Quality Assurance Project Plan

Clark County Clean Water Program Stormwater Flow Reduction Strategy Monitoring

By
Ian Wigger

Project Name:	**LID Permeable Pavers Effectiveness Study**
Project Code:	**TOYLID**
Agency Name:	**Clark County**
Agency Contact Name:	**Ian Wigger**
Department:	**Environmental Services Clean Water Program**

Version 1.0

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Clark County Clean Water Program Stormwater Flow Reduction Strategy Monitoring

Approved by:

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CWP — Clark County Clean Water Program
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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
RSMP	Regional Stormwater Monitoring Program
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 Department of Environmental Services Clean Water Program and approved by the Washington Department of Ecology as required by Section S8.C.3.b of the National Pollutant Discharge and Elimination System Phase I Permit (Permit). This QAPP describes Clark County's plan for continuing to assess the effectiveness of permeable pavers as a Low Impact Development (LID) practice. This effectiveness study is located at McCord Vancouver Toyota Dealership, Clark County, Washington.

The Clean Water Program will monitor precipitation volume, stormwater runoff volume, and onsite conditions at this site in accordance of this QAPP until September 2017. This study will assess the effectiveness of a permeable pavers parking area to infiltrate runoff over time and compare the results to its design standard. The county will also examine maintenance-related characteristics such as moss growth, maintenance performed, and infiltration testing results in an effort to better understand maintenance needs for this BMP in the Pacific Northwest. Hydrologic parameters recorded will include stage and discharge of the out fall pipe vault, and precipitation at the site. Base course will be monitored through observation wells on the site (Figure 9). Maintenance will be tracked and documented to establish the typical maintenance practices. Inspections will be performed to establish an extent of clogging and moss coverage of the paver void filling aggregate. Infiltration monitoring of the pavers will attempt to quantify the impact of clogging on the site.

The primary goal of this QAPP is to assure the delivery of defensible monitoring data 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 Department of Environmental Services 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 (county), under National Pollutant Discharge Elimination System (NPDES) Section S8.C.3.b. of the phase I municipal stormwater Permit (Permit). The primary goal of this QAPP is to assure the delivery of defensible data and decisions by documenting the quality and integrity of the monitoring data, the representativeness of the results, the precision and accuracy of the analyses, and the completeness of the data.

This QAPP was developed following 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 necessary data
D. Quality assurance and quality control procedures to ensure the QAPP 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

Standard Operating Procedures (SOPs) will provide guidance to users of this QAPP.

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:

- Background and Problem Statement
- Project Description
- Organization and Schedule

1. Background and Problem Statement

An effectiveness study is required under the Effectiveness Studies Option #3 of Section S8.C.3 of the Permit. This effectiveness study involves the continuation and refinement of earlier pavement hydrologic monitoring as well as evaluation of site conditions and maintenance operations. The overall project is described in a study proposal approved by Ecology in September 2014.

The use of permeable pavement is a requirement of the Permit. A unique opportunity was available to build upon three years of monitoring at an auto dealership with a large installation of Interlocking Concrete Pavement Blocks (ICPB). Given maintenance costs, limited resources, and increasing usage, there is a need for specific information on when ICPB corrective maintenance work is required. This need is especially strong given the Pacific Northwest’s unique climate that often promotes moss growth that could accelerate clogging between pavers. As pervious paving systems come into widespread use under LID requirements, knowledge of long-term performance and maintenance needs will become increasingly important to stormwater managers.

Permeable pavement effectiveness studies are generally of limited duration and scope in the Pacific Northwest. There is relatively little documentation of how well modular permeable paver systems perform during their estimated 20-year design life, especially for Pacific Northwest applications with prolonged wet seasons and ubiquitous moss growth.

In 2010, the county began monitoring a newly built 7.5 acre modular permeable paver parking lot the McCord Vancouver Toyota dealership (site). The site was designed to infiltrate the 100-yr, 24-hour rainfall event of slightly more than 4 inches. The paver system includes a base layer large enough to hold the infiltrated volume from the 100-yr storm rainfall and permeable pavers as the wearing course. The monitoring study included a field inlet system capable of collecting and routing any onsite runoff to a single outfall fitted with a flume to measure flow, a rainfall gage to measure, and six observation wells installed in the base course to observe water depth below the pavers.

Routine maintenance activities have not prevented the establishment of moss in multiple areas of the site. As the moss density increases, infiltration rates in these areas have been reduced and may require corrective maintenance to sustain the designed infiltration rate. The county has monitored runoff from the site to compare actual performance to the design criteria of full onsite retention of rainfall up to the 100-year 24-hour design storm. As of October 2014, only small amounts of runoff have occurred, and these occurred during intense short storms or a rare heavy multi-day rainy period. The small amounts of runoff from the site may be indicative of reduced infiltration rates caused by moss becoming established between the voids of the pavers.

The county's infiltration tests of the site showed the lowest measured infiltration rate was 4 inches per hour at a location with almost 100% moss coverage. Four inches per hour is approximately 50% of the paver design rate of 7.8 inches per hour and well below the 10 inches per hour rate specified in the Washington State Department of Ecology (Ecology)

LID Operation and Maintenance manual (Herrera 2013) for corrective maintenance. Corrective maintenance to restore infiltration rates as designed may include removing clogged material from between pavers and replacing with clean aggregate.

Based on the county's observations of infiltration rates and runoff behavior at the site, Washington State Department of Ecology LID Operation and Maintenance manual (Herrera 2013) recommended corrective maintenance standard of 10 inches per hour infiltration rate may be an overly conservative standard. Additionally, adhering to the Ecology LID Operation and Maintenance manual would require up to 38 infiltration tests (at this 326,700 square foot concrete paver site) making infiltration testing at this site time consuming and costly. Creating a defensible alternative approach to annual infiltration testing regimen at large sites such as this site with vegetation would be beneficial.

1.1. Permit Overview and Monitoring Requirements

Ecology issued the Permit with three different Effectiveness Studies Options. The county chose option #3. This option states that the Permittee will pay into a collective fund to implement the Regional Stormwater Monitoring Program effectiveness studies at a reduced rate and will independently conduct an effectiveness study program.

The county will conduct an independent effectiveness study that follows a proposal describing the purpose, objectives, design, and the method of the independent effectiveness study, with expected outcome modifications to the Permittee's stormwater management program and relevance to other Permittees.

The county will describe interim results and the status of the study in annual reports within the duration of the study.

A final report of results and recommendation of future actions will be delivered to Ecology and put on the county's webpage within six months of the completion of the study.

1.2. Historical Hydrologic Management Effectiveness Monitoring

Section S8.F.7 of the 2007 Permit, required the county to monitor the effectiveness of one flow reduction strategy. The county used the McCord Vancouver site to meet these requirements. The county was required to monitor the effectiveness of a flow reduction strategy installation within the county. Monitoring of the flow reduction strategy included continuous rainfall and surface runoff monitoring. Flow reduction strategies were against a predicted outcome.

The county accomplished these requirements by monitoring, characterizing, and assessing the effectiveness of pervious pavers of a LID flow reduction strategy, and compared results against a predicted outcome.

1.3. Study Area

The project addresses the stormwater design for McCord Vancouver Toyota 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 project is the McCord Vancouver Toyota lot (Figure 2), and includes the main building, the car parking/storage area, and the sidewalk along NE 53rd Avenue. The size of the commercial building is approximately 41,985 square feet, and the total size of the site is 10.33 acres (Figure 3).

<u>Predevelopment Conditions</u>

The past 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 onsite: 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 although, fill material for the site development may have altered groundwater conditions.

As part of site development, subsurface conditions were explored for site development with fourteen soil test borings and were generally divided into three strata; non-native soil (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).

<u>Developed Site</u>
Stormwater runoff from the building rain harvesting overflow, parking lot, and the landscaped area are infiltrated onsite using Eco-Loc Permeable Concrete Pavers. Stormwater is treated by means of infiltration through the base course and the underlying imported sandy soils (Figures 5 and 6).

The 12 inch a gravel filled reservoir (base course) is designed to store the 100-year storm (24-hr rainfall = 4.30 inches). The 100-year storm runoff design fill the base to 0.4 feet above bottom of the base course. The design infiltration rate for the subgrade is 0.5 in/hr and the void ratio is 0.10 (Appendix C).

Design for the pervious pavers, base and bedding materials is described in Appendix A.

The site includes an overflow drainage system. Runoff that does not infiltrate, flows downhill to the south and west margins. Curbs along these margins prevent flow from exiting the site. A series of inline concrete field inlets with slotted grates along the south and west perimeter of the site convey any stormwater runoff to a single outfall in the southwest corner of the property (Figure 8). This outfall drains into a drainage ditch at the southwest corner of the property (Figure 7).

Observation wells are used to monitor saturation of the base materials (Figures 9 and 10).

1.3.1. Permeable Pavement

The term permeable pavement, or pervious pavement, is used to describe pavements that allow stormwater to infiltrate into base course below the pavement surface. The base material provides temporary storage of stormwater before it infiltrates into the subgrade. Pervious pavement in this study refers to modular interlocking concrete block pavers (figure 5 and 6).

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 filled with aggregate. The pavement sections for the site were designed to recommendations of the Interlocking Concrete Pavement Institution Permeable Interlocking Concrete Pavement Design Manual (Smith, 2006).

1.3.2. 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. Recent studies indicate that infiltration rates can be reduced due to clogging material filling voids within pervious pavement (Dierkes et al., 2002 and Hunt et al., 2002). Field observations of this site show that despite routine maintenance (sweeping), moss establishment has potentially impaired the effectiveness of the pervious paver site, reducing the infiltration of stormwater runoff. Little data exist in assessing the impact of moss growth over time. The county will evaluate the effectiveness of pervious pavement and infiltration reduction over time.

2. 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.

2.1. Study Goal and Objectives

The goals of this study are to assess the effectiveness of a maturing permeable paver system at infiltrating stormwater over time, examine the effectiveness of routine maintenance activities, assess how moss coverage affects infiltration rates, and propose a practical alternative to the standard infiltration testing.

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

- Collect runoff rate and precipitation data at the site throughout duration of the study
- Document maintenance activities by the site owner
- Monitor water levels in observation wells in paver base course
- Collect infiltration rate data
- Quantify the effectiveness of pervious pavers over time as a flow reduction strategy and compare results against a predicted outcome
- Evaluate simple alternatives to standard infiltration tests that mimic real world rainfall events
- Validate and report the monitoring results to Ecology

2.3. Information Requirements

Information needed to meet the study objectives includes:

- Continuous record of rainfall data
- Continuous record of outfall flow data
- Onsite surface ponding and runoff monitoring observations
- Infiltration rate test data at various locations

2.4. Data Collection

The monitoring design contains three primary components that will be conducted.

- Hydrologic monitoring
- Onsite observations
- Infiltration tests

<u>Hydrologic Monitoring</u>

Precipitation and outfall flow data are needed for verification of the design criteria regarding the effectiveness of the pervious pavers and to observe any changes with time due to clogging.

Hydrologic monitoring will provide the following:

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

<u>Onsite Observations</u>

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

Onsite observations will involve the following measurements:

- Depth of water in base material using observation wells
- Ponding water on pavers
- Seepage from site perimeter
- Tracking of maintenance activities
- Mapping of moss extent
- Infiltration tests

2.6. Study Area

The study area is the McCord Vancouver Toyota car dealership (Figure 3). The site includes the building, sidewalk, car parking/storage area, and landscaping. The total size of the parcels is 10.33 acres.

The stormwater runoff from the parking lots and the landscaped area is infiltrated onsite using Eco-Loc Permeable Concrete Pavers. The pavement sections for the site were designed using 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).

2.7. Practical Constraints

Monitoring may be constrained by specific characteristics of the storm drain system and site hydrology. The underdrain pipes and vault containing the outfall pipe allows for a small volume of retention with the potential that there could be small runoff events not observed as flow. The site allows for accurate measurement of flow leaving the site through the outfall.

The limits on reliability of precipitation forecasts create inherent logistical challenges for onsite monitoring during storm events. Field crew mobilization for a potential onsite monitoring event will be restricted to 1 day to a few hours prior to the storm.

2.8. Data Collection

The county will collect continuous precipitation and outfall flow data for the duration of the study. Twelve site visits, one per month and preferably after a rainfall event, will be made each year. 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 anomaly that may indicate that pervious pavers are not functioning as designed.

Moss extent mapping will be performed to establish the effect of the potential differences in infiltration rates and the rate of change in the moss coverage.

2.9. Use of Data for Management Decisions

This study will allow the county to evaluate how well the site continues to infiltrate compared to its design rate. The study will analyze maintenance and infiltration rate monitoring to refine the standards in the LID Operation and Maintenance manual (Herrera 2013).

3. 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.

3.1. Roles and Responsibilities

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

Table 1. Project roles and responsibilities for the Effectiveness Study.

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 Supervisor	Overall management of the County's NPDES Phase I compliance activities. Monitor and assess the quality of work. Comply with corrective action requirements.
Ian Wigger CWP 360-397-6118 ext. 4282	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. Manage and oversee monitoring activities, including data management.
Bob Hutton CWP 360-397-6118 ext. 4583	Quality Control Coordinator	Validate and verify data collected, and initiate corrective action as appropriate.
Ecology 360-690-7120	SW Region Permit Manager	Review and approval of the final QAPP
McCord Vancouver Vancouver Toyota Site Representative	Site Manager	Provide information describing maintenance actions.

3.2. Schedule

The following table indicates the estimated study schedule. The schedule may change due to unforeseen reasons.

Table 2. Anticipated program schedule for the effectiveness study.

<table>
<tr><th>Activity</th><th>Anticipated Date of Initiation</th><th>Anticipated Date of Completion</th><th>Deliverable</th><th>Deliverable Due Date</th></tr>
<tr><td colspan="5"></td></tr>
<tr><td>Submission of draft QAPP</td><td>12 Sep 2014</td><td>09 Jan 2015</td><td>Draft QAPP</td><td>09 Jan 2015</td></tr>
<tr><td>Approval of draft QAPP</td><td>Jan 2015</td><td>May 2015</td><td>Final QAPP</td><td>May 2015</td></tr>
<tr><td>Interim results and status of study implementation</td><td>Annually</td><td>Annually</td><td>Annual Report</td><td>Annually</td></tr>
<tr><td>Precipitation monitoring</td><td rowspan="5">May 2015</td><td rowspan="5">30 Sep 2017</td><td>Rainfall data</td><td rowspan="5">Reported to Ecology each year starting 2016</td></tr>
<tr><td>Outfall flow monitoring</td><td>Flow data</td></tr>
<tr><td>Observation well depth</td><td>Well Measurements</td></tr>
<tr><td>Seepage from site perimeter</td><td>Visual Observations</td></tr>
<tr><td>Maintenance records/actions</td><td>Records</td></tr>
<tr><td>Data validation</td><td></td><td></td><td>QC Report</td><td>Feb each year</td></tr>
<tr><td>Report of final results</td><td>Oct 2017</td><td>Jun 2018</td><td>Final Report</td><td>31 Jul 2018</td></tr>
</table>

3.3. 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. In addition to technical training, field personnel will receive guidance that addresses specific monitoring issues that may impact their health and safety.

3.4. Revisions

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

4. 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.

4.1. Data Quality Objectives

Data quality objectives may be either qualitative or quantitative, and describe the type, quality, and quantity of data that are required to fulfill the program objectives. The data quality objectives are 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
- Mapping of moss coverage will be of sufficient accuracy to describe clogging at site scale

4.2. 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 (Table 3). Additional criteria for completeness and representativeness of the monitoring data collected are also required.

Table 3. Effectiveness study 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 model H-340	0.01 inches	0.01 in. (@ 4 inches / hr)	WaterLOG Series, Model H-340 Tipping Bucket Rain Gage Manual
Data Recording (precipitation, stage, and calculated discharge)	Digital Data Logger: Campbell Scientific CR200X data logger, and Raven XT modem	Sensor dependent (e.g. 12.34 ft)	Not applicable	CR200/CR200X Series Dataloggers Manual
Stage recorder automated	Stage Sensor : Campbell Scientific CS450 pressure transducer	0.0001 psi (sensor input) or 0.01 feet of water depth (default digital sensor output)	±0.01 feet	CS450/CS455 Pressure Transducer Manual
Observation well crest gage	Cork dust and measure down	Visit frequency	Not applicable	Not applicable

5. Monitoring Program Design

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

5.1. Hydrologic Monitoring

A continuous rainfall and discharge record will be utilized to observe and interpret any rainfall/runoff relationship for this site.

