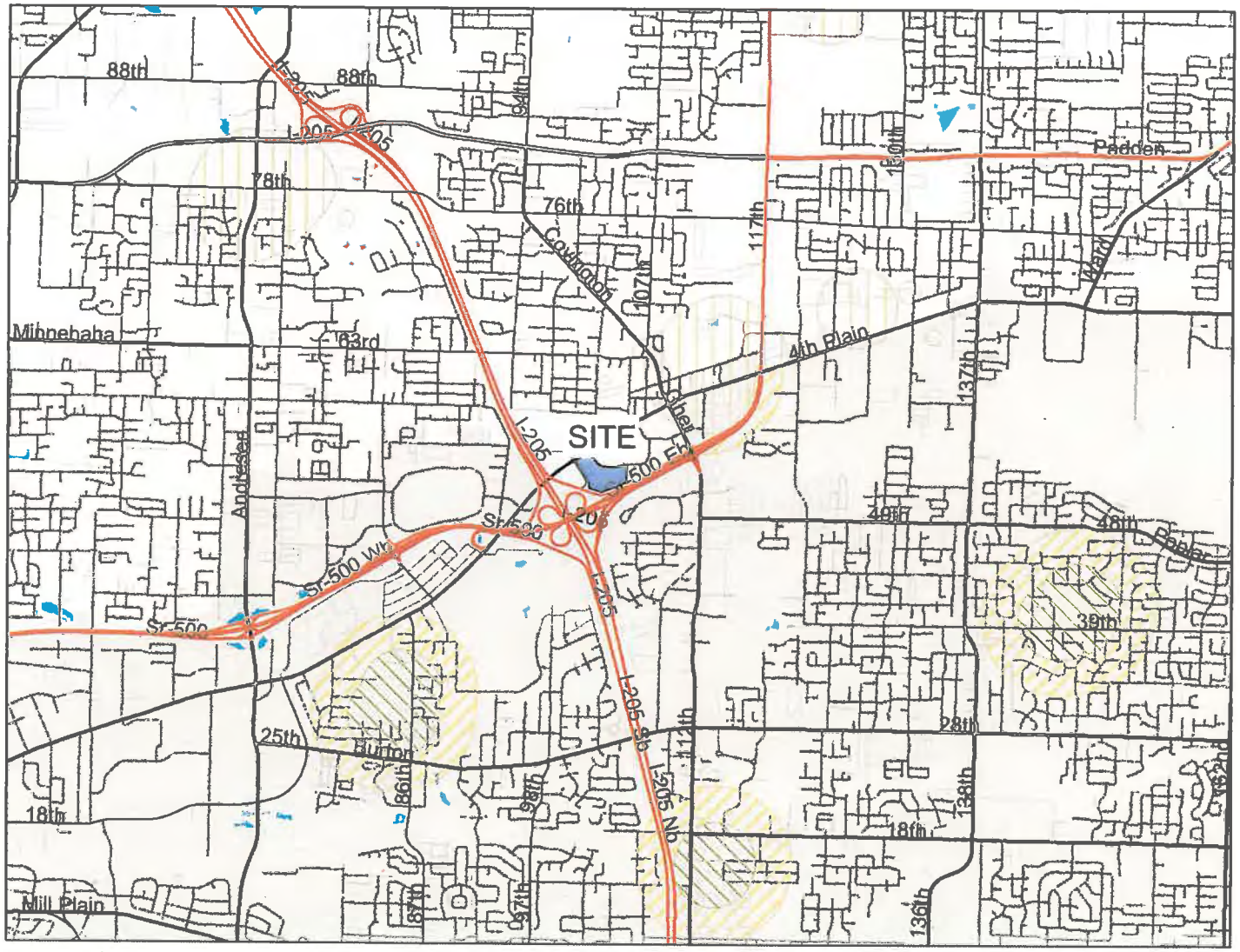
0 2000 4000 6000 ft.

Map center: 45° 39' 8" N, 122° 34' 26" W

Scale: 1:20,000

Information shown on this map was collected from several sources. Neither Clark County, Washington, nor the producer of this document accept responsibility for any inaccuracies that may be present.

Groundwater Protection Area Map



Legend

Roads
- Alley
- Arterial
- DNR
- DNR (Private Land)
- Driveway
- Interstate
- Interstate Ramp
- Primary Arterial
- Private Roads
- Private Roads w/o Names
- Public Roads
- SR Ramp
- State Route

Groundwater Protection Areas
- Category 1 Recharge Areas
- Category 2 Recharge Areas
- Within 1,000-foot buffer
- Within 1,900-foot buffer

Waterbodies

City Boundaries

Urban Growth Boundaries

County Boundary

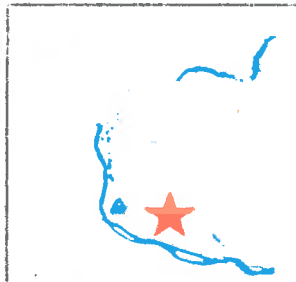
0 5000 10000 15000 ft.

Map center: 45° 39' 35" N, 122° 33' 58" W

Scale: 1:50,000

Information shown on this map was collected from several sources. Neither Clark County, Washington, nor the producer of this document accept responsibility for any inaccuracies that may be present.

Vancouver Toyota Aerial Photo



Legend

- Parcels
- Aerial Photography
- Waterbodies
- City Boundaries
- Urban Growth Boundaries
- County Boundary

0 250 500 750 ft.

Map center: 45° 39' 36.9" N, 122° 34' 3.4" W

Scale: 1:2,500

Information shown on this map was collected from several sources. Neither Clark County, Washington, nor the producer of this document accept responsibility for any inaccuracies that may be present.

1974

NE 1/4 of Section 16 T2R2E WM

Serial Number: 159839-010
Owner: MNM PROPERTIES LLC
Address: 0

Serial Number: 159839-005
Owner: MNM PROPERTIES LLC
Address: 0

Serial Number: 159839-000
Owner: MNM PROPERTIES LLC
Address: 0

Major Roads
- State
- State On-Ramp
- Interstate
- Interstate On-Ramp
- Primary Arterial
- Arterial
- Roads

Quarter Section Quadrant

Municipal Jurisdiction
- Unincorporated
- Incorporated

(Scale 1:3600)

100 0 100 200 300 400 Feet

Assessment & GIS

proud past, promising future

CLARK COUNTY
WASHINGTON

N

Plot Date: Aug 22, 2007 Map produced by: lm

Information shown on this map was collected from several sources. Neither Clark County or the agency producing this document accept responsibility for any inaccuraces that may be present.

Appendix B: Hydrologic Calculations of Vancouver Toyota Final Stormwater Report

APPENDIX B

Hydrologic Calculations

Soil Characteristics - B1-2

Isopluvial Maps (100-year) - B3

SCS Western Washington Runoff Curve Numbers - B5

HydroCAD Santa Barbara Urban Hydrograph Printouts - Inserts

Vancouver Toyota
Project #14292

MacKay & Sposito, Inc.

Soil Characteristics

Lauren Series

The Lauren series consists of deep, somewhat excessively drained, nearly level to gently sloping soils on terraces 50 to 300 feet above the Columbia River. In a few places, on terrace fronts, the soils are steep to very steep. These are very gravelly soils that formed in mixed Columbia River alluvium that contained some volcanic ash. Lauren soils are in the southwestern part of the county, in the vicinity of Mill Plain, Orchards, and Fourth Plain. The original vegetation was Douglas-fir, grand fir, big leaf maple, vine maple, salal, and ferns. The average annual precipitation is about 48 inches. Nearly all the acreage is cleared and in cultivation or suburban development. There are a few stands of second growth Douglas-fir in farm woodlots.

Lauren gravelly loam, 0 to 8 percent slopes (LgB).-This soil occurs on terraces. The slopes are generally less than 4 percent and approach 8 percent only along the terrace breaks. In a typical profile the surface layer is very dark brown gravelly and very gravelly loam about 20 inches thick. Below the surface layer is friable, dark-brown very gravelly loam about 13 inches thick. The next layer is dark-brown very gravelly coarse sandy loam about 11 inches thick. The underlying material, to a depth of 70 inches, is dark-brown very gravelly loamy coarse sand. Included in mapping were a few small areas where very gravelly loamy coarse sand is within 30 inches of the surface. This soil is somewhat excessively drained and easily tilled. Permeability generally is moderately rapid, but it is rapid in the substratum. The available water capacity is moderate. Fertility is moderate. Surface runoff is slow, and the erosion hazard is slight. Representative profile of Lauren gravelly loam, about 8 miles east of Vancouver, 600 feet north of southeast 15th Street, and 250 feet east of 164th Avenue, SE1/4SE1/4SE1/4 sec. 36, T. 2 N., R. 2 E.

Tisch Series

The Tisch series consists of deep, poorly drained, highly organic soils underlain by stratified layers of diatomaceous earth and peat to a depth of about 4 feet or more. These are medium-textured soils that formed in alluvium in shallow depressions. They are generally ponded. The relief is level to slightly concave. The original vegetation was sedges, reeds, deciduous trees, hardhack, and other water-tolerant shrubs and grasses. The annual precipitation is 40 to 60 inches. Tisch soils are used for hay, pasture, mint, and truck crops.

Tisch silt loam, 0 to 3 percent slopes (ThA) -This soil occupies depressions, mainly along Burntbridge Creek. In most places the slope is less than 1 percent. In a typical profile the surface layer is silt loam about 27 inches thick. It is very dark gray in the upper part and mottled, very dark gray, very dark brown, and pale brown in the lower part. The subsurface layer is very dark brown mottled silt loam about 4 inches thick. The next layer is very dark brown muck about 14 inches thick. The underlying material, to a depth of 53 inches, is very dark grayish-brown peat. Included in mapping were a few areas where gravel and stones are less than 40 inches below the surface and other areas where the surface layer is silty clay loam. Also included were small areas where thin layers of volcanic ash or diatomite occur within the surface layer. This soil is poorly drained and is ponded much of the year. Permeability is moderately slow, and fertility is moderate. The available water capacity is very high. There is no erosion hazard. The soil is easily tilled, and drained areas can be cultivated throughout a wide range of moisture content. Artificial drainage is needed to make the soil suitable for farming, but drainage is difficult. Lateral permeability is slow and necessitates closely spaced open or closed drains. Representative profile of Tisch silt loam, in a pasture about 30 feet north of Burntbridge Creek drainage ditch and 1/8 mile east of NE. 137th Avenue in NE1/4NE1/4NAV1/4 sec. 14, T. 2 N., R. 2 E.

Wind River Series
The Wind River series consists of deep, somewhat excessively drained, nearly level to very steep soils. These are gravelly soils that formed in Columbia River alluvium of mixed origin. They occur at elevations of 150 to 500 feet, in the southwestern part of the county, between Vancouver and Orchards. The original vegetation was Douglas-fir, grand fir, and Oregon white oak. The understory was hazel, dogwood, vine maple, salal, and ferns. The annual precipitation is about 40 inches. Nearly all the acreage has been cleared and cultivated. It is rapidly being converted to urban development. The few areas in crop production are used mainly for pasture, tree fruits, nuts, and row crops.

Wind River gravelly loam, 0 to 8 percent slopes (WrB).-
This is the dominant soil in the area between Vancouver and Orchards. In most places the slope is nearly level and is generally less than 3 percent. It is similar to Wind River sandy loam, 0 to f percent slopes, except for the texture of the surface layer. Nearly all of this soil has been cleared and is in cultivation or suburban development. The principal crops are pasture, tree fruits, and row crops. Red clover and white clover are the legumes commonly grown; orchardgrass, tall fescue, and ryegrass are the common grasses. Pears, apples, and hazelnuts are the tree crops; pole beans and strawberries are the common row crops. (Capability unit IIIe- 3; woodland suitability group 3fM3; wildlife site 6)

CLARK COUNTY, WASHINGTON

100 YEAR 24-HOUR ISOPLUVIALS

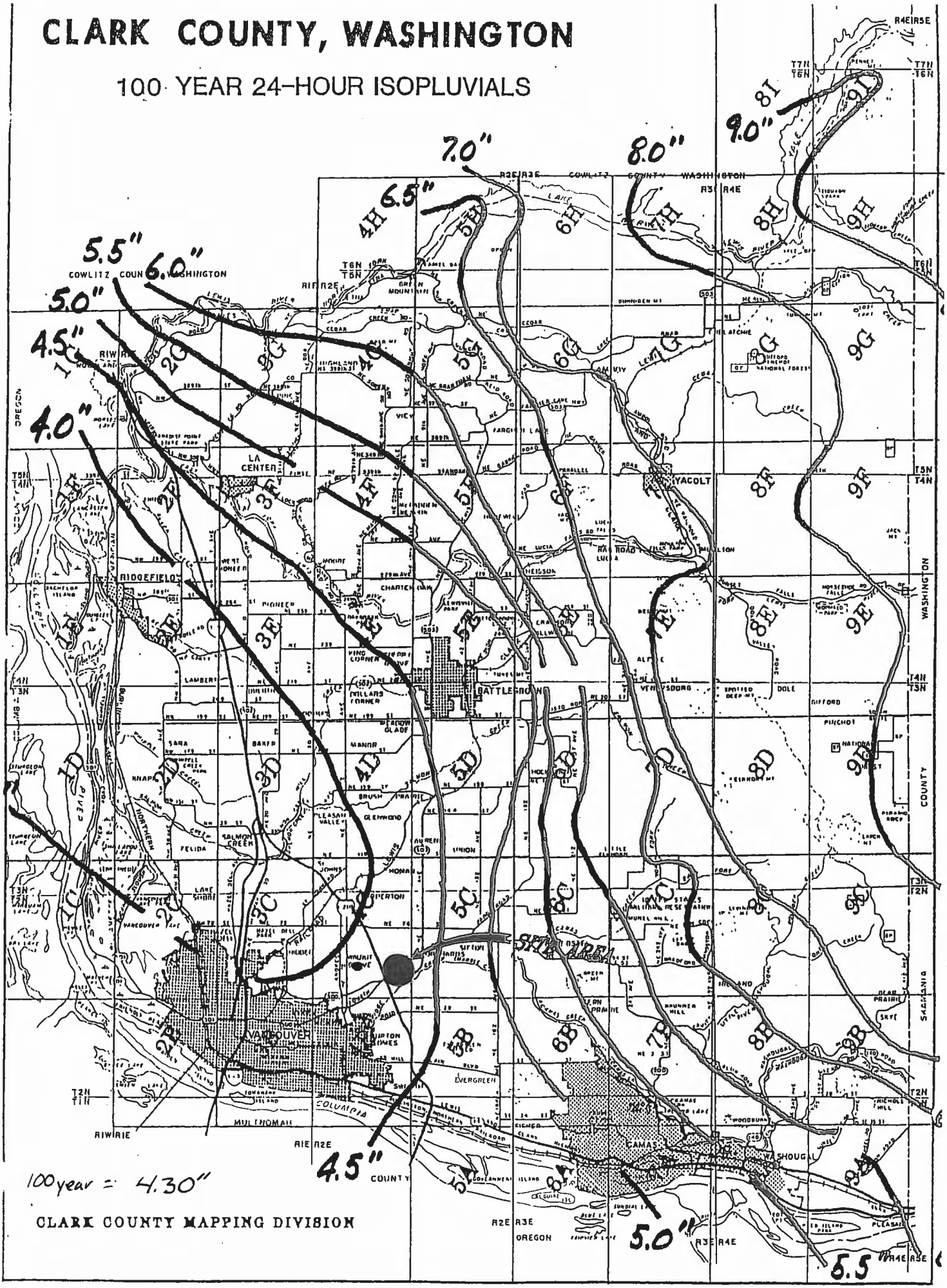


Exhibit B

Hydrologic Soil Groups for Soils in Clark County

U.S. Department of Agriculture
Soil Conservation Service

WATER FEATURES

Survey Area—CLARK COUNTY, WASHINGTON

Map symbol and soil name		Hydrologic group	Flooding			High water table		
			Freq	Duration	Months	Depth (Ft)	Kind	Months
BpB	BEAR PRAIRIE	B	NONE		—	6.0—6.0		—
BpC	BEAR PRAIRIE	B	NONE		—	6.0—6.0		—
CnB	CINEBAR	B	NONE		—	6.0—6.0		—
CnD	CINEBAR	B	NONE		—	6.0—6.0		—
CnE	CINEBAR	B	NONE		—	6.0—6.0		—
CnG	CINEBAR	B	NONE		—	6.0—6.0		—
CrE	CINEBAR	B	NONE		—	6.0—6.0		—
CrG	CINEBAR	B	NONE		—	6.0—6.0		—
CsF	CISPUS	B	NONE		—	6.0—6.0		—
CtA	CLOQUATO	B	OCCA		NOV—MAR	6.0—6.0		—
CvA	COVE	D	OCCA		DEC—APR	0—1.0	PERCH	DEC—JUN
CwA	COVE	D	OCCA		DEC—APR	0—1.0	PERCH	DEC—JUN
DoB	DOLLAR	C	NONE		—	1.5—3.0	PERCH	NOV—APR
Fn	FILL LAND		NONE		—	2.0—2.0		—
GeB	GEE	C	NONE		—	2.0—4.0	PERCH	NOV—APR
GeD	GEE	C	NONE		—	2.0—4.0	PERCH	NOV—APR
GeE	GEE	C	NONE		—	2.0—4.0	PERCH	NOV—APR
GeF	GEE	C	NONE		—	2.0—4.0	PERCH	NOV—APR
GuB	GUMBOOT	D	NONE		—	0—1.5	APPAR	DEC—APR
HcB	HESSON	C	NONE		—	6.0—6.0		—
HcD	HESSON	C	NONE		—	6.0—6.0		—
HcE	HESSON	C	NONE		—	6.0—6.0		—
HcF	HESSON	C	NONE		—	6.0—6.0		—
HgB	HESSON	C	NONE		—	6.0—6.0		—
HgD	HESSON	C	NONE		—	6.0—6.0		—
HhE	HESSON	C	NONE		—	6.0—6.0		—
HiA	HILLSBORO	B	NONE		—	6.0—6.0		—
HiB	HILLSBORO	B	NONE		—	6.0—6.0		—
HiC	HILLSBORO	B	NONE		—	6.0—6.0		—
HiD	HILLSBORO	B	NONE		—	6.0—6.0		—
HiE	HILLSBORO	B	NONE		—	6.0—6.0		—
HiF	HILLSBORO	B	NONE		—	6.0—6.0		—
HoA	HILLSBORO	B	NONE		—	6.0—6.0		—
HoB	HILLSBORO	B	NONE		—	6.0—6.0		—
HoC	HILLSBORO	B	NONE		—	6.0—6.0		—
HoD	HILLSBORO	B	NONE		—	6.0—6.0		—
HoE	HILLSBORO	B	NONE		—	6.0—6.0		—
HoG	HILLSBORO	B	NONE		—	6.0—6.0		—
HsB	HILLSBORO	B	NONE		—	6.0—6.0		—
HtA	HOCKINSON	D	NONE		—	0.5—1.5	APPAR	NOV—APR
HuB	HOCKINSON	D	NONE		—	0.5—1.5	APPAR	NOV—APR
HvA	HOCKINSON	D	NONE		—	0.5—1.5	APPAR	NOV—APR
	DOLLAR	C	NONE		—	1.5—3.0	PERCH	NOV—APR

WATER FEATURES

Survey Area—CLARK COUNTY, WASHINGTON

Map symbol and soil name	Hydrologic group	Flooding			High water table		
		Freq	Duration	Months	Depth (Ft)	Kind	Months
KeC KINNEY	B	NONE		—	6.0—6.0		—
KeE KINNEY	B	NONE		—	6.0—6.0		—
KeF KINNEY	B	NONE		—	6.0—6.0		—
KnF KINNEY	B	NONE		—	6.0—6.0		—
LaE LARCHMOUNT	B	NONE		—	6.0—6.0		—
LaG LARCHMOUNT	B	NONE		—	6.0—6.0		—
LcG LARCHMOUNT	B	NONE		—	6.0—6.0		—
LeB LAUREN	B	NONE		—	6.0—6.0		—
LgB LAUREN	B	NONE		—	6.0—6.0		—
LgD LAUREN	B	NONE		—	6.0—6.0		—
LgF LAUREN	B	NONE		—	6.0—6.0		—
LiB LAUREN	B	NONE		—	6.0—6.0		—
LrC LAUREN	C	NONE		—	1.5—3.0	PERCH	DEC—MAR
LrF LAUREN	C	NONE		—	1.5—3.0	PERCH	DEC—MAR
McB McBEE	C	FREQ		NOV—MAY	2.0—3.0	APPAR	NOV—APR
McB McBEE	C	OCCA		NOV—MAY	2.0—3.0	APPAR	NOV—APR
MeA McBEE	C	FREQ		NOV—MAY	2.0—3.0	APPAR	NOV—APR
MiA McBEE VARIANT	D	RARE		—	—	APPAR	—
MnA MINNIECE	D	NONE		—	0—2.0	PERCH	NOV—MAY
MnD MINNIECE	D	NONE		—	0—2.0	PERCH	NOV—MAY
MoA MINNIECE VARIANT	D	NONE		—	0—2.0	PERCH	NOV—MAY
MsB MOSSYROCK	B	NONE		—	6.0—6.0		—
NbA NEWBERG	B	OCCA		DEC—MAR	6.0—6.0		—
NbB NEWBERG	B	OCCA		DEC—MAR	6.0—6.0		—
OdB ODNE	D	NONE		—	0—1.5	APPAR	OCT—APR
OeD OLEQUA	B	NONE		—	6.0—6.0		—
OeE OLEQUA	B	NONE		—	6.0—6.0		—
OeF OLEQUA	B	NONE		—	6.0—6.0		—
OhD OLEQUA VARIANT	C	NONE		—	2.0—3.0	APPAR	NOV—MAY
OhF OLEQUA VARIANT	C	NONE		—	2.0—3.0	APPAR	NOV—MAY
OiB OLYMPIC	B	NONE		—	6.0—6.0		—
OiD OLYMPIC	B	NONE		—	6.0—6.0		—
OiE OLYMPIC	B	NONE		—	6.0—6.0		—
OiF OLYMPIC	B	NONE		—	6.0—6.0		—
OmE OLYMPIC	B	NONE		—	6.0—6.0		—
OmF OLMPIC	B	NONE		—	6.0—6.0		—
OpC OLYMPIC VARIANT	C	NONE		—	6.0—6.0		—
OpE OLYMPIC VARIANT	C	NONE		—	6.0—6.0		—
OpG OLYMPIC VARIANT	C	NONE		—	6.0—6.0		—
OrC OLYMPIC VARIANT	C	NONE		—	6.0—6.0		—
PhB PILCHUCK	C	OCCA		NOV—APR	2.0—4.0	APPAR	NOV—APR
PoB POWELL	C	NONE		—	1.5—2.0	PERCH	DEC—APR
PoD POWELL	C	NONE		—	1.5—2.0	PERCH	DEC—APR
PoE POWELL	C	NONE		—	1.5—2.0	PERCH	DEC—APR

WATER FEATURES

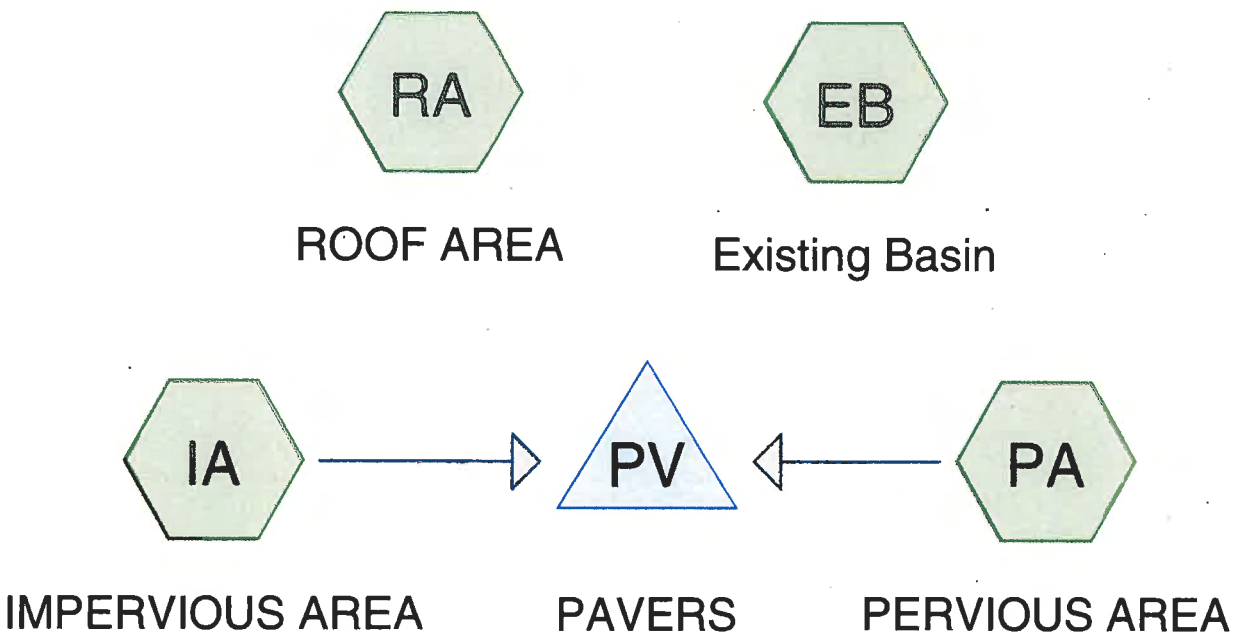
Survey Area—CLARK COUNTY, WASHINGTON

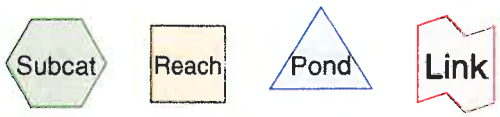
Map symbol and soil name		Hydrologic group	Flooding			High water table		
			Freq	Duration	Months	Depth (Ft)	Kind	Months
PuA	PUYALLUP	B	OCCA		NOV—APR	6.0—6.0		—
Ra	RIVERWASH	D	FREQ	VERY LONG	OCT—JUL	0—2.0	APPAR	JAN—DEC
Rc	RIVERWASH	D	FREQ	VERY LONG	OCT—JUL	0—2.0	APPAR	JAN—DEC
Rk	ROCK LAND	D	NONE		—	6.0—6.0		—
Ro	ROUGH BROKEN LAND	A	NONE		—	6.0—6.0		—
SaC	SALKUM	B	NONE		—	6.0—6.0		—
SiB	SARA	D	NONE		—	1.0—2.0	PERCH	DEC—APR
SiD	SARA	D	NONE		—	1.0—2.0	PERCH	DEC—APR
SiF	SARA	D	NONE		—	1.0—2.0	PERCH	DEC—APR
SmA	SAUVIE	B	RARE		—	6.0—6.0		—
SmB	SAUVIE	B	RARE		—	6.0—6.0		—
SnA	SAUVIE	D	FREQ		DEC—JUN	0—1.0	APPAR	MAY—JUN
SpB	SAUVIE	B	RARE		—	6.0—6.0		—
Sr	SEMIAHMOO	C	RARE		—	1.5—3.0	APPAR	NOV—MAY
Su	SEMIAHMOO VARIANT	D	NONE		—	—	APPAR	—
Su	SEMIAHMOO VARIANT	C	NONE		—	1.5—3.0	APPAR	NOV—MAY
SyA	SIFTON	B	NONE		—	6.0—6.0		—
ThA	TISCH	D	NONE		—	—	APPAR	—
ThA	TISCH	D	NONE		—	0—1.0	APPAR	DEC—APR
VaB	VADER	B	NONE		—	6.0—6.0		—
VaC	VADER	B	NONE		—	6.0—6.0		—
WaA	WASHOUGAL	B	RARE		—	6.0—6.0		—
WgB	WASHOUGAL	B	NONE		—	6.0—6.0		—
WgE	WASHOUGAL	B	NONE		—	6.0—6.0		—
WhF	WASHOUGAL	B	NONE		—	6.0—6.0		—
WnB	WIND RIVER VARIANT	B	NONE		—	6.0—6.0		—
WnD	WIND RIVER VARIANT	B	NONE		—	6.0—6.0		—
WnG	WIND RIVER VARIANT	B	NONE		—	6.0—6.0		—
WrB	WIND RIVER VARIANT	B	NONE		—	6.0—6.0		—
WrF	WIND RIVER VARIANT	B	NONE		—	6.0—6.0		—
YaA	YACOLT	B	NONE		—	6.0—6.0		—
YaC	YACOLT	B	NONE		—	6.0—6.0		—
YcB	YACOLT	B	NONE		—	6.0—6.0		—

Table III-1.3 SCS Western Washington Runoff Curve Numbers
(Published by SCS in 1982) Runoff curve numbers for selected agricultural, suburban and urban land use for Type 1A rainfall distribution, 24-hour storm duration.

LAND USE DESCRIPTION		CURVE NUMBERS BY HYDROLOGIC SOIL GROUP A	B	C	D
Cultivated land(1):	winter condition	86	91	94	95
Mountain open areas:	low growing brush & grasslands	74	82	89	92
Meadow or pasture:		65	78	85	89
Wood or forest land:	undisturbed	42	64	76	81
Wood or forest land:	young second growth or brush	55	72	81	86
Orchard:	with cover crop	81	88	92	94
Open spaces, lawns, parks, golf courses, cemeteries, landscaping.					
Good condition:	grass cover on ≥75% of the area	68	80	86	90
Fair condition:	grass cover on 50-75% of the area	77	85	90	92
Gravel roads & parking lots:		76	85	89	91
Dirt roads & parking lots:		72	82	87	89
Impervious surfaces, pavement, roofs etc.		98	98	98	98
Open water bodies:	lakes, wetlands, ponds etc.	100	100	100	100
Single family residential(2):					
Dwelling Unit/Gross Acre	%Impervious(3)				
1.0 DU/GA	15	Separate curve number shall be selected for pervious & impervious portions of the site or basin			
1.5 DU/GA	20				
2.0 DU/GA	25				
2.5 DU/GA	30				
3.0 DU/GA	34				
3.5 DU/GA	38				
4.0 DU/GA	42				
4.5 DU/GA	46				
5.0 DU/GA	48				
5.5 DU/GA	50				
6.0 DU/GA	52				
6.5 DU/GA	54				
7.0 DU/GA	56				
PUD's, condos, apartments, commercial businesses & industrial areas	%impervious must be computed				

(1) For a more detailed description of agricultural land use curve numbers refer to National Engineering Handbook, Sec. 4, Hydrology, Chapter 9, August 1972.
(2) Assumes roof and driveway runoff is directed into street/storm system.
(3) The remaining pervious areas (lawn) are considered to be in good condition for these curve numbers.





Drainage Diagram for 14292_FINAL TIR CALCS
Prepared by Mackay & Sposito, Inc. 5/5/2008
HydroCAD® 8.00 s/n 001662 © 2006 HydroCAD Software Solutions LLC

Area Listing (all nodes)

	Area (acres)	CN	Description (subcats)
EX	0.210	65	Meadow or Pasture (EB)
EX	0.820	78	Meadow or Pasture (EB)
PROP	1.500	89	<50% Grass cover, Poor, HSG D (PA)
EX	9.300	89	Meadow or Pasture (EB)
PROP	0.980	98	(RA)
~~PROP~~	~~0.980~~	~~98~~	~~Building Area (IA)~~
PROP	7.660	98	Permeable Paver Area (IA)
PROP	0.190	98	Sidewalks (IA)
	~~21.640~~		

Time span=0.00-24.00 hrs, dt=0.05 hrs, 481 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment EB: Existing Basin Runoff Area=10.330 ac Runoff Depth>2.94"
Flow Length=750' Tc=50.7 min CN=88/0 Runoff=4.55 cfs 2.531 af

Subcatchment IA: IMPERVIOUS AREA Runoff Area=8.830 ac Runoff Depth>4.06"
Tc=6.0 min CN=0/98 Runoff=8.86 cfs 2.985 af

Subcatchment PA: PERVIOUS AREA Runoff Area=1.500 ac Runoff Depth>3.10"
Tc=6.0 min CN=89/0 Runoff=1.19 cfs 0.387 af

Subcatchment RA: ROOF AREA Runoff Area=0.980 ac Runoff Depth>4.06"
Tc=0.0 min CN=0/98 Runoff=1.01 cfs 0.332 af

Pond PV: PAVERS Peak Elev=0.37' Storage=0.286 af Inflow=10.05 cfs 3.372 af
Outflow=3.93 cfs 3.370 af

Total Runoff Area = 21.640 ac Runoff Volume = 6.235 af Average Runoff Depth = 3.46"
54.67% Pervious Area = 11.830 ac 45.33% Impervious Area = 9.810 ac

Subcatchment EB: Existing Basin

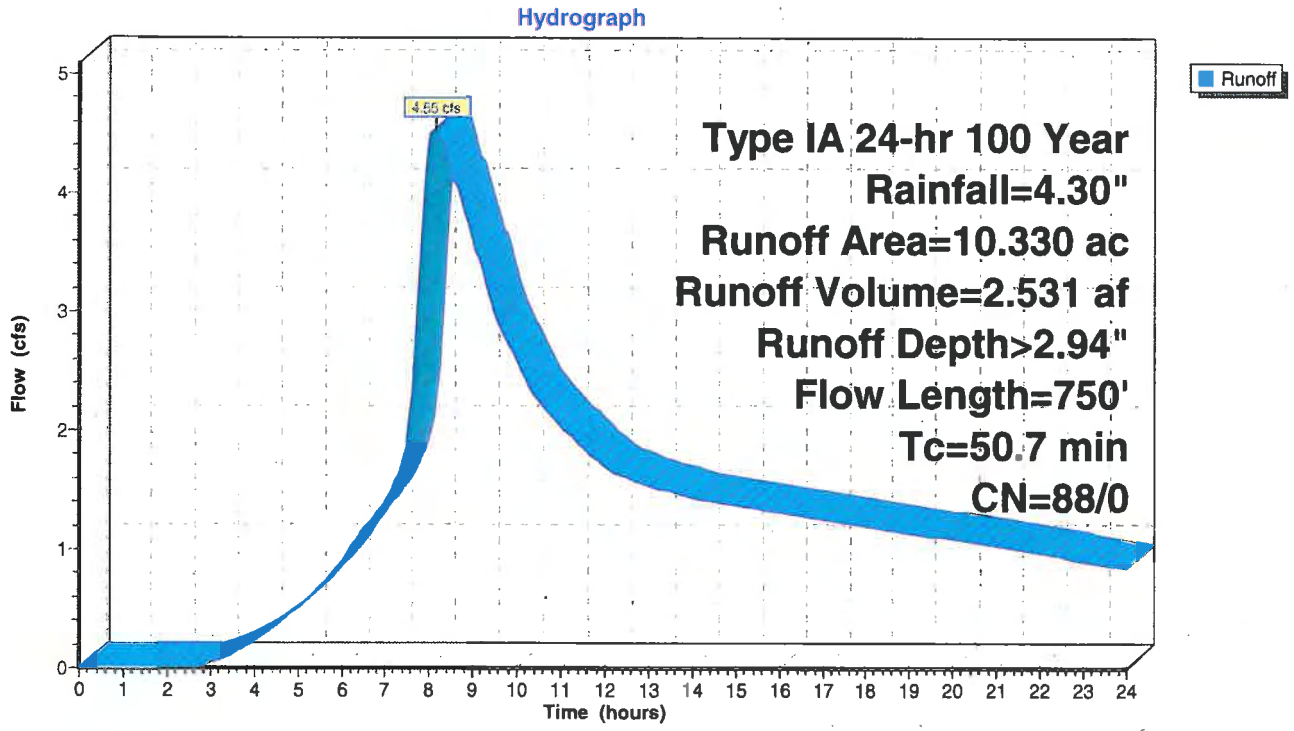
Runoff = 4.55 cfs @ 8.15 hrs, Volume= 2.531 af, Depth> 2.94"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100 Year Rainfall=4.30"

Area (ac)	CN	Description
9.300	89	Meadow or Pasture
0.820	78	Meadow or Pasture
0.210	65	Meadow or Pasture
10.330	88	Weighted Average
10.330	88	Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
41.3	300	0.0056	0.12		**Sheet Flow, First 300** Range n= 0.130 P2= 2.30"
5.4	200	0.0031	0.61		**Shallow Concentrated Flow, Next 200 Feet** Kv= 11.0 fps
4.0	250	0.0088	1.03		**Shallow Concentrated Flow, Last 250** Kv= 11.0 fps
50.7	750	Total			

Subcatchment EB: Existing Basin



Subcatchment IA: IMPERVIOUS AREA

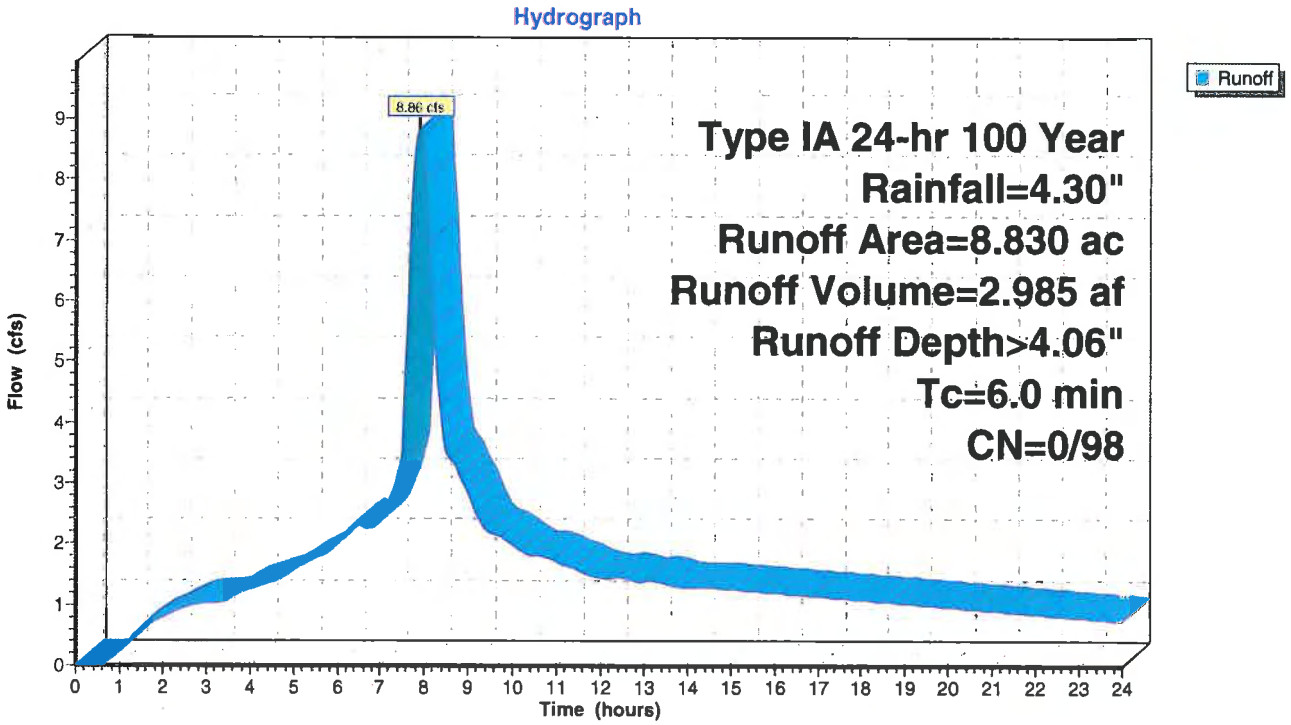
Runoff = 8.86 cfs @ 7.92 hrs, Volume= 2.985 af, Depth> 4.06"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100 Year Rainfall=4.30"

Area (ac)	CN	Description
0.980	98	Building Area
7.660	98	Permeable Paver Area
0.190	98	Sidewalks
8.830	98	Weighted Average
8.830	98	Impervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment IA: IMPERVIOUS AREA



Subcatchment PA: PERVIOUS AREA

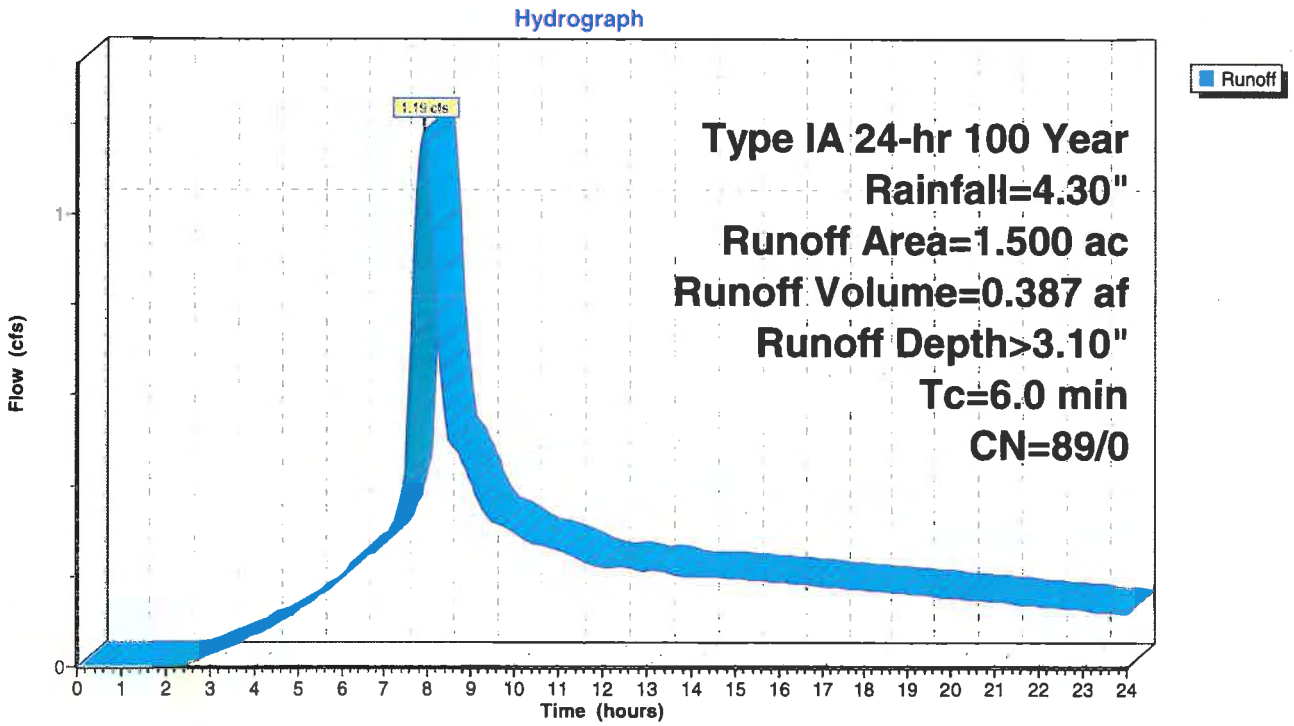
Runoff = 1.19 cfs @ 7.95 hrs, Volume= 0.387 af, Depth> 3.10"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100 Year Rainfall=4.30"

Area (ac)	CN	Description
1.500	89	<50% Grass cover, Poor, HSG D
1.500	89	Pervious Area

Tc (min)	Length (feet)	Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
6.0					**Direct Entry,**

Subcatchment PA: PERVIOUS AREA



Subcatchment RA: ROOF AREA

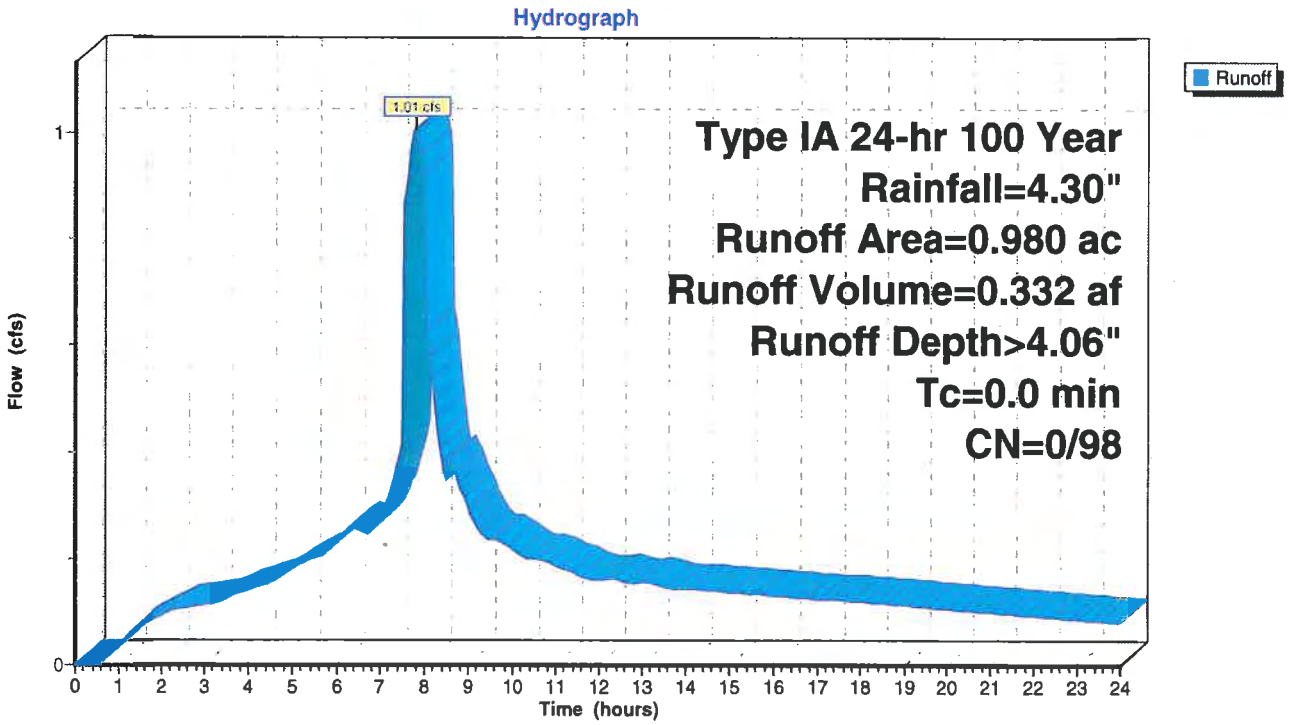
[46] Hint: Tc=0 (Instant runoff peak depends on dt)

Runoff = 1.01 cfs @ 7.80 hrs, Volume= 0.332 af, Depth> 4.06"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Type IA 24-hr 100 Year Rainfall=4.30"

Area (ac)	CN	Description
0.980	98	
0.980	98	Impervious Area

Subcatchment RA: ROOF AREA



Pond PV: PAVERS

Inflow Area = 10.330 ac, Inflow Depth > 3.92" for 100 Year event
Inflow = 10.05 cfs @ 7.92 hrs, Volume= 3.372 af
Outflow = 3.93 cfs @ 7.55 hrs, Volume= 3.370 af, Atten= 61%, Lag= 0.0 min
Discarded = 3.93 cfs @ 7.55 hrs, Volume= 3.370 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.05 hrs
Peak Elev= 0.37' @ 8.55 hrs Surf.Area= 7.800 ac Storage= 0.286 af

Plug-Flow detention time= 12.1 min calculated for 3.370 af (100% of inflow)
Center-of-Mass det. time= 11.7 min (678.9 - 667.2)

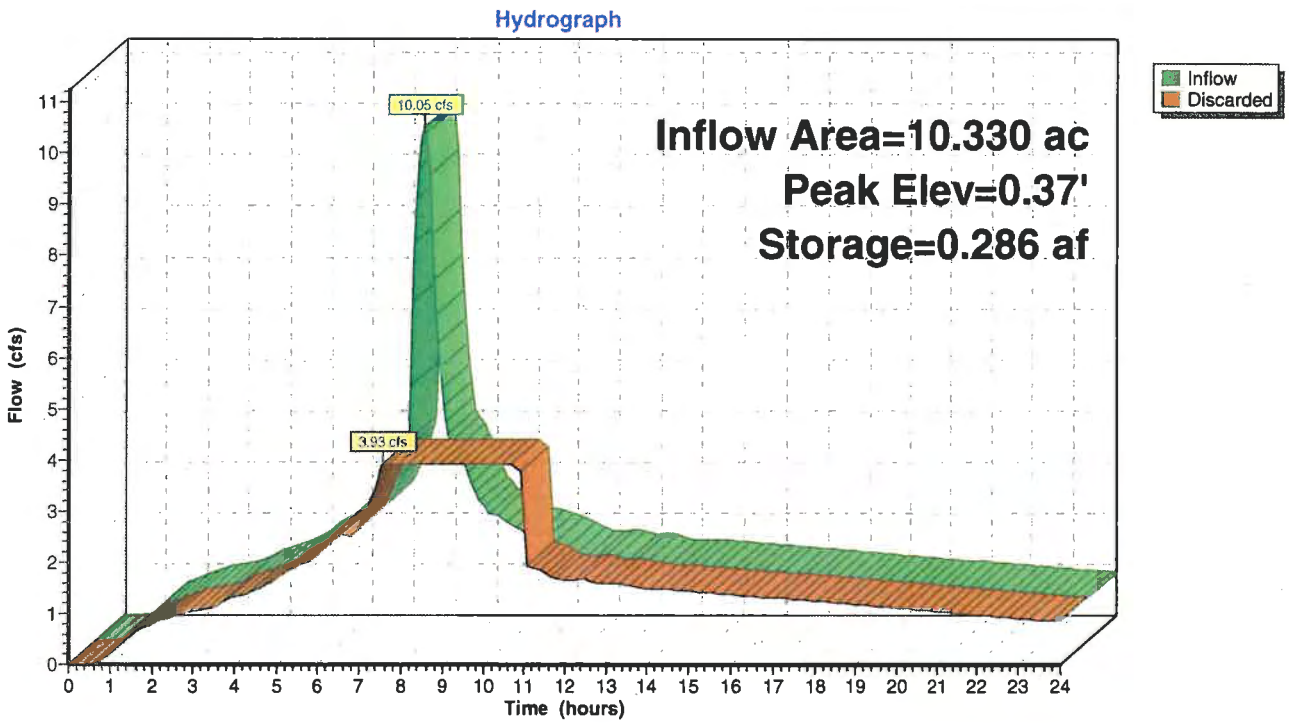
Volume	Invert	Avail.Storage	Storage Description
#1	0.00'	0.780 af	**Custom Stage Data (Prismatic)** Listed below (Recalc) 7.800 af Overall x 10.0% Voids

Elevation (feet)	Surf.Area (acres)	Inc.Store (acre-feet)	Cum.Store (acre-feet)
0.00	7.800	0.000	0.000
1.00	7.800	7.800	7.800

Device	Routing	Invert	Outlet Devices
#1	Discarded	0.00'	**0.500 in/hr Exfiltration over Surface area**

Discarded OutFlow Max=3.93 cfs @ 7.55 hrs HW=0.01' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 3.93 cfs)

Pond PV: PAVERS



Appendix C: Hydraulic Calculations of Vancouver Toyota Final Stormwater Report

APPENDIX C

Hydraulic Calculations

PCSWMM Calculation Printout - C1

Pavement Section Detail - C2

Roof Drainage Conveyance - C3

Storm Roof Infiltration Design - C4

Vancouver Toyota
Project #14292

MacKay & Sposito, Inc.

