Development Services

From Concept to Construction

Phone: 503-823-7300 Email: bds@portlandoregon.gov 1900 SW 4th Ave, Portland, OR 97201
More Contact Info (http://www.portlandoregon.gov//bds/article/519984)

APPEAL SUMMARY

Status: Decision Rendered - Reconsideration of 14825

Appeal ID: 14919	**Project Address:** 4540 SE Milwaukie Ave
Hearing Date: 4/12/17	**Appellant Name:** Martha Williams, PE
Case No.: P-001	**Appellant Phone:** 503-946-6690
Appeal Type: Plumbing	**Plans Examiner/Inspector:** Joe Blanco
Project Type: commercial	**Stories:** 4 **Occupancy:** R-2 **Construction Type:** V-A
Building/Business Name:	**Fire Sprinklers:** Yes - in building
Appeal Involves: Erection of a new structure	**LUR or Permit Application No.:** 17-111682-CO
Plan Submitted Option: pdf [File 1] [File 2] [File 3] [File 4] [File 5] [File 6]	**Proposed use:** Multi-family housing

APPEAL INFORMATION SHEET

Appeal item 1

Code Section	Oregon Plumbing Specialty Code - Storm Drainage 1101.5.3.2
Requires	Oregon Plumbing Specialty Code requires that no drywell shall be located closer than 5 feet (1524 mm) of a property line nor closer than 10 feet (3048mm) to a building unless approved by the building official.
Proposed Design	The applicant is proposing the use of a drywell system to be installed underneath the building structure for multiple reasons; see ‘Reason for Alternative’ section for more information. Drywells located under the structure have been taken into account by the Geotechnical engineer and structural engineer per attached documents. The drywell system proposed for the building was sized to infiltrate the 10-year storm since there is a path for safe overland flow. The building roof area would produce 0.21 cfs of runoff during the 10-year storm. During larger storm events, the rim of the drywell will act as an overflow outlet, which is located in the safe overland flow path (see attachment for path). The sizing of the drywell system was done using HydroCAD®. A design infiltration rate of 4.5 in/hr was used for calculations. The drywell system will be tested at the time of installation to verify infiltration capacity. Feasibility of on-site infiltration: The feasibility of the drywell system location is based on infiltration testing, maintenance, structural design, safe overland flow, and strength of soils. Infiltration testing was performed by Hardman Geotechnical Services Inc. and documented in the "Geotechnical Engineering Report” dated July 18, 2016. This report shows measured infiltration rates of 9 in/hr onsite at a depth of 21 feet. The drywells are deep and will be discharging stormwater 5-15 feet below the bottom of the footings. The infiltration rate of the deep soils will prevent saturation of the shallow soils

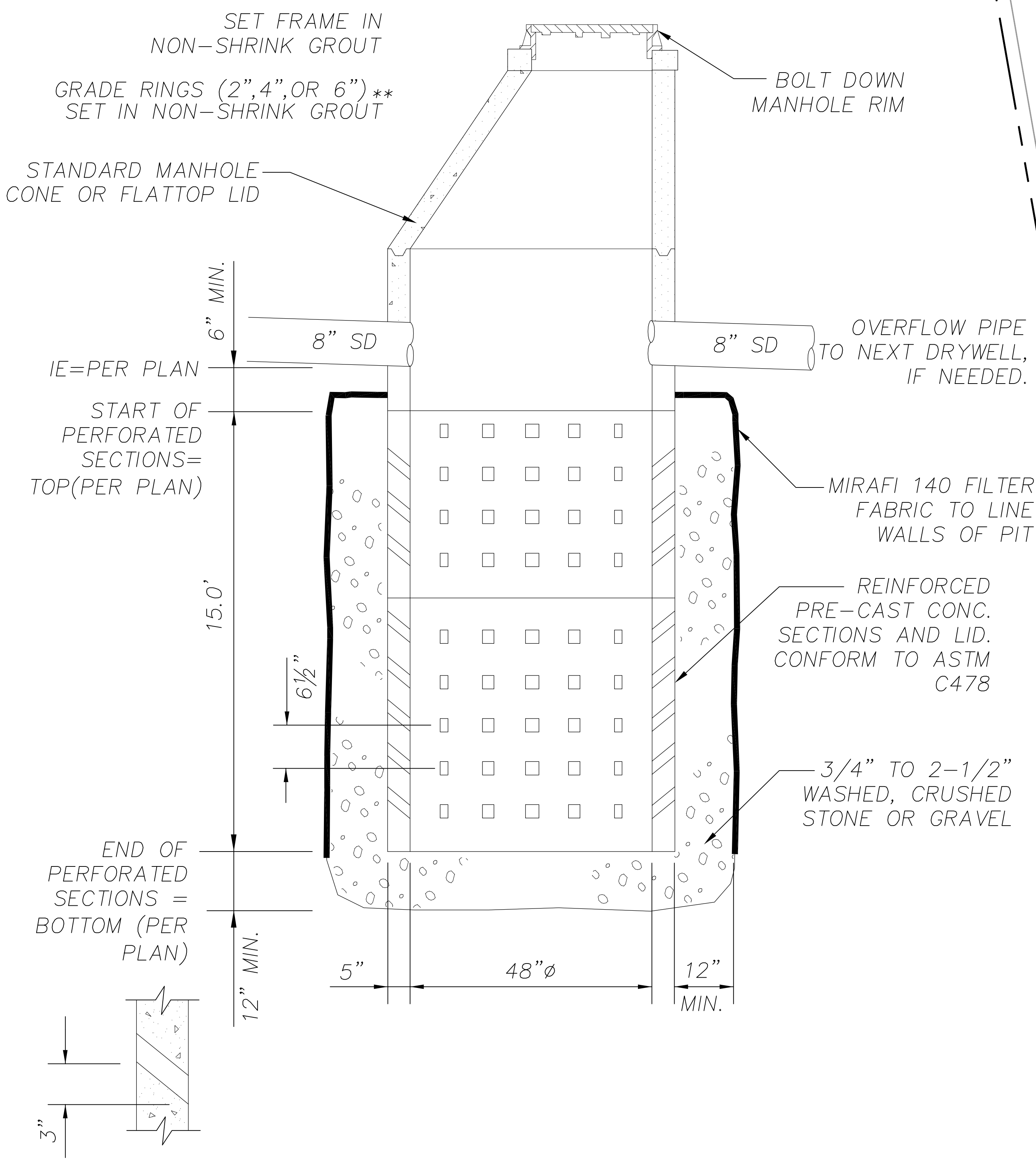
	directly underneath the building. See attachments for supporting data on the effectiveness of infiltration for the site.
Reason for alternative	The applicant proposes the drywell system to be installed underneath the building structures due to space limitations on the site (e.g. the vertical construction inhabits the property footprint, which eliminates any potential for locating the drywell system outside of the building) preventing location of drywells in accordance with the OPSC. Mitigation of Maintenance and Overflow Concerns: • All the drywells will have accessible, bolt down manhole rims located in open vehicle drive aisles or loading areas to allow for maintenance as required by Oregon Department of Environmental Quality (ODEQ). Maintenance will be performed in the same manner as if the drywell was located outside the building. The applicant has confirmed with a local company (River City Environmental Inc.) that a vacuum truck is capable of reaching lengths up to 300 feet for drywell maintenance. The drywells will be maintained by a professional management company who will follow the county recorded operations and maintenance plan for the drywells. Mitigation of Soil Bearing Concerns • The strength of the soils will not be affected by the infiltration of stormwater runoff as explained in the attached memo from Hardman Geotechnical Services, Inc. dated March 9, 2017.

APPEAL DECISION

Drywell system located beneath the building: Denied.

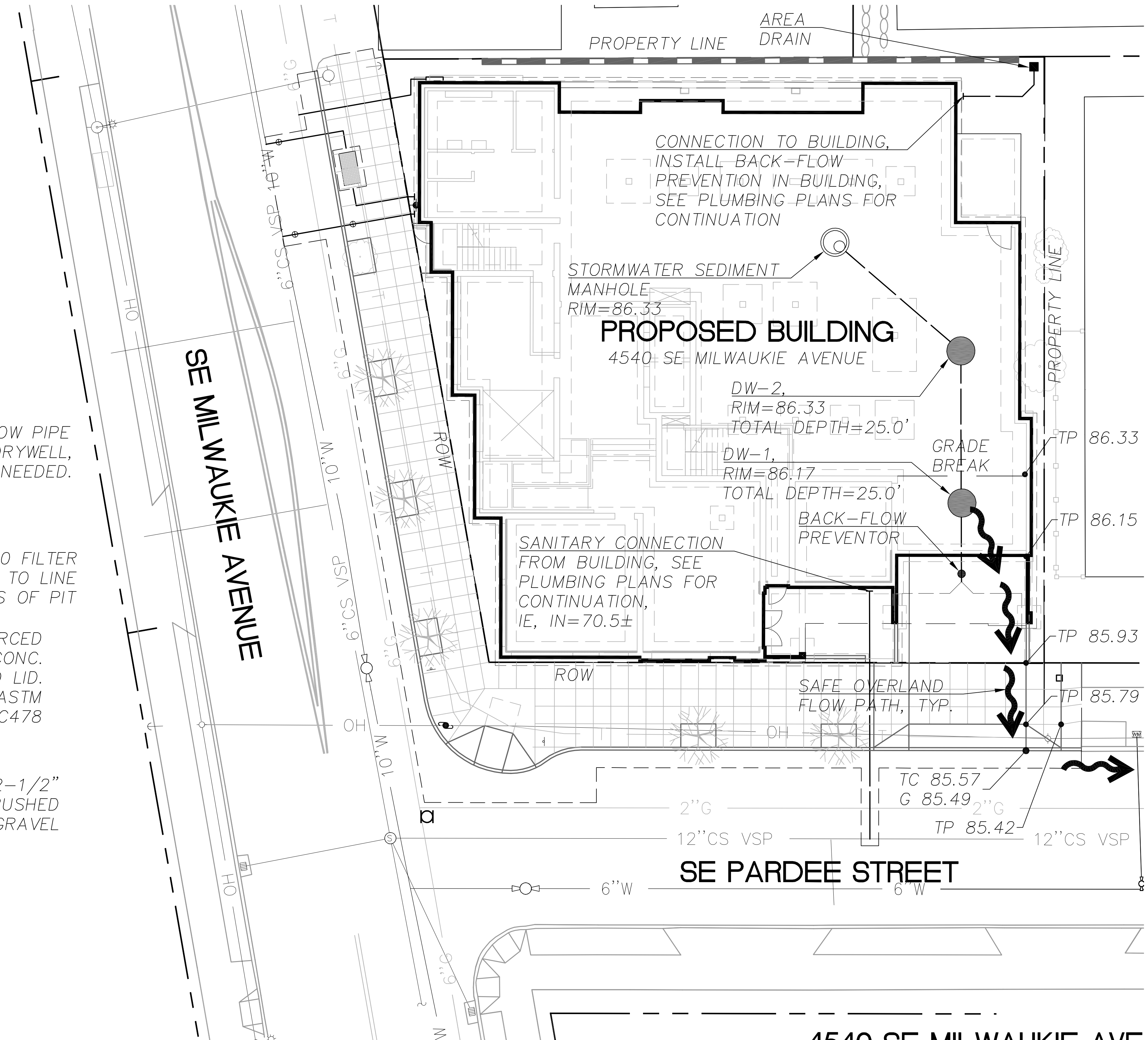
Appellant may contact Joe Blanco (503) 823-2059 for more information.

Pursuant to City Code Chapter 25.07, you may appeal this decision to the Plumbing Code Board of Appeal within 180 calendar days of the date this decision is published. For information on the appeals process and costs, including forms, appeal fee, payment methods and fee waivers, go to www.portlandoregon.gov/bds/appealsinfo, call (503) 823-7300 or come in to the Development Services Center.



1 **DRYWELL**

NTS



4540 SE MILWAUKIE AVE

SCALE: 1"=20'

GRAPHIC SCALE

0 20 40

1 inch =20 ft.

PROJECT NO.:	SMI001
DRAWN BY:	AFW
DESIGN BY:	AFW
REVIEWED BY:	MSW
DATE:	03/10/2017

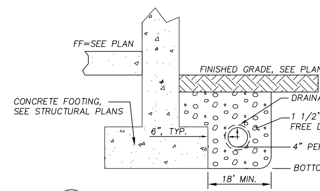
Humber Design Group, Inc.