Water stage will be measured inside of the monitoring vault outlet structure (Figures 11 and 12) using a Campbell Scientific CS450 pressure transducer, which is calibrated to a

Type “A” staff gage mounted to the vault interior wall. Discharge will be measured with a 10 inch Palmer-Bowlus flume and the pressure transducer. Rainfall will be measured using a Design Analysis Model H-340 tipping bucket rain gage. Air temperature will be measured at the rain gage using a Campbell Scientific 109-L temperature sensor. A Campbell Scientific CR200X data logger will record the measured stage, rainfall, air temperature data, and calculate any discharge. A 12-volt battery with a battery charger and dedicated outlet will provide the power for all instrumentation.

The rain gage is attached to a 12” x 12” steel plate welded to the top of a 4” steel “I” beam. The “I” beam is 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”) is attached to the north side of the “I” beam at eye level. The data logger, modem, and battery is installed inside of the locked enclosure.

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

Data sheet	Manual
https://s.campbellsci.com/documents/us/product-brochures/b_cr200x.pdf	https://s.campbellsci.com/documents/us/manuals/cr200.pdf
https://s.campbellsci.com/documents/us/product-brochures/b_cs450-cs455.pdf	https://s.campbellsci.com/documents/us/manuals/cs450-cs455.pdf
http://waterlog.com/downloads/Brochures/H340Brochure.pdf	http://waterlog.com/downloads/manuals/Entire%20H-340%20Manual.pdf
https://s.campbellsci.com/documents/us/product-brochures/b_109.pdf	https://s.campbellsci.com/documents/us/manuals/109.pdf
http://www.plasti-fab.com/wastewater-products/palmer-bowlus-flumes	(special order - no manual)

5.2. Onsite Observations and Data Collection

Onsite visits will be conducted monthly for the duration of the study. Onsite visits will include downloading data loggers, as needed, 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. Additional site visits will occur for infiltration testing of the site.

5.2.1 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. The infiltration test has no bias because the infiltration rate of the pervious pavers is defined only in terms of the ASTM C1701/C1701M test method.

5.2.2. 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. The flume precision is calculated by the manufacture, and making sure repeatable stage data is recorded will ensure the precision of the flume flow values. Replicate measurements of stage will be conducted in the field of the stage in the outfall vault box. The field measurement and the electronic data logger (EDL) will be compared to provide a measure of potential instrument drift.

Repeatability of infiltration testing was performed by a single laboratory by making two replicate measurements at three locations on a newly placed pervious concrete pavement. The replicate measurements were repeated daily from day one to day 10. The single-operator coefficient of variation of the infiltration rate at one test location was found to be 4.7% (ASTM Standard Test Method for Infiltration Rate of in Place Concrete, ASTM C1701/C1701M, 11.1).

5.2.3. Accuracy

Accuracy is 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 the design of the flume and the sensor calibration to operate within or a deviation from ideal conditions, such as backwater, clogging or debris in outlet, etc. Maximizing accuracy of discharge and rainfall is achieved by appropriate selection of measurement technology for the conditions that will occur. Infiltration accuracy is performed by following the ASTM C1701/C1701M method as specified.

5.2.4. 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.

Missing or gaps in continuous flow data may be substituted with onsite visual observation data and crest gage 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 gages in the area.

During the data validation process, an assessment will be made whether sufficient valid data exist to meet the requirements of the project. If insufficient valid data are obtained, corrective actions will be initiated by the principle investigator or a designee.

5.2.5. Representativeness

The study hydrology data are intended to be representative of conditions at the outfall monitoring station. The county 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 5 minute intervals to compute flow variations. Precipitation recordings are triggered by 0.01 inches of rain to ensure precise rainfall measurements over specific timeframes.

Infiltration tests represent values different from the true vertical hydrologic conductivity due to the horizontal flow below the pavers. ASTM C1701/C1701M will be used to standardize this lack of representativeness.

Section C. Measurement procedures

6. Sampling procedures

Field operations follow the *Standard Procedures for Monitoring Activities*, Clark County Public Works Water Resources (June 2003), as described in the county Hydrology Monitoring QAPP (June 2014), and by following the methods outlined in ASTM C1701/C1701M.

Field logs consist of standardized field sheets activities are documented in detail (Appendix B) 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. Records are cross-checked for consistency between data sheets, field logs, and other relevant data. Log books are archived in the county's files.

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

6.1. Precipitation and Flow Monitoring

Data from the rain gage and flow meter will be downloaded hourly via modem and onsite to address data drop that may occur. The rain gage and flow meter will be inspected during site visits and serviced as needed; calibration will be conducted according to the recommended method and frequency determined by the manufacturer.

Discharge through the outlet pipe is computed using an equation for a Palmer-Bowlus flume provided by the flume manufacturer allowing for a stage/discharge relationship. The stage will be measured inside the concrete storm outlet structure.

Stage, discharge, precipitation, and battery voltage will be measured every 5 minutes. Discharge, however, will only be recorded when the stage is high enough to flow through the outlet pipe.

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

6.2. Onsite 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. The project will evaluate several approaches to infiltration testing that attempt to mimic rainfall intensities in the northern Willamette Valley. Onsite data gathering will include:

- Monitoring water depths below pavers from six observation wells
- Moss and vegetation extent mapping within the site
- Onsite infiltration testing using several methods in areas with and without vegetation
- Attempted observation of paver surfaces during intense and heavy rain events
- Documentation of maintenance activities

6.2.1. Clogging Mapping

Mapping of the site will be performed to establish the moss extent. Establishing the extent of the moss will help understand the possible impacts that the moss could have on the infiltration rates at a site scale. Moss extent mapping will allow the development of categories to base infiltration testing. Mapping of areas with total, partial, and little to no clogging will be performed using GPS and mapped in GIS to establish the extents of the clogging on the site. Mapping will be done during the winter months in order to better establish the moss extent while the moss is in its growing phase.

6.2.2. Observation Wells

The site has 6 wells that allow the observation of the base course at various locations on the site (Figure 10). Each of the observation wells has a cork dust placed in it to establish a monthly crest of the depth of water in each of the wells. These depths of water in the observation wells will help to establish the saturation differences of the base course at different locations on the site.

6.2.3. Infiltration Test

Infiltration tests will be performed in accordance to ASTM C1701/C1701M. Test locations will be established using the categorization of the moss extent mapping.

6.3. Maintenance

Monitoring of the maintenance of the site will be established with the McCord Vancouver Toyota site manager. A list of maintenance actions will be established. Visits with the site manager will be to determine the time and extent of maintenance activities.

Section D. Quality Assurance and Quality Control Procedures

7. Quality Control

This section describes the QC requirements for all field activities conducted by this program. Data quality will be evaluated according to the stated MQO's.

7.1. 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.

7.1.1. Field Quality Control Procedures

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

Installation and calibration of all automated stage and precipitation gage recording stations are generally performed according to accepted USGS standard operating procedures (USDI, 1982, Vol. 1). Automated stage and precipitation sensors are certified to be calibrated and validated with onsite observations.

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

The monitoring station is equipped with a 10 inch Palmer-Bowlus flume, pressure transducer, staff gage, crest-stage gage, and data logger to allow comparisons and maintain station reliability. The gage is programmed to record the entire range of expected stages. The automated stage recorder’s latest stage reading is compared to the current water surface level at the gage to ensure all instrumentation is working properly at each field visit. 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. The monitoring station has a crest-stage gage to mark high-water levels for information backup in case of equipment failure or to mark stage peaks between automated data recordings.

Infiltration test QC methods will be followed as described in ASTM C1701/C1701M.

7.1.2. Corrective Actions

Data quality problems encountered during the measurement and calculation of discharge and precipitation will be addressed and corrected. Generally, this will 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.

8. Data Management Methods

Stage, calculated discharge and precipitation data will be transferred via telemetry from the EDL daily. Field site visits will include reading the staff gage, checking the battery and the inspection of the crest-stage gage. The rain gage will be kept clean and level.

Recorded data will be input into Aquatic Informatics Aquarius time-series software. Plots and tables of all recorded data will be reviewed with corrections applied as needed. Water discharge leaving the site is computed using a flume equation provided by the flume manufacturer for each 5 minute stage reading. Precipitation rates and totals will be computed and can be compared to rainfall data collected at the nearby a rain gage if accuracy is questioned. Tables and graphs of the data will be provided as requested.

The effectiveness study data and field notes are recorded or retrieved, stored, and managed in both hardcopy and digital form by the county. Applicable data are entered into spreadsheets for summary statistic calculations including total instantaneous discharge. 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. Automated outfall stage, precipitation, and discharge measurements are digitally recorded on the data logger and then downloaded via modem from the monitoring station to the Aquarius database for data storage. Any flow leaving the site is computed with the flume manufacturer supplied equation for each five minute stage value.

Infiltration test monitoring will recorded by field staff onsite and entered into spreadsheets. Calculations of the infiltration rates will be included in the annual and final report.

Observation well crest data will be recorded in to field sheets. The data will be used to analyze potential differences in retention of water in the base course. These data will be included in the final report.

Clogging mapping will be performed with hard maps and GPS. The data will be prepared in ArcGIS. The GIS data will be used on conjunction with the infiltration rate and base course saturations data to help understand the site dynamics. Maps of the clogging extents and changes of the extents will be included in the annual and final reports.

Section E. Assessment Procedures

9. 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.

9.1. 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 the monitoring supervisor, QC coordinator, or project manager. 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.

9.2. Reports

Annual status reports and the project final report will be generated for this study.

9.2.1. Annual Status Report

Annual status reports compiled by the county 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 county staff. Annual status reports are submitted as attachments to the county's annual NPDES permit compliance report.

9.2.2. Final Report

NPDES permit requirement S8.C.3.b.v. requires final results, including future recommendations, to be submitted to Ecology and the county's webpage within 6 months of study completion.

10. 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.

10.1. Summary of Procedures

During each monitoring trip, field staff reviews field logs to confirm that all necessary field measurements have been collected. Field results are reviewed, verified, and documented by staff in data reports to the county. 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 by county staff. 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 criteria and 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 project manager is responsible for ensuring that field data are reviewed and verified. After each successful download event, the project manager will review rainfall and flow data for gross error (e.g. outliers or data gaps) to verify the completeness of the data, and check to see that flow measurements were collected in accordance with required criteria.

10.2. 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.

10.2.1. 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

10.2.2. Data Verification

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.

10.2.3. Data Validation

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 by the project manager and QA coordinator during the preparation of annual reports. To evaluate whether precision targets have been met, stage measurement results are compared to the EDL. Stage measurements will consist of staff gage observations and crest gage observations compared to the EDL stage.

11. 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 4) must be satisfied in order for the data to be considered usable for meeting program objectives. The main purpose of this assessment is to determine if the data meet the quantity of measurements required and if it representative of stormwater runoff conditions of the site.