PCSWMM for Permeable UNI ECO-STONE® Pavements

File: 14292_Med SCENARIO.PCS **Date:** 8/9/2007 8:50:48 AM

1.0 Input Parameters

Paver Description:

Clogging Potential	**Medium**
Void condition	New Installation
Infiltration rate	7.8 in/hr
Area	339768 ft²
Slope	0.5 %
Length of overland flow	1 ft

Run-on Description:

Type of surface	**No run-on**
Area	0 ft²
Slope	0.5 %
Length of overland flow	0 ft
Manning's n	0.014
Depression storage	0.02 in

Base Description:

Base material	**Open graded**
Depth of base	12 in
Porosity	0.38
Saturated H.K.	3500 in/hr
Field capacity	0.05
Curve fitting parameter	10
Tension / soil moisture	15 ft/fraction
Initial moisture content	5 %
Initial depth of water	0 in

Drainage Description:

Drainage type	**No drainage**
Threshold elevation	0 in
Flow coefficient	0 in/hr-ft^exp
Flow exponent	0

Subgrade Description:

Subgrade soil type	**Silty Gravels to Silts (GM,SM,ML,MH,OL)**
Percolation coefficient	0.4 in/hr

Design storm:

Rainfall time step	5 minutes
Rainfall values (in/hr)	0.14, 0.14, 0.14, 0.54, 0.86, 2.13, 1.15, 0.54, 0.54, 0.14, 0.14, 0.14

Evaluation Criteria:

Allowable surface runoff	0 % (0 ft³)
Allowable base water depth	85 % (10.2 in)

2.0 Computational Results

Maximum depth of groundwater in base material: 1.572 in

Overall runoff coefficient (C=R/P): 0

Surface summary:	**Volume**	**Depth**
Total rainfall	15572.7 ft^3	0.550 in
Total infiltration	15454.73 ft^3	0.546 in
Total evaporation	117.9749 ft^3	0.004 in
Total runoff	0 ft^3	0.000 in
Remaining surface storage	0 ft^3	0.000 in

Subsurface summary:	**Volume**	**Depth**
Total lateral base drainage	0 ft^3	0.000 in
Total deep percolation	13907.78 ft^3	0.491 in
Initial storage in base	16988.4 ft^3	0.600 in
Final storage in base	18393.68 ft^3	0.650 in

Continuity errors in computation:

Surface continuity	0.000 percent
Channel continuity	0.000 percent
Groundwater continuity	-0.010 percent

Notice:

The PCSWMM for Permeable Pavements software package is only a tool to aid design and for general guidance. The results given above are not a substitute for engineering skill and judgement and in no way replace the services of experienced and professionally qualified civil engineering consultants. Further, PCSWMM for Permeable Pavements is an interface for the USEPA Stormwater Management Model (SWMM) program - the results above are produced by the SWMM program and no guarantee is made by Computational Hydraulics Int. or F. VON LANGSDORF LICENSING LTD. as to the validity of these results. Full responsibility for the use of these results and this software package for any project remains wholly with the user.

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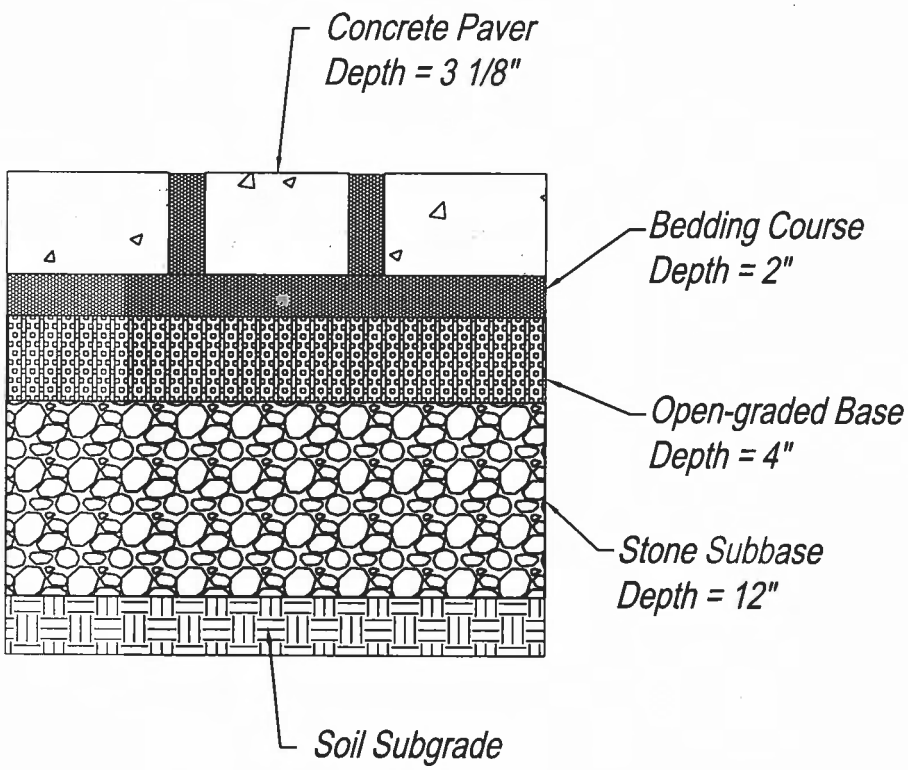
PCSWMM™ is a trademark of Computational Hydraulics Int.



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Permeable Concrete Pavement Section

6" ROOF DRAINAGE CONVEYANCE
Worksheet for Circular Channel

Project Description	
Worksheet	14292 ROOF DRAIN CONVE
Flow Element	Circular Channel
Method	Manning's Formula
Solve For	Channel Depth

Input Data		
Mannings Coefficient	0.013	
Slope	0.020000	ft/ft
Diameter	6	in
Discharge	0.73	cfs

Results		
Depth	0.38	ft
Flow Area	0.2	ft²
Wetted Perimeter	1.05	ft
Top Width	0.43	ft
Critical Depth	0.43	ft
Percent Full	75.6	%
Critical Slope	0.015715	ft/ft
Velocity	4.59	ft/s
Velocity Head	0.33	ft
Specific Energy	0.70	ft
Froude Number	1.33	
Maximum Discharg	0.85	cfs
Discharge Full	0.79	cfs
Slope Full	0.016928	ft/ft
Flow Type	Supercritical	

STORM ROOF INFILTRATION SYSTEM DESIGN

ROOF1

GIVEN:

1. Soil:	ThA, WrB, LgB	
2. 100 Year Design Storm Flow:	1.01	c.f.s.
3. Tested Infiltration Rate:	360.0	in./hr.
4. Design Infiltration Rate (50%):	180.0	in./hr.
5. Flow Conversion - inches/hour to cfs:	180.0 in. X ft. X hr. X min. / hr. X 12 in. X 60 min. X 60 sec. =	0.00417 c.f.s./s.f.

Perforated Pipe Capacity (per lineal foot):

1. Infiltration Trench Width:	3.0	ft.
2. Infiltration Trench Depth:	3.0	ft.
3. Wetted Perimeter Per Lineal Foot Of Trench:	9.0	s.f.
4. Flow Per Lineal Foot Of Trench:	0.03750	c.f.s./l.f.

Lineal Feet of Infiltration Trench Required:

1. Storm Flow to Infiltration Trench:	1.01	c.f.s.
2. Infiltration Rate Per Lineal Foot of Trench:	0.03750	c.f.s./l.f.
3. Lineal Feet of Trench Required:	26.9	l.f.

Job Number:	14,292	Designed By:	CNK
Job Name:	MacKay & Sposito	Checked By:	
Date:	3/11/2008		
Revised:			
Sheet:			

Roof Infiltration System Summary:

Roof Area (sq.ft.)	Trench length required
1000	27 ft.
2000	54 ft.
3000	81 ft.
4000	108 ft.
5000	135 ft.

Appendix D: Geotechnical Report of Vancouver Toyota Final Stormwater Report

APPENDIX D

Geotechnical Report

Vancouver Toyota
Project # 14292

MacKay & Sposito, Inc.

psi Information To Build On

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14292 - Van. Toyota

GEOTECHNICAL ENGINEERING SERVICES REPORT

For the

Proposed Vancouver Toyota Dealership Expansion
10009 NE Fourth plain Boulevard
Vancouver, Washington

Prepared for

Mr. Jeff Smith
Building Structures, Inc.
PO Box 69
Boring, Oregon

Prepared by

PROFESSIONAL SERVICE INDUSTRIES, INC.
6032 North Cutter Circle
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Telephone (503) 289-1778
Fax (503) 289-1918

PSI Report No. 704-75065-1

March 30th, 2007

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Engineering Associate
Geotechnical Services

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Figure No.1: SITE LOCATION PLAN
Figure No. 2 BORING LOCATION PLAN

APPENDICES

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Appendix B: RECORDS OF SUBSURFACE EXPLORATION
Appendix C: LABORATORY TESTING RESULTS
Appendix D: LOCAL WELL LOG RECORDS

1.0 EXECUTIVE SUMMARY

An exploration and evaluation of the subsurface conditions have been completed for the proposed Toyota dealership expansion located at 10009 NE Fourth Plain Boulevard in Vancouver, Washington. Soil borings and test pits have been conducted in the field and selected soil samples tested in the laboratory. In general, the borings and test pits conducted at the site revealed the presence of 4 to 7 feet undocumented fill materials mainly consisting of sandy silt to silty sand with gravels. At some locations, cobble sized particles and concrete/asphalt debris are presented in the fill materials. Underlying the fill materials, silt with gravel deposit with old buried topsoil or peat layer at the upper portion are encountered and extended to approximately 8 to 12 feet in depth below the existing grade. The silt with gravel deposit was underlain by sandy gravel which extends to at least 15 feet below the existing ground surface. During our field exploration processes, groundwater seepage was observed at 7 to 8 feet in depth at some of the test pits locations. Local well logs within half mile of the property indicate a static water level at approximately 10 feet below the ground surface.

Groundwater

Results of this exploration indicate that the subsurface conditions at the site are generally suitable for the use of conventional footing foundations bearing on medium dense to dense native deposits for support of the assumed structural loads and that the floor slab can be grade supported provided that the site is developed in accordance with the recommendations presented in this report. Details related to site development, foundation and general pavement design, and construction considerations are included in subsequent sections of this report.

Features requiring special consideration at this site are the presences of relatively thick undocumented variable fill and peat deposit below the fill materials. These features are discussed further in this report.

The owner/designer should not rely solely on this Executive Summary and must read and evaluate the entire contents of this report prior to utilizing our engineering recommendations in preparation of design/construction documents.

2.0 PROJECT INFORMATION

2.1 Project Authorization

Professional Service Industries, Inc. (PSI) has completed a geotechnical exploration for the proposed expansion of the Toyota dealership located at 10009 NE Fourth Plain Boulevard in Vancouver, Washington. Our services were contracted by Ms. Diane K. Stevens, Secretary Treasurer of Building Structures, Inc. on March 12th, 2007 by signing our proposal. This exploration was accomplished in general accordance with PSI Proposal No. 704-07-P083 dated March 8th, 2007.

2.2 Project Description

Project information regarding the proposed construction was obtained from Mr. Jeff Smith of Building Structures, Inc. We understand that the proposed development will consist of an approximately 44,000 square feet Sales & Service building as well as driveways and parking spaces. In addition, a storm water infiltration system is also planed on the site. We have been furnished with some site development plans and some preliminary building plans which show the property boundaries and the proposed construction. We assume that the facility will be constructed in accordance with provisions of the International Building Code, 2003 Edition (IBC 2003).

Detailed structural loading information was not provided; however, for the purpose of this report, we have assumed that maximum column and wall loads will be on the order of 80 kips and 6.0 kips per linear foot, respectively. Also, in our analyses, floor slab loads of less than 150 psf are assumed, and less than 2 feet of cut and 2 feet of fill are anticipated for the design grade.

The geotechnical recommendations presented in this report are based on the available project information, building locations, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform PSI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. PSI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

2.3 Purpose and Scope of Services

The purpose of this study was to explore the subsurface conditions at the site to enable an evaluation of acceptable foundation recommendations for the proposed facility. Our scope of services included drilling 4 soil test borings at the site to approximately 15 feet in depth, conducting 10 test pit explorations to depths ranging from approximately 5 and 8 feet below

the ground surface, carrying out 8 falling head infiltration tests, performing laboratory testing, and preparation of this geotechnical report. This report briefly outlines the available project information and testing procedures, addresses the site and subsurface conditions, describes the laboratory and field testing results, and presents recommendations regarding the following:

- Grading procedures for site development.
- Foundation types, depths, allowable bearing capacities, and an estimate of potential settlement.
- Recommendations for the floor slab support.
- General pavement design and pavement subgrade preparation.
- Comments regarding factors that will impact construction and performance of the proposed construction.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

As directed by the client, PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Location and Description

The expansion for the existing Toyota dealership facilities will be constructed at the existing auto and RV trailer parking lot and the adjacent vacant lot located southeast of the existing RV service building. The total area of the site is about 15 acres, and is bordered by the existing dealership to the northwest, NE 53rd Street to the north, a commercial building to the east, and SR-500 to the south. The approximate location of the site can be seen in Figure 1, "Site Location Plan" of this report.

Currently, the west portion of the site is covered with crushed gravels and is used as parking spaces for the dealership, the east portion is undeveloped and generally grass covered with some concrete rubbles exposed at the ground surface.

3.2 Site Geology

According to the Clark County Soil Survey (USDA, 1972), the subject property is mapped within the Lauren gravelly loam (LgB) and Wind River gravelly loam (WrB). The Lauren gravelly loam, found on terraces of 0 to 8 percent slopes, consists of friable, dark-brown very gravelly loam at the surface to very gravelly loamy coarse sand at depth. The Wind River gravelly loam is much sandier, and is composed of friable, dark red-brown coarse sand loam at the surface to coarse sand at depth. Surface runoff in this unit is slow as well, and hazard to erosion is slight.

The property is located in the Portland-Vancouver basin, a low-lying area affected by the periglacial deposits from glacial outburst floods of Glacial Lake Missoula during the upper Pleistocene. The geologic unit mapped in the project area, according to the Geologic Map of the Vancouver Quadrangle, Washington and Oregon (Washington State Department of Natural Resources, OFR 87-10, 1987), is gravel sized flood deposits (Qg). The Washington State Department of Natural Resources describes the gravel sized deposits as well-rounded, well-sorted and stratified pebble and cobble gravel with angular to subangular boulders. The gravels are supported with a sandy matrix composed of mafic volcanic fragments. Thicknesses of the deposits vary from 9 meters to more than 75 meters in the center of the valley, and thin toward valley margins.

3.3 Subsurface Materials

The site subsurface conditions were explored with 4 soil test borings for the proposed building area and 10 test pits for the general site development. Our field exploration

depths ranged from 5 to 15 feet below the ground surface. The boring and test pit locations were located in the field by surveyors. The borings were advanced utilizing hollow-stem auger. During our drilling processes, soil samples were obtained at frequent intervals of depth through the Standard Penetration Test (SPT) method, as specified in ASTM D1586, using an automatic hammer. During our drilling processes soil samples were routinely obtained. Drilling and sampling techniques were accomplished generally in accordance with ASTM procedures. The 10 test pits conducted on the site were excavated using a backhoe, and disturbed bulk samples were taken during the excavation for further laboratory analyses. The boring and test pit exploration records are presented in the Appendix B of this report, and their locations are plotted in Figure 2 – Site exploration Plan.

Select soil samples were tested in the laboratory to determine materials properties for our evaluation. The laboratory testing program consisted of visual and textural examinations (ASTM D2487), moisture content tests (ASTM D2216), and particle size analyses (ASTM D1140). Test results are shown in the individual subsurface exploration record in Appendix B and are presented in Appendix C of this report.

Based on their physical characteristics and engineering properties, the soils encountered in our borings and test pits can be generally divided into three strata – undocumented fill materials, buried peat and topsoil deposits, and native gravelly deposits.

Fill: Below 3 to 7 inches of surficial topsoil at the east portion and approximately 2 inches of crushed gravels at the west portion of the site, undocumented fill materials mainly consisting of sandy silt to silty sand with gravels were encountered. At some locations, cobble sized particles and concrete/asphalt debris are presented in the fill materials. The fill materials were found to extend to depths ranging from 4 to 7 feet below the existing ground surface. Based on the SPT N-values recorded in these fill deposits which ranging from 5 to 26 blows/foot, the fill materials are not considered properly compacted when they were placed.

Buried Peat and Topsoil: Underlying the fill materials, about 6 to 12 inches of highly humified peat deposit was encountered mainly in the east portion of the site (TP-1, TP2, TP4 and TP6). In TP-8, located in the existing gravel parking area at the west portion of the site, up to 20 inches of peat deposit was found below the fill materials. Within most of the remaining test pits and borings, buried topsoil deposit was observed during our explorations. The present of the buried peat and topsoil deposits at the site may indicate the original ground surface level before fill was placed.

Native Gravelly Deposits: Below the fill materials and the buried peat/topsoil layer, silt with gravel deposit was encountered to approximately 8 to 12 feet in depth, overlying sandy gravel deposit extending to at least 15 feet below the existing ground surface. The

Standard Penetration Tests in these gravelly deposits yield N-values generally ranging from 10 to 45 blows/foot, indicating a medium dense to dense relative density.

3.4 Groundwater Information

During our field exploration processes, groundwater seepage was observed at 7 to 8 feet in depth at some of the test pits locations. Local well logs within half mile of the property indicate a static water level at approximately 10 feet below the ground surface. Copies of these groundwater logs consulted have been included in Appendix D of this report. In addition, discontinuous zones of perched water may exist within the fill materials as evidenced by some wet soils above the more impervious buried peat/topsoil deposits.

Fluctuations of groundwater levels should be anticipated with changing climatic conditions and should be expected to be at a higher elevation after a prolonged period of precipitation.

3.5 Seismic Considerations

In accordance with Table 1615.1.1 of the 2003 International Building Code (IBC), we recommend a Site Class D (stiff Soil Profile) for this site. According to the 1996 United States Geological Survey (USGS) Earthquake Hazards website http://eqint.cr.usgs.gov/eq/html/lookup-2002-interp.html, the Peak Ground Acceleration (PGA) is 0.38g, and the maximum considered earthquake (MCE) ground motions for the site are S_S=1.00g and S_1=0.32g (for Site Class B and 5 percent critical damping). The USGS website values are a more accurate interpolation of the values presented in Figure 1615(1) and Figure 1615(2) of the IBC. In accordance with Tables 1615(1) and 1615(2), Site Coefficients F_a and F_v are 1.10 and 1.76, respectively for a Site Class D. Therefore the adjusted MCE ground motions are S_{MS}=1.10g and S_{M1}=0.56g (for Site Class D). The return interval for these ground motions is 2 percent probability of exceedance in 50 years.

3.6 Liquefaction Analysis

Liquefaction involves the substantial loss of shear strength in saturated soil, usually taking place within a soil medium exhibiting a uniform fine-grained characteristic such as sand or silty sand, loose consistency, and low confining pressure when subjected to impact by seismic or dynamic loading. Based on our geotechnical evaluation including area seismicity, on-site soil conditions, SPT N-values, laboratory test results, and depth to groundwater, the site is considered to have low risk potential for soil liquefaction. We determined the risk potential is low primarily because of the medium dense to dense soil conditions.

4.0 INFILTRATION TESTING

4.1 Test Specification & Procedure

Falling head infiltration tests were conducted in general accordance with the EPA Falling Head Percolation Test Procedure found in the EPA Design Manual of Onsite Wastewater Treatment and Disposal Systems (October, 1980). The infiltration tests were conducted using six-inch diameter PVC pipes that were installed and seated approximately 2 to 3 inches into the underlying undisturbed soils with downward pressure from our track mounted excavator. Approximately 2 to 3 inches of clean rock was placed in the bottom of the standpipe at each test location to protect bottom of boring from scouring and sediment during the introduction water. Before the test, each location was presoaked as per the EPA specification. Infiltration tests were conducted inside the stand pipe beginning with a 6-inch head. The reduction of the water level (infiltration) into the soil was recorded over multiple test runs until repeating values were obtained. Samples from near the base of each infiltration elevation were brought back from the field for laboratory gradation analysis (ASTM C117-04/C136-06).

4.2 Test Results

Infiltration rates were determined using the last two successive readings, or in cases where successive readings could not be obtained, the final water level drop was used. Results are presented in Table 4.1.

Table 4.1: Infiltration Test Data

Test ID	Infiltration Rate (in/hr)	Depth of Test (in)	Percent Passing No. 200 sieve	Soil Type
TP-2	7	72	34	Silt & Gravel
TP-4	1	54	22	Fill: Silty Sand
TP-5	3 ½	36	33	Fill: Sandy Silt to Silty Sand
TP-6	120	38	36	Fill: Silty Sand
TP-7	360	45	28	Fill: Sandy Silt to Silty Sand
TP-8	240	88	43	Silt with Gravel & Sand
TP-9	3 ½	68	48	Silt with Gravel & Sand
TP-10	42 ¼	60	35	Silt & Gravel

Our test results do not include a factor of safety. Care should be taken in the design of the infiltration system because of the possible presence of an impermeable layer below the depths of our exploration.

5.0 EVALUATION AND FOUNDATION RECOMMENDATIONS

5.1 Geotechnical Discussion

Based on the results of our fieldwork, laboratory evaluation, and engineering analyses, it is our opinion that the site is suitable for the proposed developments provided the following recommendations are incorporated into the design and construction of the project. The primary geotechnical factors influencing the design and construction of the proposed project are the presence of the undocumented variable fill and the underlying highly organic soils (i.e. peat) on site.

During our field explorations, fill materials generally consisted of a heterogeneous mixture of silty sand/sandy silt and gravels were encountered in our borings. The fill was found to extend to depths ranging from 6 to 7 feet below the existing ground surface. At some locations large pieces of concrete and asphalt debris were found in the fill materials. Although no obvious voids were observed inside the fill and the debris seems to be relatively well incorporated into the silty sand to sandy silt matrix at the exploration locations, variation in composition and compactness should always be expected within these undocumented fill deposits. Considering these and presence of underlying peat and topsoil layers which are highly compressible, the on-site fill deposits are not considered suitable as foundation bearing strata for the proposed building.

5.2 Site Preparation

For infiltration must meet Puget Sound Manual page III-3-4

Due to the presence of relatively thick variable fill materials at the site, careful observations and inspections should be made during the sub-excavation and proof rolling stages of the project to identify any soft, loose, or organic rich soils. If encountered, these deleterious soils should be over-excavated, replaced and re-compacted in accordance with our recommendations outlined in the following sections. We recommend that all topsoil, vegetation, roots, fill materials, as well as any soft/loose soils in the construction areas be stripped from the site. Our field investigation revealed the presence of about 3 to 7 inches of topsoil at east portion of the site. Utility trench excavations must be backfilled with properly compacted structural fill as outlined in Section 4.3 of this report.

After stripping and excavating to the proposed subgrade level, as required, the building and pavement areas should be proof-rolled with a heavily loaded tandem axle dump truck or similar rubber tired vehicle. Soils that are observed to rut or deflect excessively under the moving load, or are otherwise judged to be unsuitable should be undercut and replaced with properly compacted fill. Due to the presence of some large size concrete debris and weak zones within the fill materials, at some locations undercutting of these unsuitable

materials of up to 2 to 3 feet and replaced with properly backfilled structure fill should be expected. The proof-rolling and undercutting activities should be witnessed by a representative of the geotechnical engineer.

If desired, bulk samples of the site soils may be obtained by PSI for modified Proctor tests to help define the optimum moisture content of the on site soils. Based on those results more definitive statements can be made regarding the necessity to undercut and recompact the loose subgrade as well as the level of effort which will likely be required to adjust the moisture content of the in-situ soils which will be cut and used for fill. Past experience indicates that these earthwork operations may be time consuming and have the potential to add considerable cost to the earthwork portion of the project.

The silty sand to sandy silt soils encountered at this site are expected to be sensitive to disturbances caused by construction traffic and to changes in moisture content. During wet weather periods, increases in the moisture content of the soils can cause significant reduction in the soil strength and support capabilities. In addition, soils which become wet may be slow to dry and thus significantly retard the progress of grading and compaction activities. It will, therefore, be advantageous to perform earthwork and foundation construction activities during dry weather.

Proofrolling of excavation bottoms is likely not appropriate during wet weather grading. Should construction take place during wet weather, we recommend that a representative of the geotechnical engineer be present to observe the subgrade in order to evaluate whether additional preparation is indicated.

In addition, it is not uncommon for construction equipment to severely disturb the upper 1 to 2 feet of the subgrade during initial phases of site clearing especially if site preparation work is performed while the soils are wet. This may result in the need for deep undercutting and replacement of the disturbed soils. The owner may want to consider an allowance in the budget to cover this condition.

5.3 Fill Requirements

After subgrade preparation and observation have been completed, fill placement may begin. The first layer of fill material should be placed in a relatively uniform horizontal lift on the prepared subgrade. Fill materials should be free of organic or other deleterious material, have a maximum particle size less than 3 inches, be relatively well graded, and have a liquid limit less than 40 and plasticity index less than 25. The on site soils are generally considered suitable for use as structural fill, except for the peat and topsoils. However, the moisture content will most likely have to be adjusted to coincide with the moisture range required for structural fill. Structural fill should be compacted to at least 95

percent of modified Proctor maximum dry density as determined by ASTM Designation D 1557.

Fill should be placed in maximum lifts of 8 inches of loose material and should be compacted within 2 percentage points of the material's optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted engineered fill should be tested by a representative of the geotechnical engineer prior to placement of subsequent lifts. The fill should extend horizontally outward beyond the exterior perimeter of the building and footings a distance equal to the height of the fill or 5 feet, which ever is greater, prior to sloping. Also, fill should extend horizontally outward from the exterior perimeter of the pavement a distance equal to the height of the fill or 3 feet, whichever is greater, prior to sloping. All permanent fill slopes should be constructed at 2 horizontal (H) to 1 vertical (V) or flatter and should be adequately compacted. The surfaces of the slopes should be properly protected from erosion by seeding, sodding, rocking, or other acceptable means.

Fill material, if needed, during wet weather construction should consist of an all-weather, clean, granular fill containing less than 5 percent material passing the No. 200 sieve, such as course sand, crushed rock, or course sand and gravel. During wet weather grading operations, all excavations should be performed using a smooth-bladed, tracked backhoe working from areas where material has yet to be removed or from the already placed structural fill. Subgrade areas should be cleanly cut to firm undisturbed soil.

Placement of crushed rock should follow immediately after site grading in order to provide protection of the subgrade soil during construction activities. In temporary construction traffic areas, the placement of a one-foot thick granular working base is generally recommended with thicker sections (i.e. 18 to 24 inches) and/or geotextile fabrics recommended in heavily traveled construction traffic areas. Generally, three to six inches of crushed rock is sufficient in foot traffic areas.

5.4 Foundation Recommendations

Based on the subsurface condition, conventional footing foundations can be used to support the proposed structural loads. However, the existing undocumented fill, underlying peat or topsoil, and any surficially softened native soils are not considered suitable as foundation bearing strata, thus they should be overexcavated to unyielding native soils encountered at a depth of 6 to 8 feet below the existing grade. The footings can be founded directly on natural medium dense to dense silty gravelly deposit using an allowable bearing pressure of 3,500 psf.

Alternatively, a structural compacted fill (e.g. crushed rock/gravel) or a low density concrete

fill (Controlled Density Fill-CDF) can be used to back fill the orverexcavated footing area to the designed subgrade levels on which the concrete footings can then be placed. In these cases, an allowable bearing capacity of 3,000 psf can be used for these improved subgrade conditions. The dimensions of footing excavation, measured at the bottom of the excavation, should be extended at least 6 beyond the designed footing perimeters for the cases of CDF or 18 inches beyond footing perimeters for crushed rock/gravel fill, respectively.

The allowable bearing pressure includes a safety factor of 3 and is intended for dead loads and sustained live loads and can be increased by one-third for the total of all loads, including short-term wind or seismic loads. Minimum dimensions of 30 inches for square footings and 18 inches for continuous footings should be used in the foundation design process to minimize the possibility of a local bearing capacity failure. All footings should be underlain by at least 6 inches of clean, compacted crushed rock to provide protection for the subgrade soil during construction activities. Allowable lateral frictional resistance between the base of shallow foundations and the subgrade can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30. In addition, lateral loads may be resisted by a passive earth pressure based on an equivalent fluid density of 250 pounds per cubic foot (pcf) on footings poured "neat" against in-situ soils or properly backfilled with structural fill. The passive earth pressure recommendation includes a factor of safety of approximately 1.5, which is appropriate due to the amount of movement required to develop full passive resistance.

Exterior footings and foundations in unheated areas should be located at a depth of at least 18 inches below the final exterior grade to provide adequate frost protection. If the building is to be constructed during the winter months or if the foundation soils will likely be subjected to freezing temperatures after foundation construction, then the foundation soils should be adequately protected from freezing. Otherwise, interior foundations can be located at nominal depths compatible with architectural and structural considerations. Where it is necessary to place footings at different levels, the upper footing must be founded below an imaginary 10 horizontal to 7 vertical line drawn up from the base of the lower footing.

Based on the known subsurface conditions and site geology, laboratory testing and past experience, we anticipate that properly designed and constructed foundations supported on the recommended materials should experience maximum total and differential settlements between adjacent columns on the order of one inch and 1/2 inches, respectively. The foundation excavations should be observed by a representative of PSI prior to steel/concrete placement or the structural fill construction to assess that the foundation materials are capable of supporting the design loads and are consistent with the materials discussed in this report. Unsuitable soil zones encountered at the bottom of the

foundation excavations should be removed to the level of medium dense or very stiff native soils or properly compacted structural fill as directed by the geotechnical engineer. Cavities formed as a result of excavation of unsuitable soil zones should be backfilled with lean concrete or compacted structural fill.

The structured fill in the footing areas should be placed, compacted and tested in accordance with the guidelines presented in this report and the recommendations of the geotechnical engineer.
After the completion of the structural fill, the footing concrete should be placed as quickly as possible to avoid exposure of the structural fill to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond.

Care should be taken to protect prepared bearing surfaces until footing concrete can be placed. Precautions to achieve this end would consist of either:

- covering of prepared bearing surfaces with impervious membranes .
- placing a clean granular crushed aggregate blanket (2 to 4 inch thickness) over the surface.
- cessation of work during rainy weather.

Be advised that as a part of the foundation selection process, there is always a cost/benefit evaluation. Although we are recommending a specific foundation type we have not accomplished the cost/benefit evaluation.

5.5 Floor Slab Recommendations

The proposed slabs-on-grade may be supported on properly compacted structural fill or placed on the re-compacted on-site subgrade after the removal of vegetation and other deleterious materials, and after the upper soils have been proofrolled with a fully loaded tandem axle dump truck or similar rubber tired vehicle. Any soft or otherwise unsuitable areas observed during proofrolling should be over-excavated down to firm subgrade and replaced with structural fill.

Based on the existing soil conditions, the design of slabs-on-grade can be based on a subgrade modulus (k) of 100 pci; however, this value may be increased to 150 pci if a minimum 6-inch thick granular mat is placed below the floor slab as recommended below. These subgrade modulus values represent anticipated values which would be obtained in a standard in-situ plate test with a 1-foot square plate. Use of these subgrades moduli for design or other on-grade structural elements should include appropriate modification based on dimensions as necessary.

The 6 inch granular mat should consist of well-graded 1½-inch or ¾-inch-minus imported crushed rock aggregates having less than 5 percent material passing the No. 200 sieve. The crushed rock should provide a capillary break to limit migration of moisture through the slab. If additional protection against moisture vapor is desired, a vapor retarding membrane may also be incorporated into the design. Factors such as cost, special considerations for construction, and the floor coverings suggest that decisions on the use of vapor retarding membranes be made by the architect and owner.

6.0 PAVEMENT RECOMMENDATIONS

Our scope of services did not include extensive sampling and CBR testing for existing subgrade or potential sources of imported fill for the specific purpose of detailed pavement analysis. Instead, we have assumed pavement-related design parameters that are considered to be typical for the area soils types. In large areas of pavement, or where pavements are subject to significant traffic, a more detailed analysis of the subgrade and traffic conditions should be made. The results of such a study will provide information necessary to design an economical and serviceable pavement.

The thickness recommendations presented below are considered typical and minimum for the assumed parameters. We understand that budgetary considerations sometimes warrant thinner pavement sections than those presented. However, the client, the owner, and the project principals should be aware that thinner pavement sections might result in increased maintenance costs and lower than anticipated pavement life.

- **Asphalt Pavement**

The pavement subgrade should be prepared as discussed in the site preparation section of this report. We have estimated the subgrade soils will be prepared to a CBR of at least 3. Making this assumption, it is possible to use a locally typical "standard" pavement section consisting of the following:

Table 4 – Pavement Recommendations

	Thickness Recommendations (inches)	
Pavement Materials	Car Parking	Drive Lanes/Truck Routes
Asphalt Surface Course	3	4
Crushed Stone Base	8	12

Asphalt pavement base course material should consist of a well-graded, 1½-inch or ¾-inch-minus, crushed rock, having less than 5 percent material passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirements set forth in the latest Washington Department of Transportation guidelines. Base course material should be moisture conditioned to within 2 percent of optimum moisture content and compacted by mechanical means to a minimum of 95 percent of the material's maximum dry density as determined in accordance with ASTM D 1557 (Modified Proctor). Fill materials should be placed in layers that, when compacted, do not exceed about 8 inches. The asphaltic concrete material should be compacted to at least 92 percent of the material's theoretical maximum density as determined in accordance ASTM D 2041 (Rice Specific Gravity).

- **Concrete Pavement**

Rigid concrete pavement consisting of 7 inches of concrete underlain by 4 inches of granular sub-base is recommended where trash dumpsters are to be parked on the pavement or where a considerable load is transferred from relatively small steel wheels. This should provide better distribution of surface loads to the subgrade without causing deformation of the surface. Pavement may be placed after the subgrade has been properly compacted, fine-graded and proof-rolled. The work should be done in accordance with Washington Department of Transportation guidelines.

Water should not be allowed to pond behind curbs and saturate the base materials. If the base material consists of granular fill, it should extend through the slope to allow any water entering the base stone a path to exit. The project Geotechnical engineer or civil engineer should accomplish a site specific pavement design when actual traffic and loading information is available.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Excavation

Temporary earth slopes may be cut near vertical to a height of 4 feet, above which flatter slopes will be required in accordance with OSHA. Permanent earth slopes should be dressed to 2H:1V or flatter and protected from erosion. Due to the absence of groundwater within the upper portion of the soil profile, we do not anticipate the need for dewatering during construction.

Excavation and construction operations may expose the on-site soils to inclement weather conditions. The stability of exposed soils may rapidly deteriorate due to a change in moisture content (i.e. wetting or drying) or the action of heavy or repeated construction traffic. Accordingly, foundation and pavement area excavations should be adequately protected from the elements, and from the action of repetitive or heavy construction loadings.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

7.2 Construction Dewatering

Relatively shallow groundwater was encountered during our investigation. We anticipate groundwater could be as shallow as about 7 feet below existing grade. If excavations will extend below the groundwater level, pumping from perimeter ditches or well-points would likely control the expected inflows. Once excavation depths are known, we should be retained to review and update our groundwater control recommendations.

7.3 Drainage Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for floor slabs and pavements during construction. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.

The site grading plan should be developed to provide rapid drainage of surface water away from the building and pavement areas and to inhibit infiltration of surface water around the perimeter of the building and beneath the floor slabs and pavements. The grades should be sloped away from the building and pavement areas. Careful consideration should be given to the potential impact of landscaped areas and/or sprinkler systems on adjacent foundations, floor slabs, and pavements. Roof runoff should be piped to a storm sewer or approved disposal area.

7.4 Construction Monitoring

It is recommended that PSI be retained to examine and identify soil exposures created during project excavations in order to verify that soil conditions are as anticipated. We further recommend that the structural fills be continuously observed and tested by our representative in order to evaluate the thoroughness and uniformity of their compaction. Samples of fill materials should be submitted to our laboratory for evaluation prior to placement of fills on site.

It is also recommended that PSI be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. PSI cannot accept any responsibility for any conditions which deviate from those described in this report, nor for the performance of the foundation, if not engaged to also provide construction observation and testing for this project.

8.0 REPORT LIMITATIONS

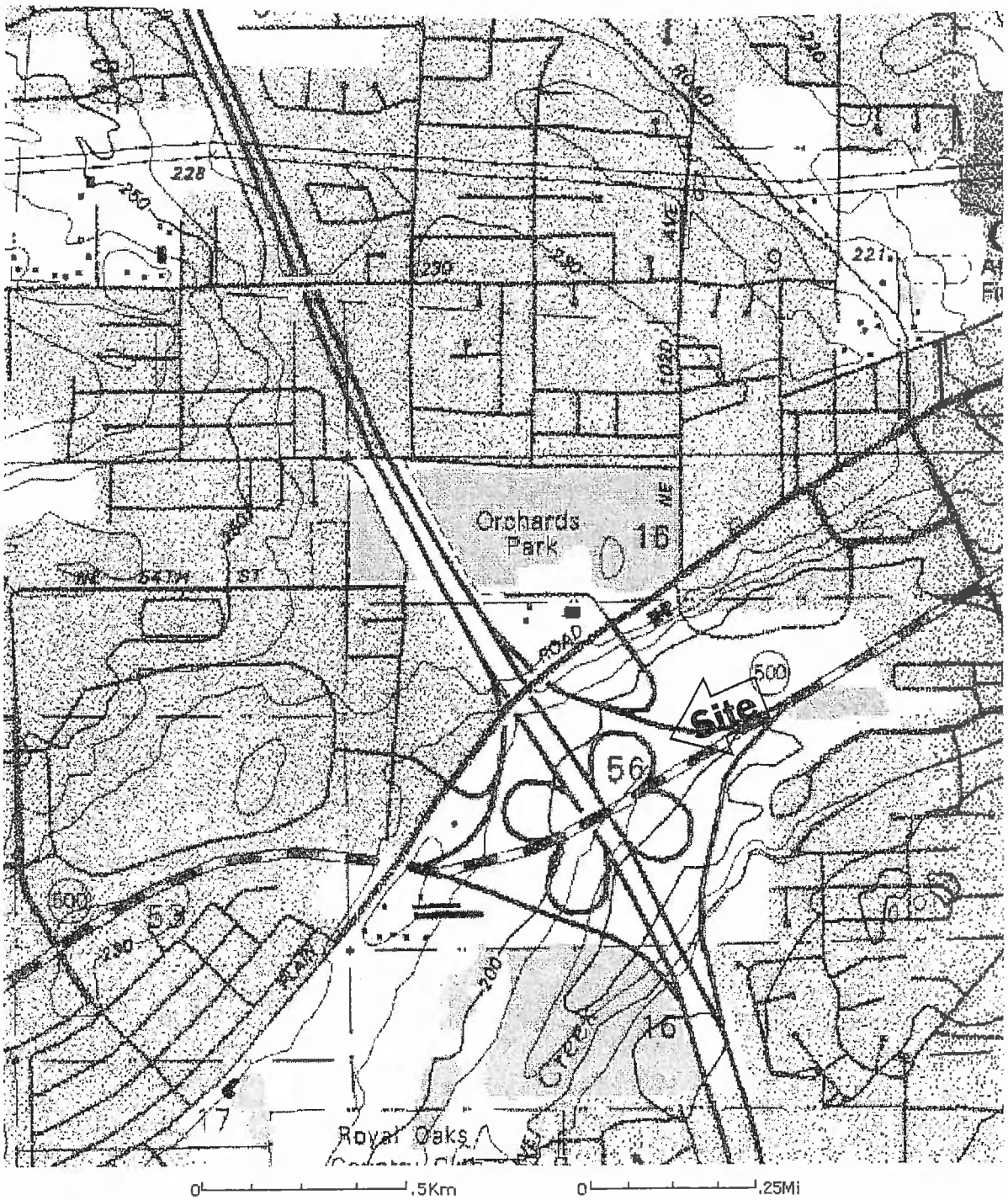
The recommendations submitted in this report are based on the available subsurface information obtained by PSI and design details furnished by our client for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation and/or pavement recommendations are required. If PSI is not retained to review these changes, PSI will not be responsible for the impact of those conditions on the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At this time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of Building Structures, Inc. for the specific application to the proposed Toyota dealership expansion located at 10009 NE Fourth Plain Boulevard in Vancouver, Washington.

FIGURES

FIGURE 1: SITE LOCATION PLAN



Source: www.TerraServer-USA.com

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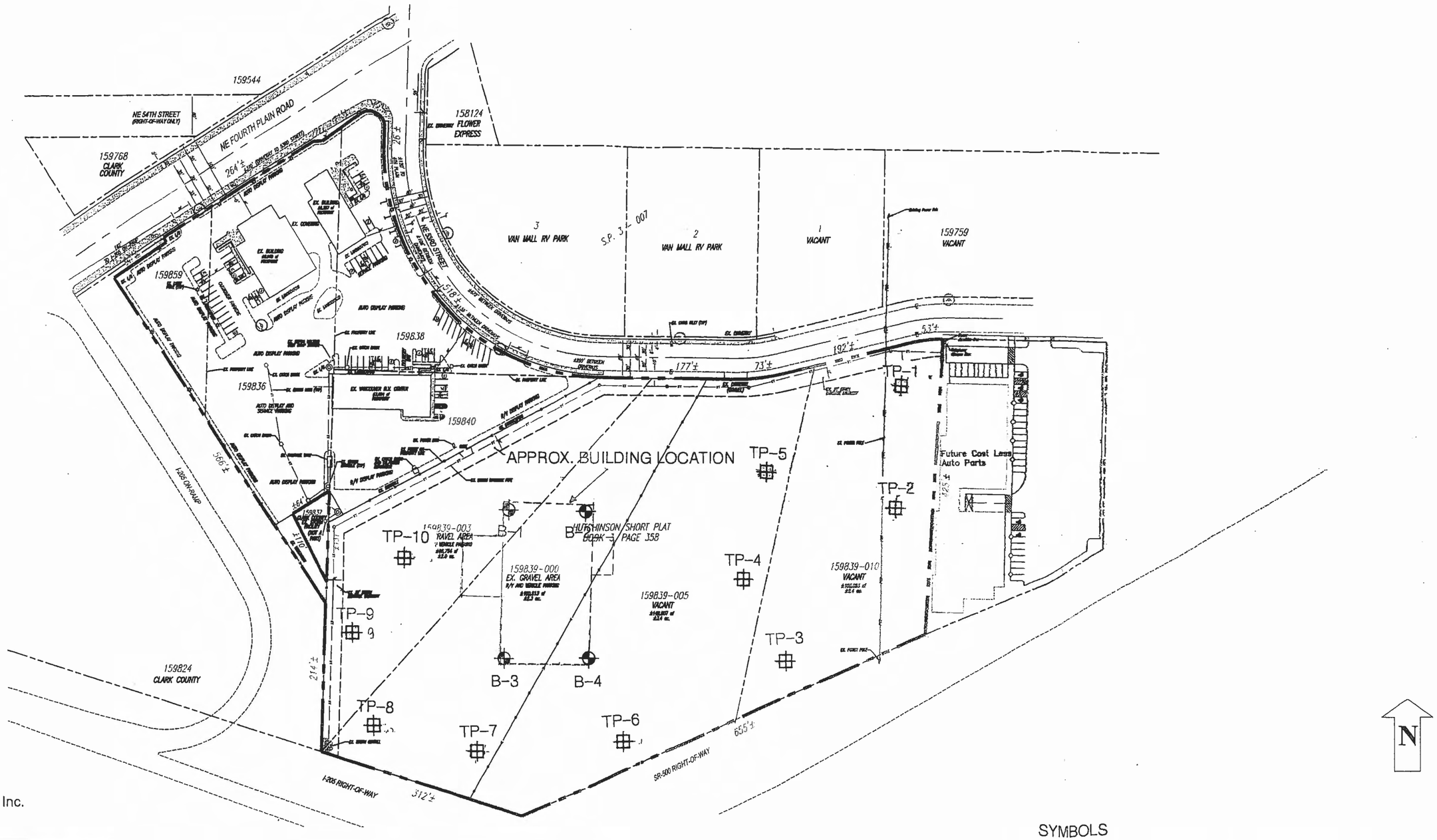
Proposed Vancouver Toyota
1009 NE Fourth Plain Road
Vancouver, Washington

File No.
704-75065

Date:
4/3/07

FIGURE 2: BORING AND TEST PIT LOCATION PLAN



Source: Mackay and Sposito, Inc.