Portland, OR • 503.946.6690 • hdgpdx.com

P:\SMI001 – 4540 SE Milwaukie\7 – CAD Drawings\SMI001-C3.00 Utilities.dwg Apr 06, 2017 – 2:05pm

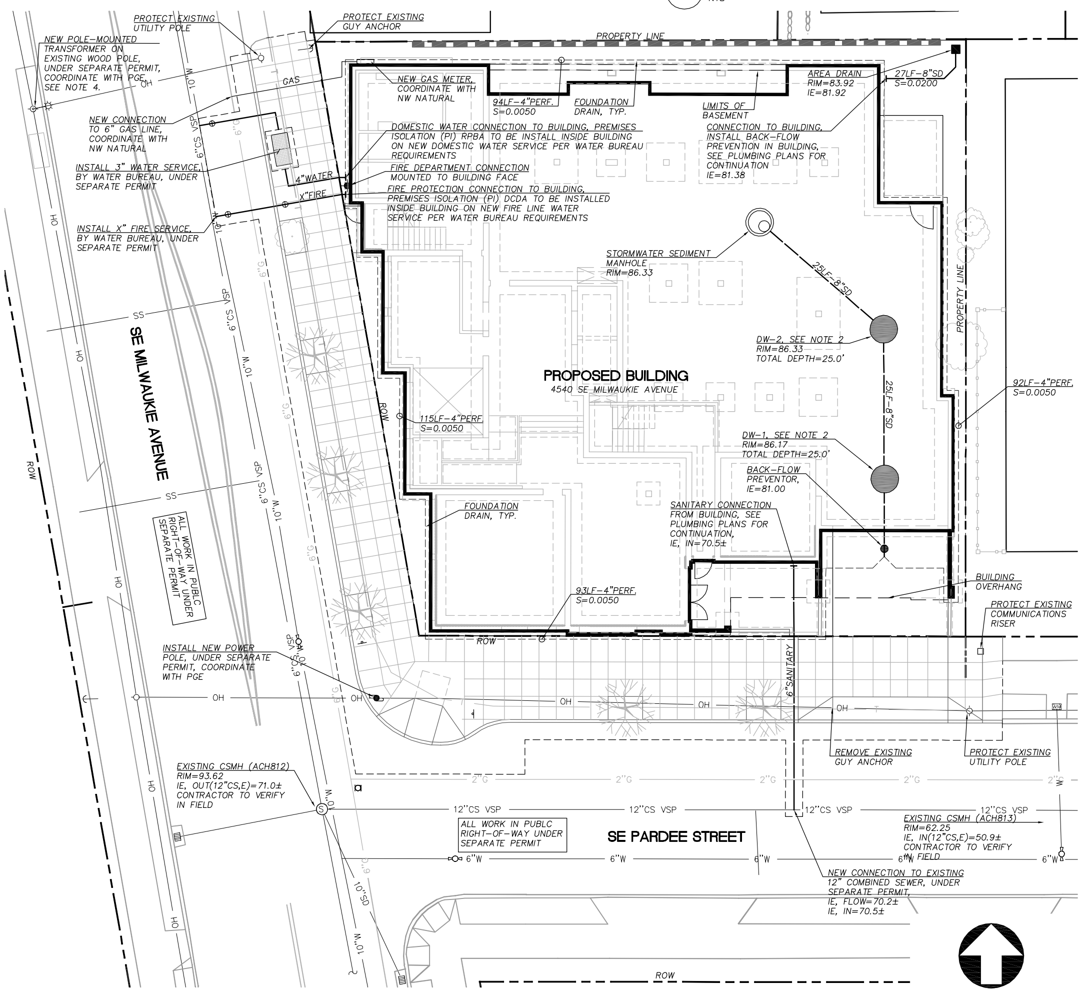
SHEET NOTES

1. ALL DOMESTIC WATER AND FIRE PROTECTION WORK IN THE PUBLIC RIGHT OF WAY BY PORTLAND WATER BUREAU AT OWNER'S EXPENSE. CONTRACTOR TO COORDINATE WORK WITH PORTLAND WATER BUREAU.
2. CONTRACTOR SHALL INSTALL AND TEST DRYWELL-1 (DW-1) PRIOR TO THE INSTALLATION OF DW-2. IF STORMWATER REQUIREMENTS ARE MET BY DW-1, DW-2 DOES NOT NEED TO BE INSTALLED.
3. ALL RUNOFF FROM ROOF AND COURTYARD ROUTED TO STORMWATER DRYWELLS IN PARKING GARAGE.
4. ELECTRICAL TRANSFORMER TO BE POLE-MOUNTED. COORDINATE WITH PGE, CONTACT CHARLES EVANS (503-258-7630).



NOTES:
1. PERFORATED PIPE AND SURROUNDING DRAIN ROCK SHOULD BE WRAPPED IN NON-WOVEN GEOTEXTILE. MARAFI 140N OR APPROVED EQUAL.

3 FOUNDATION DRAIN
NTS



UTILITY PLAN
SCALE: 1"=10'

GRAPHIC SCALE
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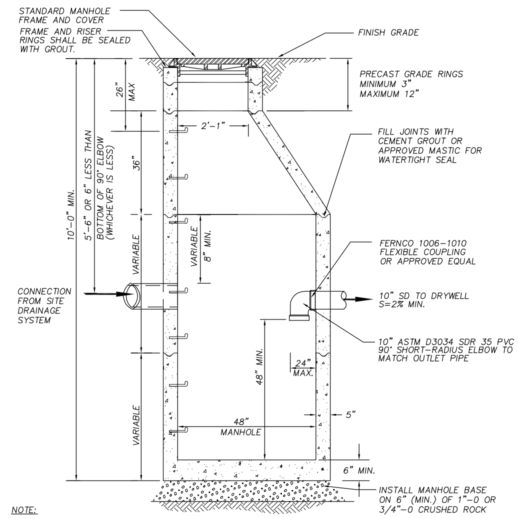
1 inch =10 ft.



NOTE:
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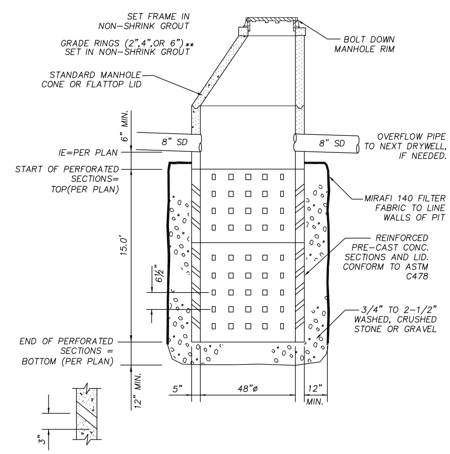
2 SEDIMENTATION MANHOLE
NTS



1 DRYWELL
NTS

DRYWELL TESTING NOTES

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5. NOTIFY CIVIL AND GEOTECHNICAL ENGINEERS 5 DAYS PRIOR TO TESTING.
6. CONTRACTOR SHALL CONTACT CITY OF PORTLAND WATER BUREAU OR APPLICABLE WATER DISTRICT TO ARRANGE FOR DRYWELL TEST WATER SUPPLY. CONTRACTOR SHALL BEAR RESPONSIBILITY FOR SECURING ALL NECESSARY PERMITS, AUTHORIZATION AND ANY FEES.
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8. CLEAN WATER SHALL BE PROVIDED TO TEST DRYWELLS, AS APPROVED. INTRODUCTION OF SEDIMENT MAY RESULT IN FAILURE OF THE DRYWELL CAPACITY TEST.
9. PROVIDE CIVIL ENGINEER WITH RECORDED TEST DATA.

REGISTERED PROFESSIONAL ENGINEER 76,071 OREGON JULY 12, 2005 MARTHA S. WILLIAMS
EXPIRES 6-30-2018

BARRY R SMITH, PC Architect
715 SW MORRISON STREET SUITE 909
PORTLAND OR 97205 503.295.6261 www.barrysmith.com

4540 SE MILWAUKIE AVENUE
PORTLAND, OR
for YOSHIDA GROUP

PERMIT SET

UTILITY PLAN
C2.0

Humber Design Group, Inc.
Portland, OR • 503.946.6690 • hdgpdx.com

01.23.2017

HARDMAN GEOTECHNICAL SERVICES INC.
Practical, Cost-Effective Geotechnical Solutions

March 9, 2017
HGSI Project No. 16-2049

Yoshida Real Estate Holdings XVI LLC
2905 SW 1st Avenue
Portland, Oregon 97201

Copy: Barry Smith, Architect

Subject: **SUPPLEMENTAL GEOTECHNICAL CONSULTATION REGARDING DRY WELLS**
SE MILWAUKIE AVE APARTMENTS
PROPERTY AT FORMER ADDRESSES 4524 AND 4540 SE MILWAUKIE AVENUE
PORTLAND, OREGON

References: 1. *Geotechnical Engineering Report, 4524 and 4540 SE Milwaukie Avenue, Portland, Oregon*; HGSI report dated July 18, 2016.

As requested, Hardman Geotechnical Services Inc. (HGSI) prepared this letter to address foundation-to-dry well setbacks for use on the project.

The attached figure shows the currently planned locations of dry wells beneath the building. The dry wells are located at least 5 feet away from the nearest building foundations, in conformance with the recommendations of the geotechnical report. Also per the recommendations of Reference 1, the dry wells will be provided with an overflow outlet to prevent flooding of the parking garage during an overflow event.

In the event that dry wells are located near the building's exterior wall, manholes should be designed and constructed in a manner which will limit disturbance of surrounding soils, and prevent the loss of soil from the surrounding area into the manhole. We recommend use of geotextile fabric between the manhole and any surrounding drain rock, and the native soils. Annular space between the manhole and surrounding soil should be backfilled tightly with compacted crushed rock or drain rock.

The potential for soil saturation has been accounted for in development of the allowable soil bearing pressures used in design, as discussed in the project geotechnical report. Therefore, soil saturation from dry well operation is not anticipated to cause significant adverse impacts on foundation support.

This letter should be considered supplemental to the above-referenced geotechnical report. The conclusions, recommendations, uncertainties and limitations of that report remain applicable, except where modified herein.

We appreciate this opportunity to be of service.

Sincerely,

HARDMAN GEOTECHNICAL SERVICES INC.

REGISTERED PROFESSIONAL
ENGINEER
64,422PE
OREGON
SEPTEMBER 26, 2000
SCOTT L. HARDMAN

3-9-17

EXPIRES: 06-30-2017

Scott L. Hardman, P.E., G.E.
Geotechnical Engineer

Attachment: Utility Plan prepared by others

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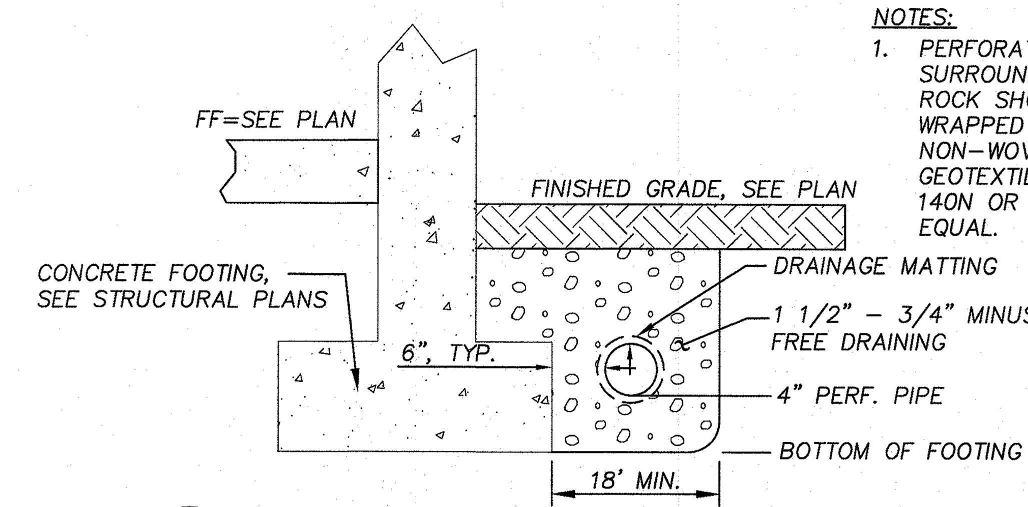