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Figures

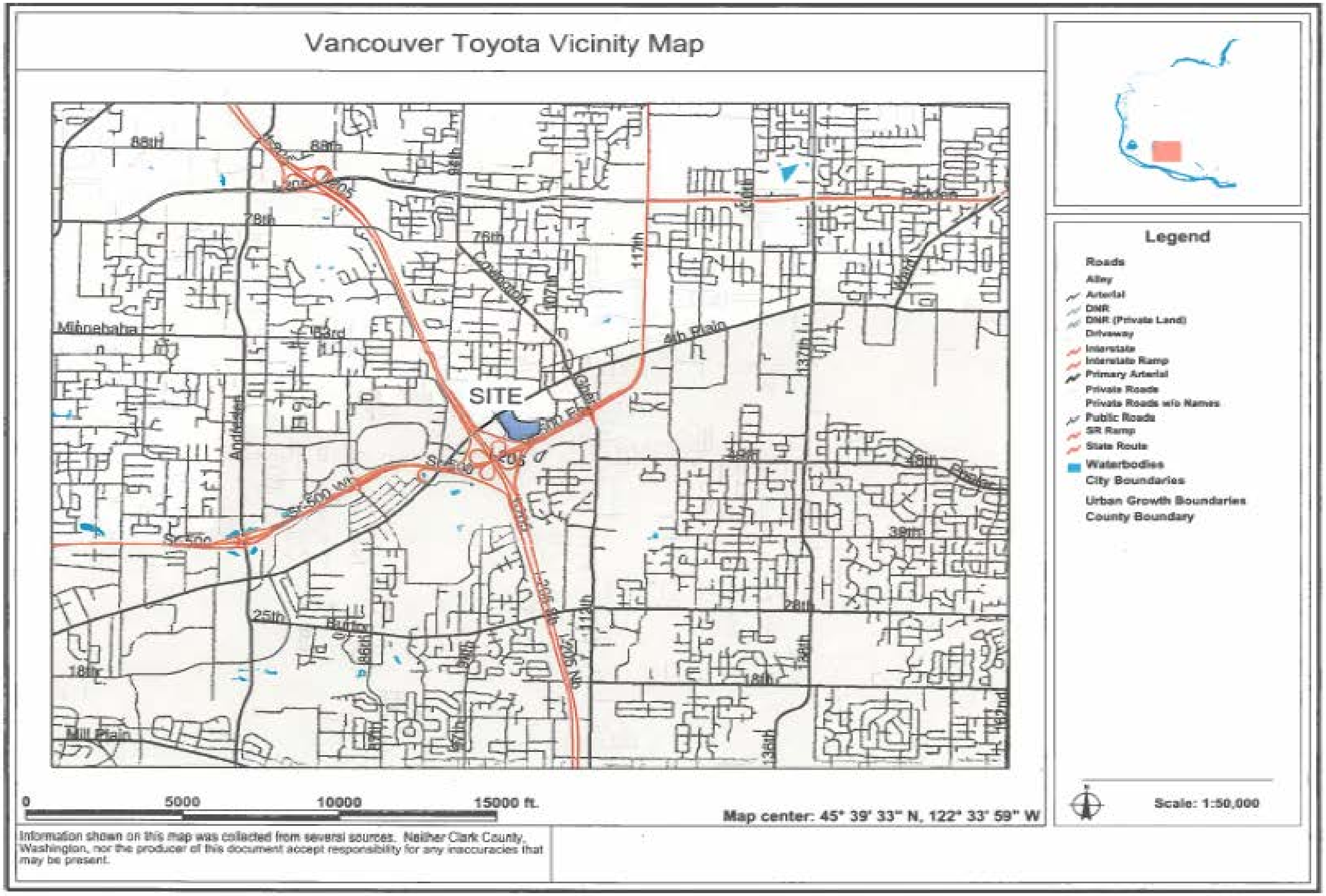

Figure 1. McCord Vancouver Toyota Site vicinity map

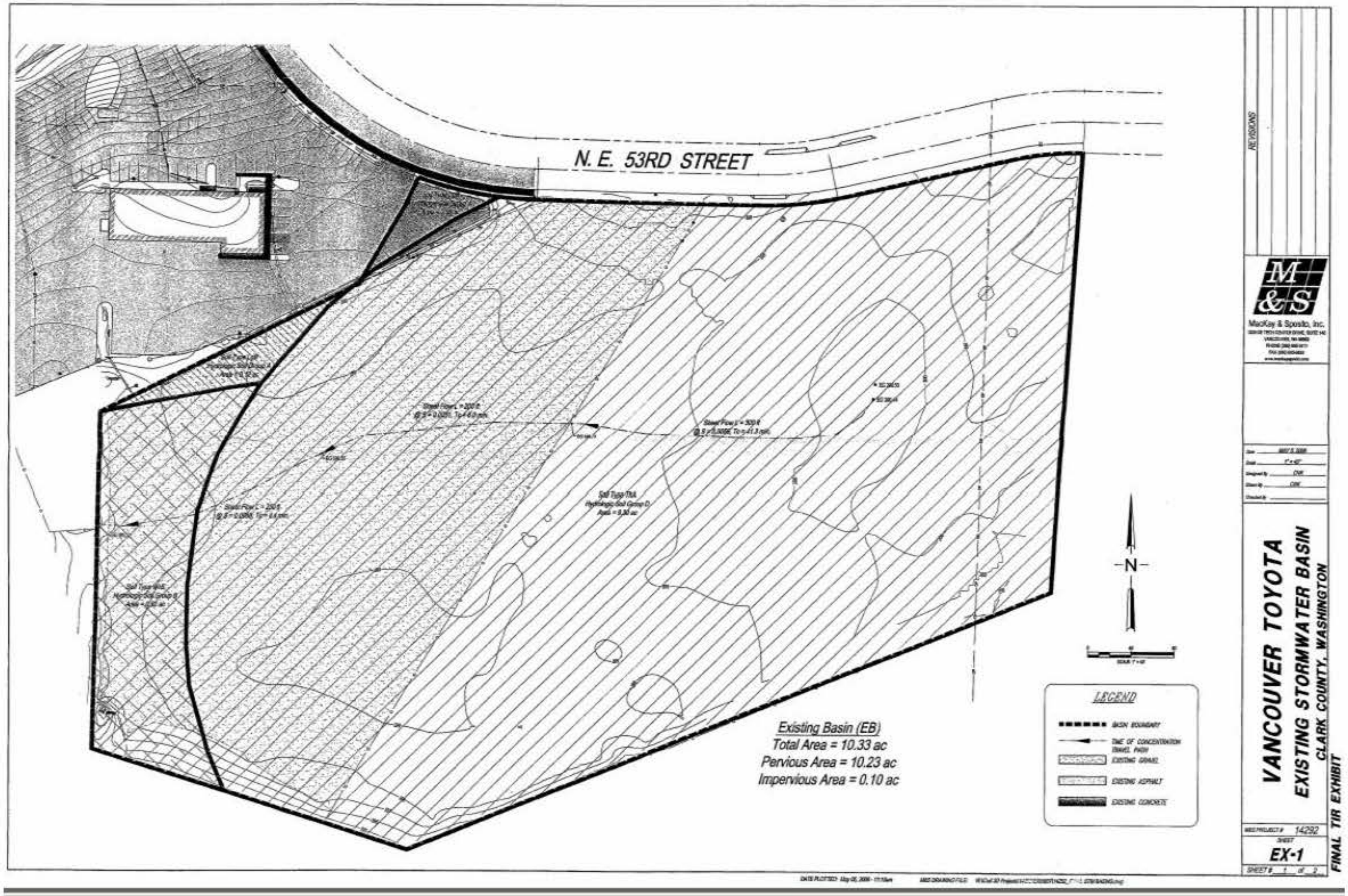

Figure 2. Site plane before McCord Vancouver Car dealership construction

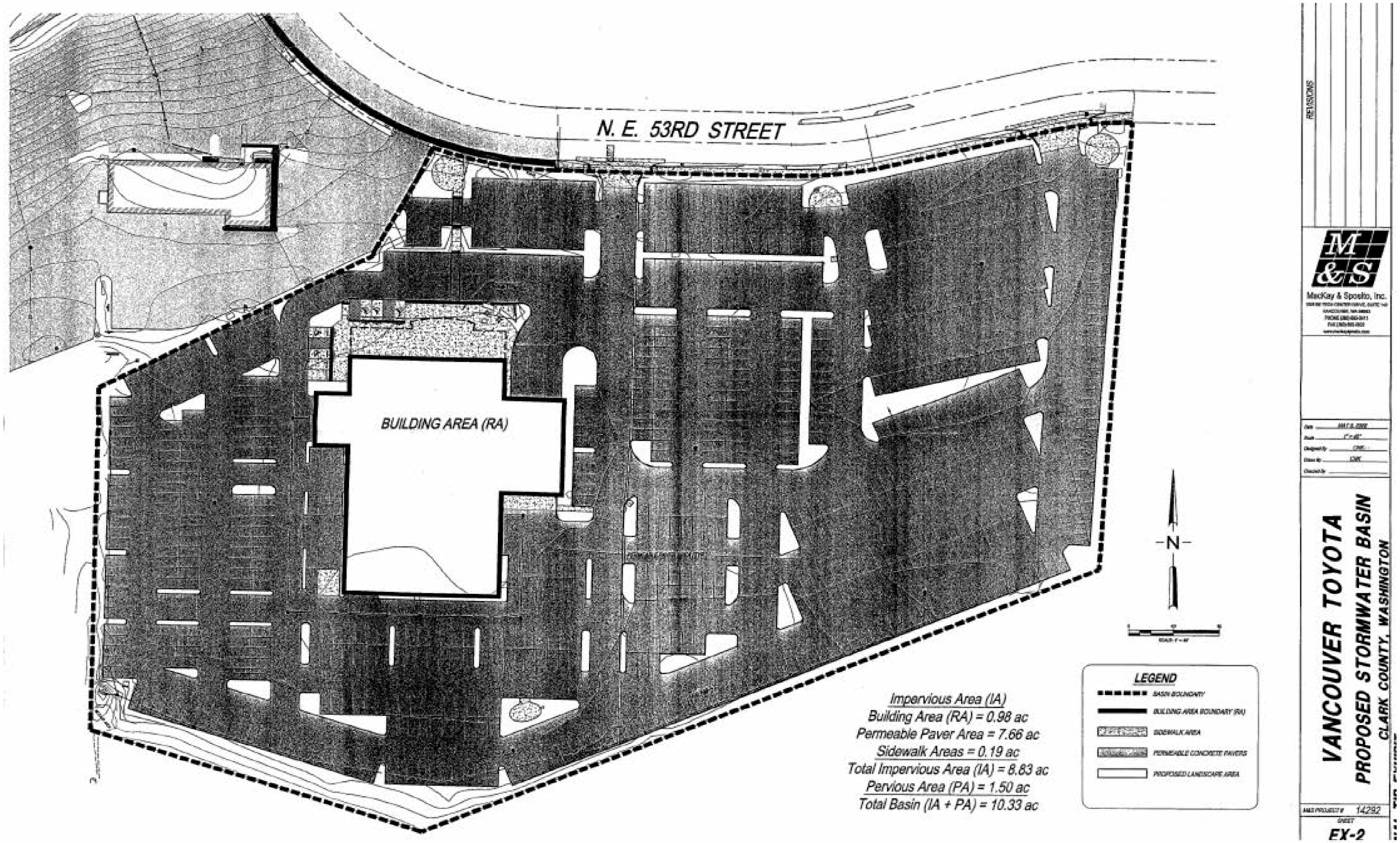

Figure 3. McCord Vancouver Toyota and associated facilities

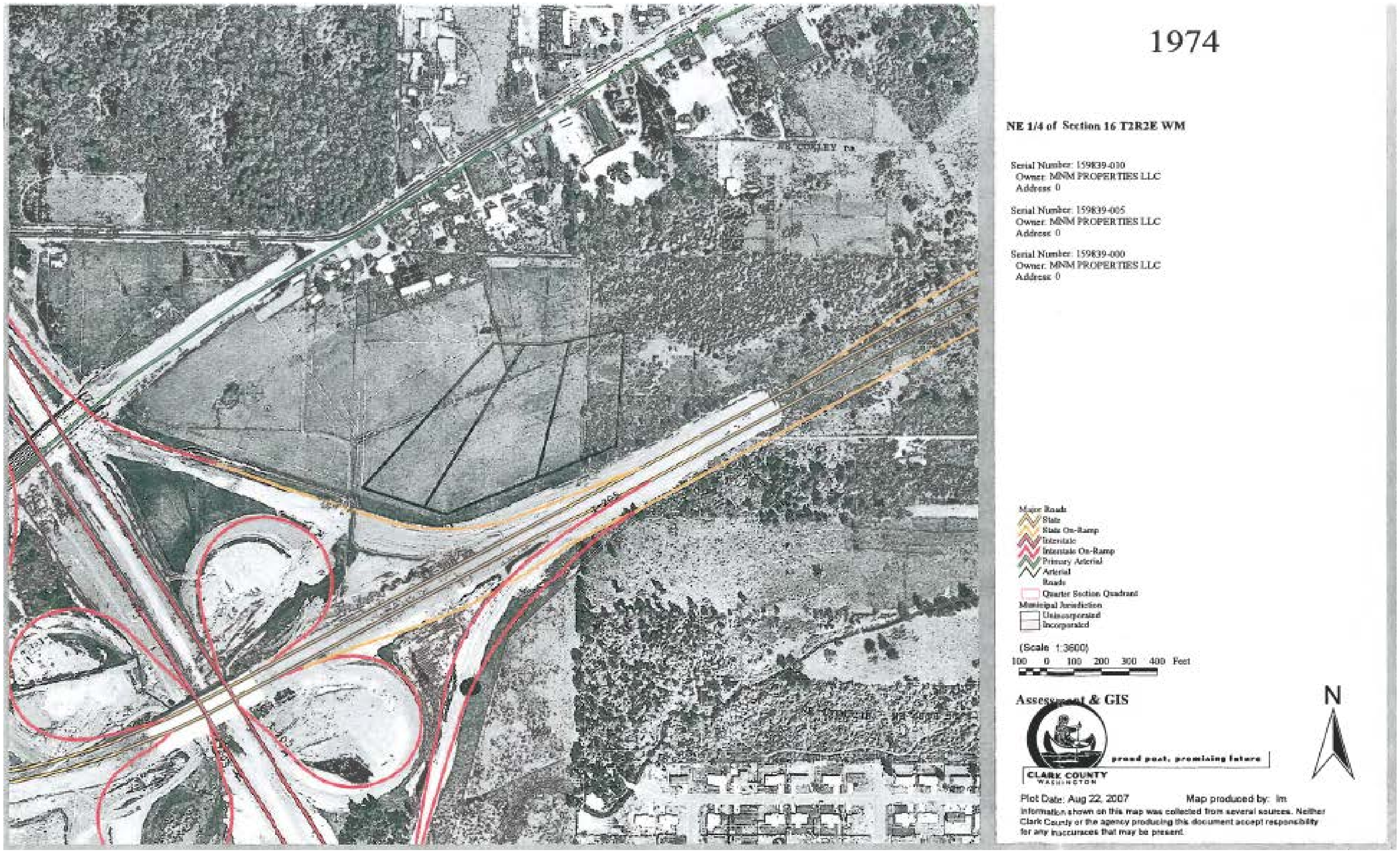

Figure 4. 1974 McCord Vancouver Toyota Site Conditions

Figure 5. Cross section of permeable concrete pavers and underlying base course

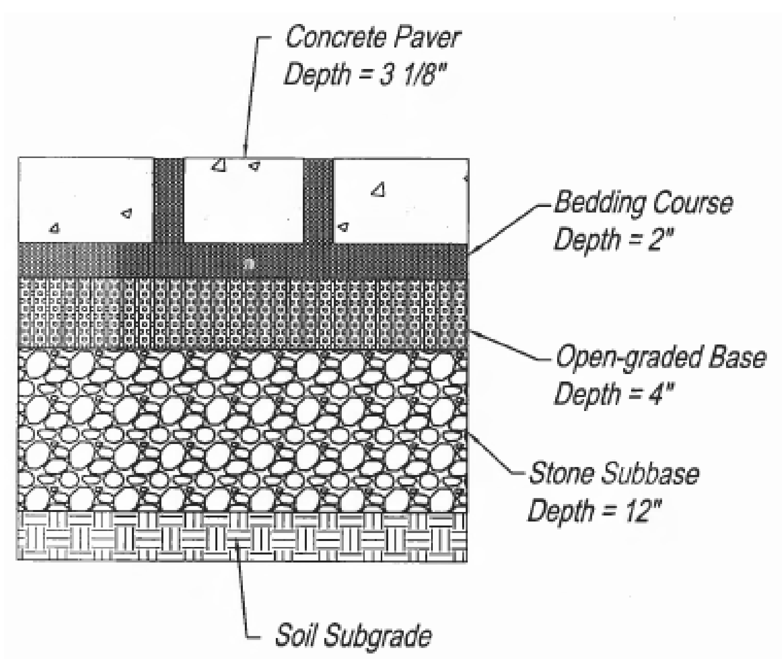