Scale: N.T.S.

SYMBOLS

- APPROXIMATE BORING LOCATION
- APPROXIMATE TEST PIT LOCATION

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(D-A)

APPENDIX A

General Notes & Soil Classification Chart

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System is used to identify the soil unless otherwise noted.

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. split-spoon.

Qu: Unconfined Compressive Strength, TSF.

Qp: Penetrometer value, unconfined compressive strength, TSF.

Mc: Water Content, %.

LL: Liquid Limit, %.

PI: Plasticity Index, %.

δd: Natural Dry Density, PCF.

▼ Apparent Groundwater Level at time noted after completion of boring.

DRILLING AND SAMPLING SYMBOLS

SS: Split-Spoon – 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube – 3" O.D., except where noted.
AU: Auger Sample.
DB: Diamond Bit.
CB: Carbide Bit.
WS: Washed Sample.

TERM (NON-COHESIVE SOILS)	STANDARD PENETRATION RESISTANCE (SAFETY HAMMER)	STANDARD PENETRATION RESISTANCE (AUTOMATIC HAMMER)
Very Loose	0-4	0-3
Loose	4-10	3-7
Medium	10-30	7-20
Dense	30-50	20-33
Very Dense	Over 50	Over 33

TERM (COHESIVE SOILS)	Qu – (TSF)
Very Soft	0-0.25
Soft	0.25-0.50
Firm (Medium)	0.50-1.00
Stiff	1.00-2.00
Very Stiff	2.00-4.00
Hard	4.00+

PARTICLE SIZE

Boulders	8 in.+	Coarse Sand	5mm-0.6mm	Silt	0.074mm-0.005mm
Cobbles	8 in.-3 in.	Medium Sand	0.6mm-0.2mm	Clay	-0.005mm
Gravel	3 in.-5mm	Fine Sand	0.2mm-0.074mm		

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

psi

(D-B)

APPENDIX B

Record of Subsurface Exploration

LOG OF TEST BORING NO. B-1

CLIENT: **Building Structures, Inc.**

PROJECT: **Proposed Vancouver Toyota**

LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA

PSI PROJECT NUMBER: 704-75065

DATE OF EXPLORATION: 3/19/2007

EQUIPMENT: CME-75 Hollow Stem Auger w/Auto SPT Hammer

LOGGED BY: T. French

BORING LOCATION: See Boring and Test Pit Location Plan

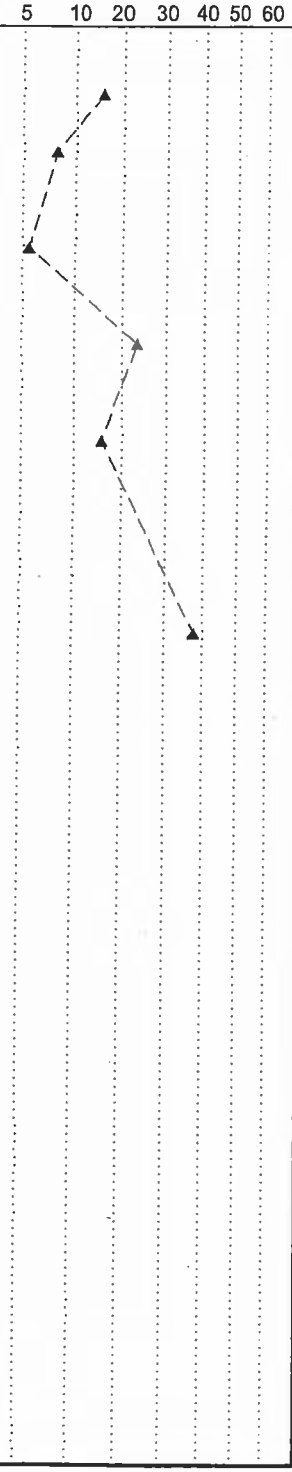
SURF. ELEV.:' GROUNDWATER: ' TERMINATION DEPTH: 16.5'

The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

DEPTH (FT)	SAMPLES	SYMBOL	U.S.C.S. CLASS	SOIL DESCRIPTION *Stratigraphic lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described below. Specific groundwater depths should be expected to vary season to season. Please refer to the report text for further explanation of soils encountered and exploration methods employed.*	MOISTURE CONTENT(%)	% PASSING #200 SIEVE	DCP (BLOWS/FOOT)	BLOWS/6"	POCKET PEN (TSF)	TORVANE SHEAR (TSF)	LIQUID LIMIT	PLASTIC LIMIT
				CRUSHED GRAVEL-2 inches thick								
	SPT 1			FILL-sandy silt, trace gravel, organic, and peat pockets, dark brown to brown, moist, medium stiff Some asphalt pieces between 2.5 and 4 feet				4-6-9	3.0			
	SPT 2							3-3-5	3.0			
5	SPT 3							2-2-3				
				OLD TOPSOIL-black/brown								
	SPT 4			SILT W/WEATHERED GRAVELS AND TRACE SAND-brown, moist to wet, medium dense				6-10-12	4.5			
10	SPT 5							4-7-8	2.5			
				SANDY GRAVEL-brown, moist, dense								
15	SPT 6							12-16-20				
				Soil boring terminated at 16.5 feet below ground surface. Groundwater was not encountered during site exploration.								
20												
25												
30												
35												

PENETRATION RESISTANCE (blows/foot)

140 pound hammer/30 inch drop



BL PDX DCP 704-75065.GPJ CURRENT PORTLAND GEOTECH TEMPLATE.GDT 4/3/07

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LOG OF TEST BORING NO. B-2

CLIENT: **Building Structures, Inc.**

PROJECT: **Proposed Vancouver Toyota**

LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA

PSI PROJECT NUMBER: 704-75065

DATE OF EXPLORATION: 3/19/2007

EQUIPMENT: CME-75 Hollow Stem Auger w/Auto SPT Hammer

LOGGED BY: T. French

BORING LOCATION: See Boring and Test Pit Location Plan

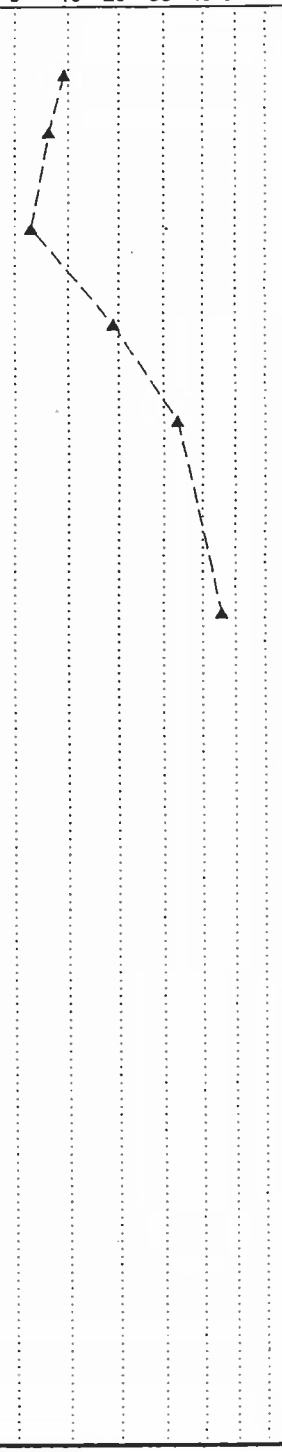
SURF. ELEV.:' GROUNDWATER:' TERMINATION DEPTH: 16.5'

The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

DEPTH (FT)	SAMPLES	SYMBOL	U.S.C.S. CLASS	SOIL DESCRIPTION	MOISTURE CONTENT(%)	% PASSING #200 SIEVE	DCP (BLOWS/FOOT)	BLOWS/6"	POCKET PEN (TSF)	TORVANE SHEAR,(TSF)	LIQUID LIMIT	PLASTIC LIMIT
				CRUSHED GRAVEL-2 inches thick								
	SPT 1			FILL-sandy silt/silty sand, some gravel, trace asphalt pieces, brown, moist, medium stiff				3-6-4				
	SPT 2							3-4-4	2.5			
5	SPT 3			OLD TOPSOIL-black/brown				2-3-3	1.5			
				SILT W/WEATHERED GRAVELS AND TRACE SAND-brown, moist to wet, medium dense								
	SPT 4							4-5-13	3.5			
10	SPT 5				▽			3-11-21				
				SANDY GRAVEL-some silt, brown, moist, dense to very dense								
15	SPT 6							31-21-23				
				Soil boring terminated at 16.5 feet below ground surface.								
20				Groundwater was encountered at 10 feet below existing site grade during site exploration.								
25												
30												
35												

PENETRATION RESISTANCE (blows/foot)

140 pound hammer/30 inch drop



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LOG OF TEST BORING NO. B-3

CLIENT: **Building Structures, Inc.**

PROJECT: **Proposed Vancouver Toyota**

LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA

PSI PROJECT NUMBER: 704-75065

DATE OF EXPLORATION: 3/19/2007

EQUIPMENT: CME-75 Hollow Stem Auger w/Auto SPT Hammer

LOGGED BY: T. French

BORING LOCATION: See Boring and Test Pit Location Plan

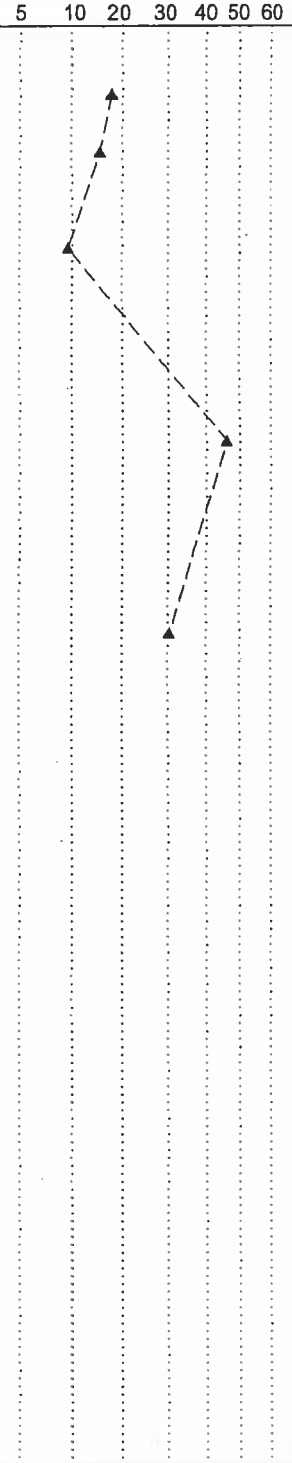
SURF. ELEV.:' GROUNDWATER: 10' TERMINATION DEPTH: 16.5'

The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

DEPTH (FT)	SAMPLES	SYMBOL	U.S.C.S. CLASS	SOIL DESCRIPTION *Stratigraphic lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described below. Specific groundwater depths should be expected to vary season to season. Please refer to the report text for further explanation of soils encountered and exploration methods employed.*	MOISTURE CONTENT(%)	% PASSING #200 SIEVE	DCP (BLOWS/FOOT)	BLOWS/6"	POCKET PEN (TSF)	TORVANE SHEAR (TSF)	LIQUID LIMIT	PLASTIC LIMIT
				CRUSHED GRAVEL-2 inches thick								
	SPT 1			**FILL**-sandy silt/silty sand, some gravel, trace organics, trace clay, brown to dark brown, moist to wet, medium stiff				7-9-8	4.5			
	SPT 2							3-8-7				
5	SPT 3							3-7-3				
				SILT W/WEATHERED GRAVELS AND TRACE SAND-brown, moist to wet, medium dense								
	SPT 4							19-35/4"				
10	SPT 5							4-25-19				
				SANDY GRAVEL-some silt, brown, moist, very dense to dense								
15	SPT 6							5-14-15				
				Soil boring terminated at 16.5 feet below ground surface.								
20				Groundwater was not encountered during site exploration.								
25												
30												
35												

PENETRATION RESISTANCE (blows/foot)

140 pound hammer/30 inch drop



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LOG OF TEST BORING NO. B-4

CLIENT: **Building Structures, Inc.**

PROJECT: **Proposed Vancouver Toyota**

LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA

PSI PROJECT NUMBER: 704-75065

DATE OF EXPLORATION: 3/19/2007

EQUIPMENT: CME-75 Hollow Stem Auger w/Auto SPT Hammer

LOGGED BY: T. French

BORING LOCATION: See Boring and Test Pit Location Plan

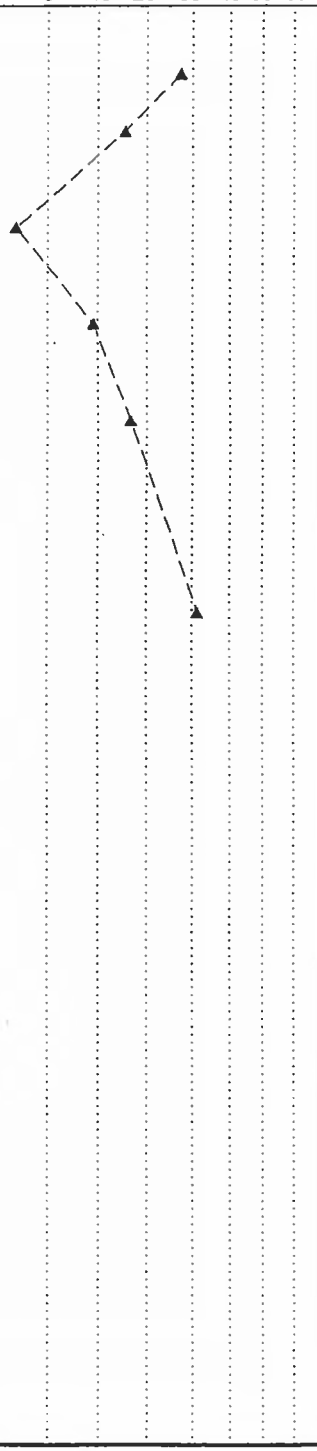
SURF. ELEV.:' GROUNDWATER:' TERMINATION DEPTH: 16.5'

The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

DEPTH (FT)	SAMPLES	SYMBOL	U.S.C.S. CLASS	SOIL DESCRIPTION	MOISTURE CONTENT(%)	% PASSING #200 SIEVE	DCP (BLOWS/FOOT)	BLOWS/6"	POCKET PEN (TSF)	TORVANE SHEAR (TSF)	LIQUID LIMIT	PLASTIC LIMIT
				TOPSOIL-3 inches thick								
	SPT 1			**FILL**-sandy silt/silty sand, some gravel, some asphalt pieces, brown to dark brown, moist, medium stiff				8-15-11	3.0			
	SPT 2							8-6-7				
5	SPT 3							3-1-1				
				SILT TO CLAYEY SILT W/WEATHERED GRAVELS AND TRACE SAND-brown, wet, soft								
	SPT 4			**SANDY GRAVEL**-some silt, brown, wet to moist, medium dense to dense				1-4-6				
10	SPT 5							7-7-9				
15	SPT 6							27-15-15				
				Soil boring terminated at 16.5 feet below ground surface.								
20				Groundwater was not encountered during site exploration.								
25												
30												
35												

PENETRATION RESISTANCE (blows/foot)

140 pound hammer/30 inch drop



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LOG OF TEST PIT NO. TP-1

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 6 inches thick								
1		**SILT W/GRAVELS**- some cobbles, brown, moist								
2										
3										
4		**PEAT**- organic material, 1 foot thick								
5		**SILT**- some gravels, trace sand, gray, moist								
6		**SANDY GRAVEL**- some silt, brown, moist to wet								
7										
8										
9		Test pit terminated at 7.5 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
10		Groundwater was encountered at a depth of 7 feet below existing site grade during site exploration.								
11		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
12										
13										
14										
15										
16										
17										
18										
19										
20										

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LOG OF TEST PIT NO. TP-2

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 6 inches thick								
1		**GRAVELLY SAND (FILL)**- some silt, brown to dark brown, moist								
2										
3										
4										
5		**PEAT**- organic material, 1 foot thick								
	GRAB 1				18				34	
6		**SILT AND GRAVEL**- trace clay, gray, moist								
7		Test pit terminated at 6 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
8										
9		Groundwater was not encountered during site exploration.								
10		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
11										
12										
13										
14										
15										
16										
17										
18										
19										
20										

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LOG OF TEST PIT NO. TP-3

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 3 inches thick								
1		**SILTY SAND/SANDY SILT (FILL)**-some gravel, occassional cobbles, brown, moist								
2										
3										
4										
5		**OLD TOPSOIL**- black/brown								
6		**SANDY GRAVEL**- some to trace silt, brown, wet to moist								
7										
8										
9		Test pit terminated at 7'4" below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
10		Groundwater was encountered at a depth of 7 feet below existing site grade during site exploration.								
11		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
12										
13										
14										
15										
16										
17										
18										
19										
20										

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LOG OF TEST PIT NO. TP-4

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 6 inches thick								
1		**SILTY SAND/SAND (FILL)**-some gravel, brown, moist to wet								
2										
3										
4										
5	GRAB 1				20				22	
6										
		PEAT- organic material, 6 inches thick								
7		**SILT W/GRAVEL**- trace sand, gray, moist to wet								
8										
9		Test pit terminated at 8 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
10										
11		Groundwater was encountered at a depth of 7.5 feet below existing site grade during site exploration.								
12		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
13										
14										
15										
16										
17										
18										
19										
20										

TP PTLD 704-75065.GPJ PSI CORP.GDT 4/3/07

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LOG OF TEST PIT NO. TP-5

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 6 inches thick								
1		**SILTY SAND/SANDY SILT (FILL)**-some gravel, brown, moist								
2										
3	GRAB 1				14				33	
4										
5										
6		**OLD TOPSOIL**- black/brown								
7		**SILT W/GRAVEL**- trace sand, brown, wet								
8										
9		Test pit terminated at 7.5' below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
10		Groundwater was encountered at a depth of 7.5 feet below existing site grade during site exploration.								
11										
12		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
13										
14										
15										
16										
17										
18										
19										
20										

TP_PTLD 704-75065.GPJ PSI CORP.GDT 4/3/07

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LOG OF TEST PIT NO. TP-6

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 7 inches thick								
1		**SILTY SAND** - some gravel, some asphalt and concrete debris, brown, moist								
2										
3	GRAB 1				33				36	
4		**PEAT**- organic material, 10 inches thick								
5		**SILT W/GRAVEL AND SAND**- gray, moist to wet								
6										
▼7										
8										
9		Test pit terminated at 7.5 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
10		Groundwater was encountered at a depth of 7 feet below existing site grade during site exploration.								
11		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
12										
13										
14										
15										
16										
17										
18										
19										
20										

TP_PTLD 704-75065.GPJ PSI_CORP.GDT 4/3/07

psi
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217-0126
(800) 783-6985

LOG OF TEST PIT NO. TP-7

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		TOPSOIL- 6 inches thick								
1		**SANDY SILT/SILTY SAND (FILL)**-some gravel, some asphalt debris, brown, moist								
2										
3										
4	GRAB 1				47				28	
5		**SANDY GRAVEL**- some silt, trace cobbles, brown, moist								
6										
7										
8										
9										
10		Test pit terminated at 8.5' below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
11		Groundwater was encountered at a depth of 8'4" feet below existing site grade during site exploration.								
12										
13		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
14										
15										
16										
17										
18										
19										
20										

TP_PTLD 704-75065.GPJ PSI_CORP.GDT 4/3/07

psi
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217-0126
(800) 783-6985

LOG OF TEST PIT NO. TP-8

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		CRUSHED GRAVEL- 2 inches thick								
1		**SILTY SAND (FILL)**- some gravels, some aphalt debris, brown to dark brown, moist								
2										
3										
4										
5										
6		**PEAT AND TOPSOIL**- 20 inches thick								
7										
	GRAB 1	**SILT W/GRAVEL AND SAND**- trace clay, brown, moist to wet			17				43	
8										
9		Test pit terminated at 8 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
10										
11		Groundwater was not encountered during site exploration.								
12		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
13										
14										
15										
16										
17										
18										
19										
20										

TP PTLD 704-75065.GPJ PSI CORP.GDT 4/3/07

6032 North Cutter Circle, Suite 480
Portland, Oregon 97217-0126
(800) 783-6985

LOG OF TEST PIT NO. TP-9

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		CRUSHED GRAVEL- 2 inches thick								
1		**SANDY SILT (FILL)**- some gravels and cobbles, dark brown								
2										
3										
4										
5	GRAB 1	**SILT W/GRAVEL AND SAND**- trace clay, gray, moist							48	
6										
7		Test pit terminated at 6 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
8										
9		Groundwater was not encountered during site exploration.								
10		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
11										
12										
13										
14										
15										
16										
17										
18										
19										
20										

TP PTLD 704-75065.GPJ PSI CORP.GDT 4/3/07

6032 North Cutter Circle, Suite 480
Portland, Oregon 97217-0126
(800) 783-6985

LOG OF TEST PIT NO. TP-10

CLIENT: Building Structures, Inc.
PROJECT: Proposed Vancouver Toyota
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA
PSI PROJECT NUMBER: 704-75065
SURF. ELEV.:

DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: Test Pit
LOGGED BY: Y. Lang
TEST PIT LOCATION: See Boring and Test Pit Location Plan

DEPTH, FT.	SAMPLES	SOIL DESCRIPTION	SYMBOL	U.S.C.S. CLASS	Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Passing 200 Sieve	POCKET PEN (tsf)
		CRUSHED GRAVEL- 2 inches thick								
1		**SILTY SAND (FILL)**- some gravels, brown to dark brown, moist								
2										
3										
4										
5		**SILT AND GRAVEL**- trace to some sand, gray, moist								
	GRAB 1				19				35	
6										
7		Test pit terminated at 5'3" below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
8		Groundwater was not encountered during site exploration.								
9		Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.								
10										
11										
12										
13										
14										
15										
16										
17										
18										
19										
20										

TP_PTLD 704-75065.GPJ PSI_CORP.GDT 4/3/07

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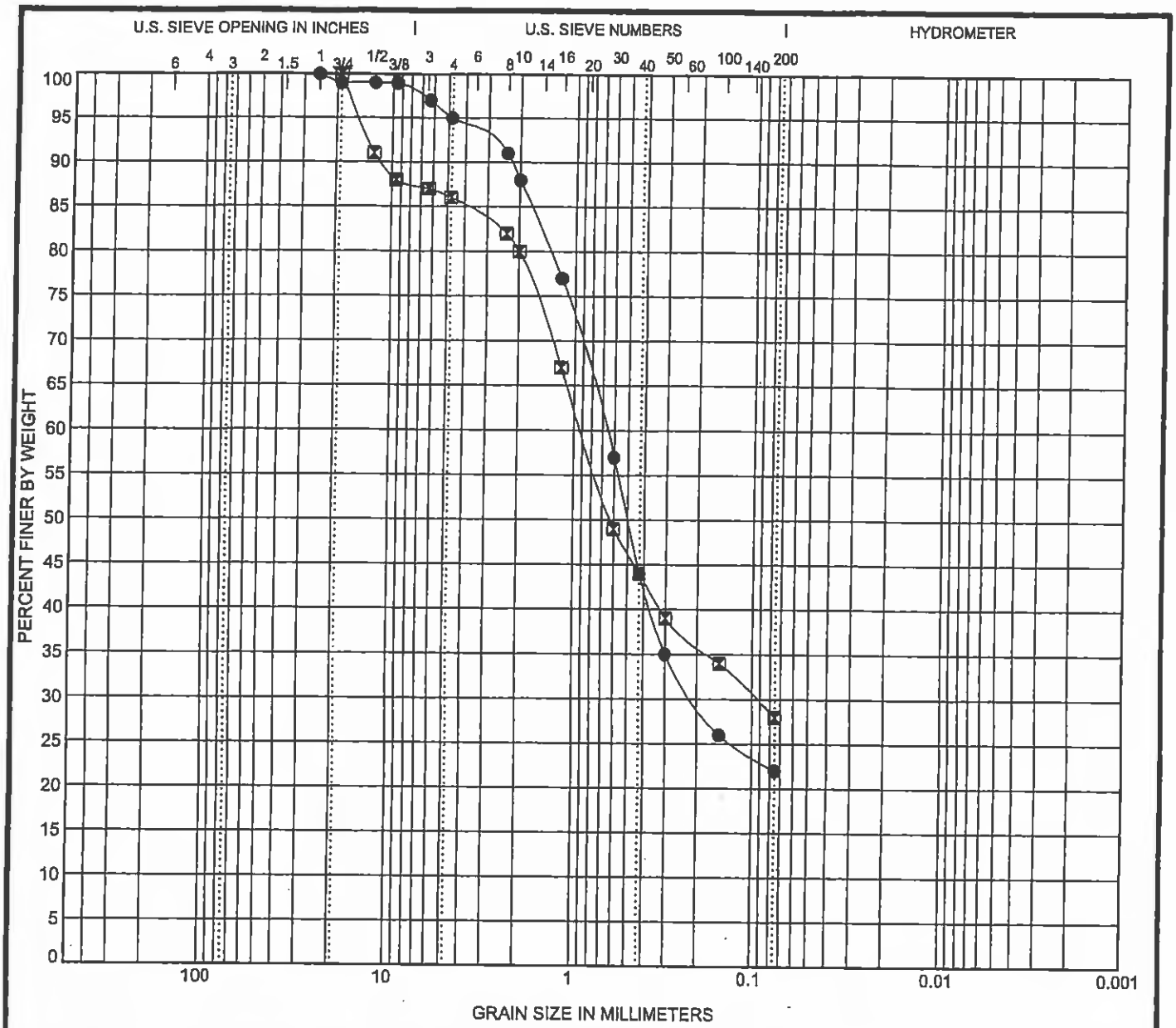
Portland, Oregon 97217-0126
(800) 783-6985

(D-C)

APPENDIX C

Laboratory Testing Results



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● TP-4	at 5.0'	SILTY SAND (SM)					
⊠ TP-7	at 4.5'	SILTY SAND (SM)					

Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● TP-4	at 5.0'	25	0.664	0.204		5.0	73.0	22.0	
⊠ TP-7	at 4.5'	19	0.907	0.094		14.0	58.0	28.0	

psi
Engineering Consulting Testing
6032 N. Cutter Circle Suite #480, Portland, Oregon 97217
Phone (503) 289-1778 Fax (503) 289-1918

GRAIN SIZE DISTRIBUTION (ASTM C136-06/C117-04)

Client: Building Structures, Inc.
Project Name:Proposed Vancouver Toyota
Project Location: 1009 NE Fourth Plain Rd., Vancouver, WA
Report Number: 704-75065

(D-D)

APPENDIX D

Local Well Log Records

RESOURCE PROTECTION WELL REPORT

(SUBMIT ONE WELL REPORT PER WELL INSTALLED)

CURRENT Notice of Intent No. 515222

Construction/Decommission *("x" in circle)*

- [x] Construction
- [ ] Decommission *ORIGINAL INSTALLATION Notice of Intent Number* 192809

Consulting Firm Cambria

Unique Ecology Well ID Tag No:

WELL CONSTRUCTION CERTIFICATION: I constructed and/or accept responsibility for construction of this well, and its compliance with all Washington well construction standards. Materials used and the information reported above are true to my best knowledge and belief.

[x] Driller [ ] Engineer [ ] Trainee Name (Print) Darryl Metzger

Driller/Engineer/Trainee Signature [signature]

Driller or Trainee License No. 2587

If trainee, licensed driller's Signature and License no.

Type of Well *("x" in circle)*

- [ ] Resource Protection
- [x] Geotech Soil Boring

Property Owner Chevron

Site Address 9414 Vancouver Mall Dr.

City Vancouver County: Clark

Location SE/4 NW/4 Sec 16 Twn 2N R 2E EWM or WWM *circle one*

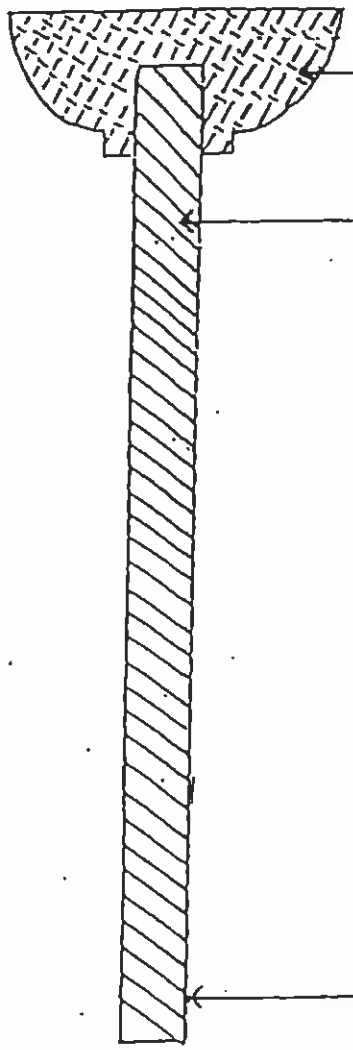
Lat/Long (s, t, r still REQUIRED) Lat Deg ____ Lat Min/Sec ____ Long Deg ____ Long Min/Sec ____

Tax Parcel No.

Cased or Uncased Diameter 2" Static Level 10'

Work/Decommission Start Date 3/20/06

Work/Decommission Completed Date 3/20/06

Construction/Design	Well Data	Formation Description
	SB-1 CONCRETE SURFACE SEAL BACKFILL 0-1' Concrete 1'-10' Bent. Chips DEPTH OF BORING 10' "	0 - 2.5' ft. Heavy Top Fill (Cobbles) 2.5' - 10' ft. Medium gravels + course sands — ft.

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Scale 1"= ____ Page ____ of ____

ECY 050-12 (Rev 2/01)

Washington State Department of Transportation

LOG OF TEST BORING

177232

Start Card S-14911

Job No. XL-0993 SR 500 Elevation 197.6 (60.2 m)

HOLE No. H-8-03

Sheet 1 of 2

Project SW Region Quality Engineering Center

Driller Vince Johnson Lic# 2832

Site Address Vic. Gher Drive

Inspector Cleo Andrews

Start April 3, 2003 Completion April 3, 2003 Well ID# Equipment CME 55 w/ autohammer

Station 40' E of W boundary Offset 131.5' S of curbline Casing HWT 4" x 25.0' Method Wet Rotary

Northing Easting Latitude Longitude

County Clark Subsection NE 1/4 NE 1/4 Section 16 Range 2E Township 2N

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				3 6 7 15 (13)		D-1		Silty GRAVEL with sand, subrounded, medium dense, brown, moist, Homogeneous, no HCl reaction. Length Recovered 0.3 ft, Length Retained 0.3 ft		
				7 11 8 7 (19)		D-2		Silty GRAVEL with sand, with cobbles, subrounded, medium dense, grayish brown, moist, Stratified, no HCl reaction. (100% drilling fluid return). Length Recovered 0.5 ft, Length Retained 0.5 ft		
5	1			8 20 18 8 (38)		D-3		Silty GRAVEL with sand, with cobbles, subrounded, dense, brown, moist, Stratified, no HCl reaction. Length Recovered 0.3 ft, Length Retained 0.3 ft		
	2					U-4		No Recovery		
						U-5 A-B C-D E	GS MC AL	CL, MC=23%, PI=16. Sandy Lean CLAY, with dark reddish brown stains, medium stiff, light gray, moist, Laminated, no HCl reaction. Length Recovered 1.7 ft, Length Retained 1.7 ft		
10	3			2 3 3 5 (6)		D-6		Sandy Lean CLAY with gravel, with dark reddish brown stains, medium stiff, light gray, moist, Laminated, no HCl reaction. Length Recovered 2.0 ft, Length Retained 1.3 ft	▽ 04/03/2003	
	4					U-7 A-B C-D E		Sandy Lean CLAY, sandy lean clay with gravel, medium stiff, olive brown, moist, Stratified, no HCl reaction, sandy lean Clay with gravel, laminated with dark reddish brown stains. Length Recovered 1.8 ft, Length Retained 1.7 ft		
15	5			2 3 5 5 (8)		D-8	GS MC AL	CL, MC=22%, PI=15. Lean CLAY with sand, olive brown stains, medium stiff, olive gray, moist, Laminated, no HCl reaction. Length Recovered 2.0 ft, Length Retained 1.3 ft		
20	6			10 25		D-9	GS MC	SM, MC=23%. Silty SAND with olive brown stains, dense, olive gray,		

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SOIL XL0993 SW REGION QUALITY ENGINEERING CENTER.GPJ SOIL.GDT 5/22/03,10:09:21 A5

Washington State
Department of Transportation

LOG OF TEST BORING

Start Card S-14911

HOLE No. H-8-03

Job No. XL-0993 SR 500 Elevation 197.6 (60.2 m)

Sheet 2 of 2

Project SW Region Quality Engineering Center

Driller Vince Johnson Lic# 2532

Depth (ft)	Meters (m)	Profile	Standard Penetration Blows/ft (10 20 30 40)	SPT Blows/6" (N)	Sample Type	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater	Instrument
				23 22 (48)				moist, Laminated, no HCl reaction Length Recovered 2.0 ft, Length Retained 1.0 ft		
								End of test hole boring at 21 ft below ground elevation. This is a summary Log of Test Boring. Soil/Rock descriptions are derived from visual field identifications and laboratory test data.		
25	7									
	8									
30	9									
35	10									
40	11									
	12									
45	13									

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Department of Ecology

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Appendix E: Low Impact Development Technical Guidance Manual for Puget Sound from Vancouver Toyota Final Stormwater Report

APPENDIX | E

Low Impact Development Technical Guidance Manual for Puget Sound

Vancouver Toyota
Project #14292

MacKay & Sposito, Inc.

6.2.3 Maintenance

- Incorporate soil amendments at the end of the site development process.
- Protect amended areas from excessive foot traffic and equipment to prevent compaction and erosion.
- Plant and mulch areas immediately after amending soil to stabilize site as soon as possible.
- Minimize or eliminate use of pesticides and fertilizers. Landscape management personnel should be trained to adjust chemical inputs accordingly and manage the landscape areas to minimize erosion, recognize soil and plant health problems, and optimize water storage and soil permeability.

6.2.4 Performance

The surface bulk density of construction site soils generally range from 1.5 to 2.0 gm/cc (CWP, 2000a). At 1.6 to 1.7 gm/cc plant roots cannot penetrate soil and oxygen content, biological activity, nutrient uptake, porosity, and water holding capacity are severely degraded (CWP, 2000a and Balousek, 2003). Tilling alone has limited effect for reducing the bulk density and enhancing compacted soil. A survey of research examining techniques to reverse soil compaction by Schueler found that tilling reduced bulk density by 0.00 to 0.15 gm/cc. In contrast, tilling with the addition of compost amendment decreased bulk density by 0.25 to 0.35 gm/cc (CWP, 2000a).

Balousek (2003) prepared combinations of deep tillage, chisel plow, and compost amended plots on an area with silt loam soil that was cleared and graded to simulate construction site conditions. The deep-tilled plots increased runoff volume compared to the control, and the combined chisel plow and deep-tilled treatment reduced runoff volume by 36 to 53 percent. With compost added to the combined plow and till treatment, runoff volume was reduced by 74 to 91 percent.

Research plots at University of Washington, prepared with various amounts and types of compost mixed with till soil and planted with turf, generated 53 to 70 percent of the runoff volume observed from the unamended control plots. The greatest attenuation was observed in treatments with a ratio of 2 parts soil to 1 part fine, well-aged compost. The study indicates that using compost to amend lawn on till soils can "significantly enhance the ability of the lawn to infiltrate, store and release water as baseflow" (Kolsti, Burges, and Jensen, 1995).

6.3 Permeable Paving

Permeable paving surfaces are designed to accommodate pedestrian, bicycle, and vehicle traffic while allowing infiltration, treatment, and storage of stormwater. The general categories of permeable paving systems include:

- *Open-graded concrete or hot-mix asphalt pavement,* which is similar to standard pavement, but with reduced or eliminated fine material (sand and finer) and special admixtures incorporated (optional). As a result, channels form between the aggregate in the pavement surface and allow water to infiltrate.
- *Aggregate or plastic pavers* that include cast-in-place or modular pre-cast blocks. The cast-in-place systems are reinforced concrete made with reusable forms. Pre-cast systems are either high-strength Portland cement concrete or plastic blocks. Both systems have wide joints or openings that can be filled with soil and grass or gravel.

Permeable paving surfaces accommodate pedestrian, bicycle, and vehicle traffic while allowing infiltration, treatment and storage of stormwater.

- *Plastic grid systems* that come in rolls and are covered with soil and grass or gravel. The grid sections interlock and are pinned in place.

6.3.1 Applications

Typical applications for permeable paving include industrial and commercial parking lots, sidewalks, pedestrian and bike trails, driveways, residential access roads, and emergency and facility maintenance roads. Highways and other high traffic load roads have not been considered appropriate for permeable paving systems. However, porous asphalt has proven structurally sound and remained permeable in a highway application on State Route 87 near Phoenix, Arizona and permeable concrete and pavers have been successfully used in industrial settings with high vehicle loads (Hossain, Scofield and Meier, 1992).

Figure 6.3.1 The residential access road at Jordan Cove Urban Monitoring Project in Connecticut is paved entirely with permeable pavers.
Photo by Tom Wagner

Benefits of permeable pavement

Initial research indicates that properly designed and maintained permeable pavements can virtually eliminate surface flows for low intensity storms common in the Pacific Northwest; store or significantly attenuate subsurface flows (dependent on underlying soil and aggregate storage design); and provide water quality treatment for nutrients, metals, and hydrocarbons (see Section 6.3.4: Performance for additional information).

Permeable paving systems have been designed with aggregate storage to function as infiltration facilities with relatively low subgrade infiltration rates (as low as 0.1 inch/hour). When water is not introduced from adjacent areas, these systems have a lower contribution to infiltration area ratio than conventional infiltration facilities (i.e., 1 to 1) and are less likely to have excessive hydraulic loading. Directing surface flows to permeable paving surfaces from adjacent areas is not recommended. If design constraints require that surface flow be introduced from adjacent areas, particular caution should be taken to ensure that excessive sediment is not directed to the system or that additional flows will not exceed the hydraulic loading capability.

The permeable paving systems examined in this section provide acceptable surfaces for disabled persons. WAC 51-40-1103 Section 1103 (Building Accessibility) states that abrupt changes in height greater than ¼ inch in accessible routes of travel shall be beveled to 1 vertical in 2 horizontal. Changes in level greater than ½ inch shall be accomplished with an approved ramp. Permeable asphalt and concrete, while rougher than conventional paving, do not have abrupt changes in level when properly installed. The concrete pavers have small cells filled with aggregate to a level just under the top of the paver, as well as beveled edges. Gravel pave systems use a specific aggregate with a reinforcing grid that creates a firm and relatively smooth surface (see Section 6.3.2: Design).

Two qualifications for use of permeable paving and disabled access should be noted. Sidewalk designs incorporate scoring, or more recently, truncated domes, near the curb ramp to indicate an approaching traffic area for the blind. The rougher surfaces of permeable paving may obscure this transition; accordingly, standard concrete with scoring or concrete pavers with truncated domes should be used for curb ramps (Florida Concrete and Products Association [FCPA], n.d.). Also, the aggregate within the cells of permeable pavers (such as Eco-Stone) can settle or be displaced from vehicle use. As a result, paver installations for disabled parking spaces and walkways may need to include solid pavers. Individual project designs should be tailored to site characteristics and local regulatory requirements.

Many individual products with specific design requirements are available and cannot all be examined in this manual. To present a representative sample of widely applied products, this section will examine the design, installation, maintenance, and performance of permeable hot-mix asphalt, Portland cement concrete, a concrete paver system, and a flexible plastic grid system.

6.3.2 Design

Handling and installation procedures for permeable paving systems are different from conventional pavement. For the successful application of any permeable paving system three general guidelines must be followed.

For successful application of any permeable paving system follow these three general guidelines:

- *Use correct design specifications.*
- *Use qualified contractors.*
- *Strictly control erosion and sediment.*

1. **Correct design specifications**
   Proper site preparation, correct aggregate base and wearing course gradations, separation layer, and under-drain design (if included) are essential for adequate infiltration, storage, and release of storm flows, as well as structural integrity. For example, over compaction of the underlying soil and excessive fines present in the base or top course will significantly degrade or effectively eliminate the infiltration capability of the system.

2. **Qualified contractors**
   Contractors must be trained and have experience with the product, and suppliers must adhere to material specifications. Installation contractors should provide data showing successful application of product specifications for past projects. If the installation contractor does not have adequate experience the contractor should retain a qualified consultant to monitor production, handling, and placement operations (U.S. Army Corps of Engineers, 2003). Substituting inappropriate materials or installation techniques will likely result in structural or hydrologic performance problems. For example, using vibrating plate compactors (typical concrete installation procedure) with excessive pressures and frequencies will seal the void spaces in permeable cast-in-place concrete.

3. **Sediment and erosion control**
   Erosion and introduction of sediment from surrounding land uses should be strictly controlled during and after construction to reduce clogging of the void spaces in the base material and permeable surface. Filter fabric between the underlying soil and base material is required to prevent soil fines from migrating up and into the aggregate base. Muddy construction equipment should not be allowed on the base material or pavement, sediment laden runoff

should be directed to pre-treatment areas (e.g., settling ponds and swales), and exposed soil should be mulched, planted, and otherwise stabilized as soon as possible.

The preceding guidelines are not optional for the installation of permeable paving systems. Past design failures are most often attributed to not adhering to the above general guidelines, and failure is likely without qualified contractors and strict adherence to correct installation specifications.

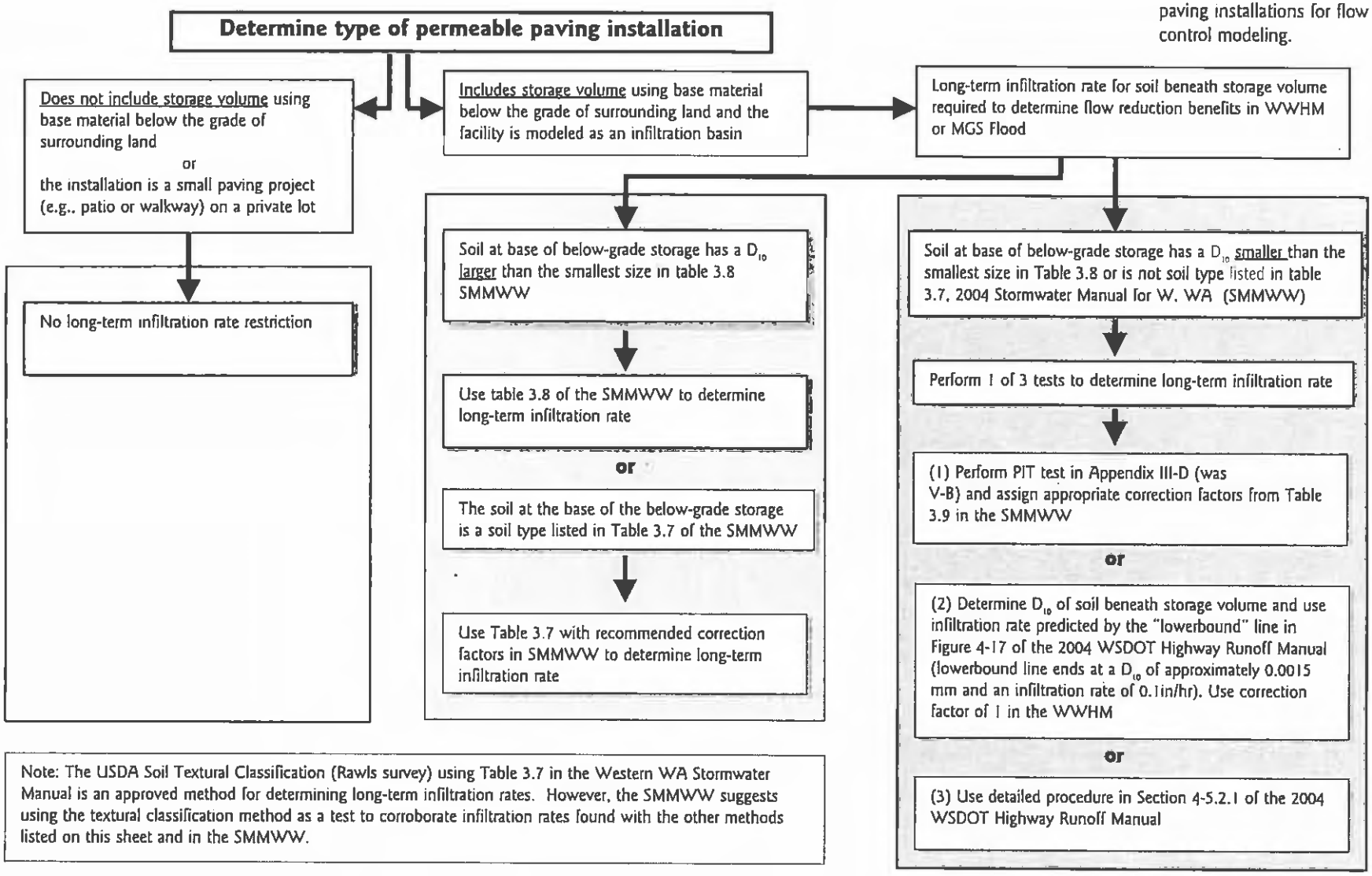
Properly designed permeable paving installations have performed well in the Midwestern and Northeastern U.S. where freeze-thaw cycles are severe (Adams, 2003 and Wei, 1986). Risk of freeze damage can be minimized by extending the base of the permeable paving system to a minimum of half the freeze depth. For example, a total minimum depth for the wearing course and aggregate base material would be 6 inches in the Seattle area, where the freeze-thaw depth is 12 inches (Diniz, 1980).