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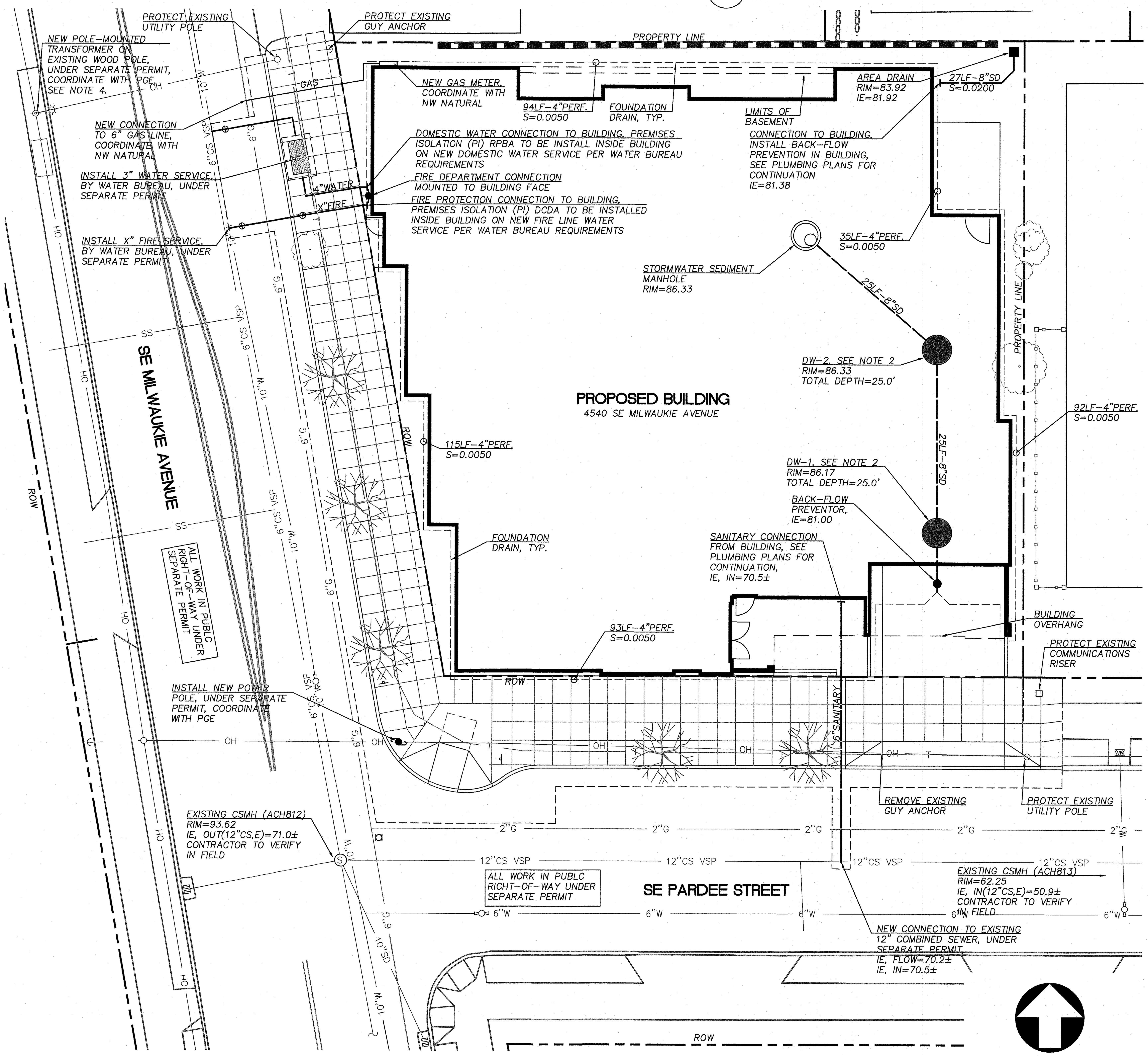

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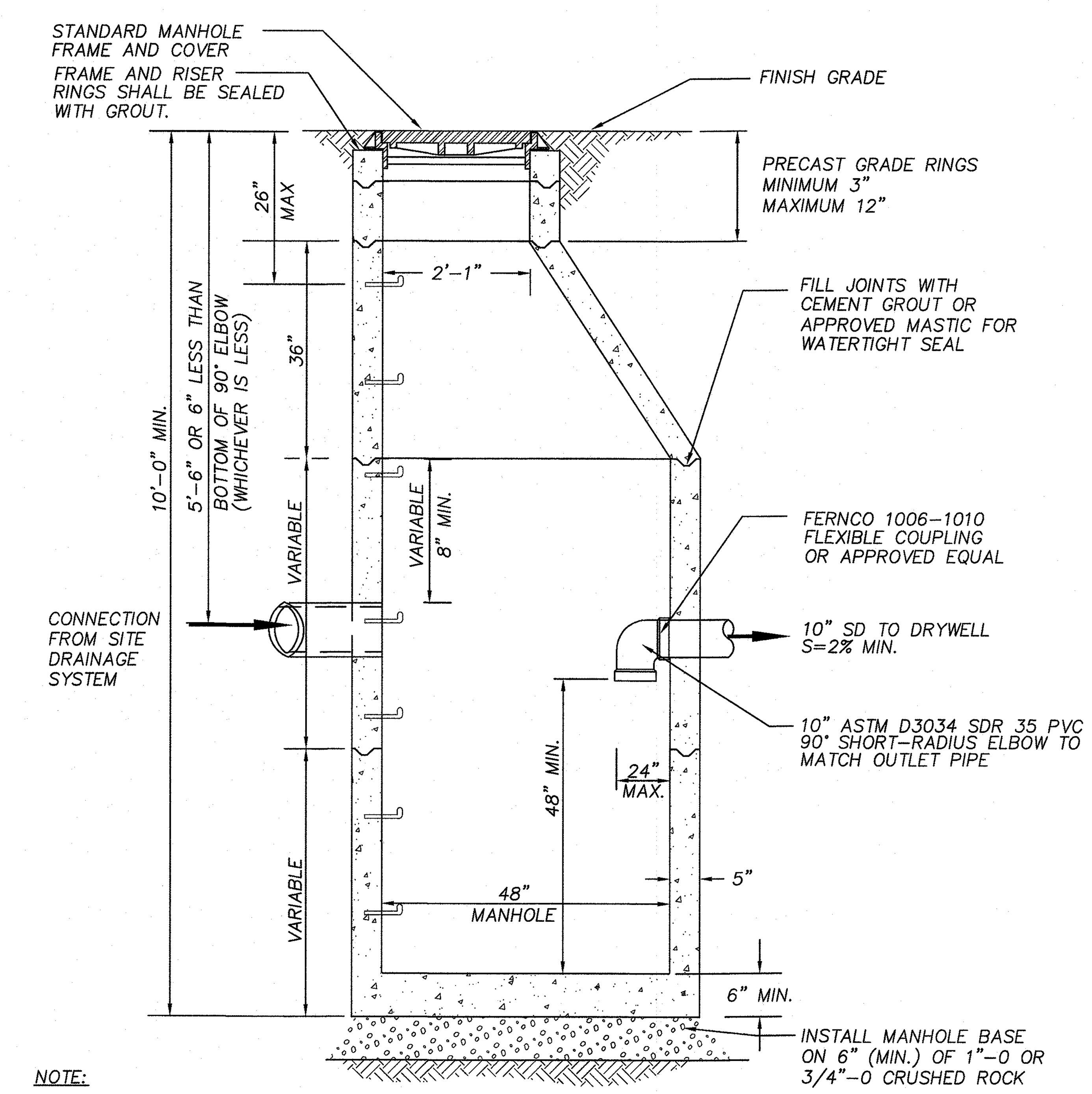

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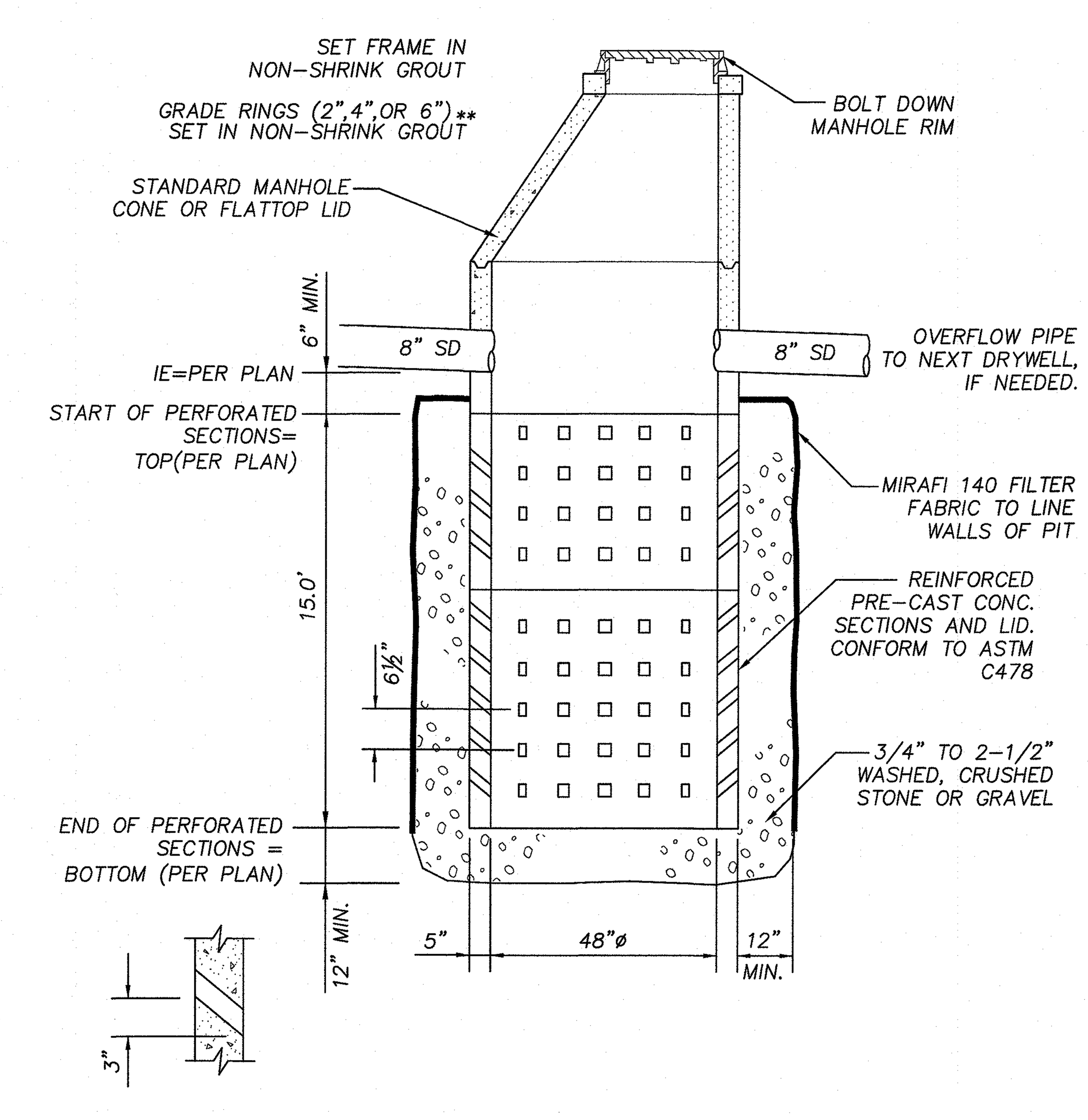

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Humber Design Group, Inc.
Portland, OR • 503.946.6690 • hdgpdx.com

REGISTERED PROFESSIONAL ENGINEER 76,071 OREGON JULY 12, 2005 MARTHA S. WILLIAMS
EXPIRES 6-30-2018

BARRY R SMITH, PC Architect
715 SW MORRISON STREET SUITE 909
PORTLAND OR 97205 503.295.6261 www.barryrsmith.com

4540 SE MILWAUKIE AVENUE
PORTLAND, OR
for YOSHIDA GROUP

PERMIT SET

UTILITY PLAN
C2.0
01.23.2017

HAYDEN ENGINEERS
STRUCTURAL | CIVIL

12480 SW 68th Ave.
Tigard, Oregon 97223
Phone: 503-968-9994
Fax: 503-968-8444

March 10, 2017

Barry R. Smith, PC Architect
715 SW Morrison St. Suite 909
Portland, OR 97205

Re: SE Milwaukie Ave. Apartments – Dry Wells
4524 and 4540 SE Milwaukie Ave.
Portland, Oregon

HCE Project #16464
Permit #: 17-111682-000-00-CO

Dear Barry,

This letter is in regards to the dry wells and foundations. Based upon the geotechnical report from Hardman Geotechnical Services Inc. dated July 18, 2016 and the supplemental letter dated March 9, 2017 it is our understanding that the dry wells can be placed 5 feet away from structural foundations without adverse effects.