Figure 6. Description of permeable concrete pavers and underlying base course

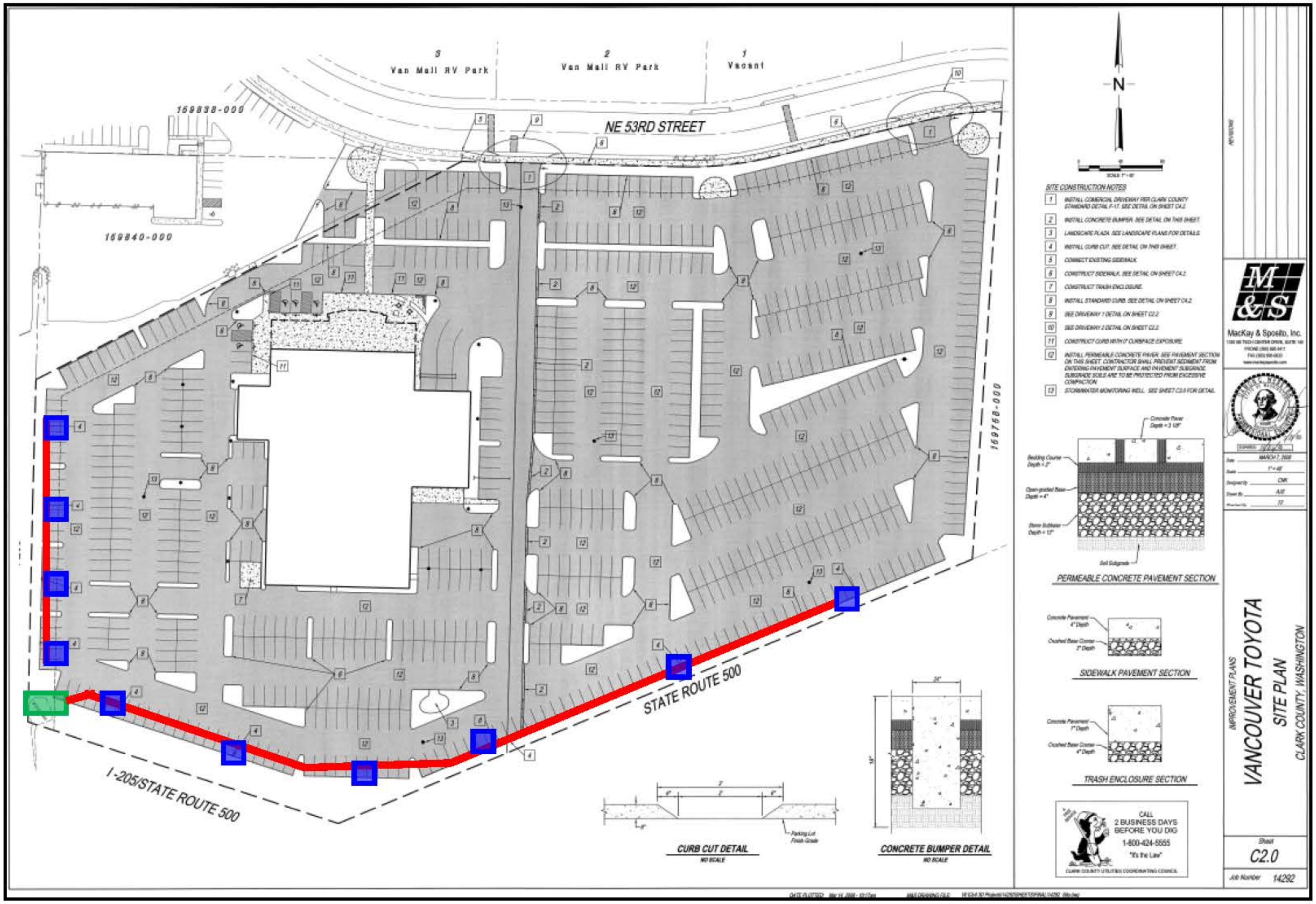

Figure 7. Field inlet system of McCord Vancouver Toyota site

Figure 8. Field inlets

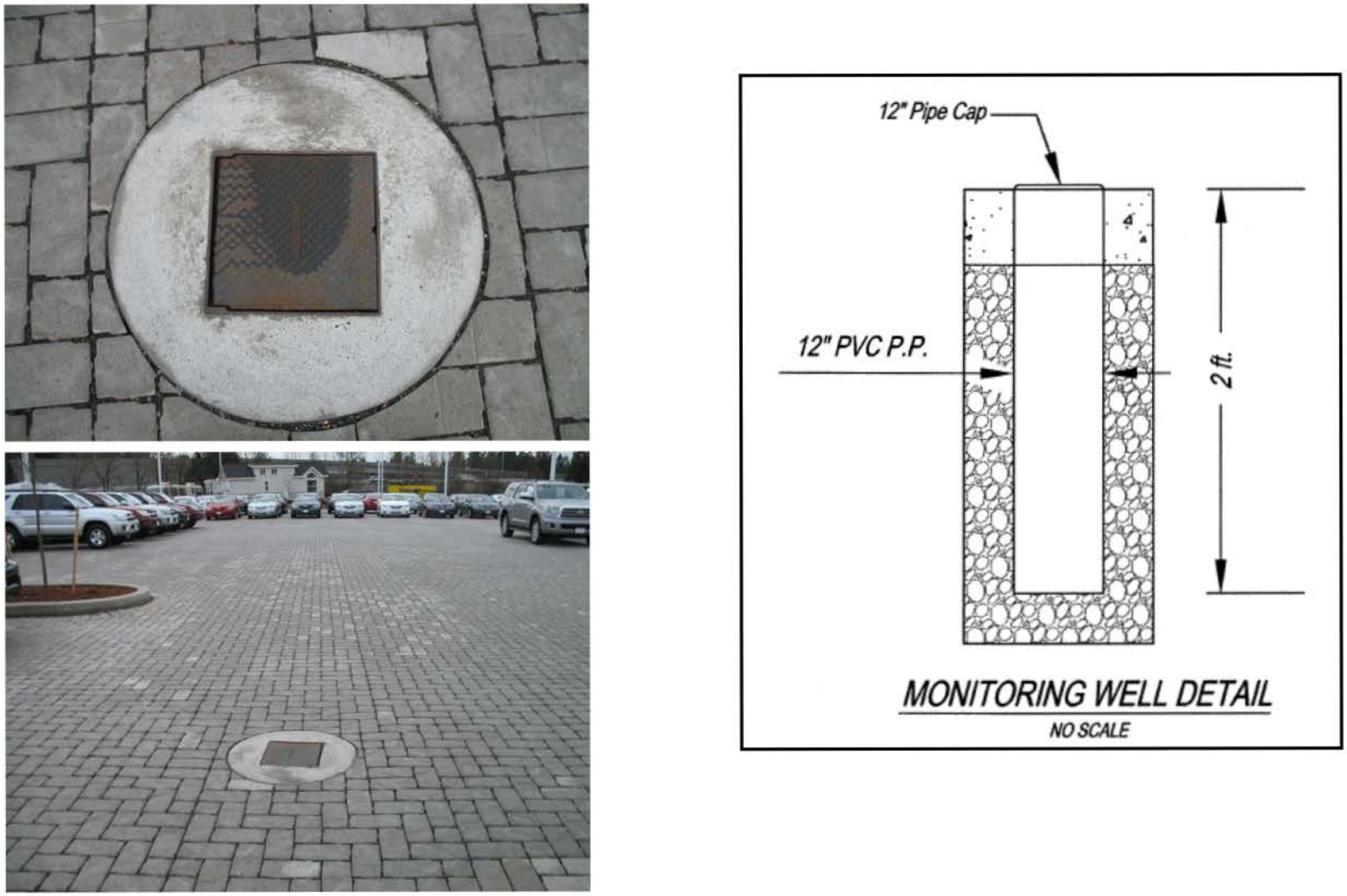

Figure 9. Observation Wells

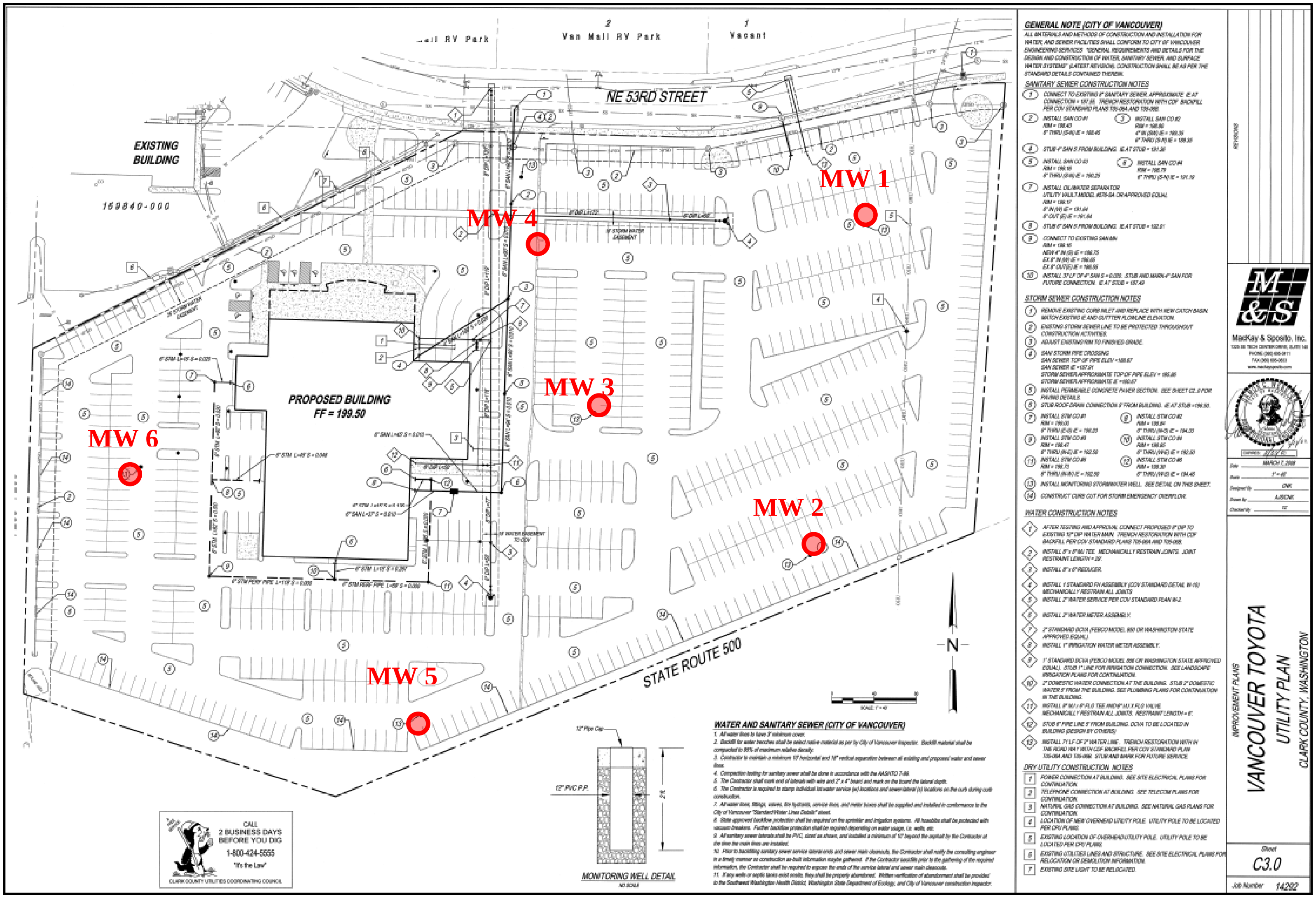

Figure 10. Observation well locations

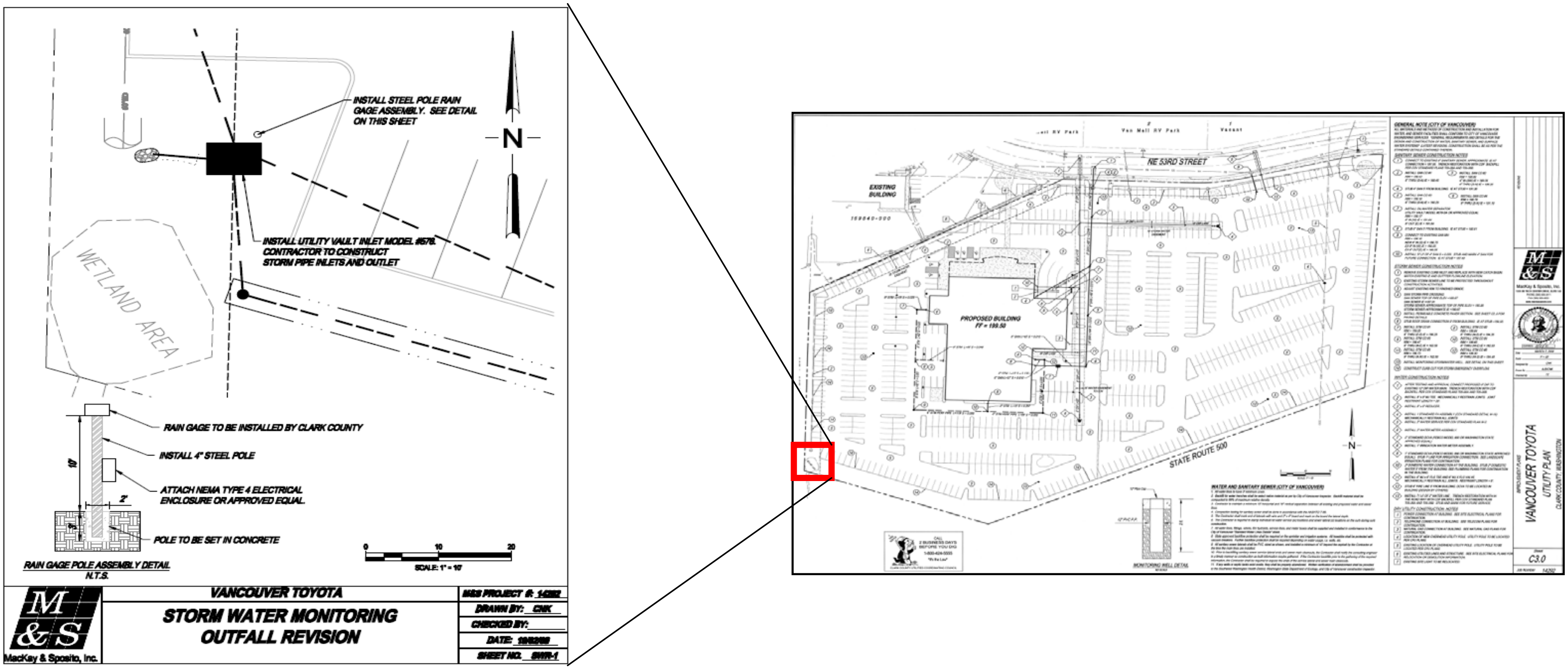

Figure 11. Site flow monitoring location and rain gage.

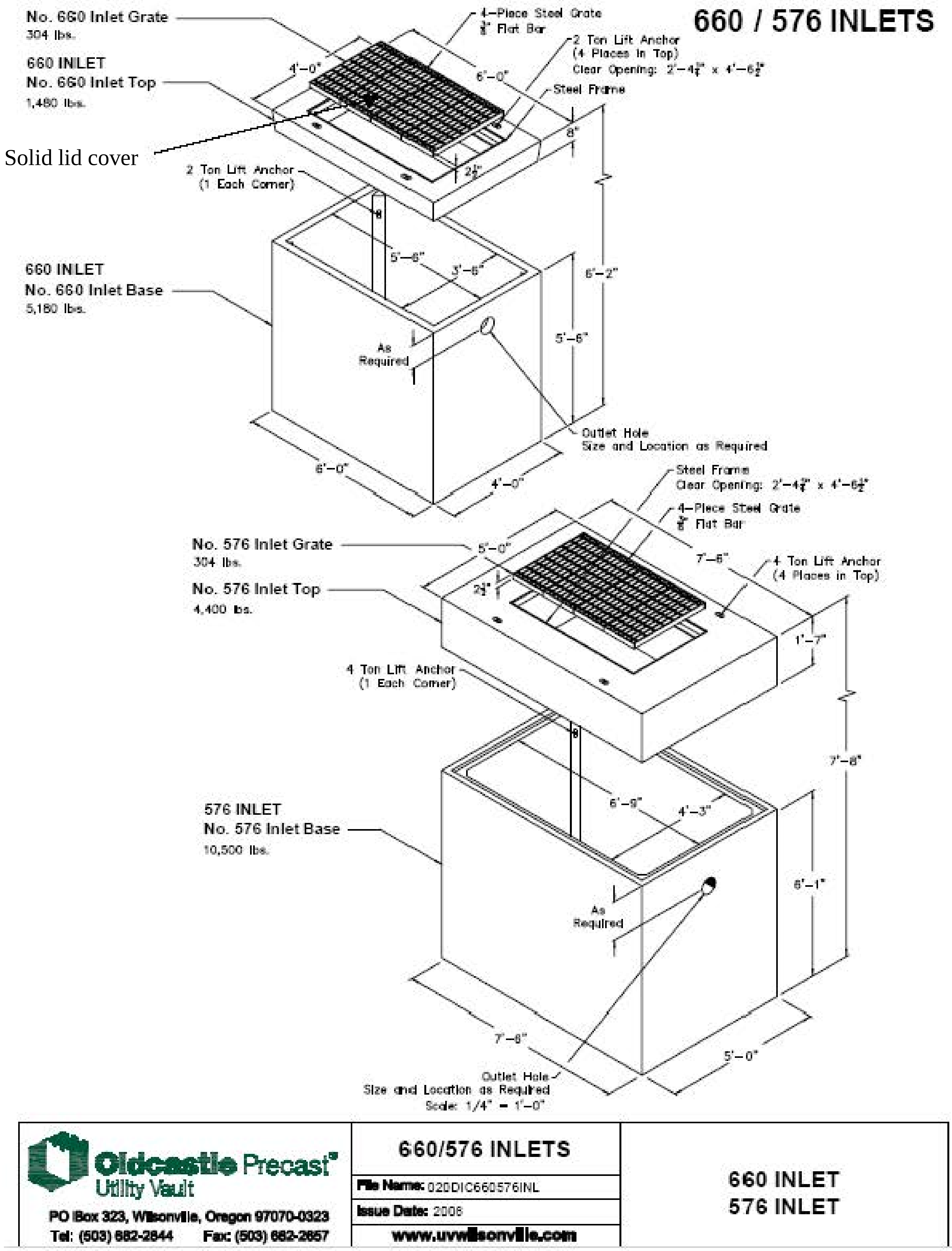

Figure 12. Vault and flow station for the McCord Vancouver Toyota site.