Determining infiltration rates

Depending on the design, permeable paving installations can be modeled as landscaped area over the underlying soil type or as an infiltration basin. If the installation is modeled as an infiltration basin, determining the infiltration rate of the underlying soil is necessary to equate flow reduction benefits when using the WWHM or MGS Flood. For details on flow modeling guidance see Chapter 7. See Figure 6.3.2 for a graphic representation of the process to determine infiltration rates. The following tests are recommended for soils below the aggregate base material:

- Small permeable paving installations (patios, walkways, and driveways on individual lots): The flow control credits on private property do not include subsurface storage; accordingly, no infiltration field tests are necessary. Soil texture, grain size analysis, or soil pit excavation and infiltration tests may still be prudent if highly variable soil conditions or seasonal high water tables are suspected.
- Large permeable paving installations (sidewalks, alleys, parking lots, roads) that include storage volume using base material below the grade of the surrounding land and the installations are modeled as an infiltration basin:
    - Method 1: Use USDA Soil Textural Classification (Rawls survey) every 200 feet of road or every 5,000 square feet.
    - Method 2: Use ASTM D422 Gradation Testing at Full Scale Infiltration Facilities every 200 feet of road or every 5,000 square feet. See the 2005 SMMWW Volume III for details on methods 1 and 2. This method uses the *2004 WSDOT Highway Runoff Manual* protocol.
    - Method 3: Use small-scale infiltrometer tests every 200 feet of road or every 5,000 square feet. Small-scale infiltrometer tests such as the USEPA Falling Head or double ring infiltrometer tests (ASTM 3385-88) may not adequately measure variability of conditions in test areas. If used, measurements should be taken at several locations within the area of interest.
    - Method 4: Pilot Infiltration Test (PIT) or small-scale test infiltration pits (septic test pits) at a rate of 1 pit/500 feet of road or 10,000 ft^2. This infiltration test better represents soil variability and is recommended for highly variable soil conditions or where seasonal high water tables are suspected. See the 2005 SMMWW Appendix III-D (formerly V-B) for PIT method description.

Figure 6.3.2 Determining long-term infiltration rates in soils under permeable paving installations for flow control modeling.



Utility excavations under or beside the road section can provide pits for soil classification, textural analysis, stratigraphy analysis, and/or infiltration tests and minimize time and expense for permeable paving infiltration tests.

Components of permeable paving systems

The following provides a general description and function for the components of permeable paving systems. Design details for specific permeable paving system components are included in the section describing specific types of permeable paving.

Wearing course or surface layer

The wearing course provides compressive and flexural strength for the designed traffic loads while maintaining adequate porosity for storm flow infiltration. Wearing courses include cast-in-place concrete, asphalt, concrete and plastic pavers, and plastic grid systems. In general, permeable top courses have very high initial infiltration rates with various asphalt and concrete research reporting 28 to 1750 inches per hour when new (see Appendix 7: Porous Paving Research for details). Various rates of clogging have been observed in wearing courses and should be anticipated and planned for in the system design (see Section 6.3.5: Performance for research on infiltration rates over time). Permeable paving systems allow infiltration of storm flows; however, the wearing course should not be allowed to become saturated from excessive water volume stored in the aggregate base layer.

Aggregate base

The aggregate base provides: (1) a stable base for the pavement; (2) a highly permeable layer to disperse water downward and laterally to the underlying soil; and (3) a temporary reservoir that stores water prior to infiltration in the underlying soil or collection in under-drains for conveyance (Washington State Department of Transportation [WSDOT], 2003). Base material is often composed of larger aggregate (1.5 to 2.5 inches) with smaller stone (leveling or choker course) between the larger stone and the wearing course. Typical void space in base layers ranges from 20 to 40 percent (WSDOT, 2003 and Cahill, Adams and Marm, 2003). Depending on the target flow control standard and physical setting, retention or detention requirements can be partially or entirely met in the aggregate base. Aggregate base depths of 18 to 36 inches are common depending on storage needs and provide the additional benefit of increasing the strength of the wearing course by isolating underlying soil movement and imperfections that may be transmitted to the wearing course (Cahill et al., 2003).

Separation and water quality treatment layer

The separation layer is a non-woven geotextile fabric that provides a barrier to prevent fine soil particles from migrating up and into the base aggregate. If required, the water quality treatment layer filters pollutants from surface water and protects groundwater quality (generally, a treatment layer will be necessary in critical aquifer recharge areas). The treatment media can consist of a sand layer or an engineered amended soil. Engineered amended soil layers should be a minimum of 18 inches and incorporate compost, sphagnum peat moss or other organic material to provide a **cation exchange capacity** of ≥ 5 milliequivalents/100 grams dry soil (Ecology, 2001). Soil gradation and final mix should provide a minimum infiltration rate of 0.5 inch/hour at final compaction.

Flow modeling guidance

See Chapter 7 for guidance and flow reduction credits for permeable paving systems when using the WWHM.

A treatment layer is not required where the subgrade soil has a long-term infiltration rate of < 2.4 inches/hour and a cation exchange capacity of ≥ 5 milliequivalents/100 grams dry soil.

Figure 6.3.3 Permeable pavers were installed at this Marysville parking lot for infiltration. Organic material was mixed with sand as part of the sub-base to enhance treatment.
Photo by Colleen Owen

Types of permeable paving

The following section provides general design specifications for permeable hot-mix asphalt, Portland cement concrete, a flexible plastic grid system, a cement paver, and a rigid plastic block product. Each product has specific design requirements. Most notably the permeable Portland cement concrete and hot-mix asphalt differ from the paver systems in subgrade preparation. Concrete and asphalt systems are designed and constructed to minimize subgrade compaction and maintain the infiltration capacity of the underlying soils. Paver systems require subgrade compaction to maintain structural support. Some soils with high sand and gravel content can retain useful infiltration rates when compacted; however, many soils in the Puget Sound region become essentially impermeable when compacted to 95 percent modified proctor or proctor rates.

The specifications below are provided to give designers general guidance. Each site has unique characteristics and development requirements; accordingly, qualified engineers and other design disciplines should be consulted for developing specific permeable paving systems.

1. Permeable hot-mix asphalt

Permeable asphalt is similar to standard hot-mix asphalt; however, the aggregate fines (particles smaller than No. 30 sieve) are reduced, leaving a matrix of pores that conduct water to the underlying aggregate base and soil (Cahill et al., 2003). Porous asphalt can be used for light to medium duty applications including residential access roads, driveways, utility access, parking lots, and walkways; however, porous asphalt has been used for heavy applications such as airport runways (with the appropriate polymer additive to increase bonding strength) and highways (Hossain, Scofield and Meier, 1992). While freeze-thaw cycles are not a large concern in

Properly installed and maintained permeable asphalt should have a service life that is comparable or longer than conventional asphalt.

the Puget Sound lowland, permeable asphalt can and has been successfully installed in wet, freezing conditions in the Midwestern U.S. and Massachusetts with proper section depths (Cahill et al., 2003 and Wei, 1986). Properly installed and maintained permeable asphalt should have a service life that is comparable or longer than conventional asphalt (personal communication, Tom Cahill, 2003).

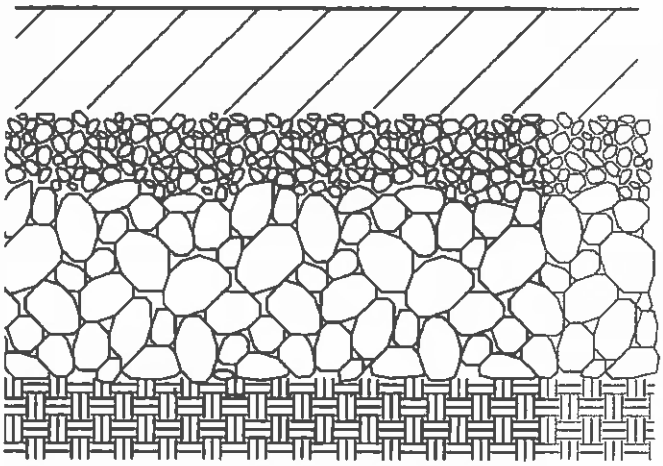


Figure 6.3.4 Permeable asphalt section.

Graphic by AHBL Engineering

Design

Several permeable bituminous asphalt mixes and design specifications have been developed for friction courses (permeable asphalt layer over conventional asphalt) and as wearing courses that are composed entirely of a porous asphalt mix. The friction courses are designed primarily to reduce noise and glare off standing water at night and hydroplaning; however, this design approach provides minimal attenuation of stormwater during the wet season in the Puget Sound region. The following provides specifications and installation procedures for permeable asphalt applications where the wearing top course is entirely porous, the base course accepts water infiltrated through the top course, and the primary design objective is to significantly or entirely attenuate storm flows.

Application: parking lots, driveways, and residential and utility access roads.

Soil infiltration rate

- As long as runoff is not directed to the permeable asphalt from adjacent surfaces, the estimated long-term infiltration rate may be as low as 0.1 inch/hour. Soils with lower infiltration rates should have under-drains to prevent prolonged saturated soil conditions at or near the ground surface within the pavement section.
- Directing surface flows to permeable paving surfaces from adjacent areas is not recommended. Surface flows from adjacent areas can introduce excess sediment, increase clogging, and result in excessive hydrologic loading. However, it may be acceptable to direct flows after treatment to the subgrade if storage volume and infiltration rates allow.

Subgrade

- Soil conditions should be analyzed by a qualified engineer for load bearing given anticipated soil moisture conditions.

- After grading, the existing subgrade should not be compacted or subjected to excessive construction equipment traffic.
- If using the base course for retention in parking areas, excavate the storage bed level to allow even distribution of water and maximize infiltration across entire parking area.
- Immediately before base aggregate and asphalt placement, remove any accumulation of fine material from erosion with light equipment and scarify soil to a minimum depth of 6 inches.

Aggregate base/storage bed

- Minimum base depth for structural support should be 6 inches (Washington State Department of Transportation, 2003).
- Maximum depth is determined by the extent to which the designer intends to achieve a flow control standard with the use of a below-grade storage bed. Aggregate base depths of 18 to 36 inches are common depending on storage needs.
- Coarse aggregate layer should be a 2.5- to 0.5-inch uniformly graded crushed (angular) thoroughly washed stone (AASHTO No. 3).
- Choker course should be 1 to 2 inches in depth and consist of 1.5-inch to U.S. sieve size number 8 uniformly graded crushed washed stone for final grading of base reservoir. The upper course is needed to reduce rutting from construction vehicles delivering and installing asphalt and to more evenly distribute loads to the base material (Diniz, 1980).

Installation of Aggregate base/storage bed

- Stabilize area and install erosion control to prevent runoff and sediment from entering storage bed.
- Install approved non-woven filter fabric on subsoil according to manufacturer's specifications. Where installation is adjacent to conventional paving surfaces, filter fabric should be wrapped up sides to top of base aggregate to prevent migration of fines from densely graded material to the open graded base, maintain proper compaction, and avoid differential settling.
- Overlap adjacent strips of fabric at least 24 inches. Secure fabric 4 feet outside of storage bed to reduce sediment input to bottom of area storage reservoir.
- Install coarse (1.5 to 2.5 inch) aggregate in maximum of 8-inch lifts and lightly compact each lift.
- Install a 1 to 2-inch choker course evenly over surface of course aggregate base.
- Following placement of base aggregate and again after placement of the asphalt, the filter fabric should be folded over placements to protect installation from sediment inputs. Excess filter fabric should not be trimmed until site is fully stabilized (U.S. Army Corps of Engineers, 2003).

Top course

- Parking lots: 2 to 4 inches typical.
- Residential access roads: 2 to 4 inches typical.
- Permeable asphalt has similar strength and flow properties as conventional asphalt; accordingly, the wearing course thickness is similar for either surface given equivalent load requirements (Diniz, 1980).

Aggregate grading:	U.S. Standard Sieve	Percent Passing
	1/2	100
	3/8	92-98
	4	32-38
	8	12-18
	16	7-13
	30	0-5
	200	0-3

- A small percentage of fine aggregate is necessary to stabilize the larger porous aggregate fraction. The finer fraction also increases the viscosity of the asphalt cement and controls asphalt drainage characteristics.
- Total void space should be approximately 16 percent (conventional asphalt is 2 to 3 percent) (Diniz, 1980).

Bituminous asphalt cement

- Content: 5.5 to 6.0 percent by weight dry aggregate. The minimum content assures adequate asphalt cement film thickness around the aggregate to reduce photo-oxidation degradation and increase cohesion between aggregate. The upper limit is to prevent the mixture from draining during transport.
- Grade: 85 to 100 penetration recommended for northern states (Diniz, 1980).
- An elastomeric polymer can be added to the bituminous asphalt to reduce drain-down.
- Hydrated lime can be added at a rate of 1.0 percent by weight of the total dry aggregate to mixes with granite stone to prevent separation of the asphalt from the aggregate and improve tensile strength.

General installation

- Install permeable asphalt system toward the end of construction activities to minimize sediment problems. The subgrade can be excavated to within 6 inches of final grade and grading completed in later stages of the project (Cahill et al., 2003).
- Erosion and introduction of sediment from surrounding land uses should be strictly controlled during and after construction. Erosion and sediment controls should remain in place until area is completely stabilized with soil amendments and landscaping.
- Adapting aggregate specifications can influence bituminous asphalt cement properties and permeability of the asphalt wearing course. Before final installation, test panels are recommended to determine asphalt cement grade and content compatibility with the aggregate (Diniz, 1980).
- Insulated covers over loads during hauling can reduce heat loss during transport and increase working time (Diniz, 1980). Temperatures at delivery that are too low can result in shorter working times, increased labor for hand work, and increased cleanup from asphalt adhering to machinery (personal communication Leonard Spodoni, April 2004).

Backup systems for protecting permeable asphalt systems

- For backup infiltration capacity (in case the asphalt top course becomes clogged) an unpaved stone edge can be installed that is hydrologically connected to the storage bed (see Figure 6.3.5).

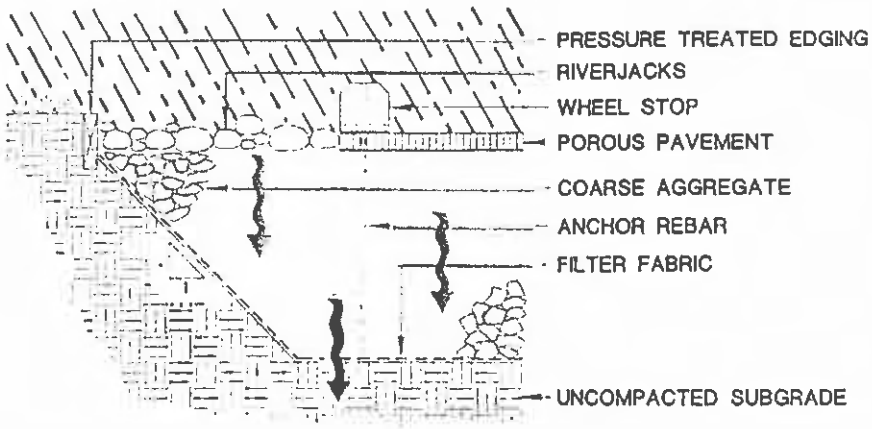


Figure 6.3.5 Unpaved section (river jacks) provides backup infiltration.
Graphic courtesy of Cahill Associates

- As with any paving system, rising water in the underlying aggregate base should not be allowed to saturate the pavement (Cahill et al., 2003). To ensure that the asphalt top course is not saturated from excessively high water levels in the aggregate base (as a result of subgrade soil clogging), a positive overflow can be installed.

For a sample specification for permeable asphalt paving see Appendix 8.

Cost

Materials and mixing costs for permeable asphalt are similar to conventional asphalt. In general, local contractors are currently not familiar with permeable asphalt installation, and additional costs for handling and installation should be anticipated. Estimates for porous pavement material and installation are approximately $.60 to .70/square foot and will likely be comparable to standard pavement as contractors become more familiar with the product. Due to the lack of experience regionally, this is a rough estimate. The cost for base aggregate will vary significantly depending on base depth for stormwater storage and is not included in the cost estimate.

2. Portland cement permeable concrete

Florida and Georgia use permeable concrete extensively for stormwater management. The material and installation specifications in Washington are derived primarily from the field experience and testing through the Florida Concrete and Products Association. In the Puget Sound region, the cities of Seattle and Olympia and Stoneway Concrete have tested materials and installed several projects including parking lots, sidewalks, and driveways.

Permeable Portland cement concrete is similar to conventional concrete without the fine aggregate (sand) component. The mixture is a washed coarse aggregate (3/8 or 5/8 inch), hydraulic cement, admixtures (optional) and water, yielding a surface with a matrix of pores that conducts water to the underlying aggregate base and soil. Permeable concrete can be used for light to medium duty applications including residential access roads, driveways, utility access, parking lots, and walkways. Permeable concrete can also be used in heavy load applications. For example, test sections in a city of Renton aggregate recycling yard have performed well

structurally after being subjected to regular 50,000- to 100,000-pound vehicle loads for the past three years (personal communication, Greg McKinnon, March 2004). Properly installed and maintained concrete should have a service life comparable to conventional concrete.

Designing the aggregate base to accommodate retention or detention storage will depend on several factors, some of which include project specific stormwater flow control objectives, costs, and regulatory restrictions. However, deeper subgrade to base courses (e.g., 12 to 36 inches) can provide important benefits including significant reduction of above ground stormwater retention or detention needs and uniform subgrade support (FCPA, n.d.). Base courses that are placed above the surrounding grade cannot be used, or given credit for, reducing retention or detention pond sizes. (See Chapter 7 for flow modeling guidance and flow reduction credits.)

Figure 6.3.6 Permeable concrete adjacent to stamped concrete in Des Moines.
Photo by Curtis Hinman

Design and installation

Three general classes of permeable concrete are prevalent: (1) the standard mix using washed course aggregate (3/8 or 5/8 inch), hydraulic cement, admixtures (optional) and water; (2) a Stoneycrete mixture which is similar to the standard mix, but incorporates a strengthening additive; and (3) Percocrete which uses a higher percentage of sand, incorporates an additive to enhance strength and the pore structure, and produces a smoother surface texture. The following design section examines the standard concrete mix. Additional information for Stoneycrete is available at Stoney Creek Materials L.L.C. Austin, Texas and for Percocrete at Michiels International Inc., Kenmore, Washington.

Application: parking lots, driveways, sidewalks, utility access, and residential roads.

Soil infiltration rate

- If runoff is not directed to the permeable concrete from adjacent surfaces, the estimated long-term infiltration rate may be as low as 0.1 inch/hour. Soils with lower infiltration rates should have under-drains to prevent prolonged saturated soil conditions at or near the ground surface within the pavement section.
- Directing surface flows to permeable paving surfaces from adjacent areas is not recommended. Surface flows from adjacent areas can introduce excess sediment, increase clogging, and result in excessive hydrologic loading.

However, it may be acceptable to direct flows after treatment to the subgrade if storage volume and infiltration rates allow.

Subgrade

- Soil conditions should be analyzed for load bearing given anticipated soil moisture conditions by a qualified engineer.
- After grading, the existing subgrade should not be compacted or subject to excessive construction equipment traffic (U.S. Army Corps of Engineers, 2003).
- Immediately before base aggregate and concrete placement, remove any accumulation of fine material from erosion with light equipment and scarify soils to a minimum depth of 6 inches if compacted (U.S. Army Corps of Engineers, 2003).

Aggregate base/storage bed

- Minimum base depth for structural support should be 6 inches (FCPA, n.d.).
- Maximum depth is determined by the extent to which the designer intends to achieve a flow control standard with the use of a below-grade storage bed. Aggregate base depths of 18 to 36 inches are common when designing for retention or detention.
- The coarse aggregate layer varies depending on structural and stormwater management needs. Typical placements include round or crushed washed drain rock (1 to 1.5 inches) or 1.5 to 2.5-inch crushed washed base rock aggregate (e.g., AASTHO No. 3).
- The concrete can be placed directly over the coarse aggregate or a choker course (e.g., 1.5 inch to US sieve size number 8, AASHTO No 57 crushed washed stone) can be placed over the larger stone for final grading.

Installation of aggregate base/storage bed

- Stabilize area and install erosion control to prevent runoff and sediment from entering storage bed.
- If using the aggregate base for retention in parking areas, excavate storage bed level to allow even distribution of water and maximize infiltration across entire parking area.
- Install approved non-woven filter fabric on subsoil according to manufacturer's specifications. Where concrete installations are adjacent to conventional paving surfaces the filter fabric should be wrapped up the sides and to the top of base aggregate to prevent migration of fines from the densely graded base to the open graded base material, maintain proper compaction, and avoid differential settling.
- Overlap adjacent strips of fabric at least 24 inches. Secure fabric 4 feet outside of storage bed to reduce sediment input to bottom of storage reservoir.
- Install coarse aggregate in maximum of 8-inch lifts and lightly compact each lift (U.S. Army Corps of Engineers, 2003).
- If utilized, install a 1-inch choker course evenly over surface of coarse aggregate base (typically No. 57 AASHTO) and lightly compact.
- Following placement of base aggregate and again after placement of concrete, the filter fabric should be folded over placements to protect installation from sediment inputs. Excess filter fabric should not be trimmed until site is fully stabilized (U.S. Army Corps of Engineers, 2003).

Top course

- Parking lots: 4 inches typical.
- Roads: 6 to 12 inches typical.
- Unit weight: 120 to 130 pounds per cubic foot (permeable concrete is approximately 70 to 80 percent of the unit weight of conventional concrete) (FCPA, n.d.).
- Void space: 15 to 21 percent according to ASTM C 138.
- Water cement ratio: 0.27 to 0.35.
- Aggregate to cement ratio: 4:1 to 4.5:1.
- Aggregate: several aggregate specifications are used including:
  - 3/8-inch to No. 16 washed crushed or round per ASTM C 33.
  - 3/8-inch to No. 50 washed crushed or round per ASTM D 448.
  - 5/8-inch washed crushed or round.
  - In general the 3/8-inch crushed or round produces a slightly smoother surface and is preferred for sidewalks, and the 5/8-inch crushed or round produces a slightly stronger surface.
- Portland cement: Type I or II conforming to ASTM C 150 or Type IP or IS conforming to ASTM C 595.
- Admixtures: Can be used to increase working time and include: Water Reducing/Retarding Admixture in conformance with ASTM C 494 Type D and Hydration stabilizer in conformance with ASTM C 494 Type B.
- Water: Use potable water.
- Fiber mesh can be incorporated into the cement mix for added strength.

Installation of top course

- See testing section below for confirming correct mixture and proper installation.
- If mixture contains excess water the cement paste can flow from the aggregate, resulting in a weak surface layer and reduced void space in the lower portion of surface. With the correct water content, the delivered mix should have a wet metallic sheen, and when hand squeezed the mix should not crumble or become a highly plastic mass (FCPA, n.d.).
- Cement mix should be used within 1 hour after water is introduced to mix, and within 90 minutes if an admixture is used and concrete mix temperature does not exceed 90 degrees Fahrenheit (U.S. Army Corps of Engineers, 2003).
- Base aggregate should be wetted to improve working time of cement.
- Concrete should be deposited as close to its final position as possible and directly from the truck or using a conveyor belt placement.
- A manual or mechanical screed can be used to level concrete at 1/2 inch above form.
- Cover surface with 6-mil plastic and use a static drum roller for final compaction (roller should provide approximately 10 pounds per square inch vertical force).
- Edges that are higher than adjacent materials should be finished or rounded off to prevent chipping (standard edging tool is applicable for pervious concrete).
- Cement should be covered with plastic within 20 minutes and remain covered for curing time.
- Curing: 7 days minimum for Portland cement Type I and II. No truck traffic should be allowed for 10 days (U.S. Army Corps of Engineers, 2003).

- Placement widths should not exceed 15 feet unless contractor can demonstrate competence to install greater widths.
- High frequency vibrators can seal the surface of the concrete and should not be used.
- Jointing: Shrinkage associated with drying is significantly less for permeable than conventional concrete. Florida installations with no control joints have shown no visible shrink cracking. A conservative design can include control joints at 60 foot spacing cut to 1/4 the thickness of the pavement (FCPA, n.d. and U.S. Army Corps of Engineers, 2003). Expansion joints can also facilitate a cleaner break point if sections become damaged or are removed for utility work.

Testing

Differences in local materials, handling, and placement can affect permeable concrete performance. The following tests should be conducted even if the contractor or consultant has experience with the material to ensure proper performance.

- The contractor should place and cure two test panels, each covering a minimum of 225 square feet at the required project thickness, to demonstrate that specified unit weights and permeability can be achieved on-site (Georgia Concrete and Products Association [GCPA], 1997).
- Test panels should have two cores taken from each panel in accordance with ASTM C 42 at least 7 days after placement (GCPA, 1997).
- Untrimmed cores should be measured for thickness according to ASTM C 42.
- After determining thickness, cores should be trimmed and measured for unit weight per ASTM C 140.
- Void structure should be tested per ASTM C 138.
- If the measured thickness is greater than 1/4 inch less than the specified thickness, or the unit weight is not within ± 5 pounds per cubic foot, or the void structure is below specifications, the panel should be removed and new panels with adjusted specifications installed (U.S. Army Corps of Engineers, 2003). If test panel meets requirements, panel can be left in place as part of the completed installation.
- Collect and sample delivered material once per day to measure unit weight per ASTM C 172 and C 29 (FCPA, n.d.).

Backup systems for protecting permeable concrete systems

- For backup infiltration capacity (in case the concrete top course becomes clogged) an unpaved stone edge can be installed that is connected to the base aggregate storage reservoir (see Figure 6.3.5).
- As with any paving system, rising water in the underlying aggregate base should not be allowed to saturate the pavement (Cahill et al., 2003). To ensure that the top course is not saturated from excessively high water levels (as a result of subgrade soil clogging), a positive overflow can be installed in the base.

Cost

Permeable concrete material and installation is approximately $3.00 to $5.00 per square foot depending on surface thickness and site conditions. Cost for base aggregate will vary significantly depending on base depth for stormwater storage and is not included in the cost estimate.

3. Eco-Stone permeable interlocking concrete pavers

Eco-Stone is a high-density concrete paver that allows infiltration through a built-in pattern of openings filled with aggregate. When compacted, the pavers interlock and transfer vertical loads to surrounding pavers by shear forces through fine aggregate in the joints (Pentec Environmental, 2000). Eco-Stone interlocking pavers are placed on open graded sub-base aggregate topped with a finer aggregate layer that provides a level and uniform bedding material. Properly installed and maintained, high-density pavers have high load bearing strength and are capable of carrying heavy vehicle weight at low speeds. Properly installed and maintained pavers should have a service life of 20 to 25 years (Smith, 2000).

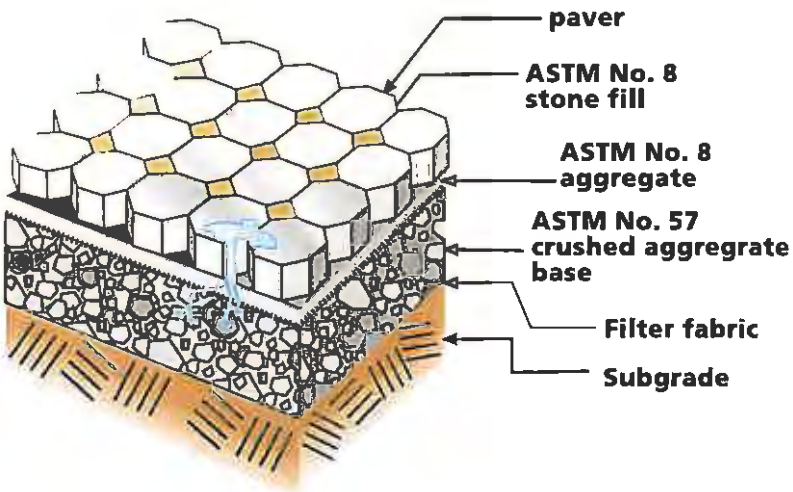


Figure 6.3.7 Permeable interlocking concrete paver section.
Graphic by Gary Anderson

Figure 6.3.8 Close-up view of permeable pavers.
Photo by Curtis Hinman

Design

Application: Industrial and commercial parking lots, utility access, residential access roads, driveways, and walkways. Experienced contractors with a current certificate in the ICPI Contractor Certification Program should perform installations.

Soil infiltration rate

- If runoff is not directed to the permeable pavers from adjacent surfaces, the estimated long-term infiltration rate may be as low as 0.5 inch/hour. Soils with lower infiltration rates should have under-drains at the bottom of the base course to prevent prolonged saturated soil conditions at or near the ground surface within the pavement section. Drain-down time for the base should not exceed 24 hours.
- Directing surface flows to permeable paving surfaces from adjacent areas is not recommended. Surface flows from adjacent areas can introduce excess sediment, increase clogging, and result in excessive hydrologic loading. However, it may be acceptable to direct flows after treatment to the subgrade if storage volume and infiltration rates allow.

Subgrade

- Soils should be analyzed by a qualified engineer for infiltration rates and load bearing, given anticipated soil moisture conditions. **California Bearing Ratio** values should be at least 5 percent.
- For vehicle traffic areas, grade and compact to 95 percent modified proctor density (per ASTM D 1557) and compact to 95 percent standard proctor density for pedestrian areas (per ASTM D698) (Smith, 2000). Soils with high sand and gravel content can retain useful infiltration rates when compacted; however, many soils in the Puget Sound region become essentially impermeable at this compaction rate. For detention designs on compacted soils that will provide very low permeability, adequate base aggregate depths and under-drain systems should be incorporated to reduce risk of continued saturation that can weaken subgrades subject to vehicle traffic (Smith, 2000).

Aggregate base/storage bed

- Minimum base thickness depends on vehicle loads, soil type, stormwater storage requirements, and freeze thaw conditions. Typical depths range from 6 to 22 inches; however, increased depths can be applied for increased storage capacity (Smith, 2000). Interlocking Concrete Paver Institute guidelines for base thickness should be followed.
- Minimum base depth for pedestrian and bike applications should be 6 inches (Smith, 2000).
- ASTM No. 57 crushed aggregate or similar gradation is recommended for the sub-base (Smith, 2000).
- ASTM No. 8 is recommended for the leveling or choker course.

Installation of aggregate base/storage bed

- Stabilize area and install erosion control to prevent runoff and sediment from entering storage bed.
- If using the base course for retention in parking areas, excavate storage bed level to allow even distribution of water and maximize infiltration across entire parking area.

- Install approved non-woven filter fabric to bottom and sides of excavation according to manufacturer's specifications. Where paver installation is adjacent to conventional paving surfaces, filter fabric should be wrapped up sides to top of base aggregate to prevent migration of fines from densely graded base to the open graded base material, maintain proper compaction, and avoid differential settling. A concrete curb the depth of the base can also be used to separate the open graded and dense graded bases.
- Overlap adjacent strips of fabric at least 24 inches. Secure fabric 4 feet outside of storage bed to reduce sediment input to bottom of area storage reservoir (Smith, 2000).
- Install No. 57 aggregate in 4 to 6-inch lifts.
- Compact the moist No. 57 aggregate with at least 4 passes of a 10-ton (minimum) steel drum roller. Initial passes can be with vibration and the final two passes should be static (Smith, 2000). Testing for appropriate density per ASTM D 698 or D 1557 will likely not provide accurate results. The Interlocking Concrete Pavement Institute specification recommends that adequate density and stability are developed when no visible movement is observed in the open-graded base after compaction (personal communication, Dave Smith ICPI).
- Install three inches of No. 8 aggregate for the leveling or choker course and compact with at least 4 passes of a 10-ton roller. Surface variation should be within ± 1/2 inch over 10 feet. The No. 8 aggregate should be moist to facilitate compaction into the sub-base (Smith, 2000).
- Asphalt stabilizer can be used with the No. 57 stone if additional bearing support is needed, but should not be applied to the No. 8 aggregate. To maintain adequate void space, use a minimum of asphalt for stabilization (approximately 2 to 2.5 percent by weight of aggregate). An asphalt grade of AC20 or higher is recommended. The addition of stabilizer will reduce storage capacity of base aggregate and should be considered in the design (Smith, 2000).
- Following placement of base aggregate and again after placement of pavers, the filter fabric should be folded over placements to protect installation from sediment inputs. Excess filter fabric should not be trimmed until site is fully stabilized.
- Designs for full infiltration of stormwater to the subgrade should have a positive overflow to prevent water from entering the surface layer during extreme events. Designs with partial or no **exfiltration** require under-drains. All installations should have an observation well (typically 6-inch perforated pipe) installed at the furthest downslope area (Smith, 2000).

Top course installation

- Pavers should be installed immediately after base preparation to minimize introduction of sediment and to reduce the displacement of base material from ongoing activity (Smith, 2000).
- Loosen and evenly smooth 3/4 to 1 inch of the compacted No. 8 stone.
- Place pavers by hand or with mechanical installers and compact with a 5000 lbf, 75 to 90 Hz plate compactor. Fill openings with No. 8 stone and compact again. Sweep to remove excess stone from surface. The small amount of finer aggregate in the No. 8 stone will likely be adequate to fill narrow joints between pavers in pedestrian and light vehicle applications. If the installation is subject

to heavy vehicle loads, additional material may be required for joints. Sweep in additional material (ASTM No. 89 stone is recommended) and use vibratory compaction to place joint material (Smith, 2000).

Figure 6.3.9 Mechanical installation of Eco-Stone pavers.
Photo by Curtis Hinman

- Do not compact within 3 feet of unrestrained edges (Pentec Environmental, 2000).
- Sand placed in paver openings or used as a leveling course will clog and should not be applied for those purposes.
- Cast-in-place or pre-cast concrete (approximately 6 inches wide by 12 inches high) are the preferred material for edge constraints. Plastic edge confinement secured with spikes is not recommended (Smith, 2000).

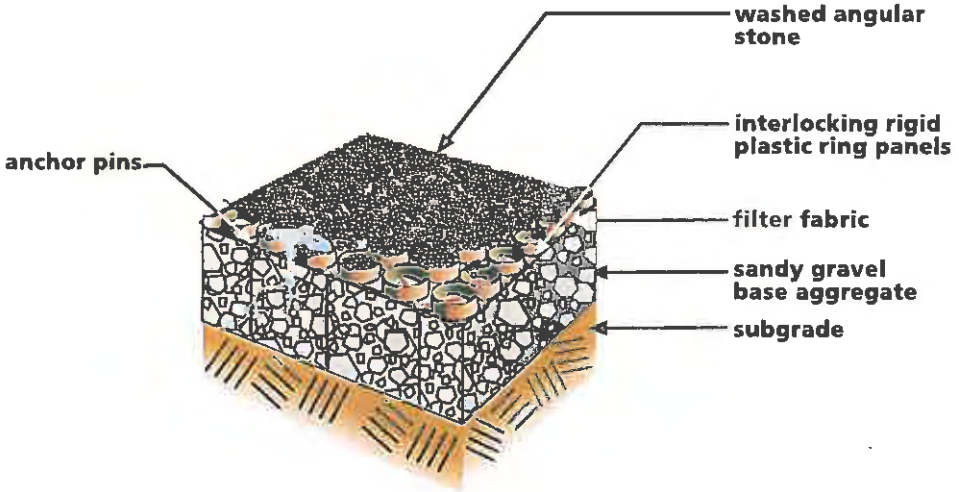
Cost

Eco-Stone material and installation costs range from $2.50 to $4.50 per square foot for the pavers, aggregate leveling layer, aggregate for the paver openings and joints, and installation. Costs for base aggregate will vary significantly depending on stormwater storage needs. Base material and installation, geotextile, excavation, and sediment controls are not included in this price estimate. Large jobs (e.g., 150,000 square feet) utilizing mechanical placement of pavers would qualify for the lower end of the cost range and smaller jobs (e.g., 40,000 square feet) with mechanical installation would likely be at the higher end of the cost range (personal communication, Brian Crooks and Dave Parisi, July 2004).

4. Gravelpave2 flexible plastic grid system

Gravelpave2 is a lightweight grid of plastic rings in 20" wide x 20" long x 1" high units with a geotextile fabric heat fused to the bottom of the grid. The grid and fabric is provided in pre-assembled rolls of various dimensions (Invisible Structures, 2003). This and other similar plastic grid systems have a large amount of open cell available for infiltration in relation to the solid support structure. Flexible grid systems conform to the grade of the aggregate base, and when backfilled with appropriate aggregate top course, provide high load bearing capability (Gravelpave2 load capacity is approximately 5700 psi) (Invisible Structures, 2003). Gravelpave2 is not impacted by the degree of freeze-thaw conditions found in the Puget Sound region. Properly installed and maintained, Gravelpave2 has an expected service life of approximately 20 years (Bohnhoff, 2001).

Figure 6.3.10 Gravelpave2 system.
Graphic by Gary Anderson



Design

Application: Typical uses include alleys, driveways, utility access, loading areas, trails, and parking lots with relatively low traffic speeds (15 to 20 mph maximum). Higher speeds may require use of a binder at 10 percent cement by weight with fill stone (Bohnhoff, 2001).

Soil infiltration rate

- If runoff is not directed to the Gravelpave system from adjacent surfaces, the estimated long-term infiltration rate may be as low as 0.5 inch/hour. Soils with lower infiltration rates should have under-drains in the base course to prevent prolonged saturated soil conditions within the top course section.
- Directing surface flows to permeable paving surfaces from adjacent areas is not recommended. Surface flows from adjacent areas can introduce excess sediment, increase clogging, and result in excessive hydrologic loading. However, it may be acceptable to direct flows after treatment to the subgrade if storage volume and infiltration rates allow.

Subgrade

- Soil conditions should be analyzed for load bearing given anticipated soil moisture conditions by a qualified engineer.
- After grading, the existing subgrade should not be compacted or subject to excessive construction equipment traffic.
- Immediately before base aggregate and top course, remove any accumulation of fine material from erosion with light equipment.

Aggregate base/storage bed

- Minimum base thickness depends on vehicle loads, soil type, and stormwater storage requirements. Typical minimum depth is 4 to 6 inches for driveways, alleys, and parking lots (less base course depth is required for trails) (personal communication, Andy Gersen, July 2004). Increased depths can be applied for increased storage capacity.

- Base aggregate is a sandy gravel material typical for road base construction (Invisible Structures, 2003).

Aggregate grading:	U.S. Standard Sieve	Percent Passing
	3/4	100
	3/8	85
	4	60
	8	15
	40	30
	200	<3

Base course installation

- Stabilize area and install erosion control to prevent runoff and sediment from entering storage bed.
- If using the base course for retention in parking areas, excavate storage bed level to allow even distribution of water and maximize infiltration across entire parking area.
- Install approved non-woven filter fabric to bottom and sides of excavation according to manufacturer's specifications. Where the installation is adjacent to conventional paving surfaces, the filter fabric should be wrapped up the sides and to the top of base aggregate to prevent migration of fines from the densely graded base to the open graded base aggregate, maintain proper compaction, and avoid differential settling.
- Overlap adjacent strips of fabric at least 24 inches. Secure fabric 4 feet outside of storage bed to reduce sediment input to bottom of area storage reservoir.
- Install aggregate in 6-inch lifts maximum.
- Compact each lift to 95 percent modified proctor.

Top course aggregate

Aggregate should be clean, washed angular stone with a granite hardness.

Aggregate grading.	U.S. Standard Sieve	Percent Passing
	4	100
	8	80
	16	50
	30	30
	50	15
	100	5

Top course installation

- Grid should be installed immediately after base preparation to minimize introduction of sediment and to reduce the displacement of base material from ongoing activity.
- Place grid with rings up and interlock male/female connectors along unit edges.
- Install anchors at an average rate of 6 pins per square meter. Higher speed and transition areas (for example where vehicles enter a parking lot with a plastic grid system from an asphalt road) or where heavy vehicles execute tight turns will require additional anchors (double application of pins).
- Aggregate should be back dumped to a minimum depth of 6 inches so that delivery vehicle exits over aggregate. Sharp turning on rings should be avoided.

- The structure of the top edge of the paver blocks reduces chipping from snowplows. For additional protection, skids on the corner of plow blades are recommended.

- Gravelpave2
  - Remove and replace top course aggregate if clogged with sediment or contaminated (vacuum trucks for stormwater collection basins can be used to remove aggregate).
  - Remove and replace grid segments where three or more adjacent rings are broken or damaged.
  - Replenish aggregate material in grid as needed.
  - Snowplows should use skids to elevate blades slightly above the gravel surface to prevent loss of top course aggregate and damage to plastic grid.

6.3.4 Limitations

Permeable paving materials are not recommended where:

- Excessive sediment is deposited on the surface (e.g., construction and landscaping material yards).
- Steep erosion prone areas that are likely to deliver sediment and clog pavement are upslope of the permeable surface.
- Concentrated pollutant spills are possible such as gas stations, truck stops, and industrial chemical storage sites.
- Seasonally high groundwater creates prolonged saturated conditions at or near ground surface and within the pavement section.
- Fill soils can become unstable when saturated.
- Maintenance is unlikely to be performed at appropriate intervals.
- Sealing of surface from sealant application or other uncontrolled use is likely. Residential driveways can be particularly challenging and clear, enforceable guidelines, education, and backup systems should be part of the stormwater management plan for a residential area utilizing permeable paving for driveways.
- Regular, heavy application of sand is used for maintaining traction during winter.
- Permeable paving should not be placed over solid rock without an adequate layer of aggregate base.

Slope restrictions result primarily from flow control concerns and to a lesser degree structural limitations of the permeable paving. Excessive gradient increases surface and subsurface flow velocities and reduces storage and infiltration capacity of the pavement system. Baffle systems placed on the subgrade can be used to detain subsurface flow and increase infiltration (personal communication, Tracy Tackett). See Chapter 7 for the flow control credit associated with permeable paving and subgrade baffles.

- Permeable asphalt is not recommended for slopes exceeding 5 percent.
- Permeable concrete is not recommended on slopes exceeding 6 percent.
- Eco-Stone is not recommended for slopes exceeding 10 percent.
- Gravelpave2 is not recommended for slopes exceeding 6 percent (primarily a traction rather than infiltration or structural limitation).

6.3.5 Permeable Paving Performance

Infiltration

Initial research indicates that properly designed and maintained permeable pavements can virtually eliminate surface flows for low intensity storms common in the Pacific Northwest, store or significantly attenuate subsurface flows (dependent on underlying soil and aggregate storage design), and provide water quality treatment for nutrients, metals, and hydrocarbons. A six-year University of Washington permeable pavement demonstration project found that nearly all water infiltrated various test surfaces (included Eco-Stone, Gravelpave, and others) for all observed storms (Brattebo and Booth, 2003). Observed infiltration was high despite minimal maintenance conducted. See Figure 6.3.11 for infiltration plotted with precipitation for one of the permeable paving test surfaces (turfstone).

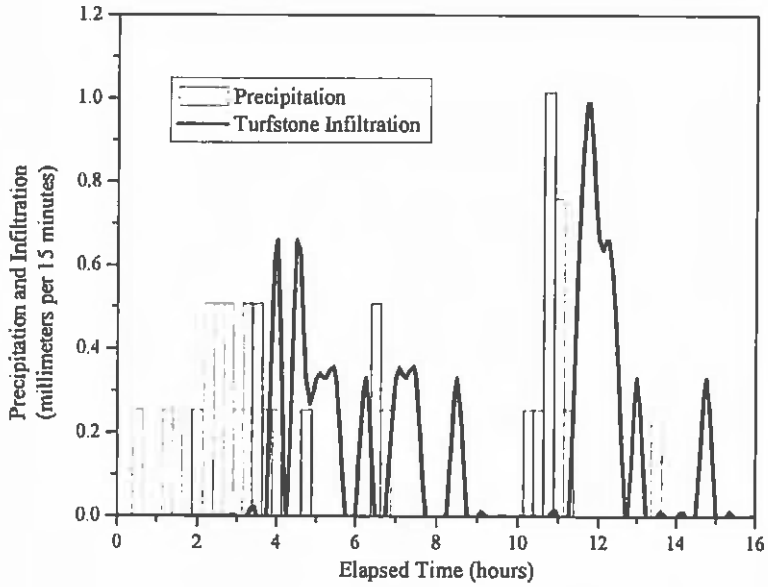


Figure 6.3.11 Infiltration plotted with precipitation at a test permeable pavement parking stall in the city of Renton. Note that essentially all precipitation infiltrates.

Source: Brattebo and Booth, 2003

Initial infiltration rates for properly installed permeable pavement systems are high. Infiltration rates for in-service surfaces decline to varying degrees depending on numerous factors, including initial design and installation, sediment loads, and maintenance. Ranges of new and in-service infiltration rates for research cited in the Appendix 7: Porous Paving Research are summarized below. To provide context for the infiltration rates below, typical rainfall rates are approximately 0.05 inch/hour in the Puget Sound region with brief downpours of 1 to 2 inches/hour.

Porous asphalt:
- highest initial rate (new installation): 1750 in/hr
- lowest initial rate (new installation): 28 in/hr
- highest in-service rate: 1750 in/hr (1 year of service, no maintenance)
- lowest in-service rate: 13 in/hr (3 years of service no maintenance)

Pervious concrete:
- highest initial rate: 1438.20 in/hr
- lowest in-service rate: 240 in/hr (6.5 years of service, no maintenance)
- Note: City of Olympia has observed (anecdotal) evidence of lower infiltration rates on a sidewalk application; however, no monitoring data have been collected to quantify observations (personal communication Mark Blosser, August 2004).

Pervious pavers:	highest initial infiltration rate (new installation): none reported lowest initial rate (new installation): none reported highest in-service rate: 2000 in/hr lowest in-service rate: 0.58 in/hr

Clogging from fine sediment is a primary mechanism that degrades infiltration rates. However, the design of the porous surface (i.e., percent fines, type of aggregate, compaction, asphalt density, etc.) is critical for determining infiltration rates and performance over time as well.

Various levels of clogging are inevitable depending on design, installation, and maintenance and should be accounted for in the long-term design objectives. Studies reviewed in the Porous Paving Research (see Appendix 7) and a review conducted by St. John (1997) indicate that a 50 percent infiltration rate reduction is typical for permeable pavements.

European research examining several permeable paver field sites estimates a long-term design rate at 4.25 inches per hour (Borgwardt, 1994). David Smith from Interlocking Concrete Pavement Institute, however, recommends using a conservative 1.1-inch per hour infiltration rate for the base course (surface intake can be higher) for the typical 20-year life span of permeable paver installations (Smith, 2000).

The lowest infiltration rate reported for an in-service permeable paving surface that was properly installed was approximately 0.58 inches/hour (Uni Eco-Stone parking installation).

Results from the three field studies evaluating cleaning strategies indicate that infiltration rates can be restored. Pervious paver research in Ontario, Canada indicates that infiltration rates can be maintained for Eco-Stone with suction equipment (see Appendix 7: Porous Paving Research). Standard street cleaning equipment with suction may need to be adjusted to prevent excessive uptake of aggregate in paver cells (Gerrits and James, 2001). Washing should not be used to remove debris and sediment in the openings between pavers. Suction should be applied to paver openings when surface and debris are dry.