Dry wells and foundation elements will be coordinated so that they have a minimum 5 feet clearance between one another. Based upon the geotechnical recommendations this clearance would prevent any adverse impacts to the foundation support.

If you have any questions, please do not hesitate to call.

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
18,525
OREGON
JULY 10, 1996
DARRON R. HAYDEN
EXP: 6/30/ **17**

Sincerely,

Hayden Consulting Engineers, Inc.

By: [signature]
Curtis McFeron
Project Manager

By: [signature]
Darron R. Hayden, P.E., S.E.
Principal

HARDMAN GEOTECHNICAL SERVICES INC.
Practical, Cost-Effective Geotechnical Solutions

July 18, 2016
HGSI Project No. 16-2049

Marty Treece
Treece and Lambert, LLC
2905 SW 1st Avenue
Portland, Oregon 97201

Via email (pdf format); hard copies provided on request

Subject: **GEOTECHNICAL ENGINEERING REPORT**
4524 AND 4540 SE MILWAUKIE AVENUE
PORTLAND, OREGON

Hardman Geotechnical Services Inc. (HGSI) performed a geotechnical engineering study for the proposed new residential project at 4524 and 4540 SE Milwaukie Avenue in the City of Portland, Oregon (see Vicinity Map, Figure 1). The purpose of the geotechnical study was to explore and evaluate the surface and subsurface conditions at the site, and to provide geotechnical recommendations for foundation design and site development.

PROJECT DESCRIPTION

The site is located northeast of the intersection of SE Milwaukie Avenue and SE Pardee Street in Portland, Oregon. Comprised of two tax lots, the subject site totals about 0.25 acres (11,065 square feet). The address of the northernmost property is 4524 SE Milwaukie Avenue, an existing home on that property was reportedly constructed in 1923. The southern property, 4540 SE Milwaukie Avenue, is occupied by a metal commercial/shop building reportedly constructed in 1954. The back of the property at 4540 SE Milwaukie Avenue has stored materials including building supplies, a travel trailer, etc. The number of and location of exploratory borings was limited by the presence of existing buildings and stored material.

Topography at the site slopes gently to moderately down to the east. The portlandmaps website indicates portions of the site have slopes steeper than 25%, primarily along the eastern margin of the property. There is an existing concrete wall along the east side of 4540 SE Milwaukie Avenue that elevates the storage yard area of that property up to about 4 feet above the neighboring property to the east.

We understand the site is to be developed as a residential apartment building or buildings, up to 5 stories in height. The ground floor will be at grade, with no substantial basement excavations planned. Some minor excavations or fills may be needed on site due to the sloping topography. We anticipate the existing wall along the east side of the site will be replaced, but at this time the replacement wall type, height and other features are not defined.

The project will also include underground utilities and stormwater disposal facilities, with on-site infiltration if feasible. Design of stormwater facilities will be performed by others. The site and grading plans have not yet been finalized. At present, specific wall and column locations, and structural loading, are being determined. HGSI should review the structural configurations and loads during the design process, and update the recommendations of this report as necessary based on specific design details.

SCOPE OF WORK AND AUTHORIZATION

Our scope of work for the project consisted of site reconnaissance, exploratory drilling, infiltration testing, geotechnical analyses, and preparation of this report. This geotechnical study was performed in accordance with HGSI Proposal No. 16-621, dated June 21, 2016, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

REGIONAL GEOLOGY AND SEISMIC SETTING

The subject site lies within the Portland Basin, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. The Portland Basin is a northwest-southwest trending structural basin produced by broad regional downwarping of the area. The Portland Basin is approximately 20 miles wide and 45 miles long and is filled with consolidated and unconsolidated sedimentary rocks of late Miocene, Pliocene and Pleistocene age.

The subject site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley, the last of which occurred about 10,000 years ago (Madin, 1990). Underlying the project site, these deposits consist of horizontally layered, micaceous, silt to fine sand.

At least three major fault zones capable of generating damaging earthquakes are known to exist in the region. These include the Portland Hills Fault Zone, Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone. These potential earthquake source zones are included in the determination of seismic design values for structures, as presented in the *Seismic Design* section. None of the known faults extend beneath the site.

FIELD EXPLORATION

EXPLORATORY BORINGS

The site-specific exploration for this study consisted of exploratory borings. On July 5, 2016, two borings, designated B-1 and B-2, were drilled to depths of approximately 21.5 feet at approximate locations shown on Figure 2. It should be noted that exploration locations were determined in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

The boreholes were drilled using a trailer mounted drill rig and solid stem auger methods. At each boring location, SPT (Standard Penetration Test) sampling was performed in general accordance with ASTM D1586 using a 2-inch outside diameter split-spoon sampler and a 140-pound hammer equipped with a rope and cathead mechanism. During the test, a sample is obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows for each 6 inches of penetration is recorded. The Standard Penetration Resistance ("N-value") of the soil is calculated as the number of blows required for the final 12 inches of penetration. If 50 or more blows are recorded within a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the number of inches driven. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. At the completion of the borings, the holes were backfilled with bentonite.

Explorations were conducted under the full-time observation of HGSI personnel. Soil samples were classified in the field and representative portions were placed in relatively air-tight plastic bags. These soil samples were then returned to the laboratory for further examination and laboratory testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in general accordance with the Unified Soil Classification System.

Summary boring logs are attached. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types. The actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

INFILTRATION TESTING

On July 5, 2016, HGSI performed open-hole falling head infiltration tests in borings B-1. The test borings were pre-saturated a minimum of 2 hours prior to testing due to the coarse grained nature of site soils. Following the soil saturation, infiltration tests were conducted. The water level was measured to the nearest 0.1 inch from a fixed point. The change in water level was recorded at intervals for a total period of at least 2 hours. Table 1 presents the results of the falling head infiltration tests.

Table 1. Summary of Infiltration Test Results

Boring	Depth (feet)	Soil Type	Infiltration Rate(in/hr)	Hydraulic Head Range (inches)
B-1	21	Silty fine Sand	9	86 - 82

SUBSURFACE CONDITIONS

The following discussion is a summary of subsurface conditions encountered in our explorations. For more detailed information regarding subsurface conditions at specific exploration locations, refer to the attached boring logs. Also, please note that subsurface conditions can vary between exploration locations, as discussed in the *Uncertainty and Limitations* section below.

SOIL

Results of the exploration program indicate that the site is underlain by silts and sands belonging to the Willamette Formation. The observed conditions and soil properties are summarized below.

> ***Silt:*** Beneath the 2 inches of gravel in borings B-1 and B-2 we encountered loose to medium stiff, moist silt. This silt was interpreted as belonging to the Willamette Formation and extended to roughly 5 to 8 feet in borings B-1 and B-2 respectively.
>
> ***Silty fine sand to sandy silt:*** Beneath the silt in boring B-1 and B-2, silty fine sand to sandy wilt was encountered. Consistency was generally medium dense / medium stiff to stiff, and slightly moist to moist. Soils appeared to coarsen with depth and extended to the termination of the borings at 21.5 feet. This material is interpreted as belonging to the Willamette Formation.

GROUNDWATER

At the time of our explorations, groundwater was not encountered beneath the site. Regional geologic mapping (Snyder, 2008) indicates that static groundwater is present at a depth of about 40 feet below the existing ground surface at the site. In our experience, it is not uncommon to encounter thin perched groundwater zones within the Willamette Formation in this area, particularly during the wet season.

The groundwater conditions reported above are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. Furthermore, it is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in land use and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Results of this study indicate that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. Spread footings are acceptable for use on this project. Additional discussion and recommendations are presented below regarding site preparation and undocumented fill removal, engineered fill, wet weather earthwork, spread footing foundations, concrete slabs-on-grade, perimeter footing drains, below-grade walls, seismic design, storm water infiltration systems, temporary excavations, utility trench backfill, and erosion control.

SITE PREPARATION AND UNDOCUMENTED FILL REMOVAL

Proposed structure and other areas of proposed improvements should be cleared of debris. Where encountered, undocumented fill within the proposed building footprint, beneath pavements or other settlement-sensitive improvements, should be completely removed and replaced with engineered fill.

Following removal of surficial debris and undocumented fill, the exposed subgrade should be evaluated by HGSI. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded dump truck. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with engineered fill, as described below. The depth of overexcavation, if required, should be evaluated by HGSI at the time of construction.

Based on the site location, it is possible that one or more old dry wells, septic systems or other below-grade structures may be present on site. In the event that old drywell(s) are encountered during site development, the following recommendations are made. Deeper portions of dry wells should be backfilled with controlled density fill (CDF), which is essentially a lean mix concrete consisting of water, sand and cement. We recommend use of “excavatable” CDF so that future excavations can be made through the dry well backfill if any new utilities or other excavations are needed in the affected areas. Above a depth of about 8 feet, at the contractor’s option, backfill may consist of granular soils such as “reject rock,” recycled concrete or similar material approved by HGSI. The granular backfill should be placed in lifts no thicker than about 18 inches and compacted with a “hoe-pac” excavator attachment to a minimum of 90 percent of Modified Proctor (ASTM D-1557). This backfill specification should also be used for any basements or other depressions that require fill during the demolition process.

ENGINEERED FILL

On-site native soils are considered suitable for use as engineered fill in dry weather conditions, provided they are relatively free of organics and are properly moisture conditioned for compaction. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 2 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill and crushed rock backfill soils should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. On-site soils may be wet of optimum; therefore, we anticipate that aeration of native soil will be necessary for compaction operations performed during late spring to early summer.

Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Field density testing should conform to ASTM D2922 and D3017, or D1556. Engineered fill should be periodically observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing.

WET WEATHER EARTHWORK

The on-site soils are moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact fill to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, HGSI should be contacted for additional recommendations.

Under wet weather, the construction area will unavoidably become wet and the condition of exposed fill and native soils will degrade. To limit the impacts of wet weather on the finished building pad surface, consideration may be given to placement of a crushed aggregate pad. Where used, we recommend the working pad be constructed using 1½"–0 crushed aggregate, and should have minimum thickness of at least 12 inches. This thickness is considered adequate to support light construction traffic, but will not be sufficient to support heavy traffic such as loaded dump trucks or other heavy rubber-tired equipment.

SPREAD FOOTING FOUNDATIONS

Spread footing foundations are acceptable for use on this project. Due to the presence of soft/medium stiff soils at anticipated foundation grades, we recommend placement of a minimum of 12 inches of crushed rock beneath structural foundations. With 12 inches of compacted crushed rock beneath footings, we recommend a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for use in design. The recommended maximum allowable bearing pressure may be increased by a factor of 1.33 for short term transient conditions such as wind and seismic loading.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of spread foundations of less than about 1 inch and differential settlement between two adjacent load-bearing components supported on competent soil of less than about ½ inch. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0.5 may be assumed along the interface between the base of the footing and subgrade soils. Passive earth pressure for buried portions of structures may be calculated using an equivalent fluid weight of 390 pounds per cubic foot (pcf), assuming footings are cast against dense, natural soils or engineered fill. The recommended coefficient of friction and passive earth pressure values do not include a safety factor. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. Loose, wet or otherwise softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. HGSI should observe foundation excavations prior to placing crushed rock, to verify that adequate bearing soils have been reached. HGSI should monitor crushed rock placement beneath foundations and perform density tests to verify compliance with the engineered fill density specification.

CONCRETE SLABS-ON-GRADE

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation* section. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content, and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 200 kcf (115 pci) should be assumed for the soils anticipated at subgrade depth. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of crushed rock of 8 inches beneath the slab.