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Appendices

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

Appendix B: Effectiveness Study Field Data Sheets /Observation Logs

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 A: Vancouver Toyota Final Stormwater Report with maps of site

Vancouver Toyota

Final
Stormwater Report

Prepared for:

Vancouver Toyota
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Vancouver, WA 98662
360.253.4440

Prepared by:

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May 12, 2008

MacKay & Sposito, Inc. Project No. 14292

Vancouver Toyota

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PAC #:	2007-00024
ENG #:	2008-00028
M&S #:	14292
DATE:	05.10.08
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Table of Contents

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

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APPENDIX | A

Maps

Vicinity Map - A1

Flood Plain Map - A2

Groundwater Protection Area Map - A3

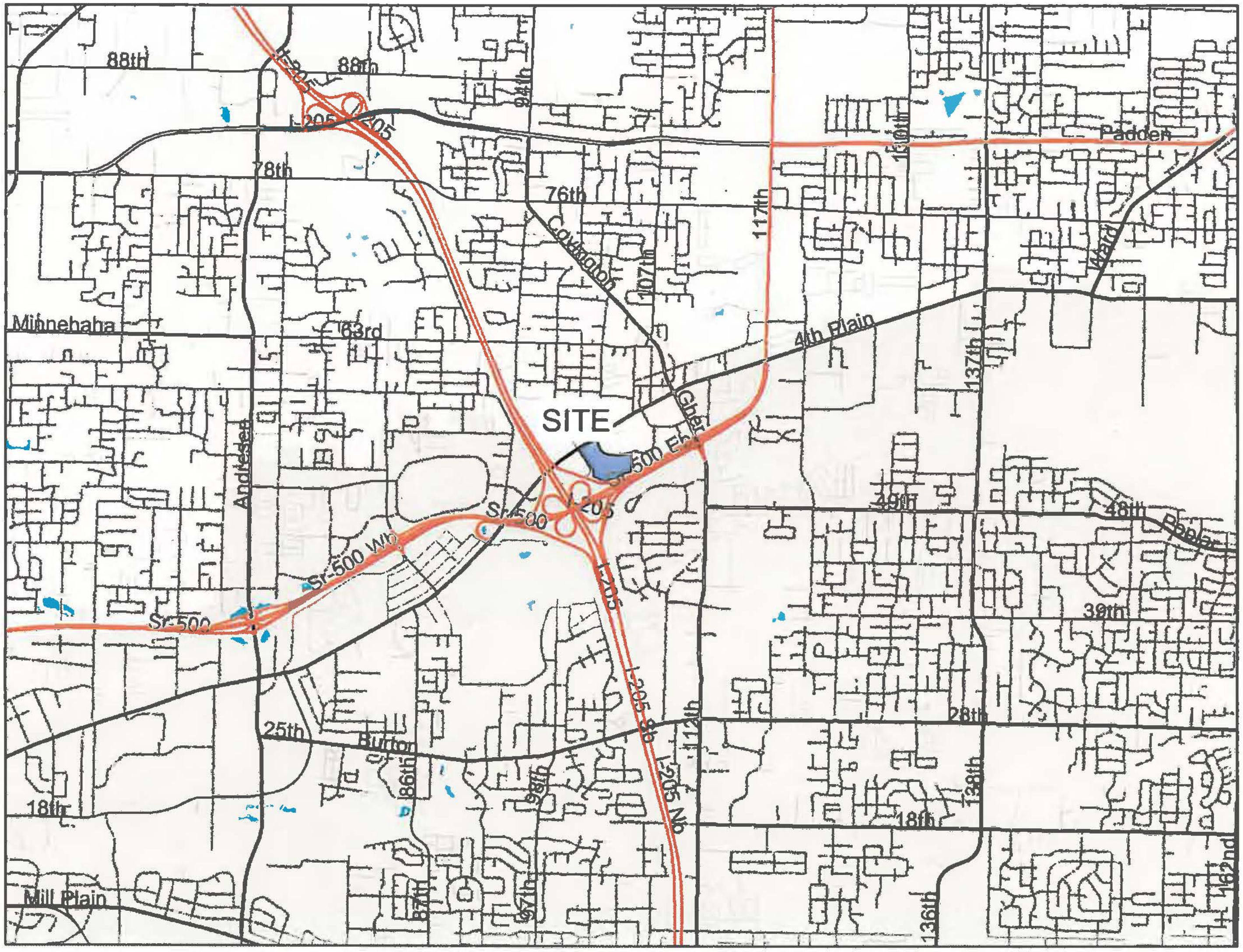
Current Aerial Photo - A4

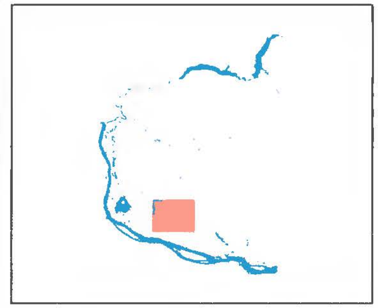
1974 Aerial Photo - A5

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.

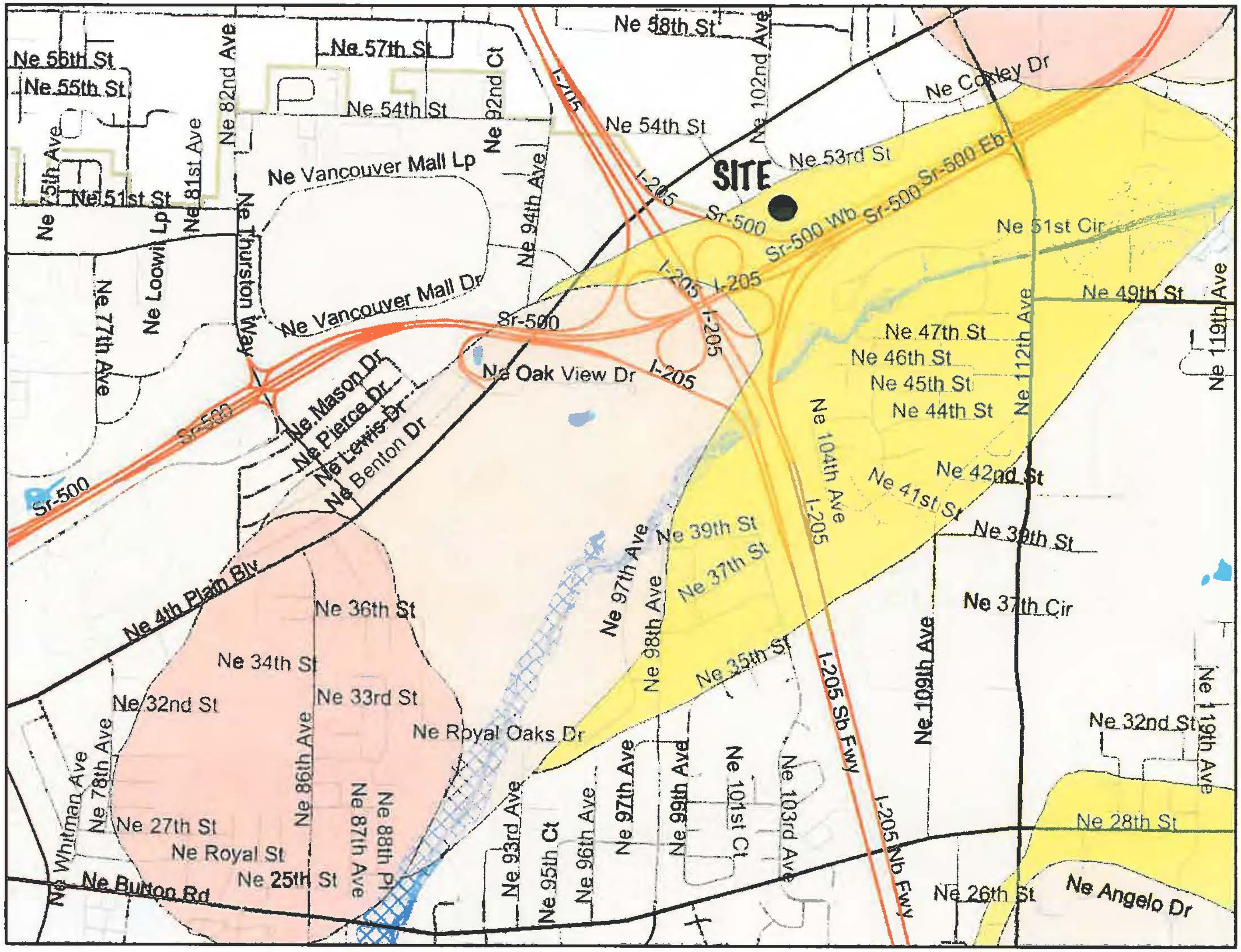
Map center: 45° 39' 33" N, 122° 33' 59" W

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



0	2000	4000	6000 ft.

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

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.

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

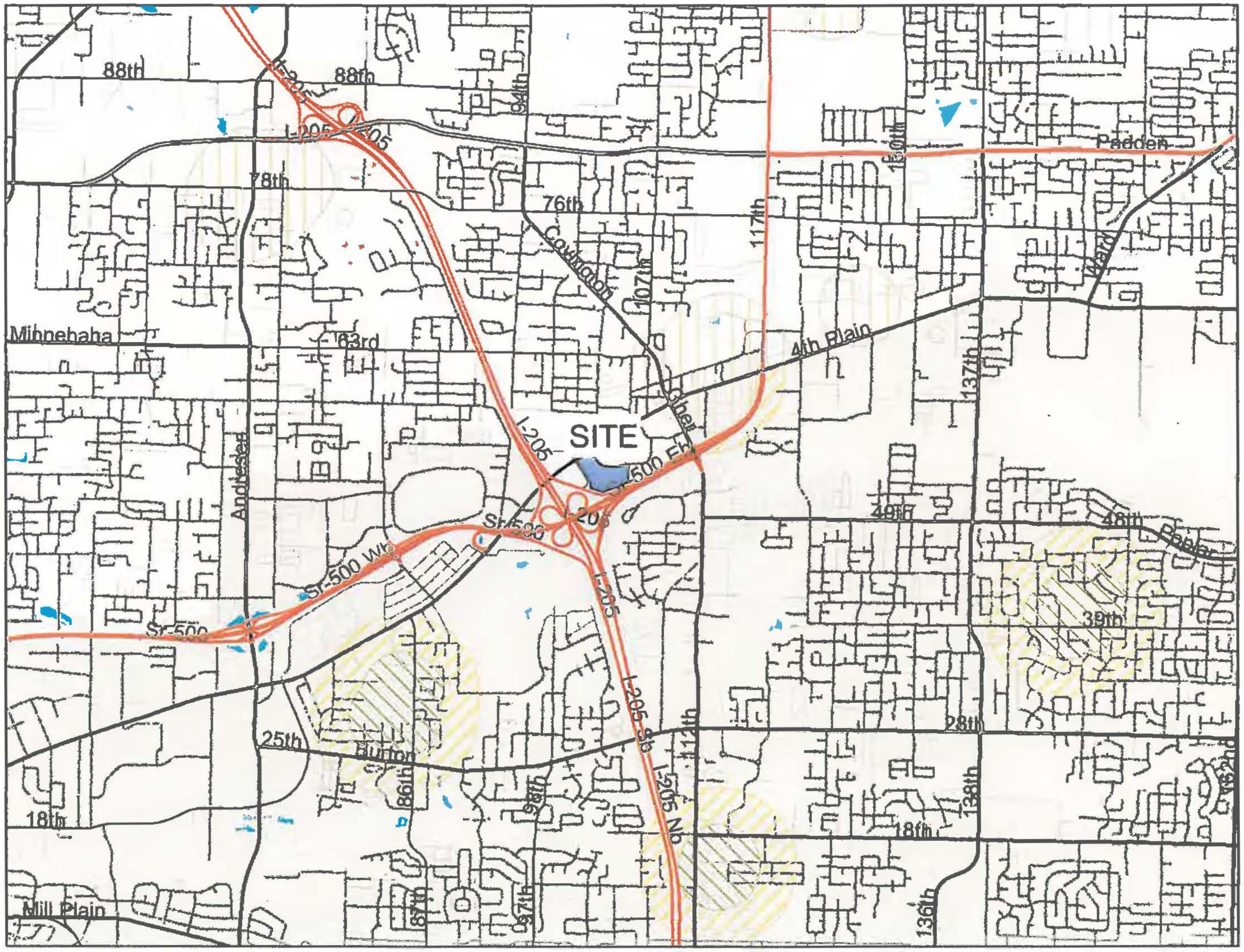
Urban Growth Boundaries

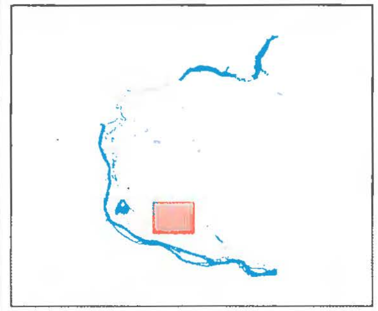
County Boundary

N

Scale: 1:20,000

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

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.