Street cleaning equipment with sweeping and suction perform adequately on moderately degraded porous asphalt while high pressure washing with suction provides the best performance on highly degraded asphalt (Dierkes, Kuhlmann, Kandasamy and Angelis, 2002 and Balades, Legret and Madiec, 1995). Sweeping alone does not improve infiltration on porous asphalt.

Water Quality

Research indicates that the pollutant removal capability of permeable paving systems is very good for constituents examined. Laboratory evaluation of aggregate base material in Germany found removal capability of 89 to 98 percent for lead, 74 to 98 percent for cadmium, 89 to 96 percent for copper, and 72 to 98 percent for zinc (variability in removal rates depended on type of stone). The same study excavated a 15-year old permeable paver installation in a commercial parking lot and found no significant concentrations of heavy metals, no detection of PAHs, and elevated, but still low concentrations of mineral oil in the underlying soil (Dierkes et al., 2002).

Pratt, Newman and Bond recorded a 97.6 percent removal of automobile mineral oil in a 780 mm (approximately 31-inch) deep permeable paver section in England. Removal was attributed largely to biological breakdown by microbial activity within the pavement section, as well as adhesion to paving materials (Pratt, Newman and Bond, 1999).

A study in Connecticut compared driveways constructed from conventional asphalt and permeable pavers (UNI group Eco-Stone) for runoff depth (precipitation measured on-site), infiltration rates, and pollutant concentrations. The Eco-Stone driveways were two years old. During 2002 and 2003, mean weekly runoff depth recorded for asphalt was 1.8 mm compared to 0.5mm for the pavers. Table 6.3.1 summarizes pollutant concentrations from the study (Clausen and Gilbert, 2003).

Table 6.3.1 Mean weekly pollutant concentration in stormwater runoff, Jordan Cove, CT.

Variable	Asphalt	Paver
TSS	47.8 mg/L	15.8 mg/L
NO_3-N	0.6 mg/L	0.2 mg/L
NH_3-N	0.18 mg/L	0.05 mg/L
TP	0.244 mg/L	0.162 mg/L
Cu	18 ug/L	6 ug/L
Pb	6 ug/L	2 ug/L
Zn	87 ug/L	25 ug/L

(Adapted from Clausen and Gilbert, 2003)

In the Puget Sound region, a six-year permeable parking lot demonstration project conducted by the University of Washington found toxic concentrations of copper and zinc in 97 percent of the surface runoff samples from an asphalt control parking stall. In contrast, copper and zinc in 31 of 36 samples from the permeable parking stall—that produced primarily subsurface flow—fell below toxic levels and a majority of samples fell below detectable levels. Motor oil was detected in 89 percent of the samples from the surface flow off the asphalt stall. No motor oil was detected in any samples that infiltrated through the permeable paving sections. (Brattebo and Booth, 2003).

6.4 Vegetated Roofs

Vegetated roofs (also known as green roofs and eco-roofs) fall into two categories: intensive and extensive. Intensive roofs are designed with a relatively deep soil profile (6 inches and deeper) and are often planted with ground covers, shrubs, and trees. Intensive green roofs may be accessible to the public for walking or serve as a major landscaping element of the urban setting. Extensive vegetated roofs are designed with shallow, light-weight soil profiles (1 to 5 inches) and ground cover plants adapted to the harsh conditions of the roof top environment. This discussion focuses on the extensive design.

Vegetated roofs improve energy efficiency and air quality, reduce temperatures and noise in urban areas, improve aesthetics, extend the life of the roof, and reduce stormwater flows.

Extensive green roofs offer a number of benefits in the urban landscape including: increased energy efficiency, improved air quality, reduced temperatures in urban areas, noise reduction, improved aesthetics, extended life of the roof, and central to this discussion, improved stormwater management (Grant, Engleback and Nicholson, 2003).

Companies specializing in vegetated roof installations emerged in Germany and Switzerland in the late 1950s, and by the 1970s extensive green roof applications were common in those countries. In 2003, 13.5 million square meters of green roofs were installed in Germany (Grant et al., 2003; Peck, Callaghan, Kuhn and Bass, 1999; and Peck, Kuhn and Arch, n.d.). While roof gardens are not as prevalent in the U.S., designers in North America are discovering the value of the technology and green

Appendix F: WSDOT Highway Runoff Manual M31-16 from the Vancouver Toyota Final Stormwater Report

APPENDIX F

WSHDOT Highway Runoff Manual M31-16

Vancouver Toyota
Project #14292

MacKay & Sposito, Inc.

IN.06, Permeable Pavement Surfaces

Introduction

General Description

Currently, this BMP cannot be considered a stand-alone runoff treatment or flow control BMP. However, when used as part of a project surface, it can reduce the total runoff, thereby providing an overall reduction to the size and placement of other acceptable runoff treatment and flow control BMPs.

Permeable (porous or pervious) *surfaces* can be applied to nonpollution-generating surfaces such as pedestrian/bike paths, raised traffic islands, and sidewalks. Permeable surfaces with a media filtration sublayer (such as sand or an amended soil) could be applied to pollution-generating surfaces (such as parking lots) for calculating runoff treatment. Permeable surfaces allow stormwater to pass through and infiltrate the soil below, thereby reducing the rate and volume of runoff associated with conventional surfacing, and fostering groundwater recharge.

The permeable concrete or asphalt pavement surface is an open-graded mix placed in a manner that results in a high degree of interstitial spaces or voids within the cemented aggregate. This technique demonstrates a high degree of absorption or storage within the voids and infiltration to subsoils. The pavement may be permeable concrete, permeable asphalt, or manufactured systems such as interlocking brick or a combination of sand and brick lattice. Geo-Cell with geotextile and aggregate material may also be considered for limited applications.

Contractors shall have prior experience with constructing permeable surfaces. If a contractor does not have this experience, the contractor shall be required to construct test panels before placement of the main surfacing to demonstrate application competency.

Permeable surfaces are vulnerable to clogging from sediment in runoff and the following techniques will reduce this potential:

- Surface runoff – Permeable surfaces should not be located where turbid runoff from adjacent areas can introduce sediments onto the permeable surface. Designs should slope impervious runoff away from permeable pavement installations to the maximum extent possible.
- Diversion – French drains, or other diversion structures, may be designed into the system to avoid unintended off-site runoff. Permeable systems can be separated using edge drain systems, turnpikes, and 0.15-foot-high tapered bumps.
- Cold climates – Snow removal activities (plowing) and the use of salt and abrasives can increase the risk of clogging.
- Slopes – Off-site drainage slopes immediately adjacent to the permeable surface should be less than 5% to reduce the chance of soil loss that would cause clogging.

Limitations

Suitable grades, subsoil drainage characteristics, and groundwater table conditions require good multidisciplinary analysis and design. Proper construction techniques and diligent field inspection during the placement of permeable surfaces are also essential to a successful installation.

- Installation works best with level, adjacent slopes (1% to 2%) and on upland soils. Permeable surface installations are not appropriate when adjacent draining slopes are 5% or greater.
- An extended period of saturation of the base material underlying the surface is undesirable. Therefore, the subsurface reservoir layer should fully drain in a period of less than 36 hours.
- The minimum depth from the bottom of the base course to bedrock and seasonally high water table should be 3 feet, unless it is possible to engineer a groundwater bypass into the system.
- Sanding or repeated snow removal can lead to a reduction in surface permeability. Permeable surfaces should not be used in traffic areas where sanding or extensive snow removal is carried out in the winter.

Examples of situations where the use of permeable surfaces is not currently recommended include the following:

underdrain system may be required. The underdrain could be discharged to a bioretention area, dispersion system, or a stormwater detention facility.

- Turbid runoff to the permeable surface from off-site areas is not allowed. Designs may incorporate infiltration trenches or other options to ensure long-term infiltration through the permeable surface.
- Any necessary boreholes must be installed to a depth of 10 feet below the base of the reservoir layer, and the water table must be monitored at least monthly for a year.
- Infiltration systems perform best on upland soils.

On-site soils should be tested for porosity, permeability, organic content, and potential for cation exchange. These properties should be reviewed when designing the recharge bed of pervious surfaces.

Once a permeable surface site is identified, contact the WSDOT Materials Laboratory to request that a geotechnical investigation be performed. The WSDOT Materials Laboratory, with assistance from the HQ Geotechnical Division (as needed), will determine the quantity and depth of borings/test pits required and any groundwater monitoring needed to characterize the soil infiltration characteristics of the site. Table IN.06.1 provides general guidance on the overall composition of permeable surfaces based on various soil conditions.

In site locations where subgrade materials are marginal, the use of a heavy-duty geogrid placed directly on subgrade may be necessary. A sand layer is placed above the heavy geogrid, followed by geotextile for drainage. Coordination with the HQ Geotechnical Division should be made for these applications.

For determining a final design-level infiltration rate, refer to the design guidance provided in Section 4-5. (Note that this guidance applies primarily to infiltration basins and may therefore exclude slower-percolating soils such as loams, which are potentially suitable for permeable surfaces.)

Table IN.06.1. Permeable surface application matrix.

Soil Characterization Chart for Design of Permeable Surface Layers					
Soil Type [A]	**A**	**B**	**C**	**D**	**Notes**
Surface Layer	1 – 4 [B]	1 – 4 [B]	1 – 4 [B]	1 – 4 [B]	4-inch depth (min.).
Base Layer	5	5	5	5	6-inch depth (min.). Aggregate base depths of 18 to 36 inches are common depending on storage needs (PSAT, 2005).
Separation Layer	7	7	7	7	The separation layer provides a permeable barrier to prevent fine soil particles from migrating up into the base aggregate.
Water Quality Treatment Layer	Not Required [C]	Not Required [C]	6 or 8	6 or 8	The treatment media can consist of a sand layer or an engineered amended soil (PSAT, 2005).
Subgrade Soil	9	9	9	9	If subgrade is overly compacted prior to constructing pavement, till soil 2 feet below the material placement to maintain the soil's permeability.
Underdrain System	No	No	To be determined	To be determined	6-inch-diameter (min.); discharged to bioretention area, natural dispersion, or a detention facility.
Edge Treatment	To be considered	To be considered	To be considered	To be considered	6-inch-diameter (min.); discharged to bioretention area, natural dispersion, or a detention facility.
Subgrade Slope	To be considered	To be considered	To be considered	To be considered	Consider slopes from 1.0% to 2.0%.
Placement Application	10 – 15	10 – 15	10 – 15	11 – 15	

Numbers Referenced in Table IN.06.1:

Surface Type
1) Portland Cement-Based Pervious Pavement Materials
2) Asphalt-Based Pervious Pavement Materials
3) Paving or Lattice Stone
4) Geo-Cell

Base Type
5) BARB (Base Aggregate for Recharge Bed) [D]

Separation
6) Sand
7) Geotextile [E]
8) Engineered Amended Soil

Miscellaneous
9) Minimum Consolidation Required

Placements
10) Residential or Access Driveways
11) Sidewalks
12) Bike Paths
13) Traffic Islands
14) Median Turn-Around
15) Parking Lots

Notes Referenced in Table IN.06.1:

[A] See Table 4B.1 in Appendix 4B for Soil Types.

[B] The separation of permeable surface installations from impermeable surface runoff may be necessary by installing an edge drain or a similar system.

[C] A treatment layer is not required where the subgrade soil has a long-term infiltration rate < 2.4 inches/hour and a cation exchange capacity greater than or equal to 5-milliequivalents/100 grams of dry soil.

[D] Variances other than that specified in the special provision. Base Aggregate for Recharge Bed, should be reviewed and included as applicable and coordinated with the WSDOT Materials Laboratory, HQ Design Office, and HQ Hydraulics Office.

[E] Permeable geotextile must be used to keep the surface layer stable and fines from migrating up through base and surface layers. To obtain geotextile classification, use Geotextile for Underground Drainage, WSDOT Standard Specification Section 9-33, as specified in the special provision Base Aggregate for Recharge Bed and WSDOT *Design Manual*, Section 530.

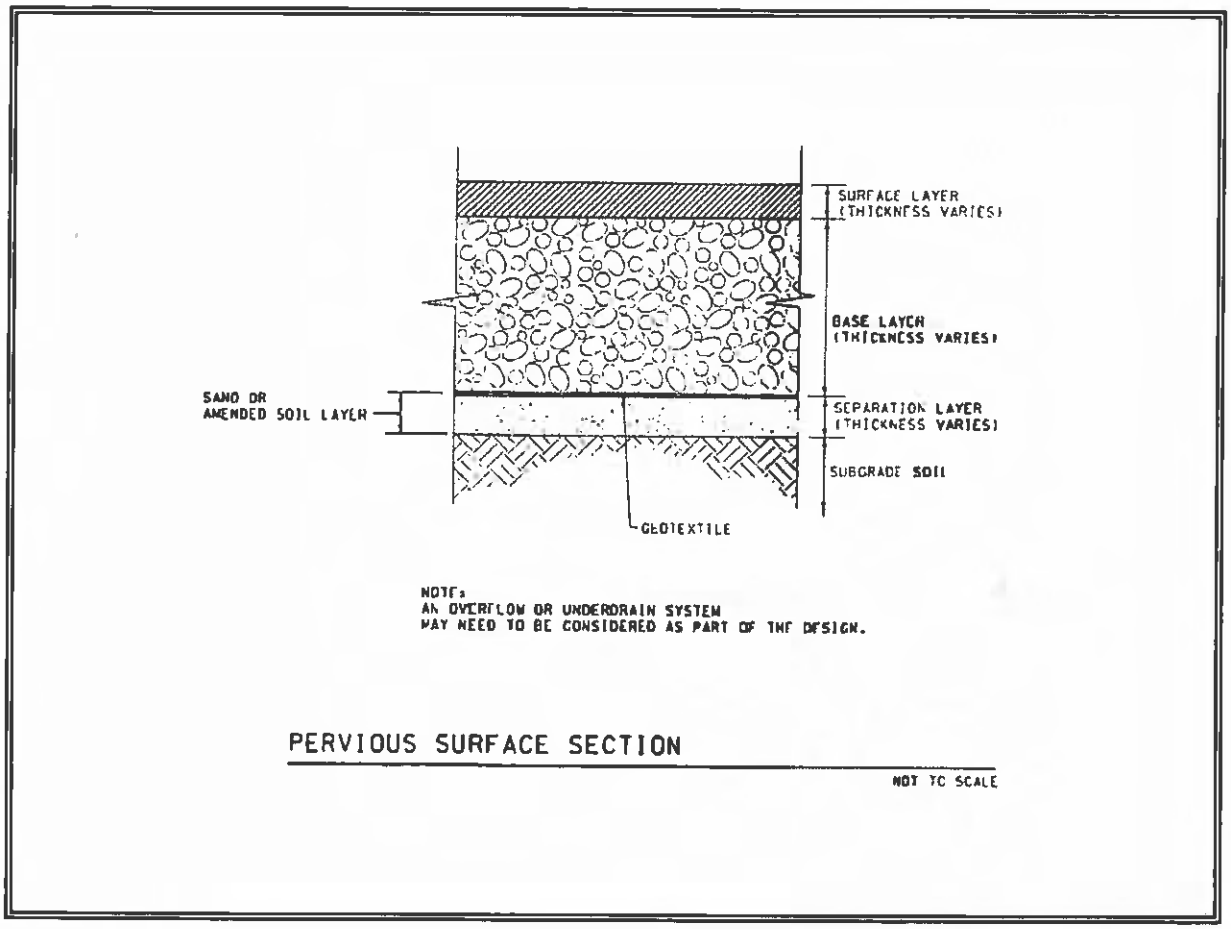


Figure IN.06.1. Permeable pavement surface detail.

Due to the relatively low water content of the concrete mix, an agent may be added to retard concrete setup time. When properly handled and installed, pervious pavement has a higher percentage of void space than conventional pavement (approximately 12% to 21%), which allows rapid percolation of stormwater through the pavement. The initial permeability can commonly exceed 200 inches per hour (Chollack et al. 2001; Mallick et al. 2000).

Asphalt-Based Pervious Pavement Materials

The surface asphalt layer consists of an open-graded asphalt mixture. The depth of the surface layer may increase from a minimum of 4 inches, depending on the required bearing strength and pavement design requirements.

Pervious asphalt pavement consists of an open-graded coarse aggregate. The pervious asphalt creates a surface layer with interconnected voids that provide a high rate of permeability.

Geo-Cell (PVC Containment Cell)

A Geo-Cell surface stabilization system consists of a high-strength, UV-resistant, PVC-celled panel that is 4 inches thick. The celled panels can be filled with soil and covered with turf by installing sod. Base gravel may also be used to fill the celled panels. Both applications create a surface layer.

The Geo-Cell creates an interlock layer with interconnected voids that provide a high rate of permeability of water to an infiltrative base layer. The common application for this system is on slopes, pedestrian/bike paths, parking areas, and low-traffic areas.

Base Layer

The underlying base material is the second component of a permeable surface's design. The base material is a crushed aggregate and provides:

- A stable base for the pavement.
- A high degree of permeability to disperse water downward through the underlying layer to the separation layer.
- A temporary reservoir that slows the migration of water prior to infiltration into the underlying soil.
- Base material is often composed of larger aggregate (1.5 to 2.5 inches) with smaller stone (leveling or choker course) between the larger stone and the wearing course. Typical void space in base layers ranges from 20% to 40% (WSDOT, 2003; Cahill, Adams, and Marm, 2003).
- Depending on the target flow control standard and physical setting, retention or detention requirements can be partially or entirely met in the aggregate base (PSAT, 2005).
- Aggregate base depths of 18 to 36 inches are common depending on storage needs, and they provide the additional benefit of increasing the strength of the wearing course by isolating underlying soil movement and imperfections that may be transmitted to the wearing course (Cahill et al. 2003).

Separation Layer

The third component of permeable systems is the separation layer. This layer consists of a non-woven geotextile fabric and possibly a treatment media base material. A geotextile fabric layer is placed between the base material and the native soil to prevent migration of fine soil particles into the base material, followed by a runoff treatment media layer if required.

- For geotextile, see WSDOT Standard Specification 9-33.

Appendix G: ICPI Permeable Interlocking Concrete Pavements Design Manual from the Vancouver Toyota Final Stormwater Report

APPENDIX G

ICPI Permeable Interlocking Concrete Pavements Design Manual

Vancouver Toyota
Project #14292

MacKay & Sposito, Inc.

Permeable Interlocking Concrete Pavements

Selection • Design • Construction • Maintenance

David R. Smith

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Every effort has been made to present accurate information. However, the recommendations herein are guidelines only and will vary according to local conditions. Professional assistance should be obtained in the design, specifications, and construction with regard to a particular project.

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Taking a project from idea to reality often takes longer than anticipated. This manual is an example. It was proposed to the Technical Committee of the Interlocking Concrete Pavement Institute in the early 1990s. The idea was well-received, but other priorities placed the project on the sidelines. Postponements, however, are often a blessing in disguise. More time was provided for research and for literature to be reviewed.

Thanks goes to the Technical Committee and the Board of Directors of the Interlocking Concrete Pavement Institute for providing me the time and funding to create this manual. Deb Stover of Image Media created the drawings and formatted this publication. Thanks is due to Tom Schueler, from the Center for Watershed Protection in Ellicott City, Maryland. His practical insights and experience with best management practices for abating water pollution, plus the library at his Center were invaluable to bringing this manual into reality.

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Introduction

Compared to North America, Germany consumes about 12 times more per capita of interlocking concrete pavement annually. At least 15% of this is permeable interlocking concrete pavement. Permeable pavements are popular because the German approach to environmental protection is not simply based on attenuating impacts from development. They consider the benefits of the natural environment to society. Development must regenerate, maintain and enhance it. This notion is rooted in their word for environment, *umwelt*. Its meaning embraces the health and well-being of people and nature. North American English would translate the German notion of a healthy environment as environmental quality.

Infiltration trenches have been in use for decades as a means to reduce stormwater runoff and pollution, and to recharge groundwater. Recent experience has demonsrated that they work successfully when runoff is filtered prior to entering the pavement. From an engineering perspective, permeable interlocking concrete pavements are infiltration trenches with paving over them to support pedestrians and vehicles. Therefore, much of this manual is borrowed from literature and experience on infiltration trench design, construction and maintenance. It also borrows engineering from other kinds of permeable pavements.

This manual is written for civil engineers, architects, landscape architects and contractors. Those who use it should be familiar with stormwater management concepts and calculations such as the Rational Method and the National Resources Conservation Service (NRCS) Technical Release (TR) 55. They should be also familiar with the design of best management practices. The references cited with numbers in parentheses (e.g., (2)) throughout the text provide a wealth of information. The Glossary of Terms in Appendix A clarifies the meaning of many words and concepts used throughout the manual.

The manual does not portend to be complete, and it does not provide a "one stop" book for design. Rather, it provides criteria for selecting appropriate sites and the basics for sizing storage areas. Detailed inflow and outflow (stage-discharge) calculations are not covered because they vary considerably from site to site. Calculations must be done by a qualified engineer familiar with hydrology and hydraulics. Construction guidelines are provided as well as a maintenance checklist.

Permeable pavements should be incorporated into broader site designs and regenerative development that improves environmental quality, i.e., the health and well-being of people and nature. Permeable interlocking concrete pavements can do this more elegantly than other pavement, permeable or impervious. For example, they visually annunciate vehicular and pedestrian circulation, reduce micro-climatic temperatures, enhance tree growth and soften harsh visual transitions between building walls and the ground. A multitude of colors and patterns define areas and tie them to surrounding buildings and landscape. Sensitivity to these design concerns will improve the health, safety, and well-being of people and nature.

David R. Smith
Washington, D.C.
2005

Section 1. Selection

Impacts from Impervious Surfaces

Urbanization brings an increasing concentration of pavements, buildings, and other impervious surfaces. They generate additional runoff and pollutants during rainstorms, causing streambank erosion, as well degenerating lakes and polluting sources of drinking water. Increased runoff deprives ground water from being recharged, decreasing the amount of available drinking water in many communities. Figure 1 summarizes the impacts of impervious surfaces.

Increased Imperviousness leads to:	RESULTING IMPACTS				
	Flooding	Habitat Loss	Erosion	Streambed alteration	Channel widening
Increased volume	*	*	*	*	*
Increased peak flow	*	*	*	*	*
Increased peak flow duration	*	*	*	*	*
Increased stream temperature		*			
Decreased base flow		*			
Changes in sediment loadings	*	*	*	*	*

Figure 1. Impacts from increases in impervious surfaces (1).

Stormwater generates intermittent discharges of pollutants into water courses. Since the pollutants in stormwater runoff are not generated by a single, identifiable point source such as a factory, but from many different and spatially separated sources, they are called non-point sources of water pollution. During and after rainstorms, non-point sources of runoff pollution flow in huge quantities that render them untreatable by conventional wastewater treatment plants. In many cases, the receiving water cannot process the overwhelming amount of pollutants either. Therefore, the breadth of pollutants are difficult to control, as well as the extent to which they can be treated through nature's process in a lake, stream, or river.

Best Management Practices (BMPs)

U.S. federal law (2) has mandated that states control non-point source water pollution through the National Pollution Discharge Elimination System (NPDES) program. The law requires, among many things, that states identify and require best management practices, or BMPs, to control non-point source pollution from new development. BMPs are implemented typically through regional and local governments charged with water quality management, planning, and regulation.

BMPs include many technologies and land management practices for reducing the quantity of pollutants in stormwater. They are used in combination at the site, development and watershed scales to attain the maximum benefits to the stormwater drainage system. BMP's are divided into structural and non-structural practices. Structural BMPs capture runoff and rely on gravitational settling and/or the infiltration through a porous medium for pollutant reduction. They include detention dry ponds, wet (retention) ponds, infiltration trenches, sand filtration systems, and permeable and porous pavements. These are often used to offset increases in pollutants caused by new development (3).

Nonstructural BMPs involve a wider scope of practices. They can range from public awareness programs about preventing non-point water pollution to the use of natural techniques such as bio-retention and stormwater wetlands to enhance pollutant removal and promote infiltration of water into the ground.

capacity of the drainage system. Economics suggests that design professionals should study the trade-offs between permeable interlocking concrete pavements and other best management practices for these areas.

Figure 4. Interlocking pavers can allow water through openings created by the paver shape.

Benefits and Limitations of Permeable Interlocking Concrete Pavements

This BMP essentially functions as an infiltration and retention area that can accommodate pedestrians, vehicular parking, and traffic. This combination of functions offers the following benefits:

- Conservation of space on the site and reduction of impervous cover
- Reduction of runoff by as much as 100% from frequent, low-intensity and short duration storms
- Reduction or elimination of unsightly retention basins in other parts of the drainage system
- Promotes tree survival by providing air and water to roots
- Preserves woods and open space that would have been destroyed for detention basins
- Reduces pollutants and improves water quality
- Reduction of runoff temperature
- Reduced peak discharges and stress on storm sewers
- Increased recharge of groundwater
- Reduction of downstream flows and stream bank erosion due to decreased peak flows and volumes
- Reduced overall project development costs due to a reduction in storm sewers and drainage appurtenances
- Eliminates puddling and flooding on parking lots
- Reduced snow plow costs due to rapid ice melt drainage
- Durable, high-strength, low-absorption concrete units resist freeze-thaw and heaving
- Reduces micro-climatic temperatures and contributes to urban heat island reduction
- Eligible for LEED• credits (see *ICPI Tech Spec 16—Achieving LEED• Credits with Segmental Concrete Pavements*)
- Immediately ready for traffic (no waiting days for curing)
- Can be placed over underground stormwater storage systems

Limitations are listed below and are addressed throughout the manual:

- Overall cost compared to other BMPs
- Greater site evaluation and design effort

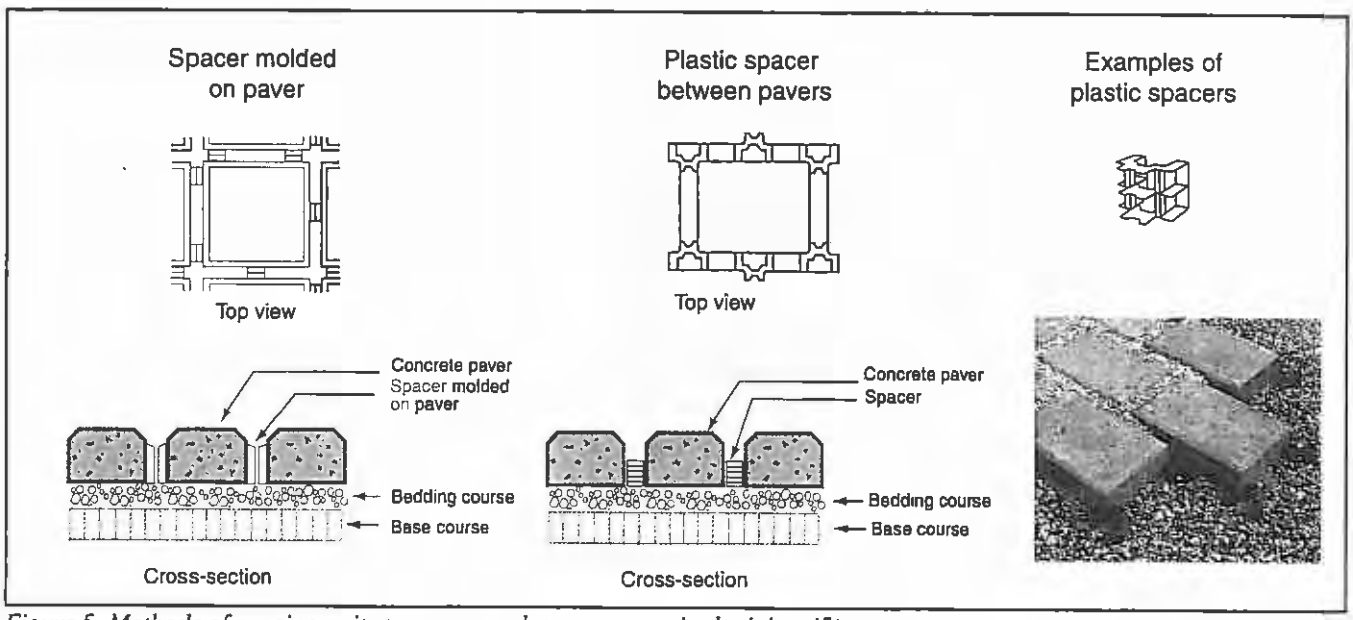

Figure 5. Methods of spacing units to accommodate aggregate in the joints (5).

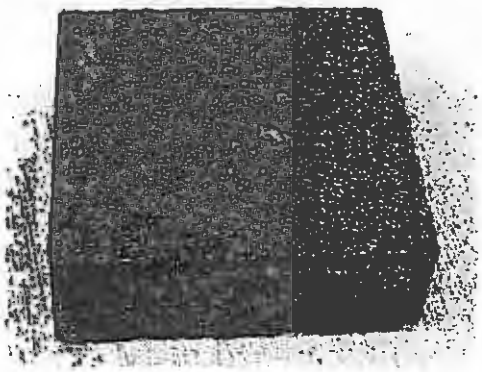

Figure 8. Porous concrete units allow water directly into open-graded concrete.

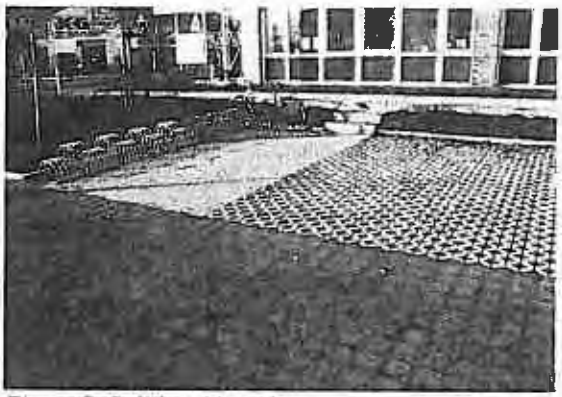

Figure 9. Solid, grid, and permeable interlocking concrete pavers working in harmony to reduce water runoff and pollution, and to provide a more comfortable microclimate.

Infiltration Practices and Municipal Regulatory Approaches

The decision regarding whether to use infiltration practices including permeable pavements is guided by municipal policy and design criteria (plus experience). Site constraints (covered in detail later) are often the most influential factors. Design criteria and regulations vary across the continent due to different rainstorms, geographic locations and land-use development patterns. In most localities, BMPs are designed to a specific storm recurrance (or return period), duration, and intensity, e.g., a 2-year, 24-hour storm of 1.5 in./hr (33 mm/hr or 106 l/s/ha), or capture the volume from the first ½ to 1 in. (13 to 25 mm).

A well-structured municipal stormwater management strategy will consider the influence of the region's range or spectrum of rainfall frequencies on the selection of BMPs. Each region has its own rainfall frequency distribution patterns. Different management practices can handle various volumes of runoff and pollution within portions of this spectrum. Figure 10 illustrates these overlapping ranges of rain storms, expressed in recurrence intervals. It also shows management objectives that can be achieved within those categories of recurrence and rainfall volume.

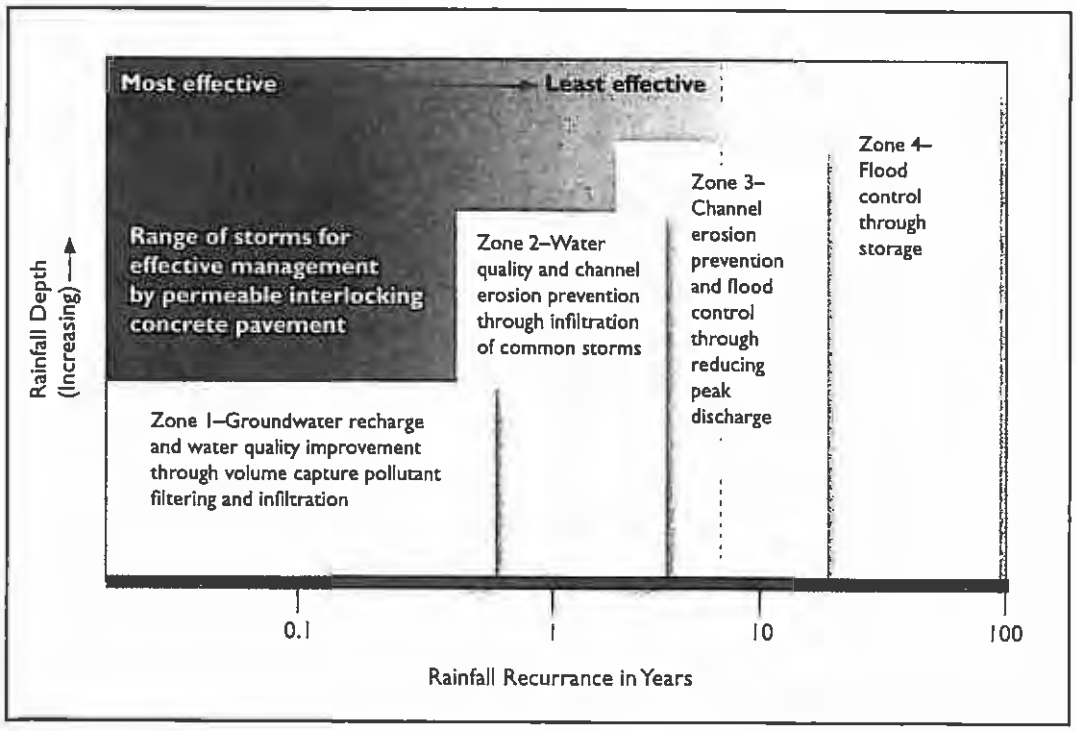

Figure 10. Rainfall Frequency Spectrum defines the distribution of all rainfall events for a region. It is a tool for applying and sizing permeable interlocking concrete pavements and other BMPs to treat pollutants and to control runoff quantities (after Schueler) (9).

Section 2. Design

Municipal Stormwater Management Objectives

The intent of many regional authorities, drainage districts, counties, cities and towns aim at preservation of natural drainage and treatment systems, or limit flows to drainage systems especially if they are working at or near capacity. Some agencies achieve this through a comprehensive stormwater management plan including operation and maintenance administered through stormwater utilities. Some governments use stormwater modeling and field calibration of watersheds and watercourses in their jurisdiction. Modeling can range from simple formulas like the Rational Method, NRCS TR-55 or more sophisticated models such as HEC or EPA SWMM. These results inform drainage design guidelines for specific site development proposals brought to a government for approval. Sophisticated modeling can also demonstrate specific downstream impacts from a specific development proposal.

In approaching site design, municipalities incorporate some or all of the following design goals for managing stormwater.

1. Reduce the generation of additional stormwater and pollutants by restricting the growth of impervious surfaces.
2. Treat runoff to remove a given percentage of a pollutant or pollutants from the average annual post-development load. Target pollutant reductions can include total suspended solids (TSS) (typically 80% reduction) and total phosphorous (TS) (typically 40% reduction) as these are primary indicators of water quality. Reductions are measured on a mass basis.
3. Capture and treat a specific water quality volume defined as the initial depth of rainfall on a site (typically ranging from 0.75 in to 1.5 in. or 18 to 40 mm). This volume generally contains the highest amount of pollutants.
4. Enhance stream channel protection through extended detention (and infiltration) of runoff volume from a given storm event, e.g., a 1 or 2 year 24-hour storm. The difference in volumes between pre- and post development is often detained, infiltrated and/or slowly released.
5. Provide streambank erosion prevention measures such as energy dissipation and velocity control plus preservation of vegetative buffers along a stream.
6. Reduce overbank flooding through reducing the post-development peak discharge rate to the pre-development rate for a given storm, e.g., a 25-year, 24-hour event.
7. Reduce the risk of extreme flooding by controlling and/or safely conveying the 100-year, 24-hour return frequency storm event. This goal is also supported by preserving existing and future floodplain areas from development or restricting it in them as much as possible.
8. Maintain groundwater recharge rates to maintain stream flows and ecosystems as well as recharging aquifers.
9. Prevent erosion and sedimentation from construction through control practices provided on site development plans inspected during construction.

Permeable interlocking concrete pavements can play an important role in reaching all of these goals. These pavements help meet these goals with full, partial or no exfiltration of the open-graded stone base into the soil subgrade.

Figure 11. Portland, Oregon renovated streets with about 20,000 sf (2,000 m²) of permeable interlocking concrete pavement after water and sewer line repairs in an older neighborhood. The city incorporated modeling to evaluate this pavement. The pavement decreased combined sewer overflows to the waste treatment plant and discharges into the Willamette River.

- Industrial sites that do not receive hazardous materials, i.e., where there is no risk to groundwater or soils from spills.
- Storage areas for shipping containers with non-hazardous contents.
- The impervious area does not exceed five times the area of the permeable pavement receiving the runoff.
- The estimated depth from the bottom of the pavement base to the high level of the water table is greater than 2 feet (0.6 m). Greater depths may be required to obtain additional filtering of pollutants through the soil.
- The pavement is downslope from building foundations, and the foundations have piped drainage at the footers.
- The slope of the permeable pavement surface is at least 1% and no greater than 5%.
- Land surrounding and draining into the pavement does not exceed 20% slope.
- At least 100 ft (30 m) should be maintained between permeable pavements and water supply wells, streams, and wetlands. (Local jurisdictions may provide additional guidance or regulations.)
- Sites where the owner can meet maintenance requirements (see maintenance section).
- Sites where there will not be an increase in impervious cover draining into the pavement (unless the pavement is designed to infiltrate and store runoff from future increases in impervious cover).
- Sites where space constraints, high land prices, and/or runoff from additional development make permeable interlocking concrete pavements a cost-effective solution.

Permeable interlocking concrete pavements are not recommended on any site classified as a stormwater hotspot, i.e., if there is any risk that stormwater can infiltrate and contaminate groundwater. These land uses and activities may include the following:

- Vehicle salvage yards, recycling facilities, fueling stations, service and maintenance facilities, equipment and cleaning facilities
- Fleet storage areas (bus, truck, etc.)
- Commercial marina service and maintenance areas
- Outdoor liquid container storage areas
- Outdoor loading/unloading facilities
- Public works materials/equipment storage areas
- Industrial facilities that generate or store hazardous materials
- Storage areas for commercial shipping containers with contents that could damage groundwater and soil
- Land uses that drain pesticides and/or fertilizers into permeable pavements (e.g., agricultural land, golf courses, etc.)
- Other land uses and activities as designated by an appropriate review authority

Design Considerations for Pedestrians and Disabled Persons

Before a parking lot or plaza is constructed, existing pedestrian paths across the lot should be studied and defined. Vehicle lanes, parking spaces, pedestrian paths, and spaces for disabled persons can be delineated with solid concrete pavers. Paths with solid units will make walking more comfortable, especially for pedestrians with high-heeled shoes and for the elderly. Likewise, parking spaces accessible to disabled persons and for bicycles should be marked with solid pavers. Permeable interlocking concrete pavers with openings or wide joints should not be used in disabled-accessible parking spaces or on pedestrian ramps at intersections.

Infiltration Rates of Permeable Interlocking Concrete Pavement Systems

A common error in designing permeable interlocking concrete pavements is assuming that the amount or percent of

open surface area is equal to the percent of perviousness. For example, an 18% open surface area is incorrectly assumed to be 18% pervious, or 82% impervious. The perviousness and amount of infiltration are dependent on the infiltration rates of joint filling material, bedding layer, and base materials, not the percentage of surface open area.

Compared to soils, permeable interlocking concrete pavements have a very high degree of infiltration. For example, a clay soil classified as CL using the Unified Soil Classification System might have an infiltration rate in the order of 1.4×10^{-5} in./hr (10^{-9} m/sec). A silty sand (SM) could have 1.4×10^{-3} in./hr (10^{-7} m/sec) infiltration rate. Open-graded, crushed aggregate placed in the openings of permeable interlocking concrete pavements will have an initial infiltration over 500 in./hr (over 10^{-3} m/sec), i.e, 10,000 times greater than the sandy soil and 100,000 times greater than the clay soil. The open-graded base material has even higher infiltration, typically 500 to 2,000 in./hr (10^{-3} to 10^{-2} m/sec). Therefore, the small percentage of open surface area is capable of providing a large amount of infiltration into the pavement.

Regardless of the high infiltration rate of the aggregates used in the openings and base, a key consideration is the lifetime *design* infiltration of the *entire* pavement cross-section, including the soil subgrade. Its infiltration rate is difficult to predict over time. There can be short-term variations from different amounts of antecedent water in it, and long-term reductions of infiltration from partially clogged surface or base, geotextiles or soil subgrade. So a conservative approach should always be taken when establishing the design infiltration rate of the pavement system.

Studies on permeable interlocking concrete pavers have attempted to estimate their long-term infiltration performance. Permeable concrete units (made with no fine aggregates) demonstrate lowest average permeability. Interlocking shapes with openings or those with enlarged permeable joints offer substantially higher infiltration performance over the long term.

Research on permeable pavements made with solid, nonporous units provides some guidance on long-term infiltration rates. German studies (6)(7)(8)(12), ICPI (43), and a review of the literature by Ferguson (44) reviewed parking lots with open-graded materials in the paver openings over an open-graded base. They showed a high initial infiltration when new and a decrease and leveling off as they aged. The decrease in infilitration is natural and is due to the deposit of fine materials in the aggregate fill and clogging of the base and geotextiles.

When tested, new pavements demonstrated very high surface infiltration rates of almost 9 in./hr (6×10^{-5} m/sec) and two four-year old parking lots indicated rates of about 3 in./hr (2×10^{-5} m/sec). Lower rates were exhibited on pavements where openings were filled with sand or aggregate and itinerant vegetation. In another study of two and five-year old parking lots, the infiltration rates were about 6 and 5 in./hr (4 and 3.5×10^{-5} m/sec) respectively. Infiltration was measured over approximately one hour for these two studies. In an ICPI study (44) ten sites indicated 1½ in./hr to over 780 in./hr. The lowest infiltration rates were sites clogged with fines.

The results of these studies confirm that the long-term infiltration rate depends on the intensity of use and the degree to which the surface and base receive sediment. This is also confirmed in the literature on the performance of infiltration trenches. Since there are infiltration differences between initial and long-term performance, construction, plus inevitable clogging, **a conservative design rate of 3 in./hr (2.1×10^{-5} m/sec or 210 L/sec/hectare) can be used as the basis for the design surface infiltration rate for a 20-year life.** This design infiltration rate will take in most storms.

Site Design Data

Desktop Assessment

A preliminary assessment should be conducted prior to detailed site and hydrological design. This initial assessment includes a review of the following:

- Underlying geology and soils maps
- Identifying the NRCS hydrologic soil groups (A, B, C, D)
- Verifying history of fill soil or previous disturbances or compaction
- Review of topographical maps and identifying drainage patterns
- Identifying streams, wetlands, wells and structures
- Confirming absence of stormwater hotspots
- Identifying current and future land uses draining onto the site

U.S.C. soil Classification	Typical ranges for Coefficient of Permeability, in./hour (approximate m/s)	Relative Permeability when compacted and saturated	Shearing strength when compacted	Compressibility	Typical CBR Range
GW-well graded gravels	1.3 to 137 (10^{-5} to 10^{-3})	Pervious	Excellent	Negligible	30-80
GP-poorly graded gravels	6.8 to 137 (5×10^{-5} to 10^{-3})	Very pervious	Good	Negligible	20-60
GM-silty gravels	1.3×10^{-4} to 13.5 (10^{-8} to 10^{-4})	Semi-pervious to impervious	Good	Negligible	20-60
GC-clayey gravel	1.3×10^{-4} to 1.3×10^{-2} (10^{-8} to 10^{-6})	Impervious	Good to fair	Very low	20-40
SW-well graded sands	0.7 to 68 (5×10^{-6} to 5×10^{-4})	Pervious	Excellent	Negligible	10-40
SP-poorly graded sands	0.07 to 0.7 (5×10^{-7} to 5×10^{-6})	Pervious to semi-pervious	Good	Very low	10-40
SM-silty sands	1.3×10^{-4} to 0.7 (10^{-9} to 5×10^{-6})	Semi-pervious to impervious	Good	Low	10-40
SC-clayey sands	1.3×10^{-5} to 0.7 (10^{-9} to 5×10^{-6})	Impervious	Good to fair	Low	5-20
ML-inorganic silts of low plasticity	1.3×10^{-5} to 0.07 (10^{-9} to 5×10^{-7})	Impervious	Fair	Medium	2-15
CL-inorganic clays of low plasticity	1.3×10^{-5} to 1.3×10^{-3} (10^{-9} to 10^{-8})	Impervious	Fair	Medium	2-5
OL-organic silts of low plasticity	1.3×10^{-5} to 1.3×10^{-2} (10^{-9} to 10^{-6})	Impervious	Poor	Medium	2-5
MH-inorganic silts of high plasticity	1.3×10^{-6} to 1.3×10^{-5} (10^{-10} to 10^{-9})	Very impervious	Fair to poor	High	2-10
CH-inorganic clays of high plasticity	1.3×10^{-7} to 1.3×10^{-5} (10^{-11} to 10^{-9})	Very impervious	Poor	High	2-5
OH-organic clays of high plasticity	Not appropriate under permeable interlocking concrete pavements				
PT-Peat, mulch, soils with high organic content	Not appropriate under permeable interlocking concrete pavements				

Figure 15. Suitability of soils (per the Unified Soils Classification System) for infiltration of stormwater and bearing capacity (21)(22)(23). This table provides general guidance. Testing and evaluation of soils are recommended.

When designing for full exfiltration in vehicular applications a minimum tested soil infiltration rate of 0.52 in./hr (3.7×10^{-6} m/sec) is required. Some sites may require higher rates and there may be cases where lower rates are used. Local requirements for the design of infiltration trenches may also specify minimum rates.

Soils with a tested permeability equal to or greater than 0.52 in./hr (3.7×10^{-6} m/sec) usually will be gravel, sand, loamy sand, sandy loam, loam, and silt loam. These are usually soils with no more than 10-12% passing the No. 200 (0.075 mm) sieve. These are characterized as A and B hydrologic group soils using the NRCS classification system. Silt and clay soils will likely have lower permeability and not be suitable for full exfiltration from an open-graded base. For cold climates in the northern U.S. and Canada, the lowest recommended design infiltration rate for the soil subgrade is 0.25 in./hr (2×10^{-6} m/sec).

used for filters, the use of geotextiles is more common. Figure 16 provides geotextile filter criteria from the U.S. Federal Highway Administration (FHWA) (25) and the American Association of State Highway and Transportation Officials (AASHTO) (26).

An aggregate subbase consisting of ASTM No. 2 crushed stone can be used in lieu of geotextile. This material ranges in size from 2 ½ in. to ¾ in. (63 to 19 mm) and provides a stable working platform for construction equipment to spread and compact the No. 57 stone base. After compacting the No. 2 stone, No. 57 stone is spread and compacted or choked into the openings of the No. 2 stone which rests directly on the soil subgrade.