Interior slab-on-grade floors should be provided with an adequate moisture break. The capillary break material should consist of ODOT open graded aggregate per ODOT Standard Specifications 02630-2. The minimum recommended thickness of capillary break materials on re-compacted soil subgrade is 8 inches. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction, and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 90% of its maximum dry density as determined by ASTM D1557 or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. With this type of system, an approximately 2-inch thick layer of sand is often placed over the vapor barrier to protect it from damage, to aid in curing of the concrete, and also to help prevent cement from bleeding down into the underlying capillary break materials. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside HGSI's area of expertise.

PERIMETER FOOTING DRAINS

To minimize soil moisture fluctuations adjacent to the building, we recommend the outside edge of perimeter footings be provided with a drainage system consisting of 3-inch minimum diameter perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, crushed drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Water collected from the footing drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. The footing drains should include clean-outs to allow periodic maintenance and inspection.

Down spouts and roof drains should collect roof water in a system separate from the footing drains in order to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

PERMANENT BELOW-GRADE WALLS

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-reset equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 5.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 390 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and HGSI should be contacted for additional recommendations.

A coefficient of friction of 0.5 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a minimum 12-inch wide zone of sand and gravel containing less than 5 percent fines against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

HGSI should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

SEISMIC DESIGN

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2012 International Building Code (IBC) with applicable 2014 Oregon Structural Specialty Code (OSSC) revisions. We recommend Site Class D be used for design per the OSSC, which references ASCE 7-10, Chapter 20, Table 20.3-1. Design values determined for the site using the USGS (United States Geological Survey) *Earthquake Ground Motion Parameters* utility are summarized on Table 2.

Table 2. Recommended Earthquake Ground Motion Parameters (2012 IBC / 2014 OSSC)

Parameter	Value
Location (Lat, Long), degrees	45.4899, -122.6510
Mapped Spectral Acceleration Values (MCE, Site Class B):	
Short Period, S_s	0.983 g
1.0 Sec Period, S_1	0.421 g
Soil Factors for Site Class D:	
F_a	1.107
F_v	1.579
$SD_s = 2/3 \times F_a \times S_s$	0.726 g
$SD_1 = 2/3 \times F_v \times S_1$	0.443 g

Potential seismic impacts also include secondary effects such as soil liquefaction, fault rupture potential, and other hazards as discussed below:

- **Soil Liquefaction Potential –** Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. The permanent ground water table is approximately 40 feet below the site (Snyder, 2008). Therefore, soils under the project site are not considered susceptible to liquefaction. It is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.
- **Fault Rupture Potential** – Based on our review of available geologic literature, we are not aware of any mapped active (demonstrating movement in the last 10,000 years) faults on the site. During our field investigation, we did not observe any evidence of surface rupture or recent faulting. Therefore, we conclude that the potential for fault rupture on site is low.
- **Seismic Induced Landslide** – Topography in the vicinity of the subject site is generally flat to gently sloping. The potential for slope instability and seismic induced landslide on site is considered very low.
- **Effects of Local Geology and Topography** – In our opinion, no additional seismic hazard will occur due to local geology or topography. The site is expected to have no greater seismic hazard than surrounding properties and the Portland area in general.

STORMWATER INFILTRATION SYSTEMS

Based on the results of the infiltration testing, deep soils on site exhibit moderate infiltration rates. Groundwater was not encountered in borings advanced to a maximum depth of 21.5 feet. No indications of seasonal high groundwater were observed in the borings. Based on the USGS mapping we anticipated seasonal high groundwater to be about 40 feet below the ground surface (Snyder, 2008).

In-situ infiltration tests were conducted to assess the infiltration capacity of deep soils on site for the use of a drywell. Design of stormwater infiltration facilities will be performed by others. The approximate location of the test is shown on Figure 2, and the test methodology is discussed above in the *Infiltration Testing* section, above. Table 1 summarizes results of the infiltration testing.

Near-surface soils were not tested due to the clients need for a drywell and the shallow depth to sand. At a depth of 21 feet in B-1, the infiltration test result was 9 inches/hour. A value of 9 in/hr may be used for design of deep stormwater facilities, such as dry wells, that extend to depths of at least 13 feet below ground surface. The infiltration rates presented herein do not incorporate a factor of safety. For the design infiltration rate, the system designer should incorporate an appropriate factor of safety against slowing of the rate over time due to biological and sediment clogging.

In the event that dry wells are located near the building's exterior wall, manholes should be designed and constructed in a manner which will limit disturbance of surrounding soils, and prevent the loss of soil from the surrounding area into the manhole. We recommend use of geotextile fabric between the manhole and any surrounding drain rock, and the native soils. Annular space between the manhole and surrounding soil should be backfilled tightly with compacted crushed rock or drain rock.

Dry wells should be located at least 5 feet from any structural foundations. The potential for soil saturation has been accounted for in development of the allowable soil bearing pressures used in design. Therefore, soil saturation from dry well operation is not anticipated to cause significant adverse impacts on foundation support.

Infiltration test methods and procedures attempt to simulate the as-built conditions of the planned disposal system. However, due to natural variations in soil properties, actual infiltration rates may vary from the measured and/or recommended design rates. All systems should be constructed such that potential overflow is discharged in a controlled manner away from structures, and all systems should include an adequate factor of safety. Infiltration rates presented in this report should not be applied to inappropriate or complex hydrological models such as a closed basin without extensive further studies.

TEMPORARY EXCAVATIONS

Site soils consist of medium stiff to stiff native silt, to fine sandy silt and silty fine sand, as described above. At this time the location and finish floor elevations of the new structure are not finalized. Depending on final structural configuration, it may be necessary to provide mechanical shoring system(s) along portions of the north, and east property lines to protect neighboring structures and properties. HGSI should be consulted to provide specific shoring recommendations if needed once the design details are better known.

Where space is available we recommend temporary excavation slopes of 1H:1V (Horizontal:Vertical). Where necessary, temporary excavations up to 1H:2V may be made, or the temporary excavation may consist of a 1H:1V slope with a maximum 4-foot-high vertical cut at the toe.

The temporary excavation slopes recommended herein are anticipated to have an adequate factor of safety considering overall (gross) failure, during the anticipate time span the temporary cut will be open, about 3 to 4 weeks. Some surficial erosion or sloughing may occur on the slope face. Surface water should not be allowed to pond above the temporary cut, nor should surface water be allowed to flow down the slope face. Consideration should be given to covering the temporary cut face with plastic sheeting in the event of rainy weather in the forecast. It is our opinion that there is a low potential for the planned excavation to impact or damage the existing driveway and home on the adjacent property.

HGSI's responsibility for temporary excavation stability includes only the evaluation summarized herein. HGSI is not responsible for any aspect of jobsite safety. The contractor is responsible to designate a "responsible person" for monitoring of temporary excavations on site. We suggest that the tops of temporary excavation slopes be observed at least once daily for indications of movement such as ground cracks. If cracking or other indications of slope movement are observed, work should be halted and HGSI contacted immediately for evaluation of the situation.

UTILITY TRENCH EXCAVATION AND BACKFILL

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that structural trench backfill be compacted to at least 90 percent of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a 3/4"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

EROSION CONTROL CONSIDERATIONS

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw, bio-bags, silt fences, or other appropriate technology. Where used, erosion control devices should be in place and remain in place throughout site preparation and construction. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HGSI should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, HGSI executed these services in accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

HARDMAN GEOTECHNICAL SERVICES INC.

REGISTERED PROFESSIONAL ENGINEER
64,422PE
OREGON
SEPTEMBER 26, 2000
SCOTT L. HARDMAN

7-18-16

EXPIRES: 06-30-2017

Scott L. Hardman, P.E., G.E.
Geotechnical Engineer

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site Plan
Logs of Borings B-1 and B-2

REFERENCES

Beeson, M.H., Tolan, T.L., and Madin, I.P., 1991, Geologic map of the Portland Quadrangle, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries Geological Map Series GMS-75, scale 1:24,000.

Madin, I.P., 1990, Earthquake hazard geology maps of the Portland metropolitan area, Oregon: Oregon Department of Geology and Mineral Industries Open-File Report 0-90-2, scale 1:24,000, 22 p.

Snyder, D.T., 2008, Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon Area: U.S. Geological Survey Scientific Investigations Report 2008–5059, 41 p., 3 plates.

HARDMAN
GEOTECHNICAL
SERVICES INC.

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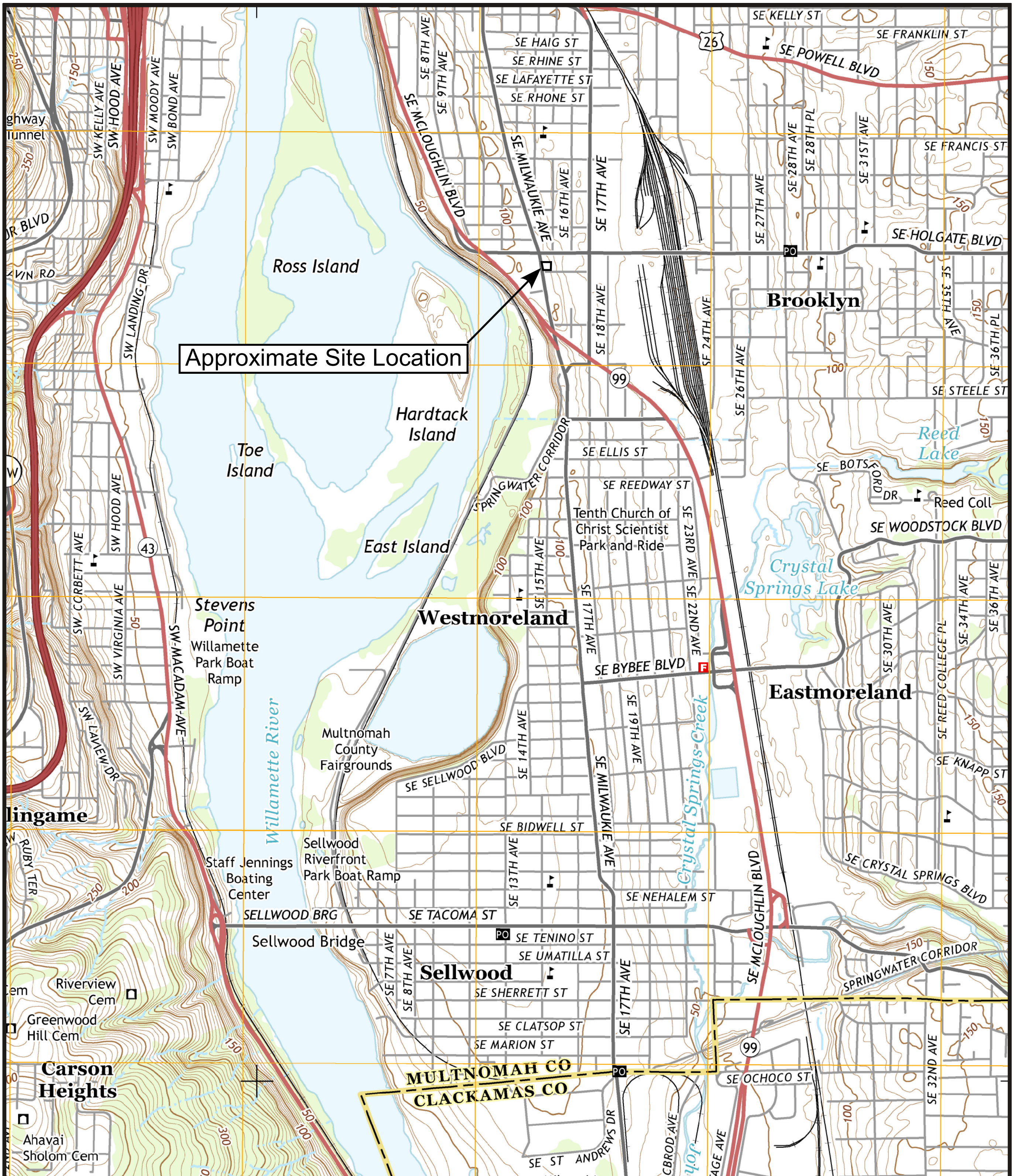
VICINITY MAP



Project: SE Milwaukie Ave Portland, Oregon	Project No. 16-2049	FIGURE 1

HARDMAN GEOTECHNICAL SERVICES INC.