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

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

N

Assessment & GIS

proud past, promising future

CLARK COUNTY
WASHINGTON

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

Appendix B: Effectiveness Study Field Data Sheets /Observation Logs

TOYLID On-Site Observation Form

Date:	Time:	Personnel:
Weather:		Last Rain Event

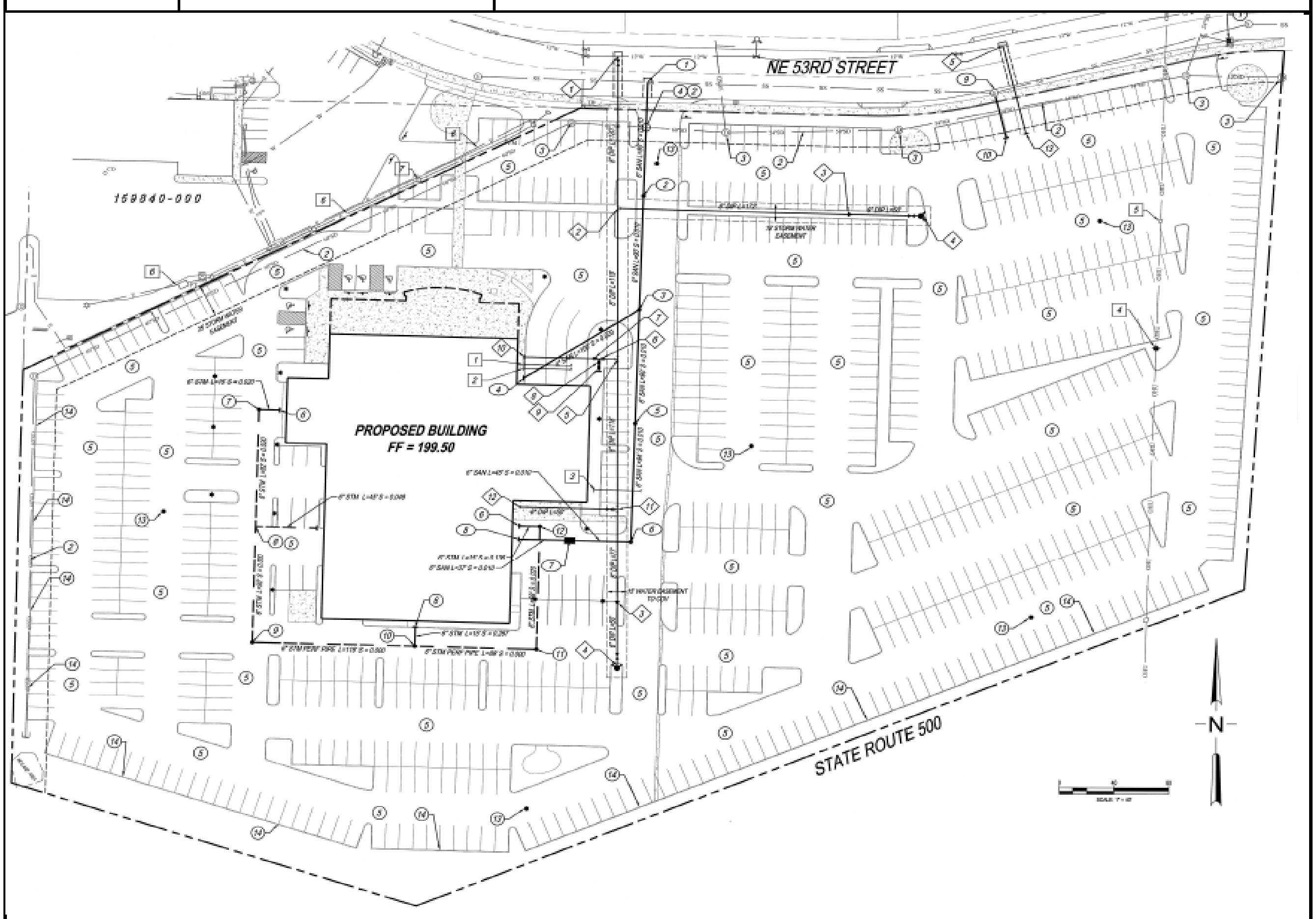
Instructions

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

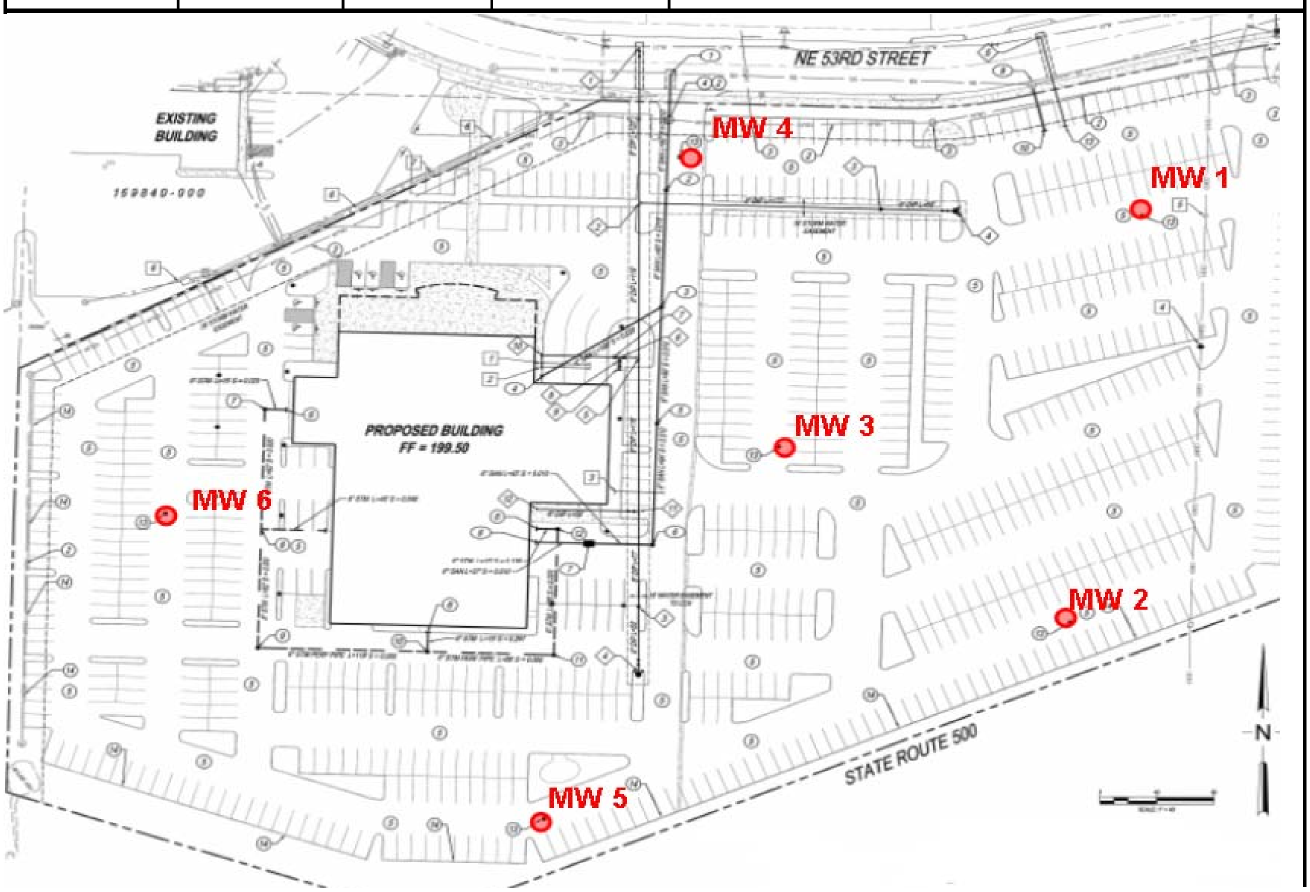
Date:	Time:	Personel:
Weather:		Last Rain Event

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				



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

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 ft^2
Slope	0.5 %
Length of overland flow	1 ft

Run-on Description:

Type of surface	**No run-on**
Area	0 ft^2
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 ft^3)
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

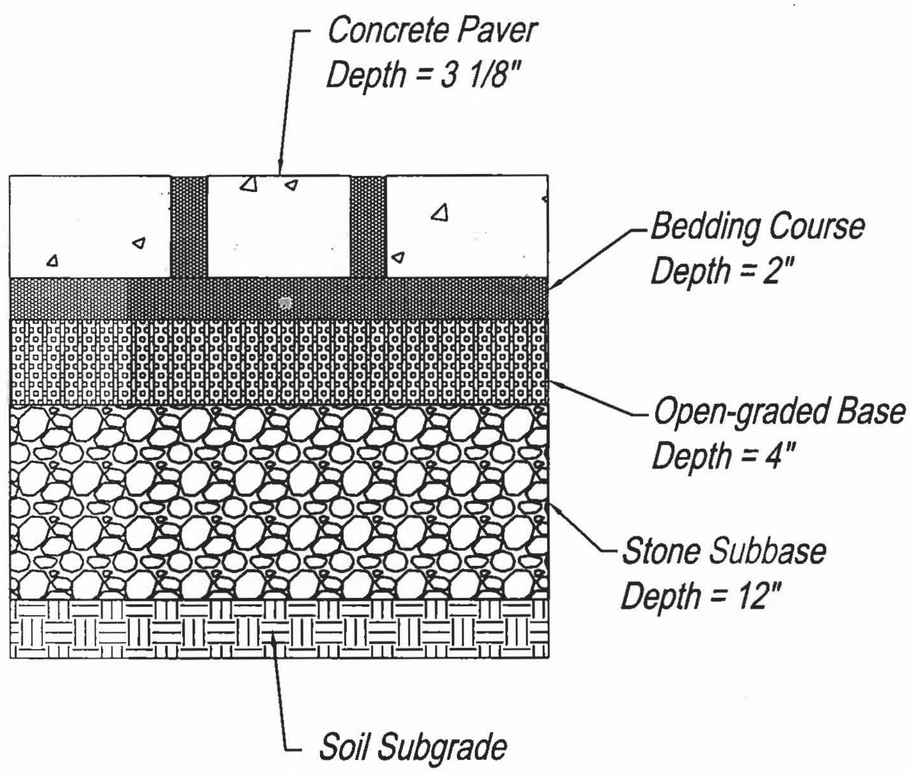
Surface summary:	**Volume**	**Depth**
Total rainfall	15572.7 ft^3	0.550 in
Total infiltration	15454.73 ft^3	0.546 in
Total evaporation	117.9749 ft^3	0.004 in
Total runoff	0 ft^3	0.000 in
Remaining surface storage	0 ft^3	0.000 in

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.



MacKay & Sposito, Inc.
M & S
ENGINEERS SURVEYORS
PLANNERS
1325 SE TECH CENTER DRIVE, SUITE 140 VANCOUVER, WA 98683
(360) 695-3411 (503) 289-6726 (888) 695-3411 FAX (360) 695-0833
www.mackaysposito.com

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:

$$\frac{180.0 \text{ in.} \times \text{ft.} \times \text{hr.} \times \text{min.}}{\text{hr.} \times 12 \text{ in.} \times 60 \text{ min.} \times 60 \text{ sec.}} = 0.00417 \text{ c.f.s./s.f.}$$

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
Engineering • Consulting • Testing

14192,- 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
Portland, Oregon 97217
Telephone (503) 289-1778
Fax (503) 289-1918

PSI Report No. 704-75065-1

March 30th, 2007

psi Information To Build On
Engineering • Consulting • Testing

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STATE OF WASHINGTON
12482
REGISTERED SINCE 1979
PROFESSIONAL ENGINEER

EXPIRES: 1-27-09

Charles R. Lane, P.E.
Senior Geotechnical Engineer
Geotechnical Services

Yuxin Lang
Engineering Associate
Geotechnical Services

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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.

Ground Water

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

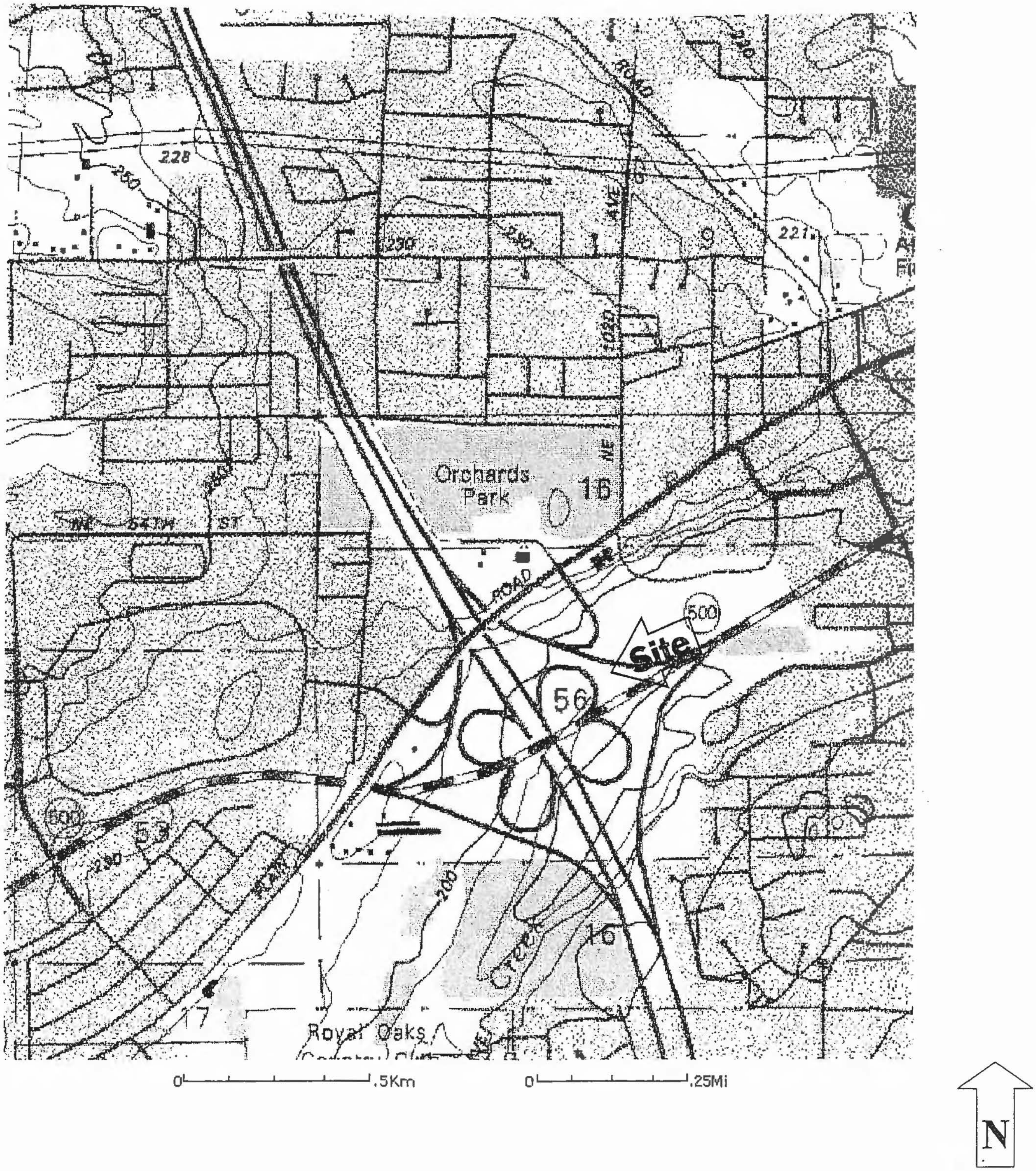
The recommendations submitted in this report are based on the available subsurface information obtained by PSI and design details furnished by our client for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report 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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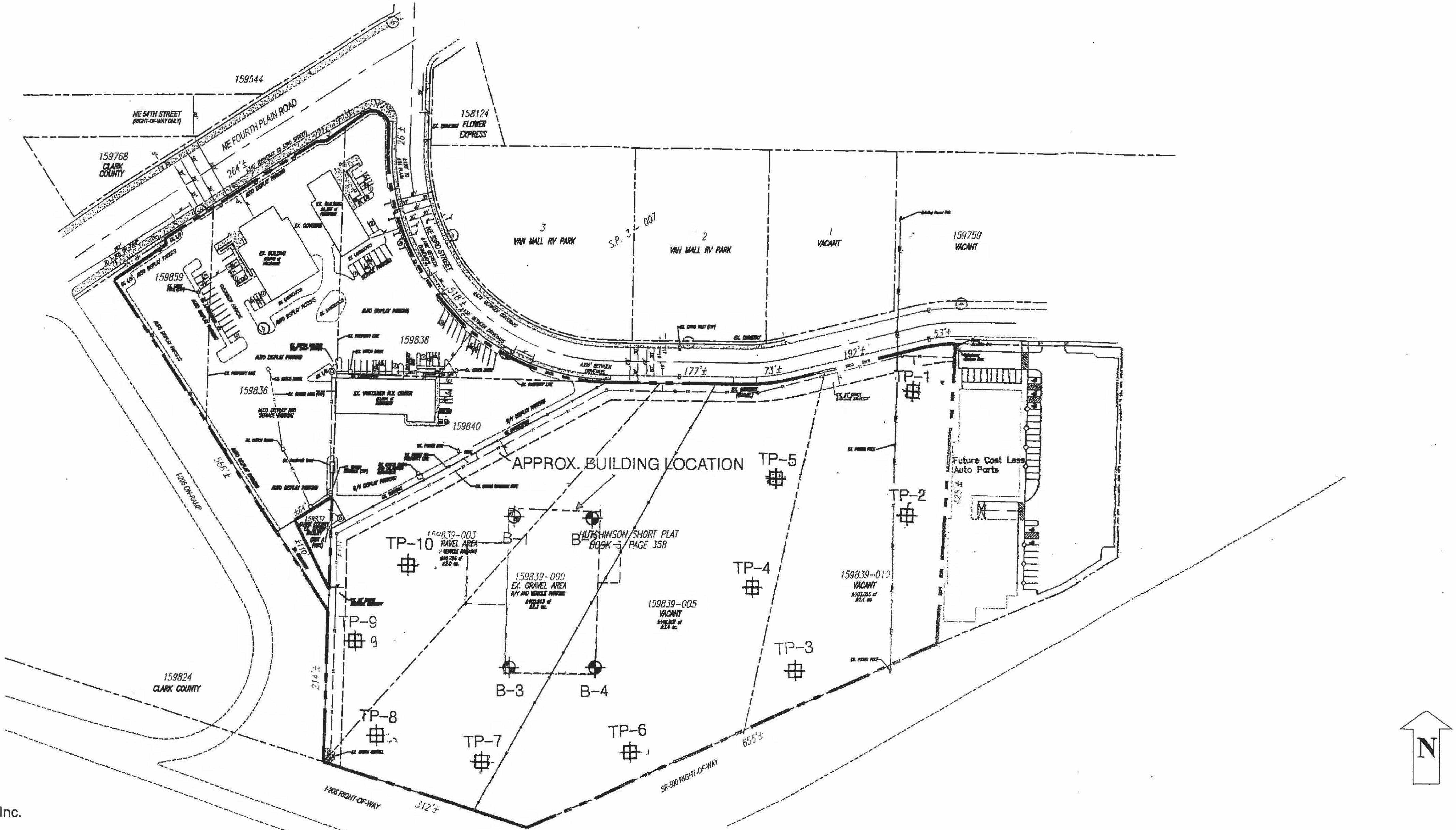
FIGURE 2: BORING AND TEST PIT LOCATION PLAN



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		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
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 (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				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

(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

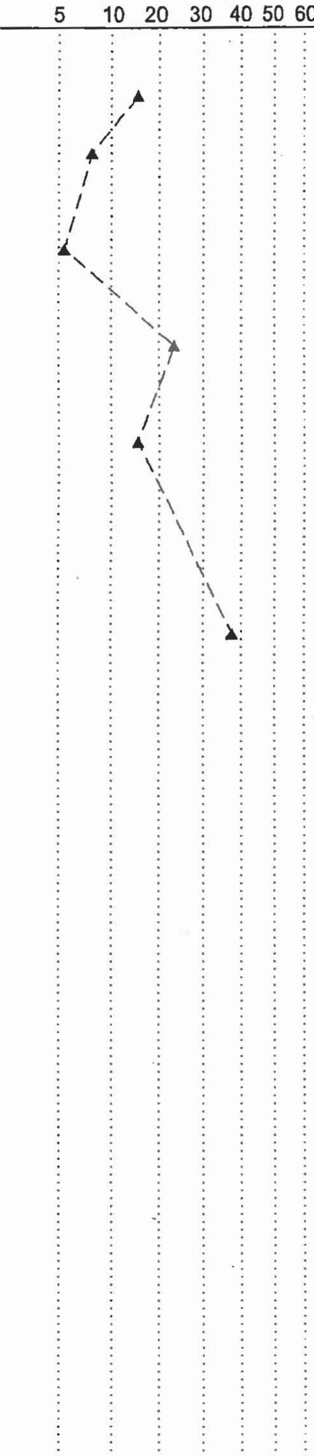
SOIL DESCRIPTION

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.