Materials for the Base, Bedding and Openings

The following data is required on materials for the base and subbase, bedding course, and aggregate in the pavement openings:

1. Sieve analysis, including washed gradations per ASTM C 136.
2. Void space in percent for the open-graded base per ASTM C 29.

Crushed stone, open-graded subbase and base—This material should be a hard, durable rock with 90% fractured faces and a Los Angeles (LA) Abrasion of < 40. A minimum effective porosity of 0.32 and a design CBR of at least 80% are recommended. A water storage capacity of open-graded

U.S. Federal Highway Administration (FHWA)

For fined grained soils **with more than 50% passing** the No. 200 (0.075 mm) sieve:

Woven geotextiles: Apparent Opening Size (AOS) $\leq D_{85}$

Nonwoven geotextiles: $AOS_{geotextile} \leq 1.8D_{85\ soil}$

AOS $\leq$ 0.3 mm or $\geq$ No. 50 sieve

For granular soils with 50% or less passing the No. 200 (0.075 mm) sieve:

All geotextiles $AOS_{geotextile} \leq B \times D_{85soil}$

Where:

$B = 1$ for $2 \geq C_U \geq 8$

$B = 0.5$ for $2 < C_U < 4$

$B = 8/C_U$ for $4 < C_U < 8$

$C_U = D_{60}/D_{10}$

Permeability criteria: k (fabric) $\geq$ k (soil)

Clogging criteria

Woven: Percent of open area $\geq$ 4%

Nonwoven: Porosity $\geq$ 30%

American Association of State Highway and Transportation Officials (AASHTO)

For soils $\leq$ 50% passing the No. 200 (0.075 mm) sieve:

$O_{95} < 0.59$ mm ($AOS_{fabric} \geq$ No. 30 sieve)

For soils > 50% passing the No. 200 sieve:

$O_{95} < 0.30$ mm ($AOS_{fabric} \geq$ No. 50 sieve)

Notes:

1. D_x is particle size at which x percent of the particles are finer. Determined from gradation curve. Example: D_{10} is the size particle of a soil or aggregate gradation for which 10% of the particles are smaller and 90% are coarser.
2. O_x is geotextile size corresponding to x particle size base on dry glass bead sieving. Hence O_{95} is the geotextile size opening for which 95% of the holes are smaller.
3. AOS is apparent opening size is essentially the same but normally defined as a sieve number rather than as a size (ASTM D 4751). POA is percent open area for (woven fabrics only). Permeability, k of the soil and geotextile (nonwoven only) are designated k_S and k_G respectively.

Figure 16. Geotextile filter criteria

surface of the No. 57 and provides some filtering of water. Therefore the No. 8 choke stone should meet the following criteria:

$$D_{15\ \text{open-graded base}} / D_{50\ \text{choke stone}} < 5 \text{ and } D_{50\ \text{open-graded}} / D_{50\ \text{choke stone}} > 2$$

D_x is the particle size at which x percent of the particles are finer. For example, D_{15} is the particle size of a soil or aggregate gradation for which 15% of the particles are smaller and 85% are coarser.

If the bedding material can't meet this filter criteria (i.e., the bedding stone is smaller or the base material is larger), a layer of geotextile may be used between the bedding and base course. This adds stability to the structure. Geotextile has been shown to accelerate digestion of oils through moisture and microbial action (45).

Besides use as a bedding material, No. 8 crushed stone aggregate is also recommended for fill material in the paver openings. Smaller sized aggregate such as No. 89 may be needed to enter narrow joints between interlocking shapes. Ferguson (43) provides additional filter criteria for aggregate layers. The void space in the bedding and joints is not considered in water storage calculations. Nonetheless, they provide an additional factor of safety since they have capacity for storing water.

Concrete units for permeable pavement—The following data is needed on the pavers:

1. Minimum thickness = $3^1/_8$ in. (80 mm) for vehicular applications and $2^3/_8$ in. (60 mm) for pedestrian applications. For pedestrian applications joint widths should be no greater than $^3/_4$ in. (15mm). Smaller stone such as No. 9 or No. 10 should be considered for filling the joints as this will lend greater interlock.

2. Percent of open area of the surface.

3. Test results indicating conformance to ASTM C 936, *Standard Specification for Solid Interlocking Concrete Paving Units* (27), or CSA A231.2, *Precast Concrete Pavers* (28) as appropriate. If the dimensions of the units are larger than those stated in these standards, then CSA A231.1, *Precast Concrete Paving Slabs* (29) is recommended as a product standard.

Sizing an Open-Graded Base for Stormwater Infiltration and Storage

The following design method is adapted from *Standard Specifications for Infiltration Practices* (30) and the *Maryland Stormwater Manual* published by the State of Maryland, Department of the Environment (31). The procedure is from "Method for Designing Infiltration Structures." This method assumes familiarity with NRCS TR 55 method (32) for calculating stormwater runoff. References 11, 33, 34, and 35 provide other methods. Provinces, states, and cities may mandate the use of other methods. The Maryland method is provided because it has been refined over many years and it illustrates important aspects of infiltration design.

Like porous asphalt pavement, permeable interlocking concrete pavement relies on an open-graded aggregate base into which water rapidly infiltrates for storage. The pavement base functions as an underground detention structure. Therefore, pavement base storage can be designed with the same methods as those used for stormwater management ponds. The design method in this section assumes full exfiltration, e.g., removal of water from the base by infiltration into the underlying soil subgrade.

The catchment for permeable interlocking concrete pavement consists of the surface area of the pavement and an area that contributes runoff to it. A schematic cross-section and the design parameters are shown in Figure 17. The base is sized to store the runoff volume from the pavement area and the adjacent contributing areas.

Soil with infiltration rates or permeability less than 0.27 in./hr (2×10^{-6} m/sec) are generally silt loam, loam, sandy loam, loamy sand, and sand. Soils with lower permeability will limit the flow of water through the soil. They will require a high ratio of bottom surface area to storage volume. Therefore, careful consideration should be given to designing drain pipes to remove excess water in these situations.

The method described below does not provide guidance on drain pipe design within the base. This can be found in reference 35. Reference 36 includes methods for determining the diameter and spacing of pipes in open-graded bases for highway pavement drainage, as well as general guidance on pavement drainage design. This method accounts for monthly variations in the water generated from background flows in the soil and infiltration area, as well as that from the runoff from the

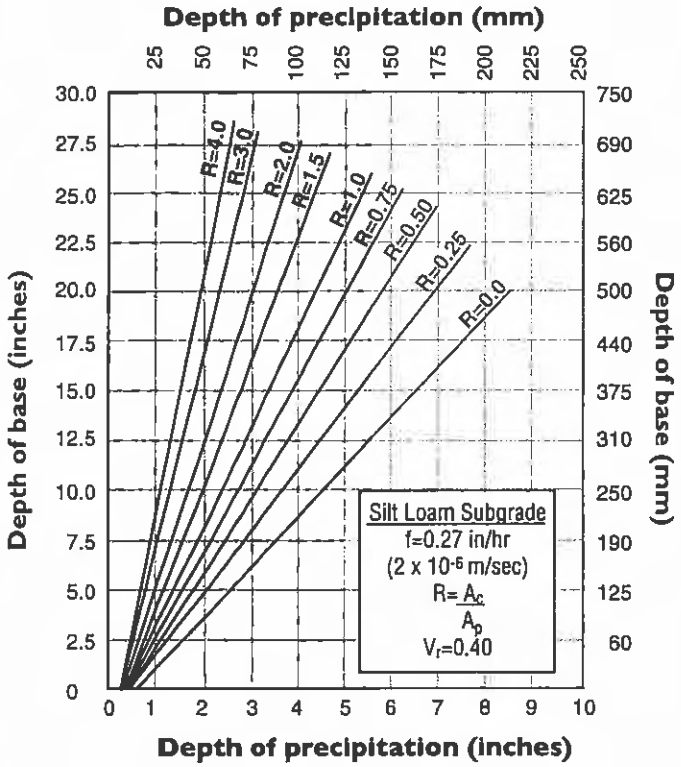

Figure 20. Open-graded base and subbase depth for silt loam subgrade.

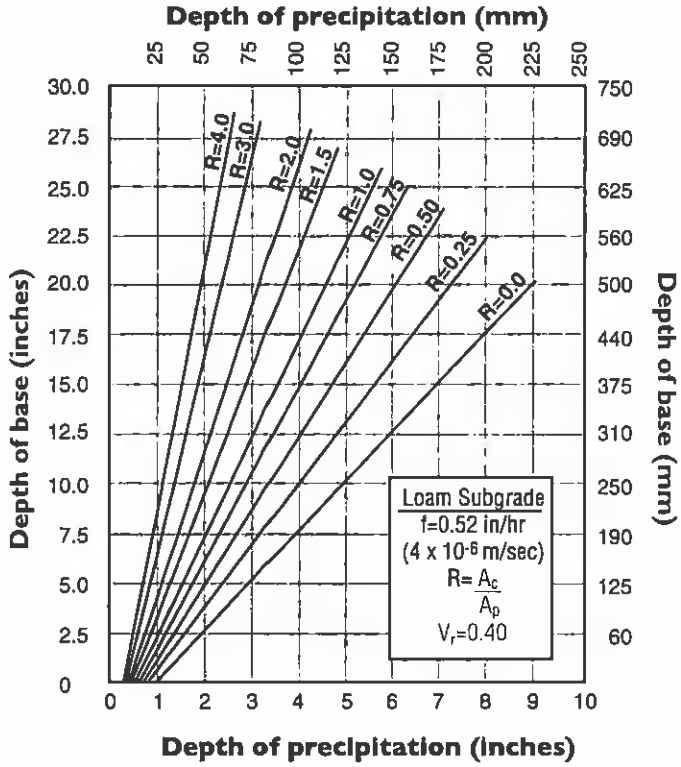

Figure 21. Open-graded base and subbase depth for loam subgrade.

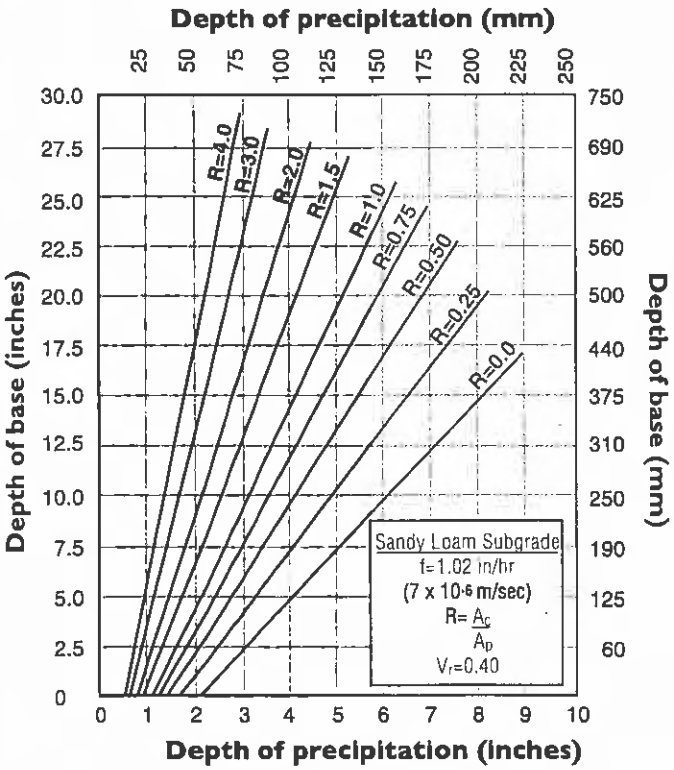

Figure 22. Open-graded base and subbase depth in sandy loam subgrade.

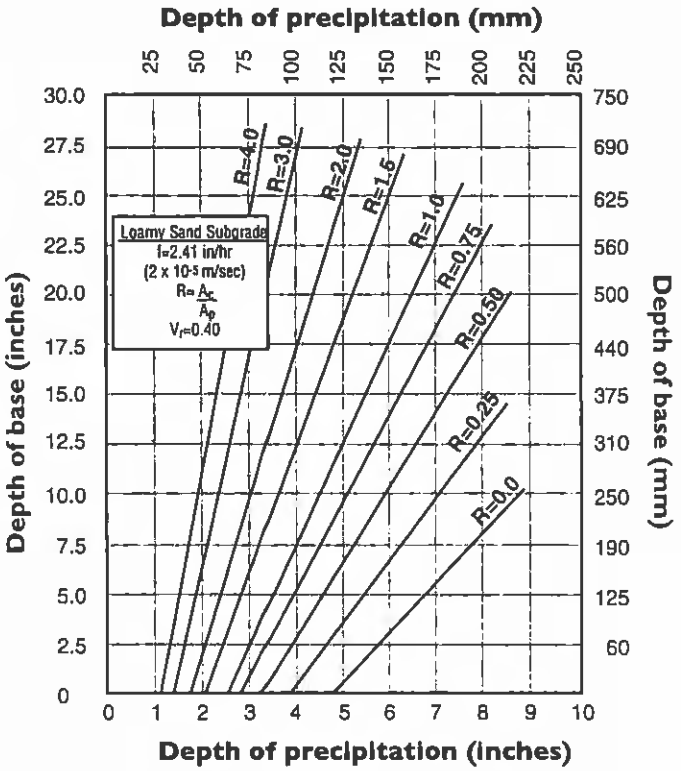

Figure 23. Open-graded base and subbase depth for loamy sand subgrade.

Note: In Figures 20 – 23 the CN = 98 of contribuitng areas

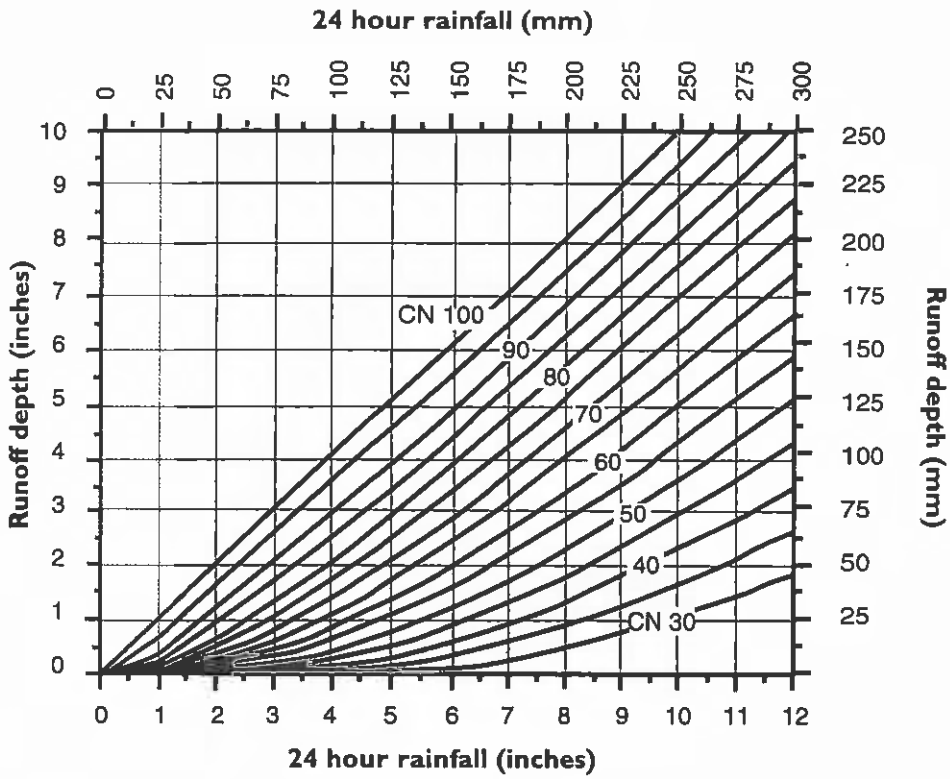

Figure 24. NRCS chart for finding runoff depth for various curve numbers.

an impervious area, CN = 98. The NRCS method offers a chart to assist in finding the depth of runoff from a given 24 hr. design storm for less than completely impervious areas, i.e., curve numbers lower than 98. This chart shown in Figure 24 is for 24-hour storms since many localities use this event for storm water management.

Design Procedure—There are two methods to design the base storage area. The first method computes the minimum depth of the base, given the area of the permeable pavement. This is called the *minimum depth method.* The other is compute the minimum surface area of the permeable pavement given the required design depth of the base. This is the *minimum area method.* The minimum depth method generally will be more frequently used.

Minimum Depth Method

1. From the selected design rainfall (**P**) and the NRCS runoff curve number, compute the increased runoff volume from the contributing area (ΔQ_c).

2. Compute the depth of the aggregate base (d_p) from Equation 3:

   Figures 20 through 23 may be used to determine the approximate stone base and subbase depth if the total runoff depth (Q_c) is to be stored.

3. Compute the maximum allowable depth (d_{max}) of the aggregate base and subbase by the feasibility formula:

$$d_{max} = f \times T_s / V_r$$

   where d_p must be less than or equal to d_{max} and at least 2 feet (0.6 m) above the seasonal high ground water table. If d_p does not satisfy this criteria, the surface area of the permeable pavement must be increased or a smaller design storm must be selected.

Minimum Area Method

1. From the selected design rainfall (**P**) and the NRCS runoff curve number for the contributing area to be drained, compute the increased runoff depth from the contributing area (DQ_c).

Step 5—Compute the maximum allowable depth (d_{max}) of the base by the feasibility formula:

$$d_{max} = f \times T_s / V_r$$

where d_p must be less than or equal to d_{max} and at least 2 feet (0.6 m) above the seasonal high ground water table. If d_p does not satisfy this criteria, the surface area of the permeable pavement must be increased or a smaller design storm must be selected. The drainage time is 24 hours.

$$d_{max} = 0.0425 \text{ ft/hr} \times 24 \text{ hr}/0.40 = 2.5 \text{ ft } (0.75 \text{ m})$$

Step 6—Check the structural base thickness to be sure it has sufficient thickness to meet the storage requirements plus function as a base for 300,000 ESALs. The Frost Condition side of Figure 18 under sand with interpolation yields a thickness close to 18 in. (0.45 m). This is slightly thicker than what is required, 16.3 in. (0.4 m), to infiltrate and store the water in the base.

In no case should the structural thickness be reduced for the sake of economy. In some cases, the designer may wish to provide a thicker base due to expected heavy loads, or from spring thawing conditions that leave the soil completely saturated and weak. A frost protection layer of sand with drains can be placed under the base (separated by geotextiles) to reduce heave from highly susceptible soils in freeze-thaw conditions. This layer of sand offers additional filtering and reduction of pollutants, and construction details are discussed elsewhere.

It is very unlikely that the base and leveling courses will heave from ice. There is typically sufficient void space in them to allow frozen water to expand (9%) without heaving because it is rare that the base will be entirely and thoroughly saturated when freezing.

Step 7—Check to be sure the bottom of the base is at least 2 ft (0.6 m) from the seasonal high water table. The total thickness of the pavement will be:

3 1/8 in. (80 mm) thick concrete pavers
2 in. (50 mm) No. 8 stone leveling course
18 in. (450 mm); 4 in. (100 mm) No. 57 base and 14 in. (350 mm) No. 2 subbase
Total thickness = 23 in. (570 mm)

Approximately two feet (0.6 m) minus 10 ft (3 m) leaves 8 ft (2.4 m) to the top of the seasonal high water table. This is greater than the 2 ft (0.6 m) minimum distance required.

A somewhat hidden consideration is the storage capacity of the layer of No. 8 crushed stone. As a factor of safety, the void space in the No. 8 layer is not part of the storage calculations. This additional volume in the leveling course can serve as a safety buffer for storage in heavy rainfall.

Step 8—Check geotextile filter criteria. Sieve analysis of the soil subgrade showed that 4% passed the No. 200 (0.075 mm) sieve, and the gradation also showed the following:

	D_{10}	D_{15}	D_{50}	D_{60}	D_{85}
Soil subgrade	**0.10**	**0.12**	**0.25**	**0.32**	**0.63**

If geotextile is used the following criteria apply.
FHWA geotextile filter criteria—For granular soils with ≤50% passing the No. 200 (0.075 mm) sieve, the following selection criteria is used for geotextiles taken from Figure 18.

All geotextiles: $AOS_{geotextile} \leq B \times D_{85\,(soil)}$

$C_U = D_{60}/D_{10} = 0.32/0.10 = 3.2$

Where:
B = 1 for $2 \geq C_U \geq 8$, 3.2 is okay.
B = 0.5 for $2 < C_U < 4$, 3.2 is okay.
B = $8/C_U$ for $4 < C_U < 8$

8/3.2 = 2.5 which does not satisfy 4 < 2.5 < 8. (Do not use for **B**.)

Figure 25. Curbing and drainage swale handle flows that exceed the design rainstorm.

Cold Climate Design

The following design considerations apply to freezing climates with extended winters having large, rapid volumes of snow melt in the late winter and early spring. These areas are mostly in the northern U.S. and Canada (39).

1. Permeable interlocking concrete pavements should not be used in permafrost regions.
2. Chlorides and road abrasives (sand) can be concentrated in snowmelt. It's impossible for any best management practice, including permeable interlocking concrete pavements, to remove chlorides found in deicing materials. In addition, road sand can clog and reduce the infiltration capacity of these pavements. It is best to stockpile snow with chlorides and/or sand away from permeable interlocking concrete pavements. Possible locations include parking lot islands or bioretention areas.
3. If salts are used for deicing, then the groundwater should be monitored for chlorides. This can be done through sampling water in observation wells located in the pavement base and soil. Chloride levels in the samples should be compared to local or national criteria for the particular use of the water in the receiving lake, stream, or river (e.g., drinking water, recreation, fishing, etc.).
4. When the frost depth exceeds 3 ft. (1 m), all permeable parking lots should be set back from the subgrade of adjacent roads by at least 20 ft (6 m). This will reduce the potential for frost lenses and heaving of soil under the roadway.
5. Plowed snow piles and snow melt should not be directed to permeable interlocking concrete pavements if groundwater contamination from chlorides is a concern. However, this may not be avoidable in some situations. If high chloride concentrations in the runoff and groundwater are anticipated, then consideration should be given to using one or two design options below:
   (a) Runoff from snow melt can be diverted from the pavement during the winter. The diversion of runoff away from the pavement is typically through channels or pipes. Pipe

Pollutant Category Source	Solids	Nutrients	Bacteria	Dissolved oxygen demands	Metals	Oils (PAHs)* SOCs*
Soil erosion	*	*		*	*	
Cleared vegetation	*	*		*		
Fertilizers		*				
Human waste	*	*	*	*		
Animal waste	*	*	*	*		
Vehicle fuels and fluids	*			*	*	*
Fuel combustion		*			*	
Vehicle wear	*			*	*	
Industrial/household chemicals	*	*	*	*	*	*
Industrial processes	*	*	*	*		*
Paints and preservatives				*	*	*
Pesticides				*	*	

PAHs = polynuclear aromatic hydrocarbons
SOCs = synthetic organic compounds

Figure 26. Common sources of pollution in urban stormwater runoff (3)

Other approaches to reducing pollutants include filtering runoff from impervious areas through sand filters to help reduce sediment and oils. The typical application involves a small area that pretreats runoff prior to entering a detention or retention pond. The sand absorbs and helps treat the concentrated pollutants found in the first flush of a rainstorm. Sand filtering system design is found in reference 9.

Figure 28. Besides expected decreases in stormwater, runoff monitored from permeable interlocking concrete pavement projects such as Glen Brook Green Subdivision, Waterford, Connecticut in the Jordan Cove Watershed demonstrate substantial reductions of pollutants compared to those from conventional pavements.

The U.S. Environmental Protection Agency recognizes permeable interlocking concrete pavement as a BMP in reducing non-point source pollutants in runoff. In 2003 the U.S. EPA issued a New Development Management Measure for protection of coastal waters near urban areas (47). These measures appear in some non-coastal state and local BMP or stormwater design manuals.

Key measures require at least 80% reduction of total suspended solids (TSS) on an average annual basis, or post-development TSS loadings not exceeding predevelopment loadings. As part of that management measure for new development, to the extent practicable, post-development peak runoff rates and volumes should be similar to predevelopment levels based on rainfall from a 2-year, 24 hour storm. This helps reduce or prevent streambank erosion and scouring.

Permeable interlocking concrete pavement can achieve this reduction in peak flows and volumes. Regarding TSS reduction, several studies have demonstrated reductions at or near the 80% level:

- Rushton (48) monitored runoff and pollutants in a Tampa, Florida parking lot for two years. Eight sub-catchments included permeable pavement, concrete and asphalt pavement. Permeable pavement had the highest load removal efficiency for ammonia, nitrate, total nitrogen, total suspended solids, copper, iron, lead, manganese and zinc. Most removal rates exceeded 75%.

- Bean (49) compared runoff quantities and quality over 18 months from a small asphalt and permeable interlocking concrete pavement parking lot with an open-graded aggregate base at
  a bakery in Goldsboro, North Carolina. The study summarizes the statistical mean pollutant concentrations from 14 rainstorms and illustrates substantial pollutant reductions including 75% for TSS.

- Scholes (50) reports on pollutant removal efficiencies of various BMPs in the United Kingdom and identifies permeable paving has having an average of 82% removal efficiency with data ranging between 64% and 100% removal rates.

- Clausen (51) monitored runoff from driveways for one year in a small residential subdivision in Waterford, Connecticut. The driveways consisted of asphalt, crushed stone and permeable interlocking concrete pavement (over a dense-graded base). Annual pollutant export in kg/ha/yr was 86% lower on the paver driveways than on the asphalt ones.

- James (52) examined surface runoff from nine rainstorms over four months from asphalt, concrete pavers and permeable interlocking concrete pavers. He also measured pollutants in

Section 3. Construction

Reducing Clogging

Preventing and diverting sediment from entering the base and pavement surface during construction ***must be the highest priority***. Extra care must be applied to keeping sediment completely away from the area. Simple practices such as keeping muddy construction equipment away from the area, installing silt fences, staged excavation, and temporary drainage swales that divert runoff away from the area will make the difference between a pavement that infiltrates well or poorly. Moreover, the pavement should not receive runoff until the entire contributing drainage area is stabilized. This should be included in the construction drawings and specifications.

> Preventing and diverting **sediment** from entering the **base** and pavement **surface** during construction **must be** the **highest** priority.

One technique for reducing silting and clogging of soil during construction is to excavate the base within 6 in. (150 mm) of the final bottom elevation. This area can contain water during storms over the construction period and drain via temporary drain pipes. Heavy equipment should be kept from this area to prevent compaction. If equipment needs to traverse the bottom of the excavation, tracked vehicles can reduce the risk of soil compaction. As the project progresses, sediment and the remaining soil depth can be excavated to the final grade prior to installing the subbase and base stone. Depending on the project design, this technique might eliminate the need for a separate sediment basin during construction.

Soil Compaction

If the initial undisturbed soil infiltration can be maintained during excavation and construction, there is a high probability that the base will drain as designed. If the soil is inadvertently compacted by equipment during construction, there will be a substantial loss of infiltration. A loss is acceptable if the infiltration rate of the soil *when compacted* was initially considered during design and in drainage calculations.

Compaction of low CBR soils (<4%) may be necessary to attain sufficient structural support and to minimize rutting from vehicular traffic. These soils should be compacted to at least 95% of standard Proctor density. Drains in the open-graded base will likely be required to remove water since compaction will greatly reduce the soil's permeability.

Geotextiles

Geotextiles are used in some permeable pavement applications and are optional when using a No. 2 aggregate subbase. Specifications and minimum physical requirements for geotextiles for separation and drainage can be found in reference 26 by AASHTO Task Force 25. For vehicular applications, high-quality fabric should be specified that resists the puncturing by coarse, angular aggregate from compaction during construction and from repeated wheel loads during its service life. Bases should have their sides and bottoms wrapped in geotextile. Overlap recommendations are provided in AASHTO specifications. ICPI recommends a minimum of 1 ft (1.3 m) overlap in well-drained soils and 2 ft (0.6 m) overlap on poor-draining weaker soils (CBR<5%).

Handling Excess Water

Designs should have curb cut-outs and/or catch basins to handle emergency overflow conditions. Partial or no exfiltration designs require pipes to handle storage and outflow from design storms and those from overflow conditions. The size and placement of drain pipes should be determined by a civil engineer experienced in hydrological design and stormwater management. Pipes in bases subject to traffic should withstand repeated vehicular loads.

Perforations in pipes should be $^3/_8$ in. (10 mm) in diameter and terminate 1 ft (0.3 m) short of the sides of the opening for the base. When corrugated metal drain pipes are used, they should be aluminized, and aluminized pipe in contact with concrete should be coated to prevent corrosion. Perforated metal drain pipes should have caps fastened to the ends.

A 6 in. (150 mm) diameter vertical perforated pipe that serves as an observation well is recommended in all pavements. The pipe should be kept vertical during filling of the excavated area with open-graded aggregate and during compaction. The bottom of the pipe can be attached to a plate for stability when resting on the geotextile and held in place during base filling and compaction by

Paver Installation

After screeding the bedding material, the pavers are placed on this screeded layer joints filled with No. 8 stone, the surface swept clean and compacted with a plate compactor. For units 3 1/8 to 4 in. (80 to 100 mm) thick, the plate compactor should exert a minimum 4,000 lbf (8 kN) at 75 to 90 Hz. For units thicker than 4 in. (100 mm), the compactor should exert at least 6,800 lbf (30 kN). After initial compaction, the joints or openings are filled with No. 8 material, the paver surface swept clean and the paving units are compacted again. For vehicular areas, proof rolling is recommended with at least two passes of a 10 T rubber-tired roller.

Figure 31. Mechanized placement of pavers with wide joints.

Paver installation can be by hand or with mechanical equipment. Mechanized installation may be a cost-efficient means to install the units and will reduce the installation time. Figure 29 shows mechanized equipment placing permeable pavers for a parking lot adjacent to a baseball field. Figure 30 shows placement of another interlocking shape, while Figure 31 illustrates mechanized placement of pavers with widened joints. For further information on mechanical installation, consult ICPI *Tech Spec 11—Mechanical Installation of Interlocking Concrete Pavements* (41) and ICPI *Tech Spec 15—A Guide for the Construction of Mechanically Installed Interlocking Concrete Pavements* (54).

Units should be cut to fill any spaces along the edges prior to compaction. Cut units should be no smaller than one-third of a whole unit if subject to vehicular traffic. All installed units should be compacted into the No. 8 aggregate and joints filled with the appropriate material and pavers compacted again within 6 ft (2 m) of the laying face at the end of each day.

Section 4. Guide Specifications and Construction Checklist

SECTION 32 14 13.19 PERMEABLE INTERLOCKING CONCRETE PAVEMENT (1995 MasterFormat Section 02795)

Note: This guide specification describes construction of permeable interlocking concrete pavers on a permeable, open-graded crushed stone bedding layer (typically No. 8 stone). This layer is placed over an open-graded base (typically No. 57 stone) and sub-base (typically No. 2 stone). The pavers and bedding layer are placed over an open-graded crushed stone base with exfiltration to the soil subgrade. In low infiltration soils or installations with impermeable liners, some or all drainage is directed to an outlet via perforated drain pipes in the subbase. While this guide specification does not cover excavation, liners and drain pipes, notes are provided on these aspects.

The text must be edited to suit specific project requirements. It should be reviewed by a qualified civil or geotechnical engineer, or landscape architect familiar with the site conditions. Edit this specification term as necessary to identify the design professional in the General Conditions of the Contract.

PART 1 GENERAL

1.01 SUMMARY

A. Section Includes
   1. Permeable interlocking concrete pavers.
   2. Crushed stone bedding material.
   3. Open-graded subbase aggregate.
   4. Open-graded base aggregate.
   5. Bedding and joint/opening filler materials.
   6. Edge restraints.
   7. [Geotextiles].

B. Related Sections
   1. Section[_____]: Curbs.
   2. Section[_____]: [Stabilized] aggregate base.
   3. Section[_____]: [PVC] Drainage pipes
   4. Section[_____]: Impermeable liner.
   5. Section[_____]: Edge restraints.
   6. Section[_____]: Drainage pipes and appurtenances.
   7. Section[_____]: Earthworks/excavation/soil compaction.

1.02 REFERENCES

A. American Society for Testing and Materials (ASTM)
   1. C 67, Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units.
   2. C 131, Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine.
   3. C 136, Method for Sieve Analysis for Fine and Coarse Aggregate.
   4. C 140, Test Methods for Sampling and Testing Brick and Structural Clay Tile, Section 8 – Freezing and Thawing.
   5. D 448, Standard Classification for Sizes of Aggregate for Road and Bridge Construction.
   6. C 936, Standard Specification for Solid Interlocking Concrete Pavers.
   7. C 979, Specification for Pigments for Integrally Colored Concrete.

C. Mock-Ups:
   1. Install a 10 ft x 10 ft (3 x 3 m) paver area.
   2. Use this area to determine surcharge of the bedding layer, joint sizes, lines, laying pattern(s), color(s) and texture of the job.
   3. This area will be used as the standard by which the work will be judged.
   4. Subject to acceptance by owner, mock-up may be retained as part of finished work.
   5. If mock-up is not retained, remove and properly dispose of mock-up.

1.05 DELIVERY, STORAGE, AND HANDLING

A. General: Comply with Division 1 Product Requirement Section.
B. Comply with manufacturer's ordering instructions and lead-time requirements to avoid construction delays.
C. Delivery: Deliver materials in manufacturer's original, unopened, undamaged container packaging with identification tags intact on each paver bundle.
   1. Coordinate delivery and paving schedule to minimize interference with normal use of buildings adjacent to paving.
   2. Deliver concrete pavers to the site in steel banded, plastic banded, or plastic wrapped cubes capable of transfer by forklift or clamp lift.
   3. Unload pavers at job site in such a manner that no damage occurs to the product or existing construction
D Storage and Protection: Store materials in protected area such that they are kept free from mud, dirt, and other foreign materials.

1.06 ENVIRONMENTAL REQUIREMENTS

A. Do not install in rain or snow.
B. Do not install frozen bedding materials.

1.07 MAINTENANCE

A. Extra materials: Provide [Specify area] [Specify percentage] additional material for use by owner for maintenance and repair.
B. Pavers shall be from the same production run as installed materials.

PART 2 PRODUCTS

Note: Some projects may include permeable **and solid** interlocking concrete pavements. **Specify each product as required.**

2.01 PERMEABLE INTERLOCKING CONCRETE PAVERS

A. Manufacturer: [Specify ICPI member manufacturer name.].
   1. Contact: [Specify ICPI member manufacturer contact information.].
B. Permeable Interlocking Concrete Paver Units:
   1. Paver Type: [Specify name of product group, family, series, etc.].
      a. Material Standard: Comply with ASTM C 936 [CSA A231.2].
      b. Color [and finish]: [Specify color.] [Specify finish].
      c. Color Pigment Material Standard: Comply with ASTM C 979.

Note: Concrete pavers may have spacer bars on each unit. Spacer **bars are** recommended **for** mechanically **installed pavers. Manually installed pavers may be installed with or without spacer bars.** Verify **with manufacturers that overall dimensions do not include spacer bars.**

2.04 ACCESSORIES

A. Provide accessory materials as follows:

Note: Curbs will typically be cast-in-place concrete or precast set in concrete haunches. Concrete curbs may be specified in another Section. Do not use plastic edging with steel spikes to restrain the paving units.

1. Edge Restraints
   a. Manufacturer: [Specify manufacturer.].
   b. Material: [Pre-cast concrete] [Cut stone] [Concrete].
   b. Material Standard: [Specify material standard.].

Note: See ICPI publication, Permeable Interlocking Concrete Pavements for guidance on geotextile selection. Geotextile use is optional.

2. Geotextile Fabric:
   a. Material Type and Description: [Specify material type and description.].
   b. Material Standard: [Specify material standard.].
   c. Manufacturer: [Acceptable to interlocking concrete paver manufacturer]]

PART 3 EXECUTION

3.01 ACCEPTABLE INSTALLERS

A. [Specify acceptable paver installation subcontractors.].

3.02 EXAMINATION

Note: The elevations and surface tolerance of the soil subgrade determine the final surface elevations of concrete pavers. The paver installation contractor cannot correct deficiencies excavation and grading of the soil subgrade with additional bedding materials. Therefore, the surface elevations of the soil subgrade should be checked and accepted by the General Contractor or designated party, with written certification presented to the paver installation subcontractor prior to starting work.

A. Acceptance of Site Verification of Conditions:
   1. General Contractor shall inspect, accept and certify in writing to the paver installation subcontractor that site conditions meet specifications for the following items prior to installation of interlocking concrete pavers.

Note: Compaction of the soil subgrade should be determined by the project engineer. If the soil subgrade requires compaction, compact to a minimum of 95% standard Proctor density per ASTM C 698. Compacted soil density and moisture should be checked in the field with a nuclear density gauge or other test methods for compliance to specifications. Stabilization of the soil and/or base material may be necessary with weak or continually saturated soils, or when subject to high wheel loads. Compaction will reduce the permeability of soils. If soil compaction is necessary, reduced infiltration may require drain pipes within the open-graded sub base to conform to local storm drainage requirements.

      a. Verify that subgrade preparation, compacted density and elevations conform to specified requirements.
      b. Provide written density test results for soil subgrade to the Owner, General Contractor and paver installation subcontractor.
      c. Verify location, type, and elevations of edge restraints, [concrete collars around] utility structures, and drainage pipes and inlets.
   2. Do not proceed with installation of bedding and interlocking concrete pavers until subgrade soil conditions are corrected by the General Contractor or designated subcontractor.

d. Lay the pavers [paving slabs] in the pattern(s) and joint widths shown on the drawings. Maintain straight pattern lines.
e. Fill gaps at the edges of the paved area with cut units. Cut pavers subject to tire traffic shall be no smaller than 1/3 of a whole unit.
f. Cut pavers and place along the edges with a [double-bladed splitter or] masonry saw.
g. Fill the openings and joints with [No. 8] stone.

Note: Some paver joint widths may be narrow and not accept most of the No. 8 stone. Use joint material that will fill joints such as washed ASTM No. 9 or No. 10 stone. These smaller stone sizes are recommended for filling joints in pedestrian applications that use 2 3/8 in. (60 mm) thick pavers.

h. Remove excess aggregate on the surface by sweeping pavers clean.
i Compact and seat the pavers into the bedding material using a low-amplitude, 75-90 Hz plate compactor capable of at least 18 kN (4,000 lbs.) centrifugal compaction force. This will require at least two passes with the plate compactor.
j. Do not compact within 2 m (6 ft) of the unrestrained edges of the paving units.
k. Apply additional aggregate to the openings and joints, filling them completely. Remove excess aggregate by sweeping then compact the pavers. This will require at least two passes with the plate compactor.
l. All pavers within 2 m (6 ft) of the laying face must be left fully compacted at the completion of each day.
m. The final surface tolerance of compacted pavers shall not deviate more than ±10 mm (±3/8 in.) under a 3 m (10 ft) long straightedge.
n. The surface elevation of pavers shall be 3 to 6 mm (1/8 to 1/4 in.) above adjacent drainage inlets, concrete collars or channels.

3.05 FIELD QUALITY CONTROL

A. After sweeping the surface clean, check final elevations for conformance to the drawings.
B. Lippage: No greater than 3 mm (1/8 in.) difference in height between adjacent pavers.

Note: The minimum slope of the finished pavement surface should be 1%. The surface of the pavers may be 3 to 6 mm (1/8 to 1/4 in.) above the final elevations after compaction. This helps compensate for possible minor settling normal to pavements.

C. The surface elevation of pavers shall be 3 to 6 mm (1/8 to 1/4 in.) above adjacent drainage inlets, concrete collars or channels.

3.06 PROTECTION

A. After work in this section is complete, the General Contractor shall be responsible for protecting work from sediment deposition and damage due to subsequent construction activity on the site.

END OF SECTION

Section 5. Maintenance

Permeable interlocking concrete pavements can become clogged with sediment over time, thereby slowing their infiltration rate and decreasing storage capacity. Figure 32 shows an installation subject to nearby construction that has brought sediment into pavement openings. Clogged surface openings are a major cause of hydrological failure. The rate of sedimentation depends on the amount of traffic and other sources that wash sediment into the joints, base and soil. Since the pavement is detaining runoff that contains sediment, there may be a need to eventually remove and replace the base material when the infiltration is reduced to such a degree that the pavement is no longer performing its job in storing and exfiltrating water.

Figure 32. Unwanted sediment tracked onto permeable pavers via this gravel access ramp to a construction site will clog their openings.

Research by James (46) and practical experience have demonstrated that periodic removal of sediment in the openings will increase surface infiltration rates. Vacuum type street cleaning equipment without brooms and water spray action are the most effective at loosening and removing sediment from the openings. Regenerative air sweepers, i.e., those that blow air across the pavement surface to create a vacuum are not recommended as they tend to move the sediment rather than remove it. Likewise, brooms and water spray may move the sediment deeper into the surface openings and contribute to clogging. The frequency of vacuum cleaning will depend on the use and sources of sediment brought to the pavement openings. Vacuuming should be done at least once or twice annually and sediment/detritus deposition monitored for more frequent vacuuming. Vacuuming will have the best results when the sediment is dry which means that vacuuming during warm, dry weather will likely yield the best cleaning results. Street cleaning equipment has the potential to vacuum stones from the pavement openings, so suction adjustments may be

In-service Inspection Checklist

- ❑ Vacuum surface openings in dry weather to remove dry, encrusted sediment. These appear as small, curled "potato chips." Vacuum settings may require adjustment to prevent uptake of aggregate in the pavement openings and joints.
- ❑ Inspect after at least one major storm per year.
- ❑ Maintained vegetation around pavement to filter runoff and minimize sediment deposition on the pavement.
- ❑ No standing water on the surface after storms.
- ❑ Repair ruts or deformations in pavement exceeding $^1/_2$ in. or 13 mm.
- ❑ Repair pavers more than $^1/_4$ in. or 6 mm above/below adjacent units.
- ❑ Replace broken units that impair the structural integrity of the surface.
- ❑ Replenish aggregate joint materials as needed.
- ❑ Check drain outfalls for free flow of water.
- ❑ Check outflow from observation well annually.

The model agreement presented below is applicable to many BMPs. It can be edited to suit local situations and customized for the maintenance of permeable interlocking concrete pavement. A list of maintenance items should be an attachment to this agreement, as well as an inspection schedule. This list of items to be inspected can be developed from the in-service inspection check-list in this section as well as from requirements established by the local government. A growing number of local governments are creating databases in which to place BMP inspection data. This provides continual documentation of care and performance.

Model Maintenance Agreement

This Maintenance Agreement made this ____ day of _____, [year], by and between [property owner/s], hereinafter referred to as "Grantor," and the [city/county of state/province] hereinafter referred to as the "[city/county]."

WITNESSETH

WHEREAS, the [city/county] is authorized ad required to regulate and control disposition of storm and surface waters within the [city/county/watershed] as set forth by [city/county] [state/provincial] ordinances; and

WHEREAS, the Grantor is the owner of a certain tract or parcel of land more particularly described as [legal description].

ALL THOSE certain lots, pieces or parcels of land, together with buildings and improvements thereon, and the appurtenances thereunto belonging, lying, situated and being in the [city/county] of [state/province] as shown on [tax maps/subdivisions plats numbers and names], duly recorded in the Clerk's Office of the [court] of [city/county] in Deed Book or Plat Book [number] at page [number] reference to which the plat is hereby made for a more particular description thereof.

It being the said property conveyed unto the Grantor herein by deed dated _____ from _____ and recorded in the Clerk's office aforesaid in Deed Book_____ at Page_____ such property being hereinafter referred to as "the property."

WHEREAS, the Grantor desires to construct certain improvements on the property which will alter existing storm and surface water conditions on the property and adjacent lands; and

WHEREAS, in order to accommodate and regulate these anticipated changes in existing storm and surface water flow conditions, the Grantor, its heirs and assigns, desire to build and maintain at their expense a storm and surface water management facility and system [more particularly described as a permeable interlocking concrete pavement]. This is shown on plat titled _____ and dated _____; and

WHEREAS, the [city/county] has reviewed and approved these plans subject to the execution of this agreement.

NOW THEREFORE, in consideration of the benefit received by the Grantor, its heirs and assigns, and as a result of the [city/county] approval of its plans, the Grantor, it heirs and assigns, with full authority to execute deeds, deeds of trust, other covenants and all rights, title and interest in the property described above hereby covenant with the [city/county] as follows:

1. Grantor, its heirs and assigns shall construct and perpetually maintain, at its sole expense, the above referenced permeable interlocking concrete pavement [storm and surface management facility and system] in strict accordance with the plan approval granted by the [city/county].

2. Grantor, its heirs and assigns shall, at its sole expense, make such changes or modifications to the permeable interlocking concrete pavement [storm drainage facility and system]. Changes or modifications may, in the [city's/county's] discretion, be determined necessary to insure that the facility and system are property maintained and continues to operate as designed and approved.

3. The [city/county], it agents, employees and contractors shall have the perpetual right of ingress and egress over the property of the Grantor, its heirs assigns, and the right to inspect [at reasonable times and in a reasonable manner,] the permeable interlocking concrete pavement [storm drainage facility and system]. Inspection is in order to insure that the system is being properly maintained and is continuing to perform in an adequate manner. [Attachment A to this agreement provides a list of items to be inspected by the [city/county]].

4. The Grantor, its heirs and assigns agree that should it fail to correct any defects in the above described facility and system within [ten (10)] days from issuance of written notice, or shall fail to maintain the facility in accordance with

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Appendix A—Glossary of Terms

AASHTO—American Association of State Highway and Transportation Officials

Aquifer—A porous water bearing geologic formation that yields water for consumption.

ASTM—American Society for Testing and Materials

Best Management Practice (BMP)—A structural or non-structural device designed to infiltrate, temporarily store, or treat stormwater runoff in order to reduce pollution and flooding.

Cation—A positively charged atom or group of atoms in soil particles that, through exchange with ions of metals in stormwater runoff, enable those metals to attach themselves to soil particles.

Choke course—A layer of aggregate placed or compacted into the surface of another layer to provide stability and a smoother surface. The particle sizes of the choke course are generally smaller than those of the surface into which it is being pressed.

Clay soils—1. (Agronomy) Soils with particles less than 0.002 mm in size. 2. A soil textural class. 3. (Engineering) A fine-grained soil with more than 50% pass the No. 200 sieve with a high plasticity index in relation to its liquid limit, according the Unified Soil Classification System.

Crushed stone—Mechanically crushed rock that produces angular particles.

CSA—Canadian Standards Association

Curve Number (CN)—A numerical representation of a given area's hydrological soil group, plant cover, impervious cover, interception and surface storage. The U.S. Soil Conservation Service (SCS) originally developed the concept. A curve number is used to convert rainfall depth into runoff volume.

Dense-graded base—Generally a crushed aggregate base with fines that, when compacted, creates a foundation for pavements and does not allow significant amounts of water into it. Particle sizes can range from 1.5 in. (40 mm) to smaller than the No. 200 (0.075 mm) sieve.

Detention pond or structure—The temporary storage of stormwater runoff in an area with objective of decreasing peak discharge rates and providing a settling basin for pollutants.