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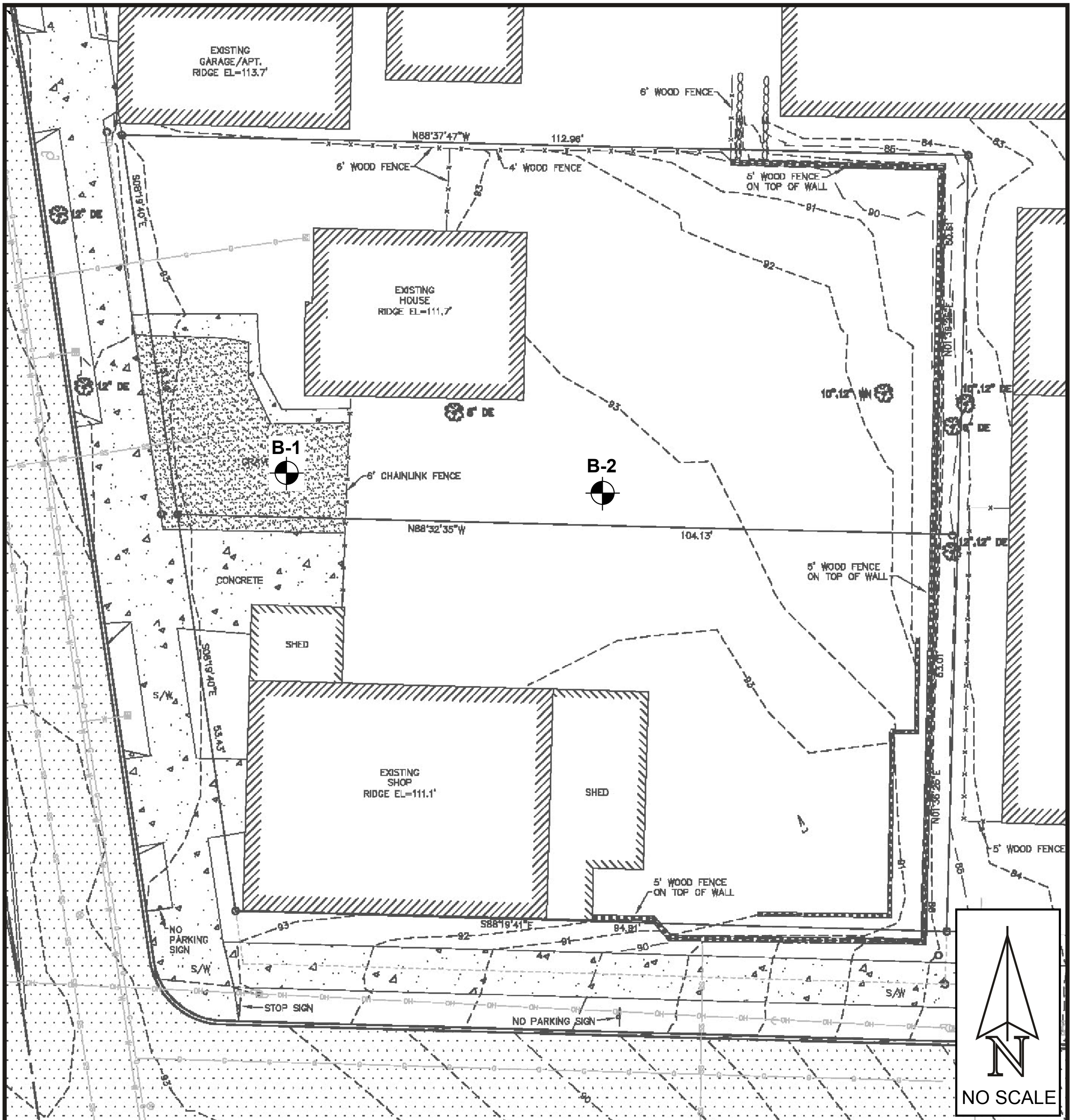
SITE AND EXPLORATION PLAN



Base map prepared by Summit Land Surveyors

Legend

B-2 Boring Designation and Approximate Location

Project SE Milwaukie Ave Portland, Oregon	Project No. 16-2049	FIGURE 2

BORING LOG

Project: SE Milwaukie Avenue, Portland, Oregon	Project No. 16-2049	Boring No. B-1

Depth (ft)	Sample Interval	SPT N-Value (Blows/foot)	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Groundwater	Material Description
						2 inches of gravel
						Medium stiff, fine sandy Silt, brown, slightly moist
	☒	4				
5	☒	6				Loose to medium dense/medium stiff to stiff, silty fine sand to sandy silt, gray and brown, slightly moist to moist. Substantial root at 6 feet.
	☒	6				
10	☒	6				
15	☒	7				
20	☒	14				
						Boring terminated at 21.5 feet No groundwater or seepage encountered
25						

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Portland, Oregon 97223
(503) 530-8076

LEGEND

☒ STP Drive Sample

▽ Water Level at Time of Drilling

Date Drilled: 7-5-16

Logged By: IDM

BORING LOG

Project: SE Milwaukie Avenue, Portland, Oregon	Project No. 16-2049	Boring No. B-2

Depth (ft)	Sample Interval	SPT N-Value (Blows/foot)	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Groundwater	Material Description
						2 inches of gravel
						Soft to medium stiff, Silt, brown, moist
	☒	3				
5	☒	5				
	☒	8				
						Loose to medium dense/medium stiff to stiff, silty fine Sand to sandy silt, gray and brown, moist.
10	☒	8				
15	☒	7				
20	☒	15				
						Boring terminated at 21.5 feet No groundwater or seepage encountered
25						

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LEGEND

☒ STP Drive Sample

▽ Water Level at Time of Drilling

Date Drilled: 7-5-16

Logged By: IDM

Stormwater Management Facilities

Private Stormwater Report

4540 SE Milwaukie Apartments

HDG Job #: SMI001

Prepared For: Yoshida Group

8440 NE Alderwood Rd, Suite A
Portland, OR 97220

Prepared By: Humber Design Group, Inc.

117 SE Taylor St. Suite 001
Portland, OR 97214
(P) 503 946 6690

'I hereby certify that this Stormwater Management Report for the 4540 SE Milwaukie Apartments project has been prepared by me or under my supervision and meets minimum standards of The City of Portland and normal standards of engineering practice.

I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.'

Date: January 23, 2017

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	Project Overview and Description	2
	Vicinity Map	3
	Methodology	4
	Analysis	5
	Engineering Conclusions	6

APPENDICES

Appendix A	Stormwater Facility Details / Exhibits	A
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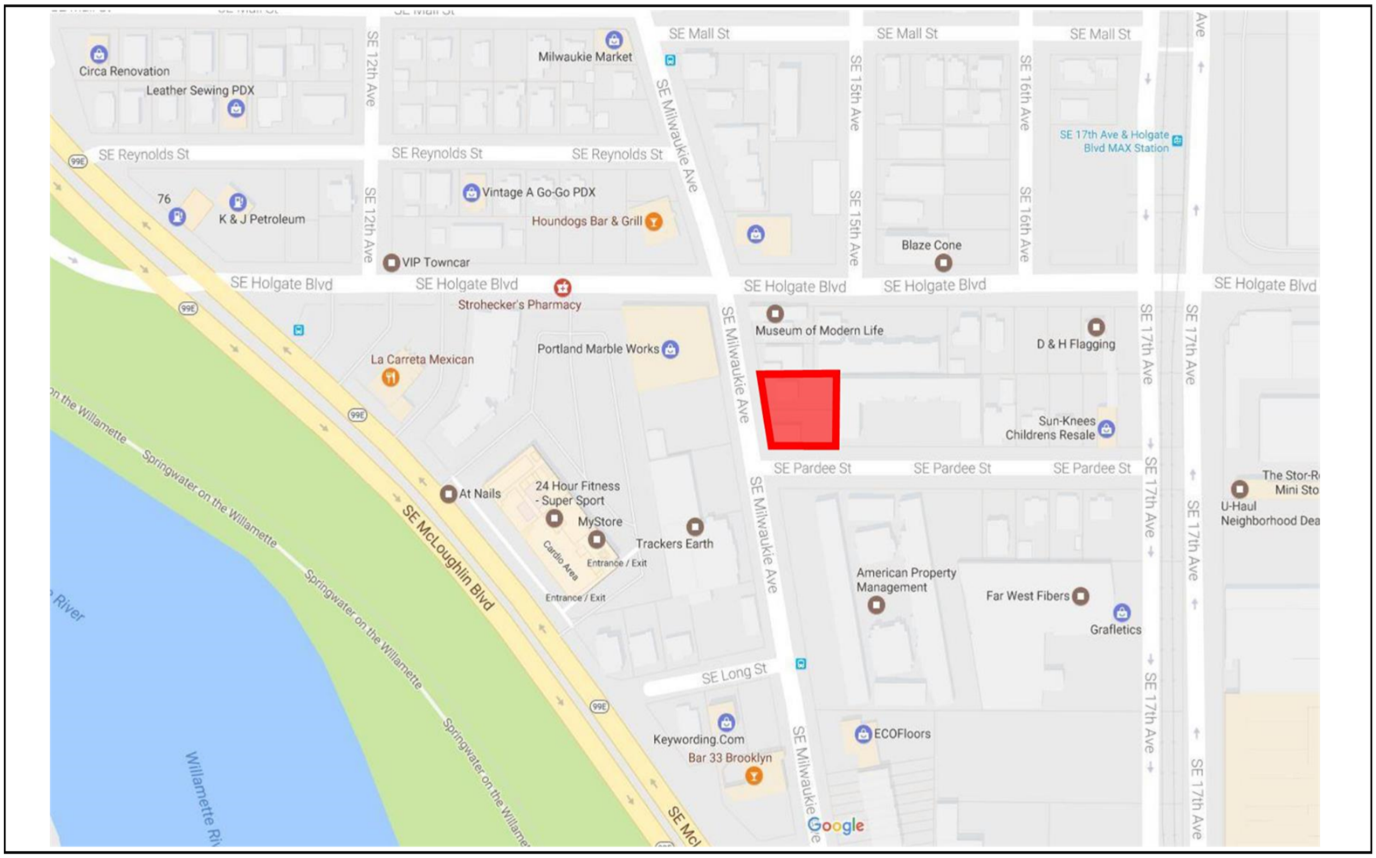
Appendix C	Operations and Maintenance Plan	C
Appendix D	Additional Forms & Associated Reports	D

Project Overview and Description

Location of Project	4540 SE Milwaukie Ave
Site Area/Acreage	0.25 ac (11,065 ac)
Nearest Cross Street	SE Milwaukie Ave, SE Pardee St
Property Zoning	CG- General Commercial
Existing Conditions	The site is currently occupied by two, single-story residential buildings with landscaping.
Proposed Development	The proposed 5-story building will have 40 apartment units, and a parking garage with 10 stalls on the ground floor.
Watershed Description	Willamette River
Subwatershed	Taggart
R#	R143025, R143024
Tax Map	1S 1E 14AB
Tax Lot	1800, 1700
Flood Zone	No
Permits Required	Building Permit Public Street Permit DEQ UIC Permit Plumbing Permit

Vicinity Map



Site Location

Methodology

Existing Drainage	The existing site currently drains to the public right-of-way to a catch basin, which connects to the 10" combined sewer in SE
Infiltration Results	The falling head infiltration test was performed by Hardman Geotechnical Services, Inc in accordance with the 2016 City of Portland SWMM guidelines. An infiltration rate of 9 inches per hour was recorded at 21 feet below grade surface.
PRIVATE Proposed Stormwater Management Techniques	Stormwater runoff from the private site will be managed with a private drywell system. Runoff from the site will be collected and piped to a sediment manhole, which will then release it to the drywell system. The sediment manhole and drywells will be located in the parking garage of the building.
PUBLIC Proposed Stormwater Management Techniques	Stormwater management is not required for the public street because the newly constructed impervious area does not include moving the exiting curb location.
Stormwater Hierarchy Justification	On-site infiltration with surface infiltration facility (Category 1) is not feasible due to proposed building covering the entire site. This project will fall under Category 2, on-site infiltration with a private drywell or soakage trench.

Analysis

Computational Method Used — HydroCAD models of a SBUH Type 1A Storm were used to calculate the stormwater management facility sizes for the catchment areas. See attached calculations. Below is a summary of the results.