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, trace gravel, organic, and peat pockets, dark brown to brown, moist, medium stiff				4-6-9	3.0			
	SPT 2			Some asphalt pieces between 2.5 and 4 feet				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.								
20				Groundwater was not encountered during site exploration.								
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

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

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

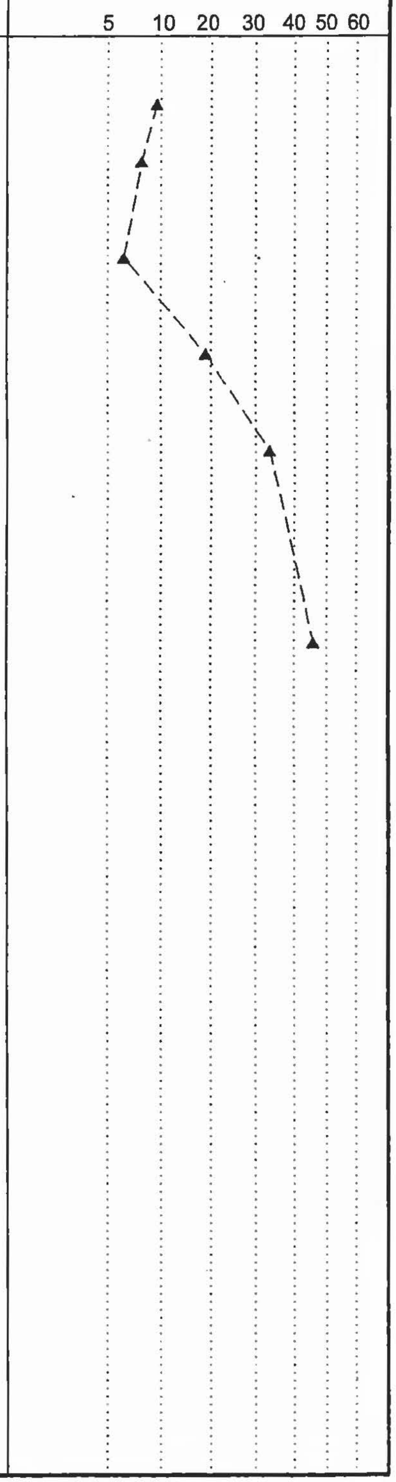
DATE OF EXPLORATION: 3/19/2007
EQUIPMENT: CME-75 Hollow Stem Auger w/Auto SPT Hammer
LOGGED BY: T. French
BORING LOCATION: See Boring and Test Pit Location Plan

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 *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 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			
	SPT 4			**SILT W/WEATHERED GRAVELS AND TRACE SAND**-brown, moist to wet, medium dense				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



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

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'

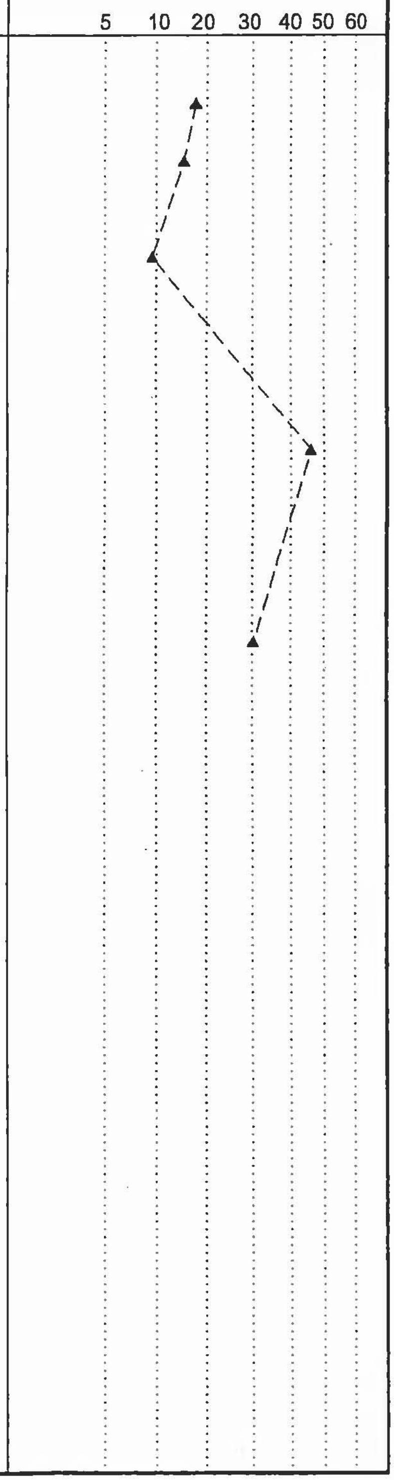
The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

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.

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
0				CRUSHED GRAVEL-2 inches thick								
				FILL-sandy silt/silty sand, some gravel, trace organics, trace clay, brown to dark brown, moist to wet, medium stiff								
	SPT 1							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				
16.5				Soil boring terminated at 16.5 feet below ground surface.								
				Groundwater was not encountered during site exploration.								

PENETRATION RESISTANCE
(blows/foot)
140 pound hammer/30 inch drop



BL_PDX_DCP 704-75065.GPJ CURRENT PORTLAND GEOTECH TEMPLATE.GDT 4/3/07

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

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'

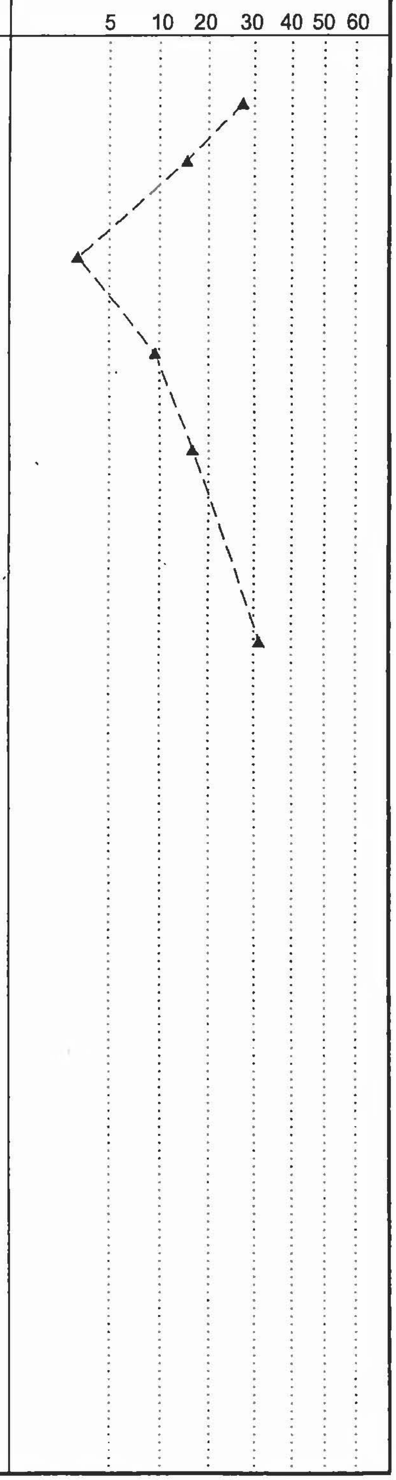
The soil boring was backfilled with auger cuttings and granular bentonite at the end of exploration

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.

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				
20				Soil boring terminated at 16.5 feet below ground surface. Groundwater was not encountered during site exploration.								
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

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

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)
0–0.5		**TOPSOIL**- 6 inches thick								
0.5–4		**SILT W/GRAVELS**- some cobbles, brown, moist								
4–5		**PEAT**- organic material, 1 foot thick								
5–6		**SILT**- some gravels, trace sand, gray, moist								
6–7.5		**SANDY GRAVEL**- some silt, brown, moist to wet								

Depth scale: 1 through 20 ft; groundwater symbol at 7 ft.

Test pit terminated at 7.5 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was encountered at a depth of 7 feet below existing site grade during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

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

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								
1–4		**GRAVELLY SAND (FILL)**- some silt, brown to dark brown, moist								
5		**PEAT**- organic material, 1 foot thick								
	GRAB 1	**SILT AND GRAVEL**- trace clay, gray, moist			18				34	

Test pit terminated at 6 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was not encountered during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

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

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)
0–0.25		**TOPSOIL**- 3 inches thick								
0.25–4.5 (1, 2, 3, 4)		**SILTY SAND/SANDY SILT (FILL)**-some gravel, occassional cobbles, brown, moist								
4.5–5.9 (5)		**OLD TOPSOIL**- black/brown								
5.9–7.3 (6, ▼7)		**SANDY GRAVEL**- some to trace silt, brown, wet to moist								

Test pit terminated at 7'4" below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was encountered at a depth of 7 feet below existing site grade during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

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

LOG OF TEST PIT NO. TP-4

CLIENT: Building Structures, Inc.	DATE OF EXPLORATION: 3/19/2007
PROJECT: Proposed Vancouver Toyota	EQUIPMENT: Test Pit
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA	LOGGED BY: Y. Lang
PSI PROJECT NUMBER: 704-75065	TEST PIT LOCATION: See Boring and Test Pit Location Plan
SURF. ELEV.:	

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)
0–0.5		**TOPSOIL-** 6 inches thick								
0.5–6.5		**SILTY SAND/SAND (FILL)-** some gravel, brown, moist to wet								
5	GRAB 1				20				22	
6.5–7		**PEAT-** organic material, 6 inches thick								
7–8		**SILT W/GRAVEL-** trace sand, gray, moist to wet								

Depth scale: 1, 2, 3, 4, 5, 6, 7, ▼ (7.5), 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

Test pit terminated at 8 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was encountered at a depth of 7.5 feet below existing site grade during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

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

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)
0–0.5		**TOPSOIL**- 6 inches thick								
0.5–5.5	GRAB 1 (~3)	**SILTY SAND/SANDY SILT (FILL)**-some gravel, brown, moist			14				33	
5.5–6.5		**OLD TOPSOIL**- black/brown								
6.5–7.5		**SILT W/GRAVEL**- trace sand, brown, wet								

Depth scale: 1–20 ft; groundwater symbol at 7.5 ft.

Test pit terminated at 7.5' below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was encountered at a depth of 7.5 feet below existing site grade during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

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-6

CLIENT: Building Structures, Inc.	DATE OF EXPLORATION: 3/19/2007
PROJECT: Proposed Vancouver Toyota	EQUIPMENT: Test Pit
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA	LOGGED BY: Y. Lang
PSI PROJECT NUMBER: 704-75065	TEST PIT LOCATION: See Boring and Test Pit Location Plan
SURF. ELEV.:	

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)
0–0.6		**TOPSOIL**- 7 inches thick								
1–4	GRAB 1	**SILTY SAND** - some gravel, some asphalt and concrete debris, brown, moist			33				36	
4–5		**PEAT**- organic material, 10 inches thick								
5–7.5		**SILT W/GRAVEL AND SAND**- gray, moist to wet								

Depth scale: 1, 2, 3, 4, 5, 6, ▼7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

Test pit terminated at 7.5 below ground surface.
Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was encountered at a depth of 7 feet below existing site grade during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

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)
0–0.5		**TOPSOIL**- 6 inches thick								
0.5–4.5		**SANDY SILT/SILTY SAND (FILL)**-some gravel, some asphalt debris, brown, moist								
4.5	GRAB 1				47				28	
4.5–8.5		**SANDY GRAVEL**- some silt, trace cobbles, brown, moist								

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

Test pit terminated at 8.5' below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was encountered at a depth of 8'4" feet below existing site grade during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

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.	DATE OF EXPLORATION: 3/19/2007
PROJECT: Proposed Vancouver Toyota	EQUIPMENT: Test Pit
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA	LOGGED BY: Y. Lang
PSI PROJECT NUMBER: 704-75065	TEST PIT LOCATION: See Boring and Test Pit Location Plan
SURF. ELEV.:	

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)
0		**CRUSHED GRAVEL**- 2 inches thick								
1–6		**SILTY SAND** (FILL)- some gravels, some aphalt debris, brown to dark brown, moist								
6–7		**PEAT AND TOPSOIL**- 20 inches thick								
7.5–8	GRAB 1	**SILT W/GRAVEL AND SAND**- trace clay, brown, moist to wet			17				43	

Test pit terminated at 8 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was not encountered during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

Depth scale: 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

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

LOG OF TEST PIT NO. TP-9

CLIENT: Building Structures, Inc.	DATE OF EXPLORATION: 3/19/2007
PROJECT: Proposed Vancouver Toyota	EQUIPMENT: Test Pit
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA	LOGGED BY: Y. Lang
PSI PROJECT NUMBER: 704-75065	TEST PIT LOCATION: See Boring and Test Pit Location Plan
SURF. ELEV.:	

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								48	
6		**SILT W/GRAVEL AND SAND**- trace clay, gray, moist								
7										
8		Test pit terminated at 6 below ground surface. Test pit loosely backfilled upon completion with excavation spoils.								
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

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

LOG OF TEST PIT NO. TP-10

CLIENT: Building Structures, Inc.	DATE OF EXPLORATION: 3/19/2007
PROJECT: Proposed Vancouver Toyota	EQUIPMENT: Test Pit
LOCATION: 1009 NE Fourth Plain Rd., Vancouver, WA	LOGGED BY: Y. Lang
PSI PROJECT NUMBER: 704-75065	TEST PIT LOCATION: See Boring and Test Pit Location Plan
SURF. ELEV.:	

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)
0		**CRUSHED GRAVEL**- 2 inches thick								
0–4.5		**SILTY SAND (FILL)**- some gravels, brown to dark brown, moist								
4.5–5.25		**SILT AND GRAVEL**- trace to some sand, gray, moist								
5.5	GRAB 1				19				35	

Test pit terminated at 5'3" below ground surface. Test pit loosely backfilled upon completion with excavation spoils.

Groundwater was not encountered during site exploration.

Stratification lines/depths shown are approximate. Actual soil conditions encountered during construction may vary from those described above.

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

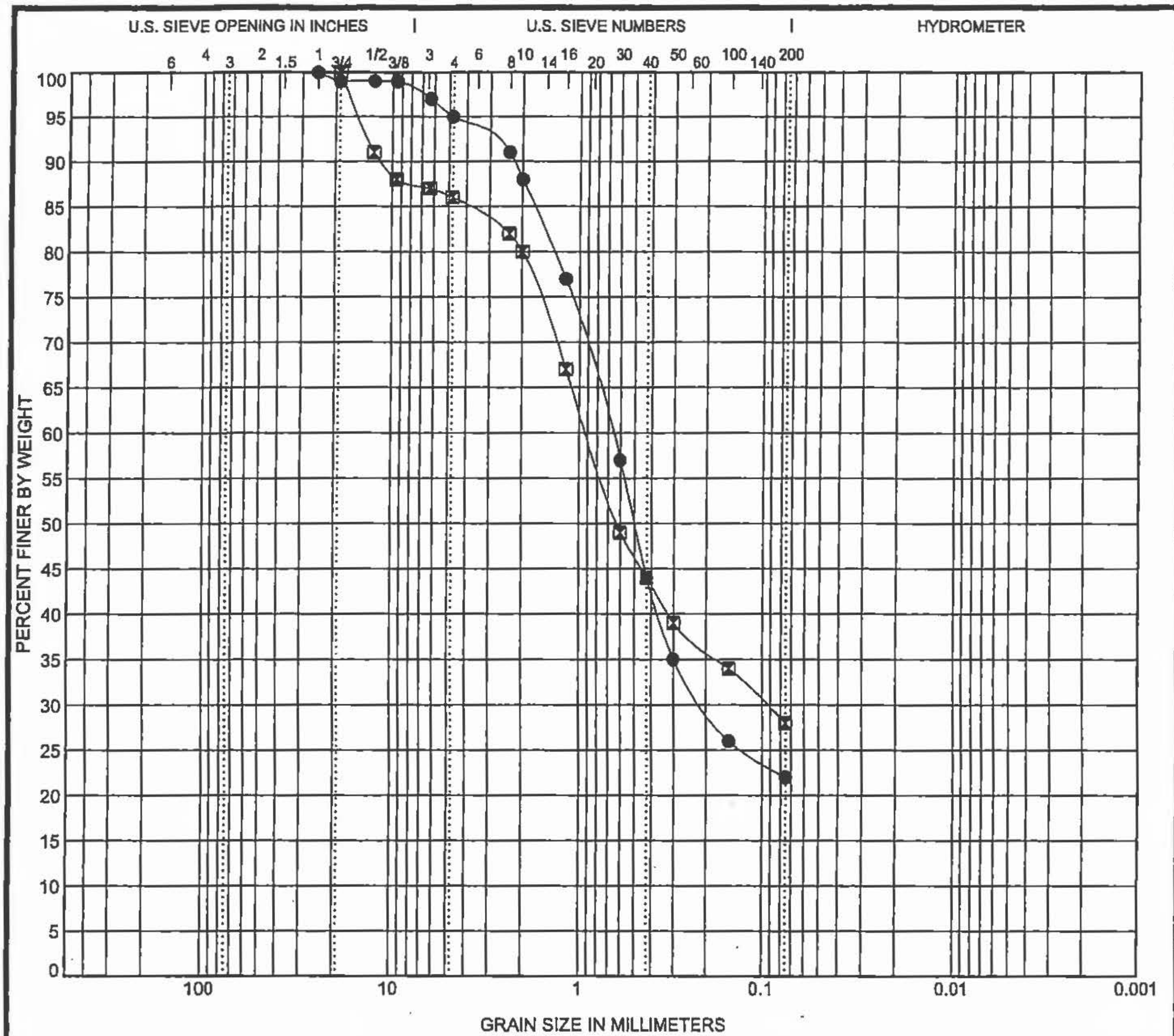
TP PTLD 704-75065.GPJ PSI CORP.GDT 4/3/07

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

(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. SI5222

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

☒ 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.