Erosion—The process of wearing away of soil by water, wind, ice, and gravity. 2. Detachment and movement of soil particles by same.

Exfiltration—The downward movement of water through an open-graded, crushed stone base into the soil beneath.

Fines—Silt and clay particles in a soil, generally those smaller than the No. 200 or 0.075 mm sieve.

Grade—1. (Noun) The slope or finished surface of an excavated area, base, or pavement usually expressed in percent. 2. (Verb) To finish the surface of same by hand or with mechanized equipment.

Gravel—1. Aggregate ranging in size from 1/4 in. (6 mm) to 3 in. (75 mm) which naturally occurs in streambeds or riverbanks that has been smoothed by the action of water. 2. A type of soil as defined by the Unified Soil Classification System having particle sizes ranging from the No. 4 sieve (4.75 mm) and larger.

Hotspot—A land use that generates highly contaminated runoff with concentrations higher than those typical to stormwater.

Hydrological Soil Group—The soils classification system developed by the U.S. Soil Conservation Service (now the Natural Resource Conservation Service) that categorizes soils into four groups, A through D, based on runoff potential. A soils have high permeability and low runoff whereas D soils have low permeability and high runoff.

Impervious cover—Surfaces that do not allow rainfall to infiltrate into the soil. Examples include pavements, roofs, sidewalks, driveways, etc.

Infiltration rate—The rate at which stormwater moves through soil measured in inches per hour or meters per second.

Karst geology—Regions of the earth underlain by carbonate rock typically with sinkholes and/or limestone caverns.

Observation well—A perforated pipe inserted vertically into an open-graded base used to monitor its infiltration rate.

One year storm—A rainfall event that occurs once a year or has a 100% chance of occurring in a given year.

One hundred year storm—A very unusual rainfall event that occurs once every 100 years or has a 1% chance of occurring in a given year.

Open-graded base—Generally a crushed stone aggregate material used as a pavement base that has no fine particles in it. The void spaces between aggregate can store water and allow it to freely drain from the base.

Outlet—The point at which water is discharged from an open-graded base through pipes into a stream, lake, river, or storm sewer.

Peak discharge rate—The maximum instantaneous flow from a detention or retention pond, open-graded base, pavement surface, storm sewer, stream or river usually related to a specific storm event.

David R. Smith is the Technical Director for the Interlocking Concrete Pavement Institute or ICPI (www.icpi.org). He and 66 companies started the ICPI in 1993, and have seen this North American industry association grow to over 500 members representing producers, contractors, and suppliers. Since 1985, he has worked closely with engineers in design and construction of every kind of concrete paver project ranging from patios to streets, plus ports and airports. In addition to publishing three books, dozens of articles, guide specifications and ICPI technical bulletins on concrete pavers, Mr. Smith has written two instructional manuals for contractors. In addition, he co-authored the ICPI manuals, *Port and Industrial Pavement Design with Concrete Pavers* and *Airfield Pavement Design with Concrete Pavers*.

As a leading authority in North America on concrete segmental paving, Mr. Smith regularly speaks at national and international conferences. He is secretary-treasurer of the Small Element Paving Technologists (www.sept.org), a group of international specialists on segmental paving. He is a past chairman of the Canadian Standard Association's Technical Committee on Precast Concrete Paving. He is an active member of ASTM, having written and revised several paving product standards for that organization. He participates as a member in the American Society of Civil Engineers, American Public Works Association, Construction Specifications Institute, and the American Society of Landscape Architects (ASLA). Mr. Smith has contributed continuing education programs to the ASLA and to the American Institute of Architects.

His education includes a Bachelor of Architecture and a Masters of Urban and Regional Planning (environmental concentration) from Virginia Tech in Blacksburg. Between earning these degrees, he taught stormwater management for landscape architecture students at Virginia Tech. He also was involved in research there on concrete grid pavements and has written extensively about their design and construction. *Permeable Interlocking Concrete Pavements* represents a synthesis of experience and insights from professional engineers, stormwater regulators, and ICPI member concrete paver manufacturers and contractors.

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Appendix H: Performance Evaluation of a Permeable Pavement and a Bioretention Swale from the Vancouver Toyota Final Stormwater Report

APPENDIX H

Performance Evaluation of Permeable Pavement and a Bioretention Swale

Vancouver Toyota
Project #14292

MacKay & Sposito, Inc.

Performance Evaluation of Permeable Pavement and a Bioretention Swale

Seneca College, King City, Ontario

Prepared by Toronto and Region Conservation

Interim Report #3

May 2007

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1.0 INTRODUCTION

1.1. Background

The adverse impacts of urban development on natural watercourses and associated infrastructure and property have been well documented. These impacts stem from the loss of natural infiltration and evapotranspiration functions as pervious vegetated areas are replaced with buildings and paved surfaces. As less rainwater infiltrates and evapotranspires, more runs off over the surface, carrying with it a variety of pollutants that ultimately end up degrading river ecosystems and contaminating swimming areas. The higher runoff volumes, in turn, cause erosion of stream channels, flooding and damage to infrastructure.

Vehicular traffic accounts for much of the build-up of contaminants on roads and parking surfaces. Wear from tires, brake and clutch linings, engine oil and lubricant drippings, combustion products and corrosion, all contribute to the build up of sediment particles, metals, and oils and grease. Degradation of road surfaces also generates derivatives from asphalt, and runoff from residential driveways and parking areas can contain driveway sealants, oil, salt, and car care products. All of these different elements and compounds can accumulate and degrade local water courses over time (Table 1.1).

Table 1.1: Typical Sources of Contaminants in Runoff from Parking Lots (Burton and Pitt, 2002)

Variable	Source
Particulates	Pavement wear, vehicles, atmosphere, maintenance
Nitrogen, phosphorus	Atmosphere, roadside fertilizer application
Lead	Tire wear (lead oxide .filler material, lubricating oil and grease, bearing wear), metal deterioration
Zinc	Tire wear (filler materials), motor oil (stabilizing additive), grease, metal deterioration
Iron	Auto body rust, steel highway structures (guard rails, etc.), moving engine parts, metal deterioration
Copper	Metal plating, bearing and bushing wear, moving engine parts, brake lining wear, fungicides and insecticides, metal deterioration
Cadmium	Tire wear (filler material), insecticide application, metal deterioration
Chromium	Metal plating, moving engine parts, break lining wear, metal deterioration
Nickel	Diesel fuel and gasoline (exhaust), lubricating oil, metal plating, bushing wear, brake lining wear, asphalt paving, metal deterioration
Manganese	Moving engine parts
Cyanide	Anti-cake compound (ferric ferrocyanide, sodium ferrocyanide, yellow prussiate of soda) used to keep de-icing salt granular
Sodium, Calcium, Chloride	De-icing salts
Sulphate	Roadway beds, fuel, de-icing salts
Petroleum, Oil, and Grease	Spills, leaks, or blow-by of motor lubricants, antifreeze and hydraulic fluids, asphalt surface leachate, fuel and oil spills and leaks
PAHs	Asphalt, fuel and oil spills and leaks
Suspended Solids	Sanding in winter, tire wear, tire tread deposits

Land use is one of the most important factors governing the level and type of pollutants present in stormwater runoff. Based on an extensive monitoring data set, the United States Environmental Protection Agency (USEPA) estimated typical pollutant yields from various land uses to local watercourses per year (Table 1.2). While pollutant loads to watercourses will vary, Table 1.2 provides

Permeable pavement systems rely almost exclusively on pollutant removal through filtration and adsorption to soil particles. Bioretention swales may not be as efficient as permeable pavements in terms of infiltration; as such systems are generally small in surface area relative to the size of the tributary paved area. However, bioretention swales offer alternative potential water quality benefits through such mechanisms as nutrient uptake and other biological processes. There is, therefore, value in carrying out monitoring of field applications of both permeable pavement and bioretention swales to assess their suitability as stormwater management options for managing existing and new development.

1.2. Study Objectives and Report Outline

The overall objective of this study is to assess the long term performance and effectiveness of permeable pavement and bioretention swales for stormwater management. While there has been a considerable amount of research conducted on these practices, there are few practical applications of such measures in Southern Ontario. Of those that have been installed, few if any have been comprehensively monitored. Even outside of Southern Ontario, data on the quality of infiltrated water and soils beneath the installations are relatively sparse. Data on the performance of permeable pavements and bioswales under winter conditions typical of a Southern Ontario climate are also very rare. This study examines permeable pavement and bioretention swales under climate and soil conditions representative of watersheds in the Greater Toronto Area (GTA).

Once the monitoring component of the project has been completed, a model will be set-up and calibrated to simulate the cumulative benefits (*e.g.* water quality, peak storm flow, water budget) of permeable pavement and bio-swales at sub-watershed and watershed scales. Results will be made available for watershed and sub-watershed planning purposes by municipalities and conservation authorities. The data will also assist in developing guidelines for meeting water budget requirements on urban lands, which are currently being developed as a separate initiative by the TRCA. Water budget guidelines will be applied to new and retrofit development through the development plan review/approvals process of stormwater management plans within the GTA.

The next section sets a context for the study through a review of available literature on permeable pavements and bioretention swales. Section 3 describes the study site and provides design details on the technologies and experimental set-up. Section 4 outlines the overall study approach, including the runoff streams to be measured and sampled, the frequency of sampling and the range of water quality variables that will be analyzed in the laboratory. Preliminary results from monitoring conducted so far are presented and discussed in Section 5, followed by conclusions and recommendations for future monitoring in Section 6.

2.0 LITERATURE REVIEW

2.1. Permeable Pavement

2.1.1. Types of Permeable Pavement

The term permeable pavement is used in general to describe pavements that allow stormwater to infiltrate into a gravel-filled reservoir (hereafter referred to as base course) below the pavement surface. This reservoir provides temporary storage of stormwater before it infiltrates into the subsoil or is drained away by perforated pipes (CWP, 2000). While all varieties of permeable pavements are designed to reduce surface runoff volumes, permeable pavement designs can differ significantly. Three main categories of permeable pavement are commonly used;

- Modular interlocking concrete block pavers
- Plastic lattice or grid systems
- Porous asphalt or pervious concrete

Modular interlocking concrete block pavement consists of impervious concrete blocks that allow water to infiltrate into the base course through voids within or between the pavers. The voids in this type of pavement may be filled with gravel or soil and grass. Plastic grid systems consist of plastic interlocking units with very little impervious surface area. Grid spaces may be planted with grass or left unplanted and filled with gravel. The grids are designed to provide structural stability and prevent settling while providing a large amount of void space for infiltration of stormwater. Porous asphalt pavement consists of standard bituminous asphalt in which the finer aggregates have been removed. Removal of these fine materials results in an asphalt with a matrix of pores that allows water to permeate through to the base course and infiltration bed. Pervious concrete works on the same principle as porous asphalt with the finer aggregates omitted from the concrete mix resulting in increased void space.

The following review of permeable pavement literature is largely limited to studies associated with interlocking concrete block pavers, as this is the type of pavement evaluated in this study.

2.1.2. Performance

2.1.2.1. Surface Runoff Reduction

Permeable pavement systems are intended to mimic natural hydrologic functions by infiltrating stormwater runoff, promoting groundwater recharge and maintaining or augmenting baseflows. Several field studies have quantified the runoff reduction benefit of concrete block pavers. At a Public Works parking lot in Renton, Washington, Booth and Leavitt (1999) reported virtually no surface runoff from planted (*i.e.* turfstone) and unplanted concrete block pavement for all rain events monitored during the autumn and early winter of 1996/97. A repeat study conducted at the same site four years later also found very low runoff from the permeable pavements. Among the 15 storms monitored, only one 44 mm rain event generated runoff. The runoff occurred on the plastic lattice Grasspave™ system filled with sand and grass, and observed runoff represented a mere 3% of the total precipitation (Brattebo and Booth, 2003). In both studies, the subsurface flows occurred after a significant delay even though the

research has demonstrated that naturally occurring microbial communities on pavement building materials helps to retain and degrade hydrocarbons within the base course layer (*e.g.* Newman et al.,2006).

Chloride is the primary constituent of road salts. It poses a significant risk to groundwater because it is extremely mobile in soil systems and is applied in large quantities to paved surfaces during the winter as a de-icing agent. Groundwater may also become contaminated with road salts from swales, road side ditches and other pervious areas that either receive drainage from, or are close to surfaces where road salts are applied.

Several studies have shown the capacity of the soils to adsorb heavy metals in urban runoff and prevent leaching to lower soil horizons and groundwater. In the Washington study discussed earlier (Brattebo and Booth, 2003); stormwater concentrations of copper, zinc and motor oil were significantly improved through infiltration via a permeable concrete block pavement installation. The study reported that 88 and 100% of asphalt runoff samples exceeded Washington receiving water standards for zinc and copper, respectively. By contrast, only 6 and 17% of permeable block pavement infiltrate samples (n=18) exceeded the standards for copper and zinc, and motor oil was consistently below analytical detection limits, even though the soil through which water infiltrated was only 10 cm deep. A study conducted at Guelph University also reported water quality improvements after infiltration of stormwater through permeable pavers, especially for zinc and iron (Shahin, 1994).

Attenuation of contaminants in infiltration systems receiving runoff from large drainage areas occurs predominantly within the upper soil layer beneath the base of the systems. In California, Nightingale (1978) found that accumulation of lead, zinc and copper in soils beneath urban runoff retention basins was limited to the upper 15 cm of soil. Salo et al (1986) also reported sharp declines in soil concentrations of lead, arsenic, nickel, and copper below five groundwater recharge basins, two of which had been in operation for more than 20 years at the time of the study. Several organic compounds were also monitored both in the soil and groundwater, including chlorinated pesticides, organo-phosphorus pesticides, chlorophenoxy herbicides and phenolic compounds. The authors concluded that there were no adverse effects on groundwater as a result of infiltrating stormwater.

A German study investigating the impacts of highway runoff on roadside soils concluded that the age of roadside soils was positively correlated with the concentrations of polycyclic aromatic hydrocarbons and several heavy metals. However, leaching of contaminants to the groundwater was limited, even for soil ages greater than 20 years. Soil characteristics such as organic content and pH were found to be important factors controlling the buffering capacity of the soils (Dierkes and Geiger, 1999).

Legret *et al* (1999) used a mathematical model to simulate the transfer of heavy metals into the soil below a porous pavement installation. The study found that soil may be contaminated by heavy metals and mineral oils over at least a one-metre radius in the long term (30 years). Contamination levels were relatively low, however, as concentrations were still below Dutch standards for unpolluted soil (Legret *et al*, 1999).

2.1.4. Clogging

The pavement openings will clog over time as dust and dirt accumulate in the pavement openings and pore spaces of the underlying granular media. Rain and traffic further exacerbate the problem by breaking up soil aggregates into finer particles that block the pores and allow for further accumulation of fines. Eventually a hard crust forms upon drying, creating a seal that can drastically reduce infiltration through the surface openings (Balades *et al.*, 1995, Pratt *et al*, 1995).

Clogging has been a serious issue in some of the early permeable pavement installations. Lindsey et al (1992) surveyed several infiltration facilities in Maryland, including 13 'porous pavement' installations, most of which were between 5 and 6 years old. Their survey found that only 2 of the 13 pavements were operating according to design, mostly due to sediment clogging. The authors did not provide details on the type of porous pavement (*e.g.* interlocking pavers, porous asphalt) or materials used in construction; hence it was difficult to evaluate the reasons for failure. Many of the early permeable pavement installations were constructed with sand as a bedding layer. Further, garden and grassed areas around the perimeter often drained onto the pavement, rather than away from it. These conditions tend to increase the potential for clogging.

More recent installations use washed stone in the pavement openings and bedding layer because these resist breakdown into smaller particles with age, and the pore spaces are large enough to transmit fine particulate matter into the base course layers, thereby reducing the potential for surface sealing. At the Guelph University experimental plots referred to earlier, Gerrits (2001) reported considerably better infiltration on 8 year old permeable pavers constructed with a bedding layer of 7.5 cm of clear washed stone than those with a 10 cm mixture of clear washed stone and sand (both installations used 40 cm of granular 'A' as the sub-base). The pure washed stone bedding layer installation also responded much more effectively to maintenance efforts directed at restoring the original surface infiltration capacity. These results are consistent with laboratory tests of interlocking pavement surface drainage materials ranging in size from 2 mm sand to 10 mm gravel (Shackel, 1995). The laboratory tests showed that uniform sized washed 2-5 mm gravel provided the best infiltration capacities.

Bean et al (2004) examined surface infiltration rates on concrete grid (n = 16) and permeable interlocking concrete pavers (n = 11) located in Maryland and North Carolina. The pavements ranged in age from six months to 20 years. The concrete grid pavers had much lower infiltration rates than the interlocking pavers. Location of pavers close to sources of fine sediment (e.g. beach, construction site, river bed) was found to dramatically reduce surface infiltration rates, although even these sites had infiltration rates greater than a grassed sandy loam soil.

2.1.5. Operation and Maintenance Considerations

Clogging of surface voids and loss of permeability over time can be minimized through regular maintenance and appropriate design of the pavement. Table 2.1 lists recommendations from various sources regarding the type and frequency of maintenance required for permeable pavements. Most sources recommend vacuum sweeping over power washing as the latter pushes sediments into the pavement rather than removing them. The frequency of vacuuming will vary depending on site conditions but once or twice a year is generally regarded as sufficient. If aggregate in the pavement openings is

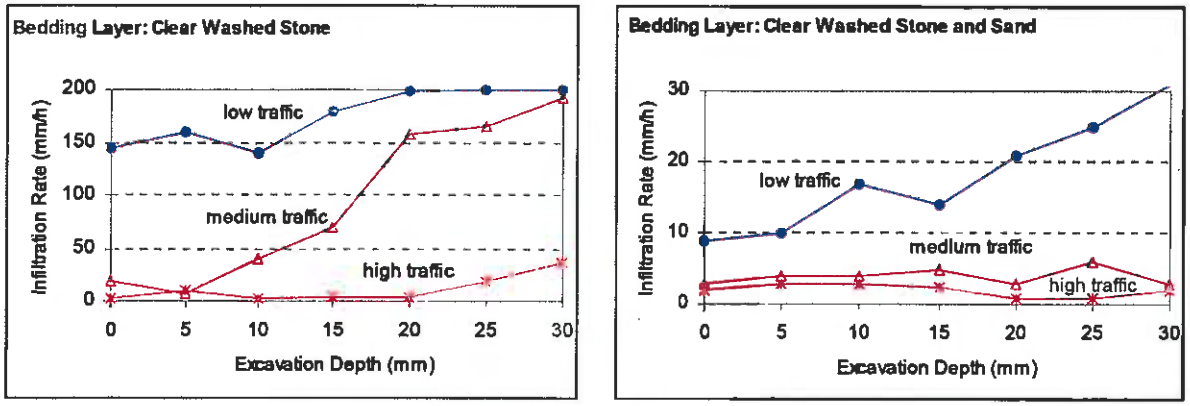

Figure 2.1: Infiltration rates on two 8 year old un-maintained permeable pavements after removal of void space material of varying depths. The bedding layers of one permeable pavement contained sand (right), the bedding layer of the other did not. Bedding layers beneath both pavements were underlain by a 40 cm sub-base of granular 'A'. Note differences in vertical scales. Source: Gerrits, 2001.

In Maryland and North Carolina, Bean et al (2004) simulated maintenance of permeable pavements using an approach similar to Gerrits (2001). Of the 14 concrete grid paver sites tested, 13 exhibited notably higher infiltration rates than the un-maintained pavers. The mean infiltration rate increased by 66%. The surface infiltration rate also increased substantially on the one interlocking concrete paver site subjected to maintenance.

2.1.6. Cost Considerations

The installed capital cost of permeable interlocking pavers will vary on a site-by-site basis, but local industry sources indicate that the cost of traditional impervious asphalt is roughly 60% that of interlocking concrete pavers. The Lake County Forest Preserves in Illinois estimates a similar installed cost ratio for a 40,000 square foot area of 0.66. This cost differential is offset in the long term by the longer life of permeable pavements and the reduced need for stormwater conveyance and treatment infrastructure (Ferguson, 2005). A survey conducted by the Lake County Forest Preserves reported that two permeable pavement sites in Pennsylvania lasted as long, or longer than impervious asphalt. (Lake County Forest Preserves, 2003). If permeable pavements are used for stormwater detention, significant savings can be achieved by reducing the amount of piping and downgrading the size of end-of-pipe facilities (James, 2004). These savings, however, are contingent upon a local regulatory framework that allows the use of permeable pavements and other infiltration facilities to meet detention requirements.

Maintenance costs for permeable pavement installations will generally be higher than for conventional asphalt because, as mentioned previously, the pavements need to be routinely cleaned to avoid clogging. However, when impervious asphalt cracks or pits, expensive sealing and patching procedures may be required. Damaged stones on permeable pavement are comparably much easier to replace (Lake County Forest Preserves, 2003). An overall comparison of maintenance costs must be conducted on a site-by-site basis due to the many variables that influence costs.

irrigating, fertilizing, trimming and overall maintenance of plant health. In circumstances when drainage appears to be comprised (*i.e.* water would pond for longer than guidelines specify), it may be necessary to investigate whether or not clogging is occurring and in which layer. The soil bed should be checked for clogging twice per year (New Jersey Department of Environmental Protection, 2003). Actions to correct clogging in a bioswale may include raking the surface, punching holes through the soil bed, or re-installing the entire bioswale as a last resort (Prince George's County Dept. of Environmental Resources, 2002). Over the long-term it may also become necessary to address soil contamination concerns that are common to stormwater infiltration practices by, for instance, excavating and replacing the soil. There are several studies that address the migration of contaminants in soils below a stormwater infiltration installation. Some of this research is discussed in section 2.1.4.

2.2.4. Cost Considerations

The cost of installing a bioswale will depend on various factors including the bioswale size, vegetation types used and whether or not the construction will be a retrofit or a new installation (USEPA, 1999). Retrofitting will increase the construction costs due to the need for demolition of existing structures and pavements (USEPA, 1999). Implementing bioretention near the source of stormwater runoff has the potential to significantly minimize the amount of storm drainage infrastructure needed. Several case studies from Prince George's County, Maryland have found that integrating bioretention facilities at a site may ultimately reduce development costs by 15-20% in comparison with costs associated with more traditional stormwater BMPs (Prince George's County Dept. of Environmental Resources, 2002).

3.0 STUDY AREA: King Campus, Seneca College

The permeable pavement and bioretention swale evaluated in this study were installed on a parking lot at Seneca College's King Campus. The campus is located at the north-west corner of Dufferin Street and 15th Sideroad in the Township of King, within the Oak Ridges Moraine (ORM) complex (Figures 3.1 and 3.2). The area drains to a tributary of the East Humber River.

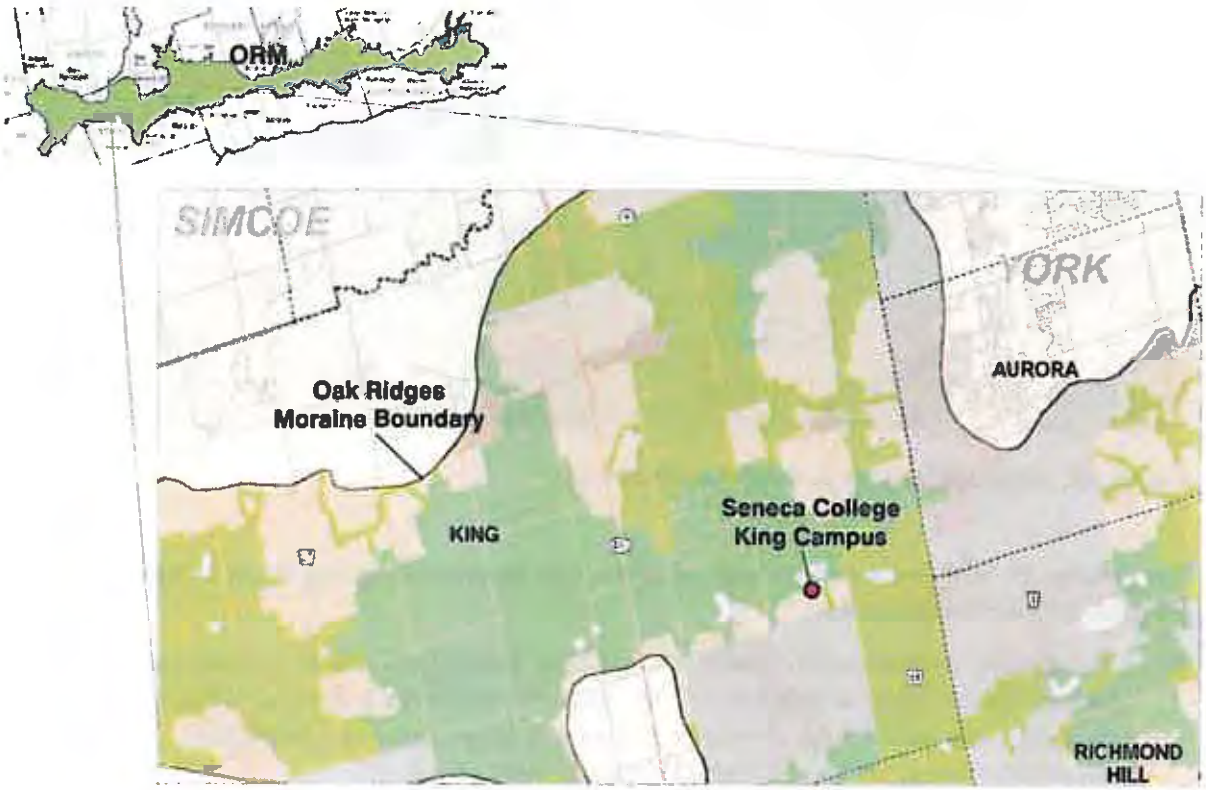

Figure 3.1: The Oak Ridges Moraine and the Township of King

4.0 STUDY APPROACH

4.1. Site Design and Construction

Planning and design of the monitoring study was completed in August 2004. For the study, the parking lot was divided into three equal areas. On one third of the total area, the asphalt was removed and resurfaced using permeable pavement (Unilock® interlocking pavers). The middle third remained unaltered and served as a control area for the study (*i.e.* conventional asphalt surface). On the final third a bio-retention swale was constructed at the drainage edge of the asphalt to treat runoff from the existing pavement (Figure 4.1).

Typically, measures such as permeable pavers and bio-retention swales do not require sewers for the collection of runoff. However, in order to evaluate the performance of each measure, a comprehensive monitoring program and collection system for both surface and subsurface flow was required. The overall study design is presented in plan view and cross section in Figure 4.1 and 4.2, respectively. Detailed descriptions of the experimental set-up are provided in the following sections.

4.1.1. Permeable Pavement

The permeable pavement portion of the study area was excavated to a depth of more than 1.5 m with a 1.5% grade towards the sampling vault. The excavation was lined with an impermeable geotextile. Three rows of weeping tile, wrapped in filter socks, were placed on top of the liner, and covered with granular for structural stability (Figure 4.3). The entire excavation was subsequently backfilled to an average depth of 1 m using the native soil. The weeping tile was connected to a porous tank collection system manufactured by Atlantis® (Figure 4.3), which in turn directs the infiltrated water to a sampling vault for quantity and quality monitoring. A similar collection trough (modified Atlantis® tank) was constructed at the edge of the parking area to collect surface runoff from the permeable pavement area during heavy rainfall. This trough also drained to the sampling vault (Figures 4.2 and 4.3).

The backfilled native soil (hereafter referred to as the subgrade) was compacted to approximately 100 % Standard Proctor Maximum Dry Density (SPMDD) in order to provide adequate structural foundation for the parking lot. A geoweb material was then placed over the compacted subgrade soil and covered with 45 cm sub-base course of granular 'A', which was also compacted to 97 % SPMDD. The granular 'A' material consisted of crushed, screened limestone with particles ranging in size up to 20 mm. A 15 cm bedding course layer composed of finer clear stone was graded and the permeable pavers were installed. The voids between the pavers were subsequently filled with screenings to allow rapid infiltration and for public safety. In addition to providing structural support, the base course (bedding and sub-base granular material) provides active storage of runoff for storms up to the 50 year design storm.

Small paved speed bumps were constructed along the perimeter of each catchment (*i.e.* permeable pavement, bioswale, control) in order to prevent intermixing of runoff. A trough on the conventional asphalt was also installed to assist with drainage into the trough (Figure 4.4).

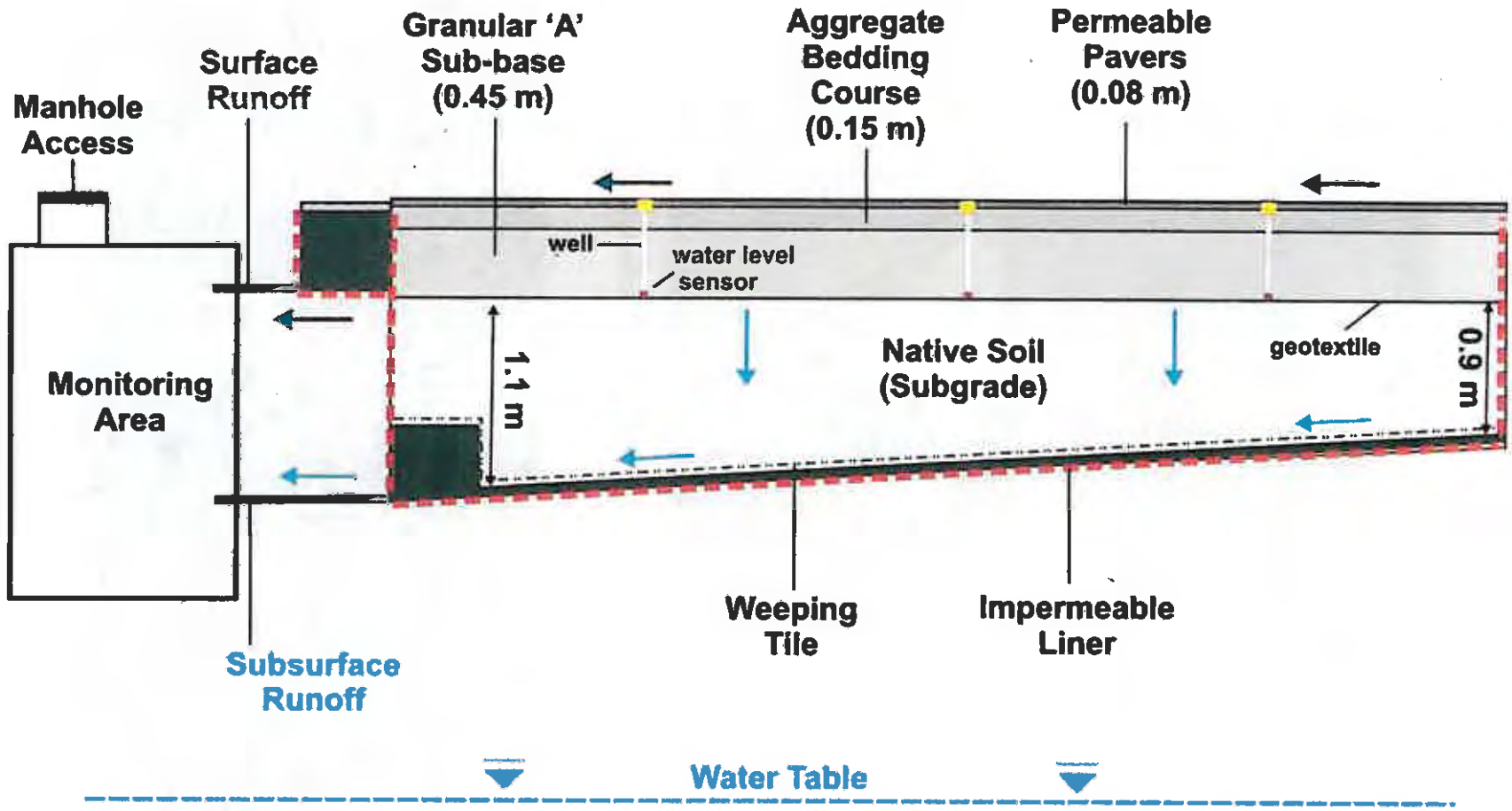

Figure 4.2: Permeable Pavement Design, Cross Section

4.1.2. Bioretention Swale

An area adjacent to the down-gradient edge of the parking lot was excavated to approximately 1m and graded towards the sampling vault. For monitoring purposes, the entire excavation was lined with an impermeable plastic liner and a weeping tile underdrain wrapped with a filter sock was installed at the bottom, and covered with granular (as under the permeable pavement soil). The native soil was replaced with screened 3:1 garden soil, lightly compacted, graded to form a shallow depression for storage, and layered with cedar mulch. Drought tolerant plants were planted on top of the swale (Figure 4.5). As in the permeable pavement design, the impermeable liner was necessary to ensure that all water passing vertically through the trench is collected in the under drain and directed to the sampling vault for monitoring.

The swale was originally sized to accommodate a ponding volume equivalent to runoff from an 11 mm storm. The depression area is overtopped during events with rainfall greater than the combined ponding volume, soil storage volume and infiltration capacity of the swale. Excess runoff flows towards grass swales by way of pre-construction flow paths and ultimately infiltrates into the ground.

Figure 4.5: Bioswale during dry weather (left) and wet weather (right)

4.1.3. Infiltration Trench

An infiltration trench was installed at the outlet to the sampling vault to ensure that flows from the study site recharge groundwater. The trench is connected to a 30 cm drainage pipe exiting the monitoring vault. It is designed to store water and gradually release it into the ground via infiltration. An overflow structure was also installed to ensure that runoff from the study areas did not backup into the monitoring vault. The overflow runoff ultimately discharges to a vegetated swale and back into the ground. Pictures of the trench installation are presented in Figure 4.6.

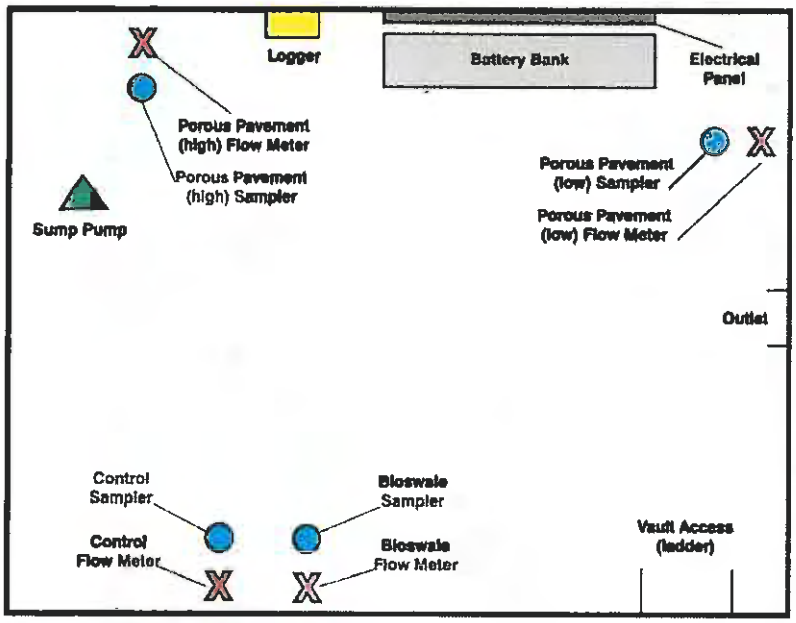

Figure 4.7: Monitoring vault floor plan

4.1.5. Power Supply

Electrical supply to operate the monitoring equipment is supplied by one 300w Southwest Windpower H40 wind turbine and 3 Sharp solar panels (1 x 165w and 2 x 170w). Designed and built by John Meulendyks of Northpoint Power Center and Seneca College, the wind turbine and solar panels are regulated using two controllers (one for the solar panels and one for the turbine) and an inverter to generate 12, 24, and 120 VDC. A battery bank consisting of 24 VDC is used to store generated energy and has a capacity of 1000 Ah which would provide just over 9 days of system operation with no energy input (Figure 4.8).

Figure 4.9: Raingauge and logger

4.2.1.2 Flow

Flow rates from surface runoff and infiltrated water are measured using four Endress and Hauser Promag 53W electromagnetic flow metres located in the underground sampling vault. Electromagnetic flow metres operate according to Faraday's principle of electromagnetic induction which states that a conductor (water) moving through an electromagnetic field generates a voltage proportional to its velocity. Proper function of the metres requires that they be continuously submerged in water. This was achieved by installing the meters within reverse slope pipes (Figure 4.10). The sensor was positioned away form the lowest point in the drain and a sediment cleaning valve was installed to avoid risk associated with solids accumulation.

All four flow metres are connected to a single Endress and Hauser Memograph logger. Data is logged continuously and recorded at a 1 minute interval. All flow is directed to a single 12" outlet and into a combined infiltration trench and overflow structure, as discussed above.

4.2.1.3 Surface Water Level and Storage

Ponding depths on the bioswale and water level changes in the permeable pavement base course are monitored continuously with five pressure transducers. Two are located in the bioswale and three are located in the permeable pavement base course, as shown in Figure 4.1. These sensors allow for an accurate determination of surface storage during rain events and indicate the time at which surface overflow occurs. These sensors are also equipped with temperature monitors, which indicate when water at the measurement point is above or below freezing in the winter. Monitoring of surface and base course water levels began in the spring of 2006.

Table 4.1: Water quality variables currently being monitored. List is subject to change.

	Variable	units	MDL	Guideline*
General Chemistry	Chloride	mg/L	0.2	230
	Mercury	ug/L	0.02	0.2
	Oxygen demand; chemical	mg/L as O2	1	
	Sodium	mg/L	0.1	
	Hardness	mg/L	1	
	Fluoride	mg/L	0.05	
	Sulphate	mg/L	2.5	
	Phenolics; 4-AAP	ug/L	0.2	
	Oxygen demand; biochemical	mg/L as O2	0.2	
	Solids; suspended	mg/L	2.5	
	Solvent extractable	mg/L	1	
	Conductivity	uS/cm	1	
	pH	none	-	6.5-9.5
	Alkalinity; total fixed endpt	mg/L CaCO3	2.5	
	Turbidity	FTU	0.01	5
Nutrients	Nitrogen; ammonia+ammonium	mg/L	0.002	1.4
	Nitrogen; nitrite	mg/L	0.001	0.06
	Nitrogen; nitrate+nitrite	mg/L	0.005	
	Phosphorus; phosphate	mg/L	5E-04	
	Phosphorus; total	mg/L	0.002	0.03
	Nitrogen; total Kjeldahl	mg/L	0.02	
Bacteria	Escherichia coli	c/100mL	-	100
	Fecal streptococcus	c/100mL	-	
	Pseudomonas aeruginosa	c/100mL	-	
Metals	Aluminum	ug/L	11	75
	Barium	ug/L	0.2	
	Beryllium	ug/L	0.02	11
	Calcium	mg/L	0.005	
	Cadmium	ug/L	0.6	0.1
	Cobalt	ug/L	1.3	0.9
	Chromium	ug/L	1.4	8.9
	Copper	ug/L	1.6	5
	Iron	ug/L	0.8	300
	Magnesium	mg/L	0.008	
	Manganese	ug/L	0.2	
	Molybdenum	ug/L	1.6	10
	Nickel	ug/L	1.3	25
	Lead	ug/L	5	5
	Strontium	ug/L	0.1	
	Titanium	ug/L	0.5	
	Vanadium	ug/L	1.5	7
	Zinc	ug/L	0.6	20

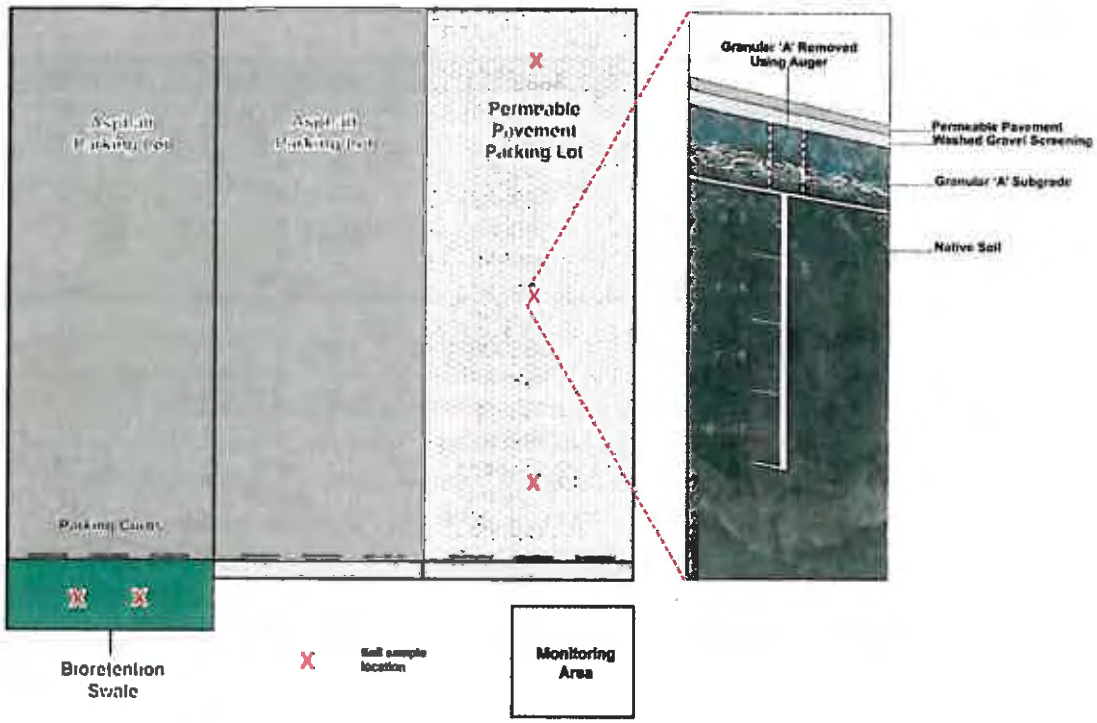

Figure 4.11: Soil sample locations in 2005

5.0 MONITORING RESULTS AND ANALYSIS

5.1. Water Quantity

A total of 37 events with precipitation depths greater than 5 mm were captured between September 15, 2005 and October 22, 2006. Twelve of these occurred during the winter (December to April). Table 1 shows preliminary hydrologic statistics for 12 runoff events that had clear start and end times. A 'runoff event' was defined as the period between the beginning of flow to the return to no flow, and as such often included several discrete rainfall periods. Surface flows from the two BMPs are not included because, as indicated previously, there was only one event with surface flow from the permeable parking lot, and bioswale overflow volumes were not monitored. Rainfall during these 12 events ranged from 6 to 35 mm.

Table 5.1: Hydrologic statistics for selected runoff events (n = 12)

Hydrologic Parameter	Control	Infiltrate	
		Permeable Pavement	Bio-retention Swale
Total Flow Volume (m^3)	37.0 [1]	33.5	16.7
Average Peak flow (L/s)	2.2	0.05	0.04
Average Flow Duration (hrs)	2.0	73.3	43.3
Average Rainfall-Runoff Lag (hrs)[2]	negligible	5.5	2.5

1. Control volumes were adjusted upwards by 20% based on known losses within the pipe connecting the collection trough to the sampling vault (this problem has since been rectified). 2. Lag times were calculated from the beginning of rainfall to the beginning of runoff.

Volumes and peak flows were greatest from the conventional asphalt surface, followed by the permeable pavement and bio-retention swale underdrains. The difference in runoff volumes between the control and permeable pavement may be a result of evaporation, but unintentional losses of infiltrated water through tears in the impermeable liner could not be ruled out. Some losses through leaks in the liner may also have occurred in the bioswale system, but most of the volume differences in this case are likely due to higher evapotranspiration rates and overflows during runoff events greater than 15 to 20 mm in size. The permeable pavement responded to rainfall events and drained more slowly than the bioswale because of the less permeable soils and lower hydraulic head (*i.e.* the smaller mass of water per unit area).

5.1.1. Hydrograph Analysis

Among the many rainfall events monitored, only one produced surface runoff from the permeable pavement. This storm was the largest event monitored, producing 72 mm of rain over a period of 5.5 hours (Figure 5.1). Rainfall events of this size and intensity are very rare in the Greater Toronto Area. Runoff from the permeable pavement measured less than 10 percent of total runoff for the event. This runoff occurred late in the storm after approximately 48 mm of rain had fallen. Over 95% of the rain had drained from the system within 24 hours. During the same event, the bioswale experienced significant overflow, infiltrating only 11% of total runoff from the contributing drainage area.

Permeable pavement infiltrate concentrations of oil and grease were frequently below laboratory detection limits, and tended to decrease over time (although the downward trend was not statistically significant). The bioswale infiltrate contained lower concentrations than the control runoff, but mean concentrations were not statistically different. The elevated levels of oil and grease in the bioswale infiltrate likely originate from undegraded oils in the manure and compost rich garden soils.

Concentrations of copper were similar in samples from all three study areas. Zinc and lead concentrations were significantly lower in the bioswale and permeable pavement infiltrate than in runoff from the control. Copper and zinc are natural micronutrients in soils; hence low concentrations would be expected even in infiltrate samples from relatively undisturbed soils. The extent to which infiltrate concentrations differ from surface runoff concentrations depend largely on the concentration at the surface. This is illustrated by the concentration ratios presented in Figure 5.4. As zinc levels in surface runoff increase beyond about 5 μg/L, levels of zinc in permeable pavement infiltrate become an increasingly smaller proportion of surface runoff levels (*i.e.* ratios in Figure 5.4 increase above 1). The relationship is similar for copper, but the threshold concentration at which ratios increase above 1 is higher, at roughly 8 μg/L. When surface runoff concentrations fall below this threshold, infiltrate concentrations tend to be greater than those measured at the surface.

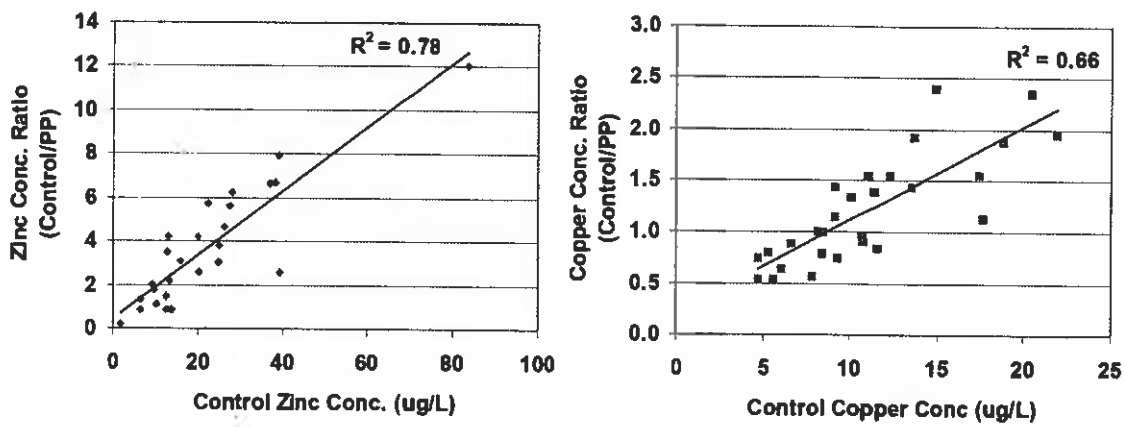

Figure 5.4: Relationship between control concentrations of zinc and copper and the ratio between control and permeable pavement concentrations for the same constituents

5.3. Winter Data

The temperature and flow data collected during the winter of 2007 had not yet been fully analyzed and quality control checked at the time of writing and, therefore, should be regarded as preliminary. Observations included in this section may be revised in the final report as new information becomes available and the data undergo more rigorous scrutiny.