Hydrologic Soil Types — Silty fine sand to sandy silt

Table 1 – Curve Numbers

Predeveloped Pervious CN	72
Predeveloped Impervious CN	98
Post-Developed Pervious CN	72
Post-Developed Impervious CN	98

Table 2 – Design Storms

WQ Storm	0.83 inches
2-year	2.40 inches
10-year	3.40 inches
25-year	3.90 inches
100-year	4.40 inches

Table 3 – Time of Concentration

Predeveloped TOC	5 min
Post-Developed TOC	5 min

Table 3– Catchment Areas and Facility Table

Catchment/ Facility ID	Source (roof, road, etc.)	Treatment Area (sf)	Ownership (private/ public)	Facility Type/ Function	Facility Size
A	Roof	9,530	Private	Drywell	2 x 48" dia. X 25' deep
B	Landscaping	1,970	Private	Drywell	2 x 48" dia. X 25' deep

Catchment/ Facility ID	WQ	25 Year
A	INF	INF

Engineering Conclusions

The preceding methodologies and calculations presented indicate compliance with the current jurisdictional stormwater management codes and requirements. A summarized breakdown is presented below:

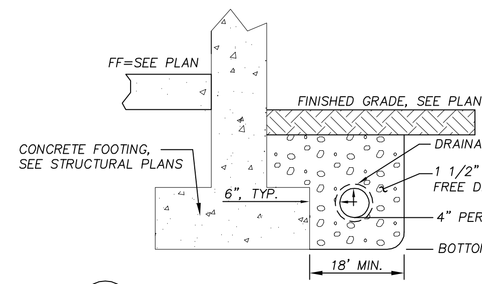
Water Quality	The proposed development will meet the provisions for water quality per the 2016 Portland Stormwater Management Manual.
Water Quantity	The proposed development will meet the provisions for water quantity per the 2016 Portland Stormwater Management Manual.
Downstream / Upstream Impacts	There are no upstream or downstream impacts created by this proposed development.
100 year storm	The 100 year storm will be safely conveyed away from structures and will overflow to the trapped sanitary drains to the combined sewer in SE Pardee St.

Appendix A

Stormwater Facility Details/Exhibits

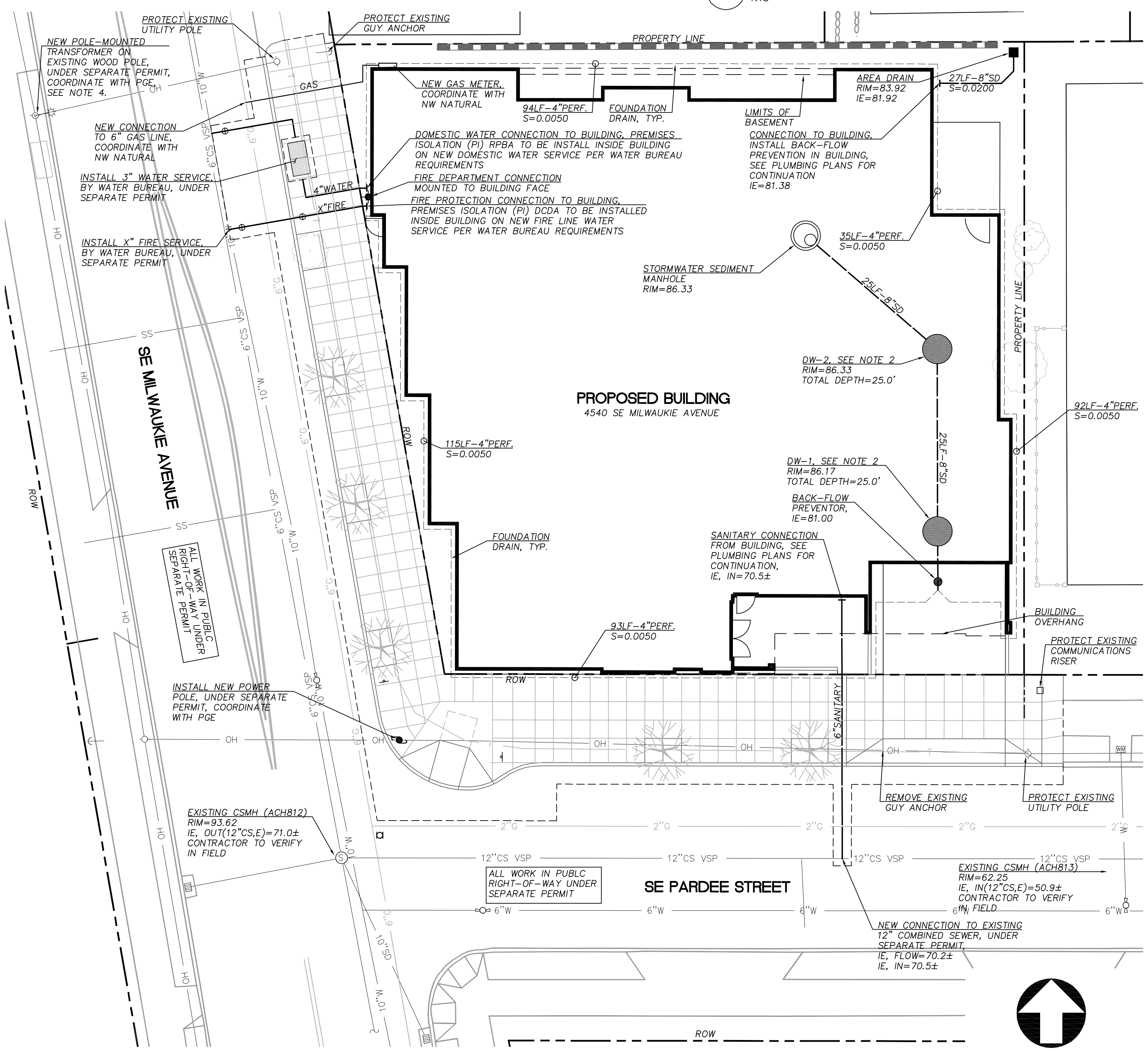
SHEET NOTES

1. ALL DOMESTIC WATER AND FIRE PROTECTION WORK IN THE PUBLIC RIGHT OF WAY BY PORTLAND WATER BUREAU AT OWNER'S EXPENSE. CONTRACTOR TO COORDINATE WORK WITH PORTLAND WATER BUREAU.
2. CONTRACTOR SHALL INSTALL AND TEST DRYWELL-1 (DW-1) PRIOR TO THE INSTALLATION OF DW-2. IF STORMWATER REQUIREMENTS ARE MET BY DW-1, DW-2 DOES NOT NEED TO BE INSTALLED.
3. ALL RUNOFF FROM ROOF AND COURTYARD ROUTED TO STORMWATER DRYWELLS IN PARKING GARAGE.
4. ELECTRICAL TRANSFORMER TO BE POLE-MOUNTED. COORDINATE WITH PGE, CONTACT CHARLES EVANS (503-258-7630).



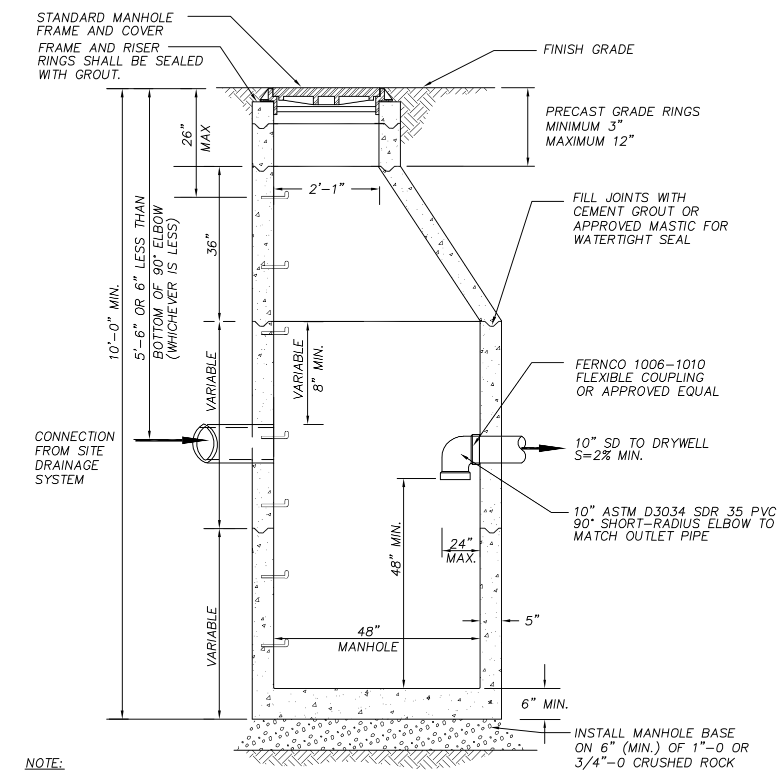
NOTES:
1. PERFORATED PIPE AND SURROUNDING DRAIN ROCK SHOULD BE WRAPPED IN NON-WOVEN GEOTEXTILE. MARAFI 140N OR APPROVED EQUAL.

3 FOUNDATION DRAIN
NTS



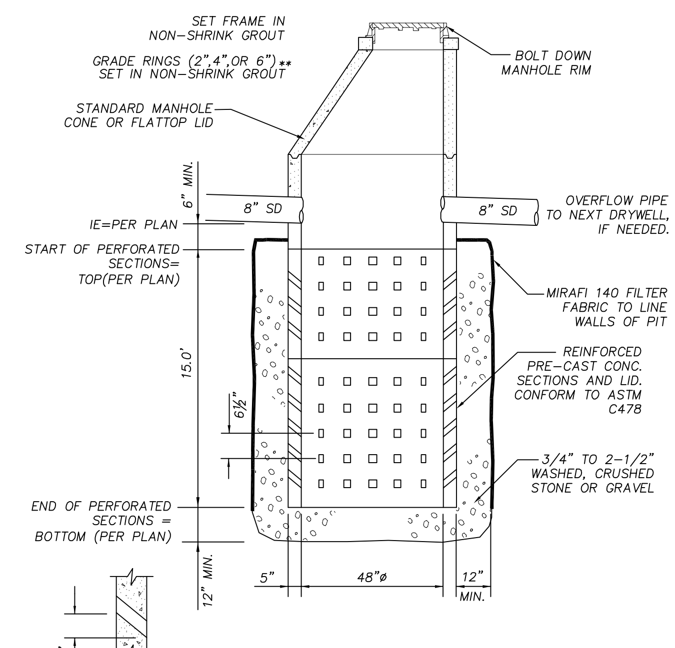
UTILITY PLAN
SCALE: 1"=10'

GRAPHIC SCALE
0 10 20
1 inch = 10 ft.



NOTE:
1. ALL PRECAST SECTIONS SHALL CONFORM TO THE REQUIREMENTS OF ASTM C478. SEE CITY OF GRESHAM STANDARDS AND SPECIFICATIONS FOR MH STEP DETAIL.
2. ALL CONNECTING PIPE SHALL HAVE A FLEXIBLE JOINT WITHIN 18" OF MANHOLE WALL.