☒ Driller ☐ Engineer ☐ Trainee Name (Print) Dannyl Metzger

Driller/Engineer/Trainee Signature Danny Metzger

Driller or Trainee License No. 2587

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

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

○ Resource Protection

☒ Geotech Soil Boring

Property Owner Chevron

Site Address 9414 Vancouver Mall Dr.

City Vancouver County: Clark

Location SE 1/4 NW 1/4 Sec 16 Twn 2N R 2E EWM or WWM *circle one*

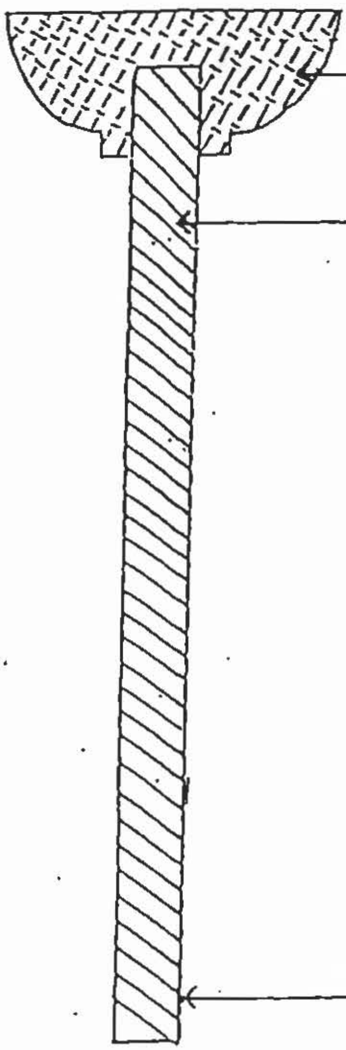
Lat/Long (s, t, r still REQUIRED) Lat Deg ______ Lat Min/Sec ______ Long Deg ______ Long Min/Sec ______

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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APR 06 2006
Washington State Department of Ecology

Scale 1"= ______ Page ______ of ______

ECY 050-12 (Rev 2/01)

Washington State Department of Transportation

LOG OF TEST BORING

177232

Start Card S-14911

HOLE No. H-8-03

Sheet 1 of 2

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

Project SW Region Quality Engineering Center

Driller Vince Johnson Lic# 2532

Site Address Vic. Gher Drive

Inspector Cleo Andrews

Start April 3, 2003 Completion April 3, 2003 Well ID#

Equipment CME 55 w/ autohammer

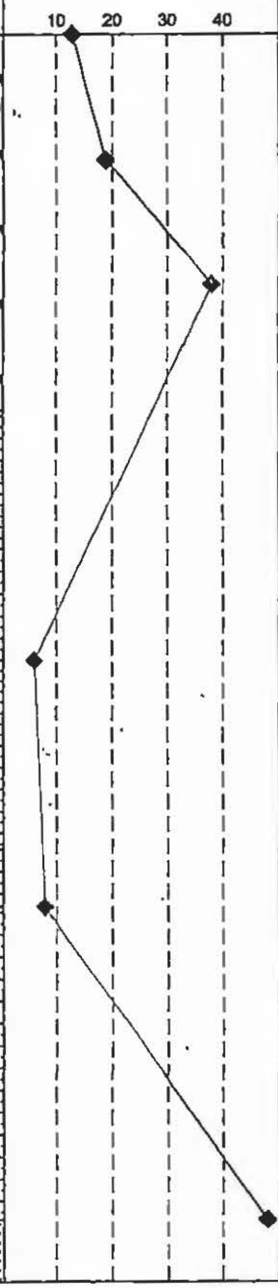
Station 40' E of W boundary Offset 131.5' S of curbline Casing HWT 4" x 25.0'

Method Wet Rotary

Northing Easting Latitude Longitude

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

Standard Penetration Blows/ft



Depth (ft)	Meters (m)	SPT Blows/6" (N)	Sample No. (Tube No.)	Lab Tests	Description of Material	Groundwater
0		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	
	1	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		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		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	
	5					
20	6	10 25	D-9	GS MC	SM, MC=23% Silty SAND with olive brown stains, dense, olive gray,	

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		
	7							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	8									
30	9									
	10									
35	11									
40	12									
	13									
45										

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.

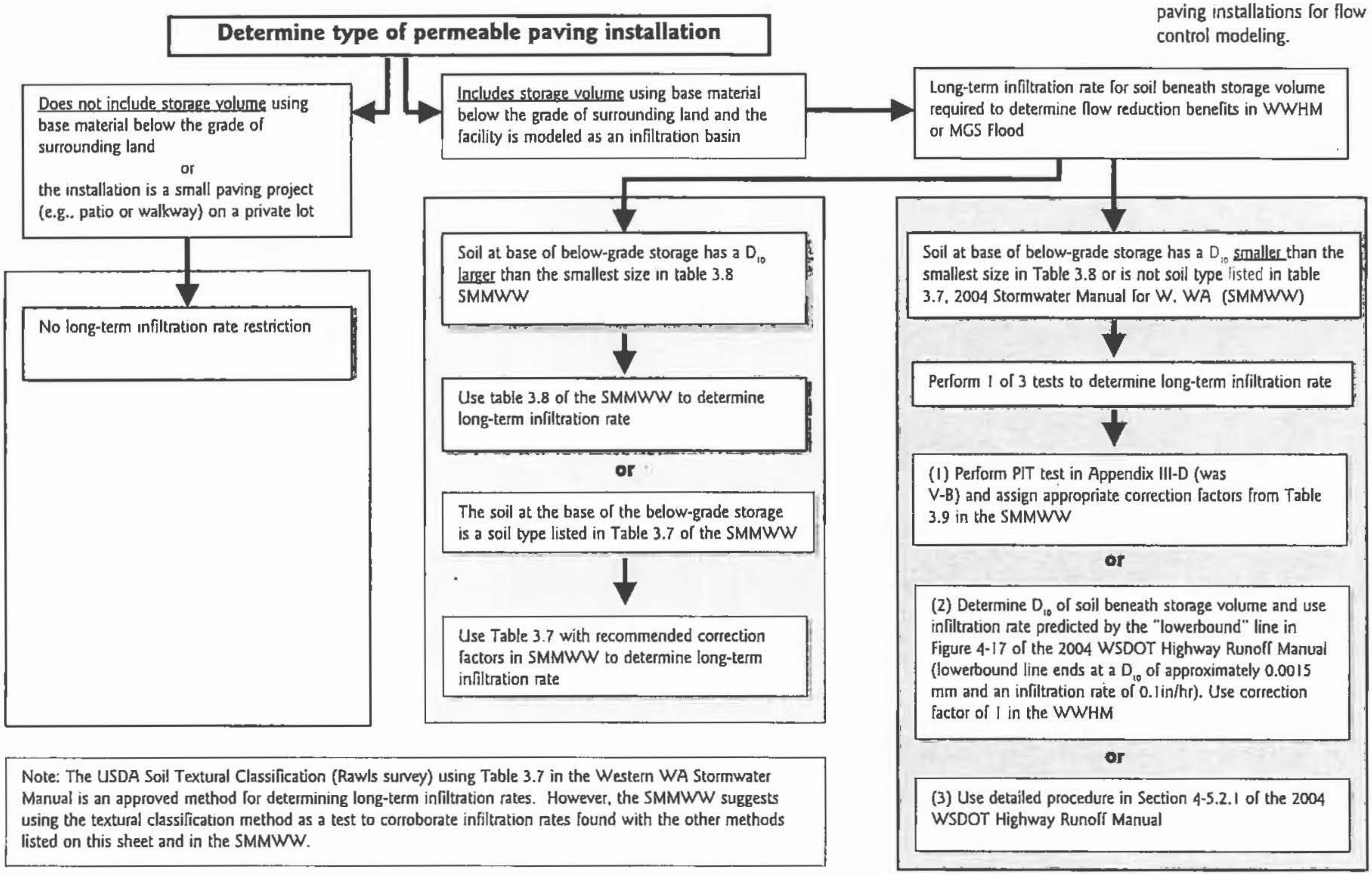
Properly designed permeable paving installations have performed well in the Midwestern and Northeastern U.S. where freeze-thaw cycles are severe (Adams, 2003 and Wei, 1986). Risk of freeze damage can be minimized by extending the base of the permeable paving system to a minimum of half the freeze depth. 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:
  - o Method 1: Use USDA Soil Textural Classification (Rawls survey) every 200 feet of road or every 5,000 square feet.
  - o 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.
  - o 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.
  - o 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).

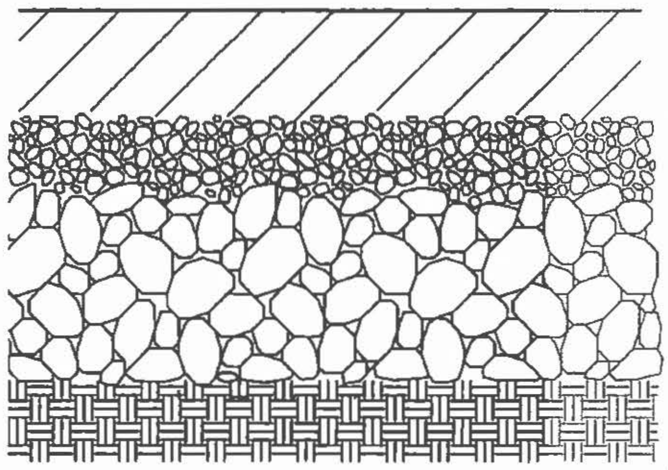


Figure 6.3.4 Permeable asphalt section.

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).

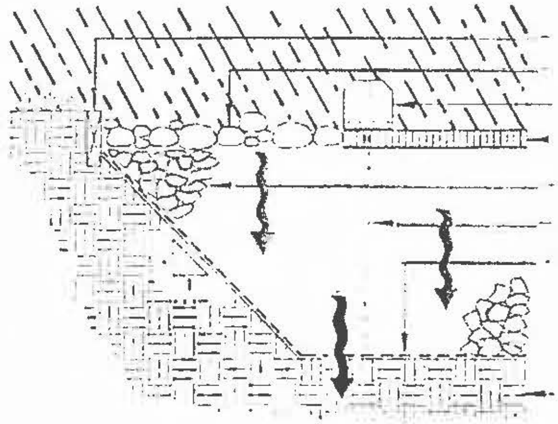


Figure 6.3.5 Unpaved section (river jacks) provides backup infiltration.
Graphic courtesy of Cahill Associates

- 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).

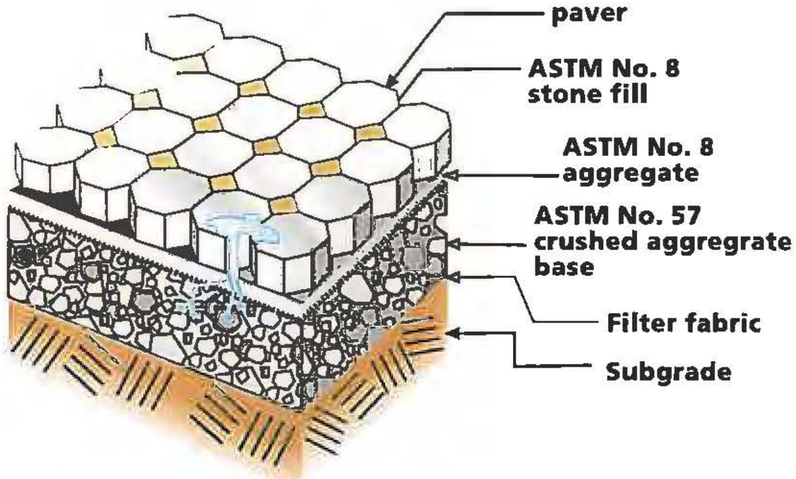


Figure 6.3.7 Permeable interlocking concrete paver section.
Graphic by Gary Anderson

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).

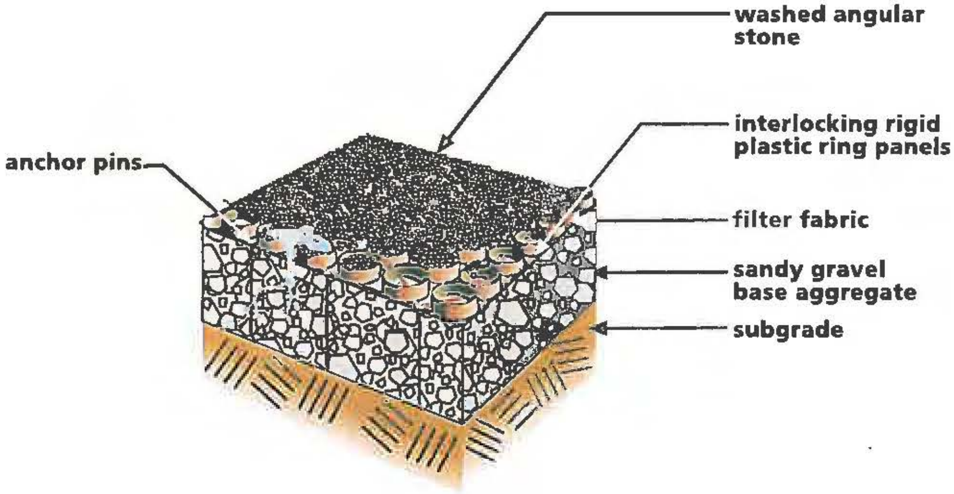
Cost

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.

- o 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
  - o Remove and replace top course aggregate if clogged with sediment or contaminated (vacuum trucks for stormwater collection basins can be used to remove aggregate).
  - o Remove and replace grid segments where three or more adjacent rings are broken or damaged.
  - o Replenish aggregate material in grid as needed.
  - o 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).

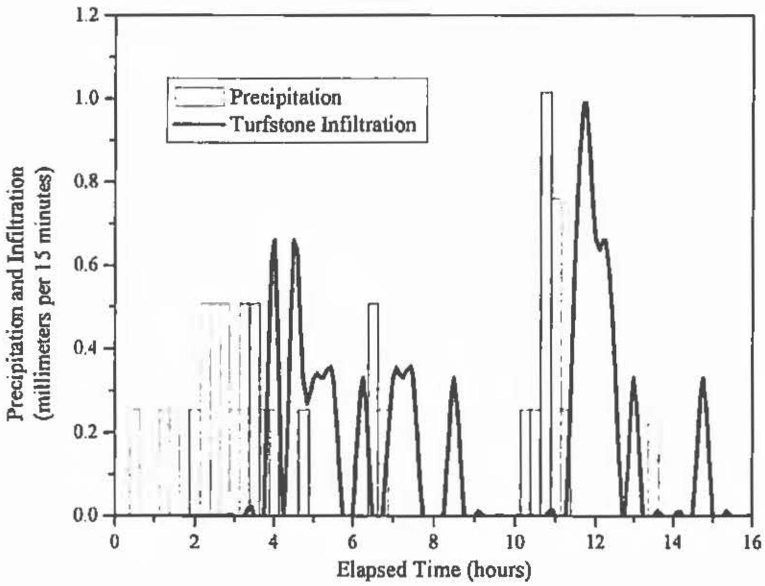


Figure 6.3.11 Infiltration plotted with precipitation at a test permeable pavement parking stall in the city of Renton. Note that essentially all precipitation infiltrates.

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