Acknowledging that the data are still in a preliminary stage of analysis, Figure 5.5 shows the precipitation, air temperature, surface temperature (in the pavement base course and bioswale surface soil), surface water level fluctuations and surface/subsurface flows for the period from January 3rd to March 6th, 2007. Additional winter monitoring data collected in 2006 are provided in Appendix B.

The early January period was unusually mild, with several periods of rain. Temperatures began to drop for a prolonged period in mid January accompanied by snow (note that the small amount of rain on January 27th was measured at the nearest airport several km from the site; the lack of any flow response on the control surface suggests that all of the precipitation at the study site was in the form of snow). The base course layer drained shortly thereafter as the snow was plowed to the eastern edge of the parking lot. It is assumed that the water in the base course drained rather than froze because temperatures in the vicinity of the well (*i.e.* near the bottom of the base course) did not fall below freezing until 9 days later. Clearly there is a substantial delay between air and subsurface temperatures as the base course remained above zero even when air temperatures fell to -20°C. On average the base course temperature was approximately 8°C higher than the air temperatures over the January 3rd to March 6th period.

Figure 5.6: Winter pictures of the permeable pavement and bioswale

The bioswale surface soils rarely fell below zero during the winter, likely due to the insulating layer of snow above the swale and microbial activity in the organic rich soils (Figure 5.6). The 'bio2' sensor registered higher temperatures than the 'bio1' sensor because it was located at a lower elevation in the swale and was therefore better insulated. Despite the warm soil temperatures, however, there appeared to be very little snowmelt or infiltration, possibly due to ice formation on top and around the perimeter of the swale, which blocked flow of melt waters from the pavement and surrounding areas. An alaternative hypothesis is that there may be a small leak in the impervious geotextile lining the swale, allowing slow flowing melt waters to infiltrate into the underlying soil, rather than (as designed) into the sampling vault where flow is being measured. Whatever the cause, surface ponding and infiltrate flows from the bioswale during the winter were measured much less frequently than from the permeable pavement. Further investigation of winter snow melt and infiltration dynamics on the bioswale will be conducted next winter.

There was some concern that ice formation within the permeable pavement base course layer would inhibit or delay infiltration when the first rain occurred after an extended cold spell, potentially causing excessive buildup of water in the base course, eventually leading to flow over the surface. This did not occur. The permeable pavement continued to function normally during winter rain events, with minor amounts of infiltrate measured even during very cold periods. Major runoff events observed on the

The quality of permeable pavement sediment samples (the subgrade, or in some cases the lower base course) was compared to samples taken from nearby reference sites to assess whether or not infiltration of road runoff contaminants had contaminated underlying soils. Results showed little variation in sediment quality with depth. Average permeable pavement concentrations were either similar to or lower than sediment concentrations from the reference sites (Figure 5.7), with no obvious relationship to pavement age, design or soil type. The one notable exception among the selected variables shown in Figure 5.7 was chloride, which is highly soluble and does not bind to soils like most other roadway contaminants. Chloride accumulates in the subgrade over time, but would be expected to eventually leach from the soil into groundwater.

These preliminary results imply that attenuation of most road runoff contaminants is occurring primarily in the bedding and upper sub-base layers, and that contamination of underlying soils is not a major concern for permeable pavements. Further sediment sampling, especially in the base course layer, on reference sites and at older bioswale sites, is planned for the summer of 2007.

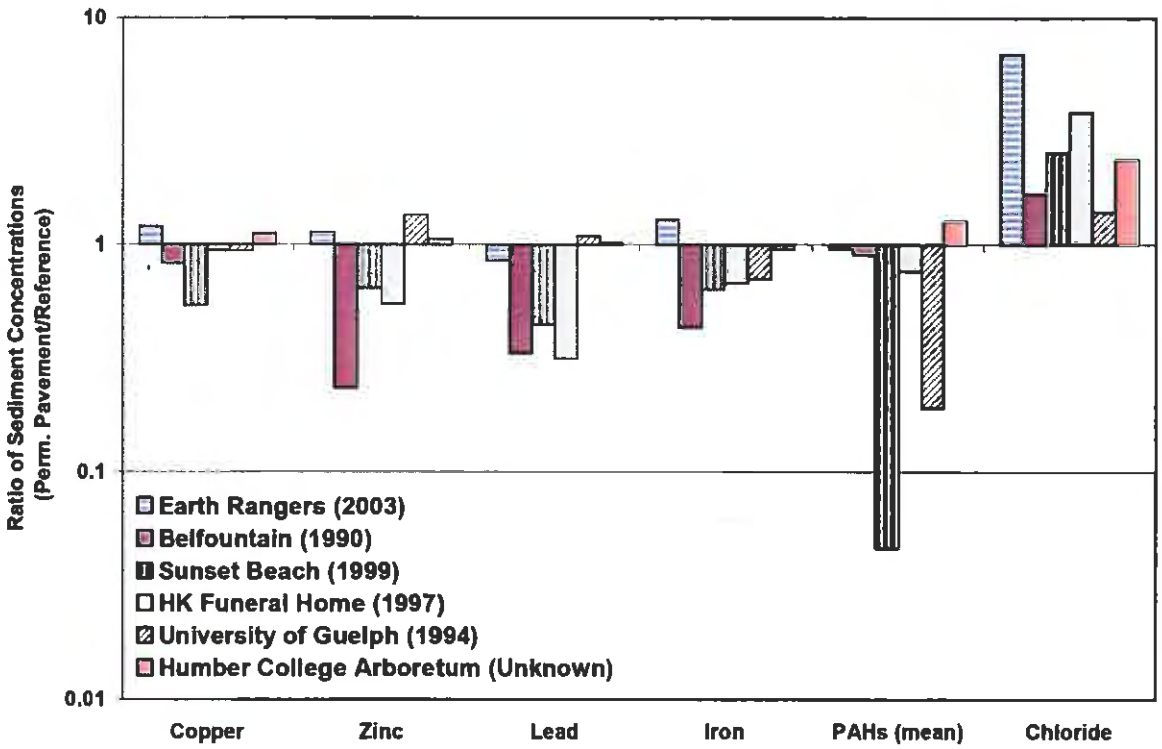

Figure 5.7: Sediment concentration ratios. Values greater than 1 indicate that permeable pavement concentrations were greater than the reference site, and vice versa.

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Appendix I: Standard Operating Procedures Under Development

Identifier:	Revision:	**Error! Unknown document property name.**	Regional Stormwater Standardization Project: Monitoring Procedures
NPDES CM 1000	**Error! Unknown document property name.**	5/27/2008	
Custodian:		Authorization Authority:	
Dean Wilson		**Error! Unknown document property name.**	

NPDES Stormwater Monitoring Program

STANDARD OPERATING PROCEDURE

Continuous Measurement of Discharge

Stormwater Monitoring Collaboration
Western Washington

This document is part of the Regional Stormwater Standardization Project and describes standard operating procedures to ensure a systematic consistent approach is followed for collecting, assessing, and documenting environmental data of known and documented quality.

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A. Scope and Application

This Standard Operating Procedure (SOP) applies to those activities that involve the collection of continuous discharge data at NPDES monitoring sites on streams, stormwater conveyance systems and BMP's. It describes equipment and site selection factors, installation, operation and field measurement techniques. A continuous flow monitoring station is commonly called a stream gage or gaging station.

The NPDES municipal stormwater permit calls for permittees to develop and implement a comprehensive long term water quality monitoring program. The monitoring program has two elements: stormwater and receiving water monitoring, and BMP effectiveness monitoring. Both elements require the production of continuous records of discharge in the stream channel, stormwater conveyance system, or BMP. The equipment and methods used must enable the collection of flow-weighted composite storm samples, base flow samples, and the production of a time series data set of flow rate. From the flow data set flow durations and volumes can be calculated and hydrographs produced.

Storm runoff and base flow is collected and transported through natural channels, ditches, culverts and engineered pipe and treatment systems. Each monitoring site will have individual characteristics that require a specific configuration of equipment and installation that best enables the collection of accurate flow data. A successful location for continuous flow monitoring features stable hydraulics and either a convenient place to directly measure discharge or the ability to install a primary flow measuring device such as a flume or weir. These factors are less important to the selection of a water quality monitoring site representative of a specific land use or activity. If an important product of the monitoring is a calculation of pollutant loadings, the importance of selecting a location that provides accurate determination of continuous flow should not be discounted.

B. Associated Documents and SOPs

The United States Geologic Survey Water Resources Division is a rich source of guidance for establishing and operating continuous discharge monitoring stations. The Techniques of Water-Resources Investigations chapters cited in the references can be accessed and downloaded at:

URL: http://pubs.er.usgs.gov/usgspubs/index.jsp

King County Water and Land Resources Division, Science Section stream gaging program guidance documents, unpublished:

- General Gage Maintenance and Data Workup Procedures
- Protocol for Measurement of Streamflow at King County Surface Water Monitoring Sites
- Protocol for Field Visits to King County Surface Water Flow Monitoring Sites
- STREAM GAGE DATA WORKUP QA/QC

Units

Hydrographers in the United States are accustomed to using English engineering units. That custom is continued in this document. Distance is in decimal feet. Volume uses cubic feet. These can be converted to S.I. for the project.

C. Method Summary

Continuous flow monitoring involves using electronic equipment to measure and record flow in a stream or other conveyance. Flow may be measured using a water level sensor, or via an ultrasonic Doppler signal or electromagnetic sensor to measure velocity directly. A programmable data logger is used to convert the signal to recorded flow values, which are recorded at set time intervals. Flow can be used to trigger an automated sampler after a set volume has passed.

In the case of a level sensor, a relationship between the water surface elevation and the flow rate (stage-discharge relationship) is developed using various generally accepted techniques. The stage-discharge relationship represents the sum of the various forces that make water move or resist movement, primarily gravity and channel friction. It is expressed as an array or a mathematical function. Continuous stream flow is calculated by using the stage-discharge relationship to match a specific water level with a corresponding rate of flow. The automatically calculated flow rate may be adequate for producing flow weighted composite samples, but post processing is usually necessary to produce an accurate flow record and may involve using velocity as an index of flow. The procedures and tasks involved with a stream gage are designed to accurately measure and record water level and determine the stage-discharge relationship at the site.

In the case of direct measurement of flow via ultrasonic or electromagnetic devices, the sensor may be integrated with a level sensor that is used in conjunction with channel geometry to calculate area; in combination these are called area/velocity (A/V) sensors). The signal is processed according to the geometry of flume, weir, or stream, in a similar manner as that used for a simple level sensor. The equation will be specific to the sensor signals and the confinement (flume, weir, or open channel) geometry.

Mechanical propeller-driven flow meters are often used for instantaneous open channel stream flow measurements, but are rarely if ever used for continuous flow logging.

D. Definitions

BMP

Best Management Practice. An engineered stormwater control or treatment designed to effect an improvement in water dynamics or quality.

Control

designates channel characteristics that determine the stage-discharge relation at the gage. Control geometry may be a consequence of natural constriction of the channel, an artificial structure, or a uniform cross section over a long reach of the channel. Other factors affecting a control include local and downstream obstructions, slope, vegetation, and shifting bed load.

Control structure

a structure on a stream or canal that is used to regulate the flow or stage of the stream or to prevent the intrusion of salt water.

Cubic foot per second

(ft^3/s also CFS) is the rate of discharge representing a volume of 1 cubic foot passing a given point during 1 second and is equivalent to 7.48 gallons per second or 448.8 gallons per minute or 0.02832 cubic meter per second.

Discharge

is the volume of water (or more broadly, volume of fluid plus suspended sediment) that passes a given point within a given period of time.

Flow Meter

An instrument for measuring the amount of fluid that travels past a given location. May be a combined flume or weir with water level sensor that can be programmed to compute discharge from the sensed water level, or may involve electronic measurement of flow via an ultrasonic signal or electromagnetic interaction. In any case, the flow measurement device is interfaced with an electronic data logger.

Gage height

(G.H.) is the water-surface elevation referred to some arbitrary gage datum. Gage height is often used interchangeably with the more general term "stage," although gage height is more appropriate when used with a reading on a gage. *See Stage*

Gaging station

is a particular site on a stream, canal, lake, or reservoir where systematic observations of water level are obtained.

Instantaneous discharge

is the discharge at a particular instant of time.

Mean discharge

Arithmetic mean (average) discharge during a specified period. Depends on averaging period. Stream discharge is usually logged at 15 minute intervals; Small area (e.g. 5 to 10 acre) stormwater discharges may need to be logged at 5 minute intervals. Daily mean is the arithmetic mean of all readings in a 24-hour period. Other means may be by month, season, wet year, or any other desired designation.

Stage

Stream stage (also called stage or gage height) is the height of the water surface, in feet, above an established datum plane where the stage is zero. The zero level is arbitrary, but is often close to the streambed (USGS ref). *See* Gage height

Stage-discharge relation

is the relation between gage height (stage) and the volume of water per unit of time (discharge) flowing in a channel.

Streamflow

is the discharge that occurs in a natural channel. Although the term "discharge" can be applied to the flow of a canal, the word "streamflow" uniquely describes the discharge in a surface stream course. The term "streamflow" is more general than "runoff" as streamflow may be applied to discharge whether or not it is affected be diversion or regulation; and runoff may be sheet flow or channelized.

Velocity meter

electronic instrument used to determine water velocity, using an ultrasonic Doppler signal, an electromagnetic field, or a propeller .

Water year

is the 12-month period, October 1 through September 30. The water year is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the year ending September 30, 1992, is called the "1992 water year."

E. Safety and Hazardous Materials Management

The following are general health and safety guidelines. These guidelines will be read and understood by all members of the sampling crew prior to any sampling activities.

- Sampling personnel will wear chemical-resistant gloves whenever coming into contact with potentially hazardous water.
- No eating, drinking, smoking, or tobacco chewing by sampling personnel will be allowed during active sampling operations.
- All sampling operations will be conducted during daylight hours.
- All accidents, "near misses," and symptoms of possible exposure will be reported to a sampler's supervisor within 24 hours of occurrence.
- All crewmembers will be aware of the potential hazards associated with any chemicals that may be used during the sampling event.

Several hazards are inherent to streams and river sampling. Vehicle safety and specific streams/rivers hazards are discussed below.

Vehicle Safety

As a provider of various public services, King County holds safe operation above all other performance criteria. All vehicles and equipment will be operated or activities performed in a manner that reflects the highest regard for safety of the public, the employees and the property of our citizens and organization. In the operation of King County vehicles and equipment, every courtesy and consideration should be given to motorists, cyclists and pedestrians with whom we share the streets and highways. No act that endangers the public well being or our employees will be tolerated. Unsafe equipment should be reported to the appropriate vehicle supervisor.

As an employee driving a King County vehicle you should be familiar with and practice the principles of defensive driving – that is, to make every effort to avoid an accident. Always be prepared to yield. Your driving safety depends on good driving attitudes consisting of skillful and defensive driving, obedience to traffic laws and courtesy to other drivers. Driving demands full concentration. The safe driver, in assuming the responsibility of operating a vehicle, knows the rules of the road, focuses full attention on driving and uses techniques that will help avoid emergency situations.

All persons operating King County vehicles should at all times drive carefully and prudently, having regard for traffic conditions and all other circumstances.

Stream and River Hazards

There are many hazards associated with stream gaging. Some of these hazards include traffic, fast moving or deep water, steep slopes to sampling sites, and hostile dogs or people. Use extreme caution when exiting vehicles, walking along busy highways or measuring from bridges. Orange reflective safety vests are required

when operating in the road right-of-way. Fast moving water can cause the monitoring personnel to lose balance and fall into the water. This can result in injury or drowning. Many of the sampling sites require personnel to walk over riprap or other extremely rough and slippery terrain. Extreme caution combined with slow movements can minimize potential injury. Be aware of your surroundings and potential presence of other people, especially under bridges or in culverts. Each field scientist will use her or his best judgment to determine the safety of sample collection at any particular site and may choose not to engage in sampling operations.

F. Stream Gage Site Selection

The key factor to siting an accurate stream gage is the hydraulic control. In a natural channel, this is the feature that physically affects the water surface elevation and creates the stage-discharge relationship. In an engineered conveyance, it is the artificial structure that controls the water level. A site with consistent hydraulic conditions will give the most accurate flow data over a range of conditions with the least effort. Other factors such as access and right of entry and potential cost of the installation must be considered.

- Open channel sites use a series of measurements made over as full a range as possible in stage of the stream plotted against the corresponding stages to define the stage-discharge relation. The stage-discharge relation will be stable if the hydraulic characteristics of the general reach of stream are unchanging and the bed material does not move appreciably. For measuring flow by wading with a meter, choose sites with a generally straight course and where the flow is contained in a single channel, free of large woody debris and large in-channel rocks that can create turbulence, back-flow, or dead-zones. Slope should be gentle; too steep slope will result in critical flow exceedance, and the stage-discharge relationship may be poor. The channel should not be altered by frequent sediment deposition or erosion. A pool behind a rock outcrop or other constriction is a good location for gauge placement, but not for flow measurement. Vegetation growth on the stream bank should not impede the water velocity or contribute flow-impeding debris to the stream. Downstream conditions such as log jams can create a variable hydraulic condition so that the stage-discharge relation will be unstable, changing with time and with the flow conditions. A channel with a stable hydraulic control may have its stage-discharge relationship defined with as few as six or eight direct measurements of discharge and produce reliable data if the measurements cover a wide representative range of stage heights. A site with unstable hydraulics may require weekly discharge measurements to produce a data set with barely acceptable reliability. In either case, a good location for making cross section measurements of discharge in all flows must be available.
- Flow through pipe systems may be measured at access points such as catch basins. Choose locations with sufficient head loss in the outlet pipe that the flow through the pipe can be calculated by measuring water level at the entrance (inlet control). Turbulence in the catch basin will degrade the accuracy of the water level measurements. It is difficult to accurately measure flow at the outlet to a pipe system.

- Primary control devices such as flumes and weirs may be constructed or installed in open channels or catch basins to provide a stable hydraulic control. Weirs are small dams with engineered openings or notches designed to accommodate the expected range of flows. Channels with a high sediment or bedload are not appropriate for weirs because the sediment will be trapped and build up behind the weir making it less accurate. Flumes are designed constrictions in the channel that produce specific hydraulic conditions that enable flow to be determined from water level. Flumes pass sediment through better than weirs, and can retain accuracy in variable backwater conditions when A/V sensors are used, but not when level sensors alone are used. Weirs and flumes take considerably more effort to establish than using existing hydraulic conditions, but they can make a poor flow measurement site into an acceptable one. Care must be taken to choose the appropriate device and size it correctly for the specific conditions at the site.

G. Apparatus, Equipment, and Consumables

Installed equipment will include a data logger or flow meter and a water level sensor. Other sensors that measure parameters such as precipitation and air and water temperature may be installed and recorded by the data logger.

- Each site will require specific tools and supplies depending on the installed equipment and design. Tools include the following:
- Portable computer, preferable weather resistant for communication with data logger
- Various interface cables for specific instruments
- Multimeter for circuit checking and monitoring battery voltage.
- Flow measuring equipment
- Water quality instruments
- Thermometer
- 25 foot steel tape, engineering units
- screw drivers, including a small one for terminal blocks
- small wrenches, hex and open end
- Five gallon bucket for flushing wells and making volume measurements
- Ladder
- Writing instruments, and field sheets and clip board
- Flash light and head lamp
- Water proof boots and chest waders
- Umbrella
- Shovel and brush removal tools
- Supplies include:
- Desiccant for the enclosure and if required for instrumentation
- Batteries
- Lubricant for locks and enclosure gaskets
- Electrical and duct tape
- Cork dust for crest stage gage
- Field Sheets and flow forms, writing instruments for waterproof paper

Procedures

Installation

Datum

For an open channel, establish a reference point (gage datum) from which to determine water surface elevation. This may be a post driven into the channel until stable, or an existing permanent structure. A staff plate may be installed to read the water level directly, or a point to measure down to the water surface may be used, e.g. from a bridge. It is important to establish this setup correctly so it can be maintained for the life of the gage. The datum may be given an arbitrary elevation or tied to a known benchmark elevation. If a staff plate is installed in the channel, then that scale becomes the reference for all other water surface measurements at this location. It is not necessary to survey the actual elevation of the gage datum unless the study requires that information. A rod and sighting level should be used to establish reference points on the gage structure, stilling well, and other points independent of the gage. The points on the gage structure will be used to measure water level. Two measurement points should be established so that field personnel have an alternative if one point is unavailable. Observing stream stage from both measurement points each site visit will verify the stage observation. The independent points can be used to reestablish the datum if the gage structure is damaged.

For a weir or flume with a level sensor, the zero water level point is synonymous with zero flow. The ideal setup is in conjunction with a submerged level sensor – behind the weir, or in a stilling well attached to a flume. In either case, the signal from the level sensor must be compensated for in the data logger, to represent zero when the water level is zero. The geometric relationship between a weir notch or flume geometry will define the stage to flow relationship. Ideally this will be calibrated by a dynamic flow test prior to installation. Weirs and flumes must be maintained at level in the field, in both horizontal axes.

Instrument Shelter

Install the housing for the recording equipment well above the highest water level expected. If the flow recorder is going to interface with an automatic sampler, cabling between the two will be necessary. At a catch basin location, all equipment may be installed below basin cover, depending on particular logistics, but access to the equipment will be difficult and everything gets wet and dirty. Confined space entry requires special safety considerations, and should be avoided whenever possible. Above grade instrument shelters are preferable.

Stilling Well

The water level sensor works best if it is installed in a stilling well. The stilling well is a chamber that is attached to the main flow channel by a small orifice. The stilling well protects the sensor and dampens the short term surges in water level due to non

flow related effects. The water level measured in the stilling well becomes the primary record of the stream gage. The stilling well should extend below the lowest expected water level so the sensor is always in water. The design should allow the well to be cleaned of accumulated sediment. The sizing and placement of the stilling well will depend on the specific site conditions.

One of the gage reference points should be the top of the stilling well so that a determination of the water level the sensor is measuring can be made.

In an open channel or ditch, a 1.25" – 2" diameter drive well point can be driven into the streambed and used as the base of a stilling well. In a catch basin a 2" – 6" diameter pipe can be bolted to the chamber wall or attached to an existing structure such as a ladder. Proper placement of the stilling will behind pipe inlets and artificial controls such as weirs and flumes is important for accurate determination of the head value used in the rating formula for the structure.

Water Level Recorder

Choose a data logger and water level sensor matched to the project requirements and monitoring site characteristics. The water level instrument should have a system accuracy of +/- 0.01 feet. Sigma, Marsh McBirney, Isco, Global Water, and Campbell Scientific flow meters, pressure sensors, and data loggers can be used to measure flow, and be programmed to drive automatic samplers and interface with a variety of telemetry options. Use a digital shaft encoder with a float and pulley to make the most accurate water level readings if that is a priority. Pressure sensors are commonly used to record water level. Properly installed with a quality data logger they can give +/- 0.01' accuracy or 0.1% of the full scale range. Submersible pressure sensors measure water level directly and are either sealed or vented. Vented sensors which automatically adjust for the effect of atmospheric pressure on the water pressure are best where the results will be used immediately to calculate discharge. Care must be taken to keep the vent tube dry to prevent sensor malfunction. Sealed or non-vented sensors must be corrected for barometric pressure before the data can be used. External pressure sensors, commonly called bubblers, measure the pressure required to push a gas (most conveniently ambient air) into the water column. The advantage of bubblers is that a small vinyl tube is the only element submerged and it may be easier to install than a submersible sensor.

An installation with wireless or telephone telemetry can improve reliability of the data collection.

Site Documentation

Create a thorough record of the site characteristics. Include qualitative and quantitative descriptions of the structures, equipment, instruments, channel or conveyance and hydraulic control. List personnel involved with the project and their contact information. Obtain latitude/longitude decimal degrees or State Plane coordinates for the location from GIS or GPS. Specify the datum type. Describe the location with address and driving directions. Include any information necessary for

accessing the site including parking location and any issues with neighbors. If the project requires altitude include that. Document the benchmarks and levels of the reference gage and any other points used to measure water level. Include the anticipated locations and methods for measuring discharge in different flow conditions. Take photographs of the site.

Station Log

Create a station log with an all weather notebook or custom form. The log is left secured at the station, and each site visit is noted with date and time, staff name, stage, and any activities, including remedial/maintenance.

Field Procedures

Site Visits

General

Field staff visits to the NPDES monitoring sites will maintain the stream gage as well as water quality sampling equipment and making in situ measurements. Stream gage maintenance can be fit in around the other activities. Briefly, the stream gage site visit includes an observation of stream stage, maintenance of the data logger and water level sensor, measurement of discharge, and maintenance of the hydraulic control. A visit may include checking the stage to flow relationship, especially after a sever storm.

Time

Time series data do not adapt to adjustments in time zone such as Daylight Savings Time (DST). Field staff should agree on a time convention and always note the time convention used, e.g. Pacific Standard Time (PST), DST, Coordinated Universal Time (UTC).

Field Notes

A standard form should be used for stream gage field notes. One can be purchased from a supplier of hydrologic equipment or one can be designed in house. In either event, forms should be printed on waterproof paper. Field notes should be legible and complete. Time should be noted to the nearest minute. Any computations used to derive values should be made on the field notes. Note the weather, general flow conditions, any adjustments made to the equipment. High water marks and debris accumulations should be noted. The field notes may be considered legal documents for the purpose of the NPDES Municipal Stormwater permit. As such, original observations should not be erased. If a notation is in error, it should be crossed out and the correct observation noted.

Observe Stage Height

Observe the stage height at the reference gage and in the stilling well before altering any equipment or conditions. Note the stage height to the nearest 0.01 feet. If there is uncertainty in the observation, note that with a +/-, e.g., 4.23' +/- 0.02'. Where stage height is determined by taping down from a point, note the datum and show the calculation. Other parameters such as air and water temperature should be noted at this point. For each site visit the stage height should be noted even if nothing else is done. This will increase the accuracy of the continuous water level record.

Data Logger

Data from the data logger may be downloaded to a laptop computer or other device at every visit, if necessary at this point. This should be done as a safety backup, even if data are normally downloaded via modem. Connect to the data logger with a field computer and note the live readings on the field sheet. The difference in the recorded elevation from measured gage height (offset) is computed and compared with previous visits. An inordinate change in offset between visits indicates that water elevation sensor installation is not working correctly and a repair should be made. Download the recorded values and view a plot of the data. The plot should be smooth and sensible. If there were any sudden jumps in water level, investigate to see if it was due to the water level sensor changing its relative position. Make sure the timestamp for the last recorded value corresponds with the expected time. The field sheet should be designed to prompt the calculations to assess sensor accuracy. This is the time to investigate any anomalies in the data.

Perform maintenance to the equipment. Change batteries and desiccant. Check electrical connections. Remove dirt and insects from the housing. Make sure flow control devices (weirs and flumes) are level. If larger repairs are needed, note the details on the field sheet. Restart the data logger if necessary, monitor and note the initial reading and observe and note the stream stage.

Measure Flow

Depending on the installation, a direct measurement of discharge may be necessary to verify or establish the stage-discharge rating. Flow measurements will use the wading cross section method and the USGS mid-section method of calculation. Most agencies have procedures established based on the specific equipment they own. Price AA and Pygmy current meters come with standard ratings and require no additional calibration as long as they pass the recommended spin tests. Other meters require that the manufacturer specified calibration procedures be followed. Use a top set wading rod to speed the placement of the current meter in the vertical profile. Make observations of gage height before and after the measurement. Do not depend on the water level recorder to provide the gage height. When measuring low flows, make a determination of the gage height or point of zero flow (gzf or pzf) to assist in extending the rating below the lowest flow measurement. Any measurement should follow the process described below.

Wading Cross Section Measurement Steps.

- ***Prepare the velocity meter. Assemble the parts and perform the manufacturer specified tests to verify operation. Note the results on the field notes. If the flow meter does not pass the operation tests or appears damaged, do not perform the measurement until a repair is made and the meter is operating correctly.***
- ***Choose a cross section that is straight and free flowing upstream and downstream. Best accuracy is obtained by measuring where the stream can be waded. There should be no tributaries or diversions to the stream between the point of interest (e.g. gage location) and the cross section. The cross section should be located in a straight reach with flow direction parallel to the channel. The channel should have a smooth stream bed and be free of obstructions that create turbulence. Water depth should be adequate for the velocity meter to work accurately. Since few streams offer an ideal cross section find the section that best meets the guidelines. Some modifications to the channel can be made to improve the measuring section. It is not necessary to use the same cross section location for all flow measurements. A particular location in the reach may be advantageous for measuring low flow but present problems measuring higher flows. Use of the same cross section for high flows may assist in estimating flows above the highest flow measurement.***
- ***When a measurement section has been found, stretch the tape across at right angles to the flow. Note the overall width and determine the average spacing of the subsections. Record the gage height, time, and edge of water (right or left facing downstream), date, party, type of meter. Place the wading rod in the first section and record the depth as read on the rod (do not read water pile-up on the rod as part of the depth) and the distance along the tape. It is good practice to minimize the distance from the edge of water to the first and last sections in order to minimize the area of flow not measured.***
- ***Use the scale located near the top of the wading rod to set the velocity meter at the proper measuring depth. If the depth is less than 2 feet, set the scale to the depth directly. This will adjust the velocity meter to the proper 0.6 of the depth below the surface. If depths are greater than 2 ft., then velocity measurements at 0.2 and 0.8 of the depth below the surface are made and the velocities averaged. To set the meter 0.8 from the surface, set the scale at half the measured depth. To set the meter for 0.2 from the surface, set the scale at twice the measured depth. Velocities are averaged over 30 seconds per***

measuring point. Note the angle of flow if it deviates noticeably from the perpendicular and multiply the subsection flow by the cosine of the angle. Angle multipliers are indexed on the edge of the standard stream gage field sheet.

- ***Proceed to measure at the rest of the sub-sections. Vary the width of the individual sections to work around obstacles. Sections should be closer in deeper, faster flow. The goal is to have no more than five percent of the total discharge from any individual section. Note the gage height, time and edge of water at the end of the measurement. The summation of the sub-interval discharges is the total discharge.***
- ***Perform a post measurement operation test of the velocity meter and note the results. Note the stream gage height.***
- ***One out of every ten integrated stream flow measurements should be repeated independently by other field staff. It is functional to have the flow measurer and data recorder exchange rolls. Integrated stream flow relative percent difference should not exceed ten percent.***
- ***During periods of rapidly changing stage, accuracy may be improved by completing the measurement quickly. Reduce the time used to average the velocity measurements and reduce the number of cross sections.***
- ***A flow calculating database routine, computer program or spreadsheet template is used to calculate and sum the sub-interval values. It is good to calculate the discharge in the field and compare the results to previous measurements and the rating curve. In any case, the flow should be worked up and reviewed as soon as possible.***
- ***The measurement quality should be rated as excellent, good, fair, or poor based on the observations that comprise the measurement: determination of depth and measurement of velocity. A measurement in ideal conditions, a smooth stream bed, adequate depth, and even sub critical flow should be rated excellent. Measurement of a shallow stream cascading over cobbles will have a large error in determining both depth and velocity and should be judged poor.***

Indirect methods.

Indirect methods of measuring discharge are available for determining peak discharge in open channels and culverts when it was not possible to measure a high flow. Careful survey of high water marks will make calculation as accurate as possible.

Volumetric measurements.

Volumetric measurements of flow are a good way to determine low flows. Discharge less than 0.4 cfs can be collected in a bucket from a pipe outfall, V-notch weir, or flume. To make a measurement, start a stop watch the instant the bucket begins to collect flow and stop the watch when the bucket is removed. Volume can be determined by weighing the water with a hanging scale or use a calibrated bucket. Multiply pounds of water by 0.016 to get cubic feet, or gallons by 0.1337 to get cubic feet. Divide by the seconds the bucket filled to produce flow in cubic feet per second. Perform the test three times to verify the measurement.

Maintain Hydraulic Control

Sticks, vegetation and garbage can accumulate on the hydraulic control and affect the stage discharge relationship. It is standard to clean this debris after any measurement of discharge so the stream stage will not be changing in response to the cleaning during the measurement. If the impoundment behind the hydraulic control will drain quickly, the control can be cleaned before the measurement as long as the stream stage is stable when the measurement is started.

It is crucial that accurate observations and notes be made documenting the conditions affecting the hydraulic control.

Data Reduction, Reporting, Review and Documentation

Data Storage

Set up a paper file folder for each site. Paper files include the field notes, station description, charts of continuous stage data, data reduction notes.

Electronic data files downloaded from data loggers should be named systematically (typically incorporating the download date/time) and stored on a computer or network hard drive that is backed up regularly.

Electronic data should be stored in a relational database. The database requires an interface application that at a minimum has the following features:

- Each site is represented uniquely and data for that site associated with it only.
- The application will create discharge rating tables from entered points, create tabular printouts of the rating for reference, and compute flow by applying the rating to a continuous gage height record.
- The application allows the user to make adjustments to the gage height record to correct stage height errors and apply rating shifts.
- The application has a system for tracking and reporting data workup settings and updates.
- The application will provide for input of field observation data and notes as well as discharge measurement summary results to separate tables.
- It will provide a method for data to be flagged with qualifiers.

- The application should report when data are provisional and final (reviewed and accepted).
- The application will create electronic data files to report out corrected gage height data and computed discharge with time stamps and data quality flags. Both continuous data and daily summaries of mean, maximum and minimum values for the day should be reported.

Flow Measurement Computation

All cross section flow measurement data should be entered into a computer program that calculates the area, velocity, and discharge for each segment and computes the total width, area, average velocity, and total discharge for the measurement. The program should allow review of the entered data to check for entry errors. If the output of the program is an electronic file, name that with the station ID and date of measurement and store with the data logger download files.

Rating Development

A stage-discharge rating must be developed before continuous flow data can be computed. If the gage is installed behind a primary control device such as a flume, weir, or standard pipe or culvert, the rating can be obtained from published handbooks and manuals. Verification discharge measurement may suggest small adjustments to the standard rating to account for variation from the specified dimensions of the device.

Open channel ratings are based on a series of direct measurements of discharge. Typically the field staff that performed the discharge measurements and maintained the gage site develops the rating for the station. An understanding of the basic concepts of simple ratings as described in USGS publication TWI 3-A1 (Kennedy, 1984) is necessary. The key to developing a rating is to have a set of discharge measurements that span the range of stage encountered at the gage. The set of measurements should be representative of the channel in a stable condition, i.e., the hydraulic control has not changed and channel has not scoured or filled significantly between measurements. It may be some weeks or months before an adequate set of discharge measurements is acquired to properly define the rating.

By convention, the set of measurements are plotted with gage height as the ordinate (Y) axis although it is the independent variable. It is a more intuitive plot with increasing stage going up the vertical axis. Use of log scale paper will combined with an appropriate offset from the gage height will produce a straight line fit through the points. The line should be at or close to excellent rated measurements and may vary from measurements with lesser quality. Subsequent flow measurements may plot off the curve. The hydrographer must make judgments about whether they accurately represent the normal stage-discharge relationship at that time or are in error. The quality rating of the discharge measurement will provide some guidance. Good quality measurements that plot off the curve significantly will require a rating shift to be applied. Consultation with experienced hydrographers is useful.

The rating curve may be extended above the highest discharge measurement and below the lowest to encompass the entire range of recorded gage heights. Consideration of the channel shape must be taken into account.

The rating curve is entered into the discharge computation program as a series of points that define the straight line segments of the rating curve. A simple trapezoidal channel with a single hydraulic control can be described with two points on the upper and lower bounds. More complex channels will require more points to describe the curve.

Process Discharge

The gage height record must be accurate for computation of continuous discharge. It is useful to process the data in four to six week increments defined by the site visit schedule. It is useful to print a paper plot of the recorded gage height for the period. Use the same gage height range for all plots. The ideal record will have the same offset from the observed stage for all observations in the period. The plot will show response to rainfall as expected and have no unexplained jumps in level. The field notes should provide some explanation about anomalies in the stage record. It is possible that equipment malfunction or some other problem leads to periods with no gage height record or one with very poor quality. Generally, gage height data is not filled or estimated.

Once the rating table has been developed, and the necessary gage height record corrections determined, the data may be input into the database program. Work from the paper plots and note the gage height corrections and any rating shifts. Also, keep a log describing the rating curve and the corrections applied for each workup period. This data workup will be reviewed and the plots and log should explain contain enough detail that the reviewer understands what was done and why.

Periods of missing gage height record can have the daily discharge estimated based on observations, rainfall records and similar gage results. The estimated must be flagged. Periods of degraded gage height record quality or should be flagged to warn users. Periods where the computed discharge exceeds the highest measured flow by a factor of two should be considered flow estimates and flagged accordingly.

It is useful for the hydrographer to perform a review of the work immediately after production. Graphical representations of the data can expose many problems. Import daily summary discharge data (daily mean, maximum value for the day, minimum value for the day) into a spreadsheet. Include nearby daily rainfall totals, and the mean daily discharge from a similar site for the same period. Plot mean daily discharge and rainfall over time. The discharge plot should reflect the rainfall as expected. Plot the minimum daily values over time. This will expose any odd drop outs in the data. Plot the mean daily discharge with the comparison gage over time. Plot discharge on a log scale to compress the range. The sites should vary together as expected. Any problem periods should be checked out and reworked. The

spreadsheet should be stored in the directory with other station files. Use a separate spreadsheet for each water year of data.

Data Review

Mistakes can be made while preparing continuous discharge data sets. Review of the data workup by a peer is key to ensuring data quality. A final review is usually performed on a complete year of data. A systematic check list should be followed.

- The paper file is organized
- Water Year spreadsheet created
- Mean, min, max checked
- Field observations are entered and accurate
- Discharge record compared to flow measurements
- Comparison gage checked
- Flow measurements reviewed
- Rating reviewed
- Workup reviewed
- Data quality flags assigned appropriately

When the reviewer is satisfied with the workup, the database flags should be turned from provisional to final for the period under review.

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Appendix J: TOYLID Field Data Sheets/Observation Logs

TOYLID On-Site Observation Form

Date:	Time:	Personnel:
Weather:		Last Rain Event

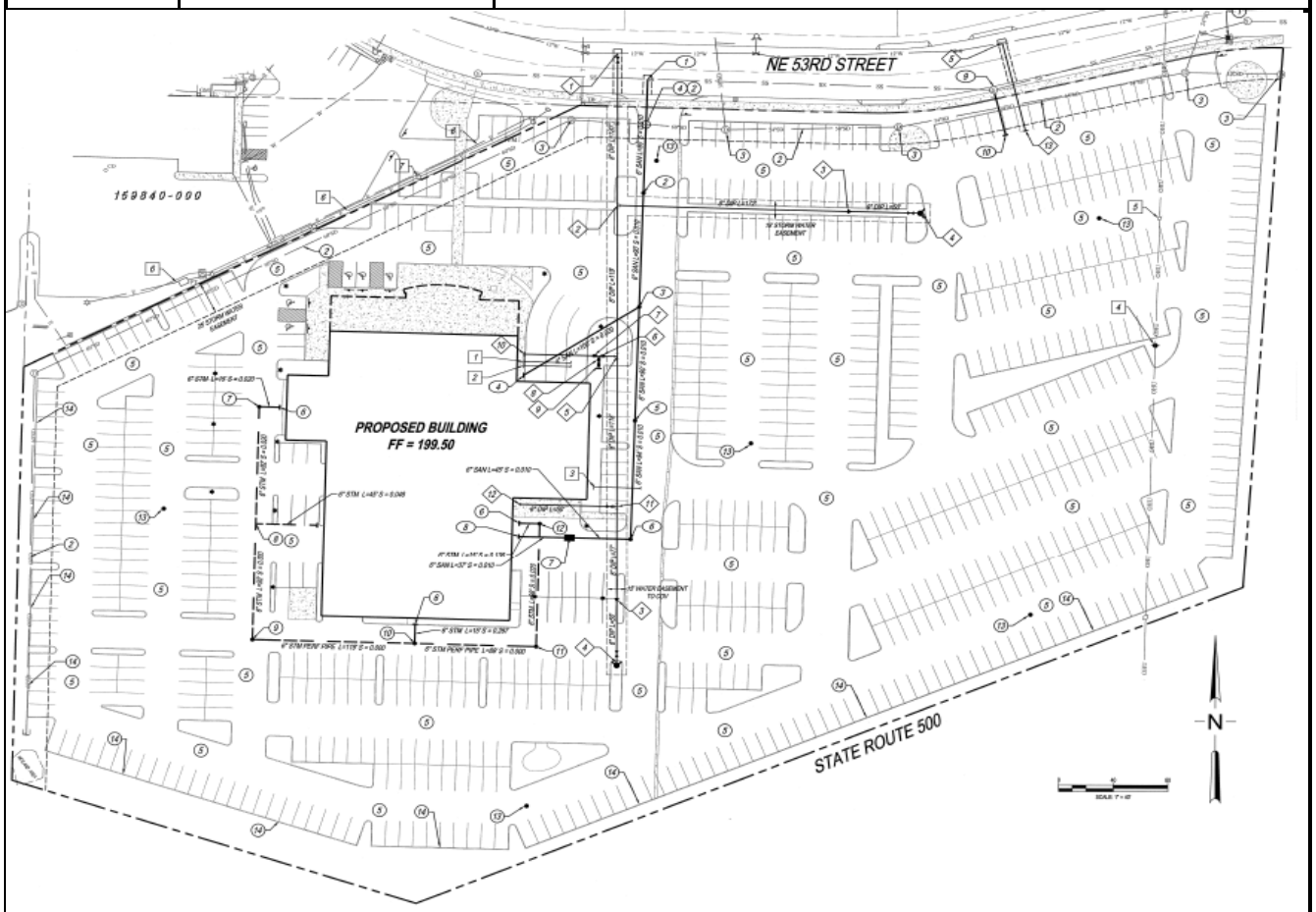
Instructions

1. Conduct observation monitoring of site
2. Not the presence of ponding of water on top of concrete pavers, seepages, contaminants (oil, grease, etc.) below in the Monitoring Log.
3. In the Monitoring Log, note the observation type, GPS location, and depict the affected area on the map.
4. Not any other unusual observations (i.e. erosion, piping, spills, etc.)

Observation Types: PW = ponding water, SE = seepage, CS =contaminants, Sp = Spills, OT = other

Monitoring Log

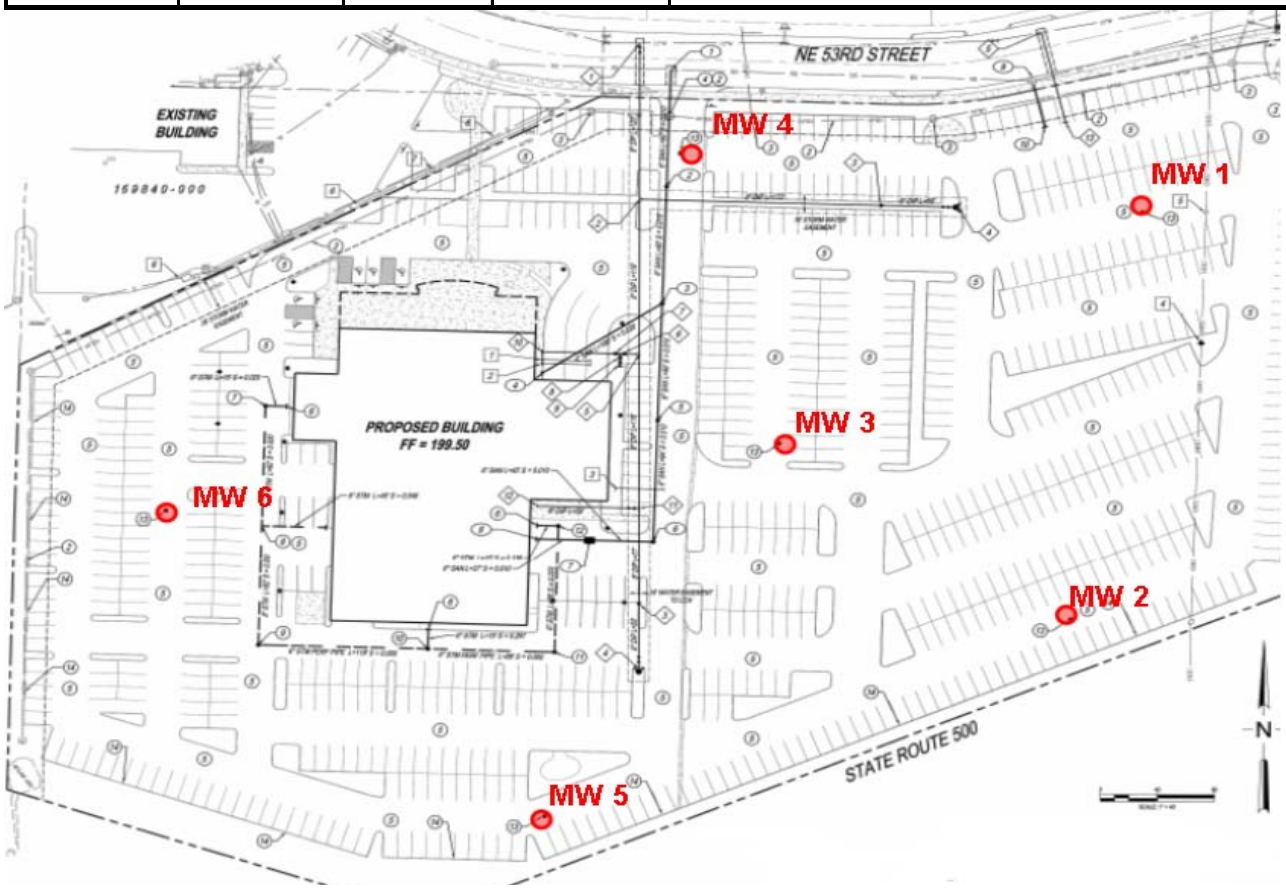
Observation Types	GPS Location	Observation Notes



Monitoring Well Observation Form		
Date:	Time:	Personel:
Weather:		Last Rain Event

Instructions

1. Locate Monitoring Well ID under monitoring well cap and record ID
2. Conduct a visual observation to determine subsoil saturation and/or presence of water
3. If water is present, take depth measurement using engineer tape (in tenths of inches)
4. Record cork dust height (high water mark) on staff gauge and rinse off cork dust
5. Not any unusual observations (i.e. color, odor, piping, etc.)

Monitoring Log

Monitoring Well ID	Water Present (yes / no)	Depth to Water (tenths of feet)	Depth to cork dust (tens of feet)	Monitoring well observation notes
MW 1				
MW 2				
MW 3				
MW 4				
MW 5				
MW 6