2 SEDIMENTATION MANHOLE
NTS



1 DRYWELL
NTS

DRYWELL TESTING NOTES

1. DRYWELL SYSTEM SHALL HAVE THE CAPACITY TO DISPOSE OF STORMWATER AT A RATE OF 359GPM FOR 2 HOURS.
2. SHOULD DRYWELL(S) FAIL TO TEST AT THE MINIMUM DESIGN RATE ADDITIONAL DRYWELLS MAY BE CONSTRUCTED, AS APPROVED, IN ORDER TO ACHIEVE THE REQUIRED MINIMUM DISPOSAL RATE. ALTERNATIVE DRYWELL LOCATIONS AND/OR ADDITIONAL DRYWELLS IN PIPED SERIES MUST BE PRE-APPROVED AND IDENTIFIED ON THE CONSTRUCTION DRAWINGS.
3. INDICATED DRYWELL(S) (AS NOTED ABOVE IN NOTE#1) SHALL BE TESTED BY THE CONTRACTOR, AS DIRECTED AND APPROVED BY THE ENGINEER.
4. DRYWELL(S) SHALL BE TESTED AFTER CONSTRUCTION OF THE DRYWELL STRUCTURE (INCLUDING DRAIN ROCK AND PERIMETER BACKFILL) BUT PRIOR TO THE CONSTRUCTION OF THE TOP SLAB AND FINISH BACKFILL.
5. NOTIFY CIVIL AND GEOTECHNICAL ENGINEERS 5 DAYS PRIOR TO TESTING.
6. CONTRACTOR SHALL CONTACT CITY OF PORTLAND WATER BUREAU OR APPLICABLE WATER DISTRICT TO ARRANGE FOR DRYWELL TEST WATER SUPPLY. CONTRACTOR SHALL BEAR RESPONSIBILITY FOR SECURING ALL NECESSARY PERMITS, AUTHORIZATION AND ANY FEES.
7. CONTRACTOR SHALL ARRANGE FOR THE PROVISION OF ALL DRYWELL TESTING EQUIPMENT, INCLUDING BUT NOT LIMITED TO FLOW METER, PIPING, AND TRAFFIC CONTROL.
8. CLEAN WATER SHALL BE PROVIDED TO TEST DRYWELLS, AS APPROVED, INTRODUCTION OF SEDIMENT MAY RESULT IN FAILURE OF THE DRYWELL CAPACITY TEST.
9. PROVIDE CIVIL ENGINEER WITH RECORDED TEST DATA.

REGISTERED PROFESSIONAL ENGINEER 76,071 OREGON JULY 12, 2005 MARTHA S. WILLIAMS
EXPIRES 6-30-2018

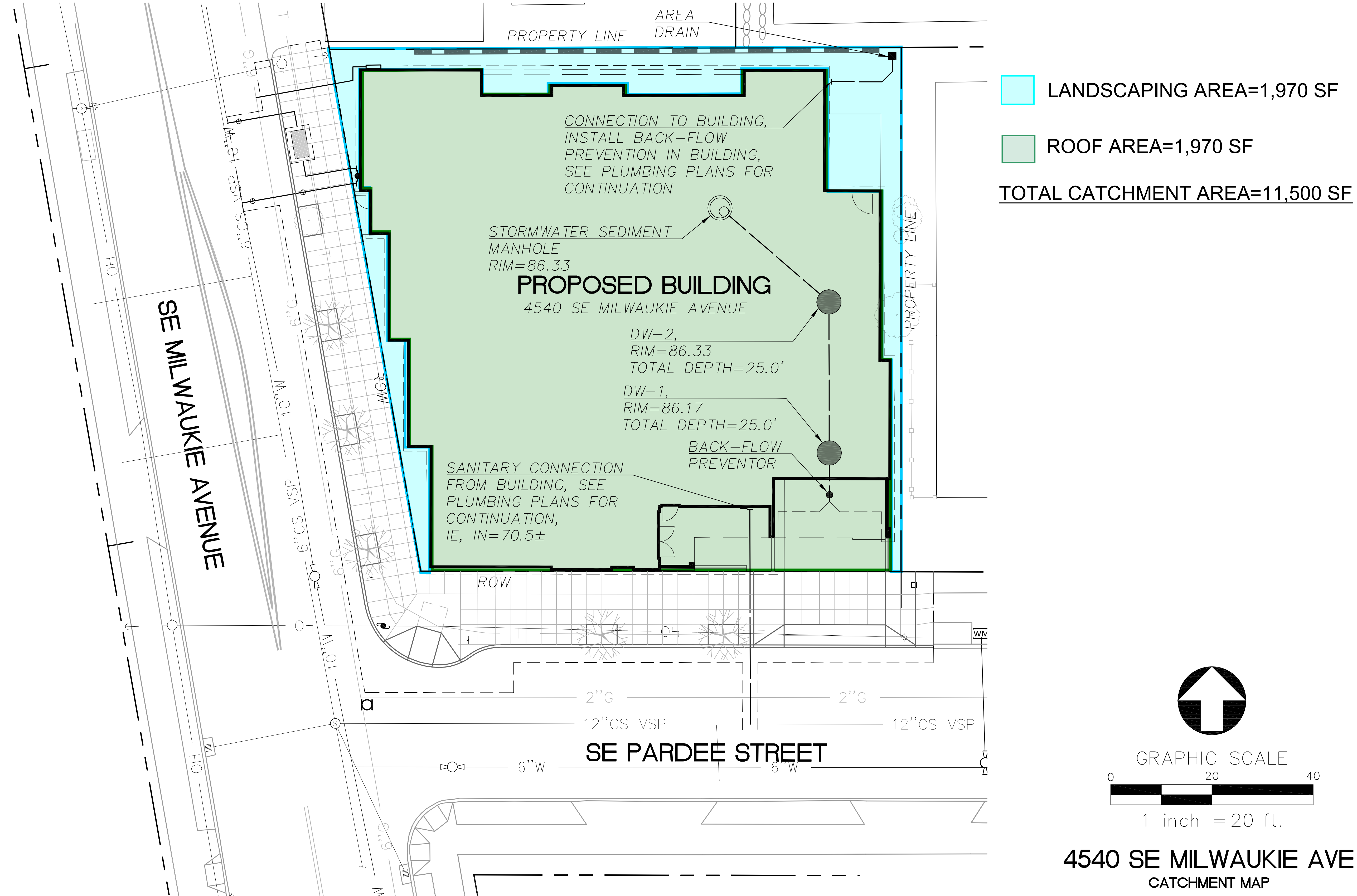
BARRY R SMITH, PC Architect
715 SW MORRISON STREET SUITE 909
PORTLAND OR 97205 503.295.6281 www.barryrsmith.com

4540 SE MILWAUKIE AVENUE
for YOSHIDA GROUP
PORTLAND, OR

PERMIT SET

Humber Design Group, Inc.
Portland, OR • 503.946.6690 • hdgpdx.com

UTILITY PLAN
C2.0
01.23.2017



CATCHMENT MAP

SCALE: 1"=20'

4540 SE MILWAUKIE AVE

CATCHMENT MAP

PROJECT NO.:	SMI001
DRAWN BY:	AFW
DESIGN BY:	AFW
REVIEWED BY:	MSW
DATE:	01/23/2017

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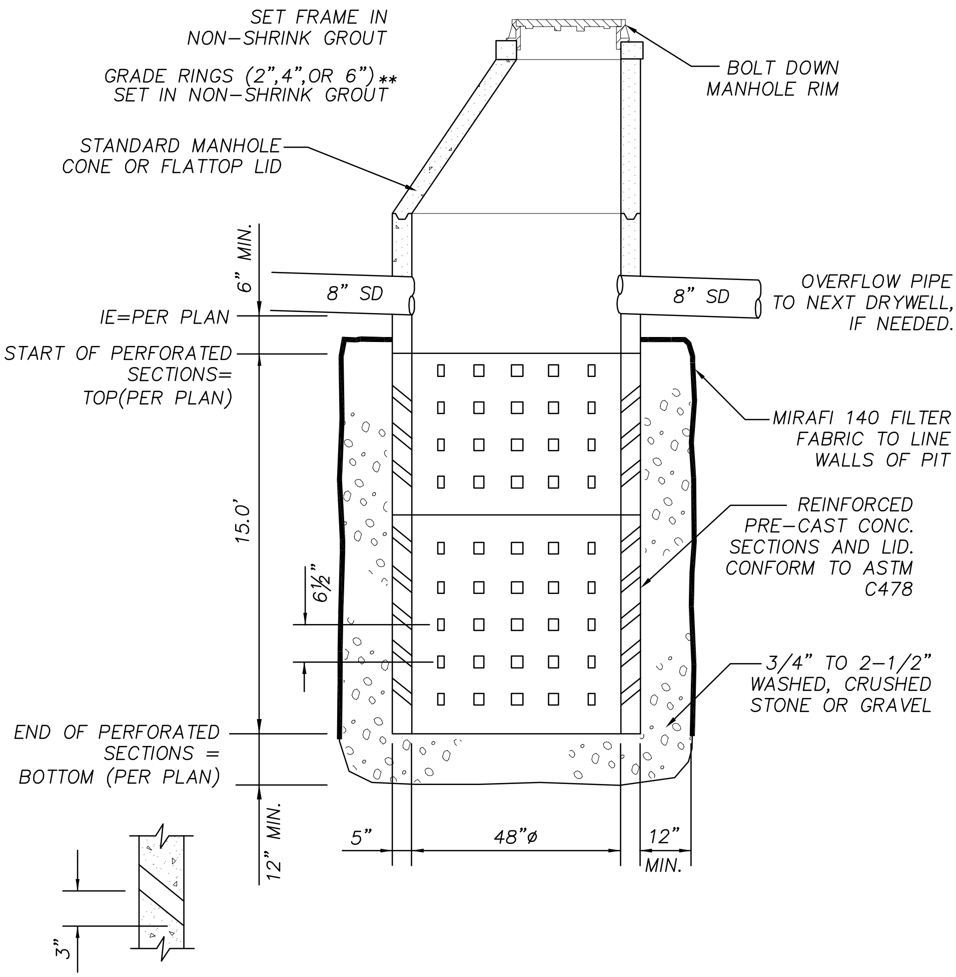
P:\SMI001 – 4540 SE Milwaukie\7 – CAD Drawings\SMI001-C3.00 Utilities.dwg Jan 24, 2017 – 1:48pm



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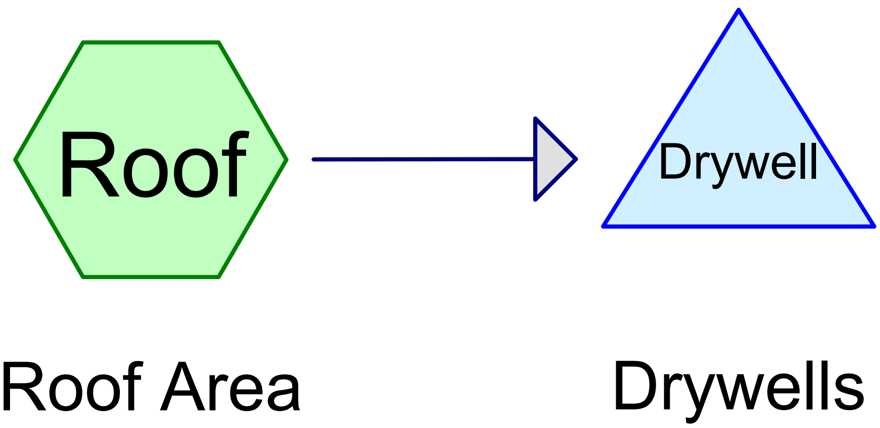
9. PROVIDE CIVIL ENGINEER WITH RECORDED TEST DATA.

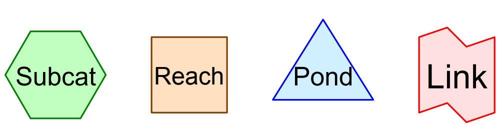
1 **DRYWELL**

NTS

Appendix B

Support Calculations





Routing Diagram for 4540 SE Milwaukie
Prepared by Humber Design Group, Inc., Printed 1/24/2017
HydroCAD® 10.00-15 s/n 09142 © 2015 HydroCAD Software Solutions LLC

Area Listing (selected nodes)

Area (sq-ft)	CN	Description (subcatchment-numbers)
11,500	98	(Roof)
11,500	**98**	**TOTAL AREA**

Soil Listing (selected nodes)

Area (sq-ft)	Soil Group	Subcatchment Numbers
0	HSG A	
0	HSG B	
0	HSG C	
0	HSG D	
11,500	Other	Roof
11,500		**TOTAL AREA**

Time span=0.00-30.00 hrs, dt=0.05 hrs, 601 points
Runoff by SBUH method, Split Pervious/Imperv.
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

SubcatchmentRoof: Roof Area Runoff Area=11,500 sf 100.00% Impervious Runoff Depth=3.17"
Tc=5.0 min CN=0/98 Runoff=0.21 cfs 3,035 cf

Pond Drywell: Drywells Peak Elev=115.45' Storage=622 cf Inflow=0.21 cfs 3,035 cf
Outflow=0.08 cfs 3,035 cf

Total Runoff Area = 11,500 sf Runoff Volume = 3,035 cf Average Runoff Depth = 3.17"
0.00% Pervious = 0 sf 100.00% Impervious = 11,500 sf

Summary for Subcatchment Roof: Roof Area

[49] Hint: Tc<2dt may require smaller dt

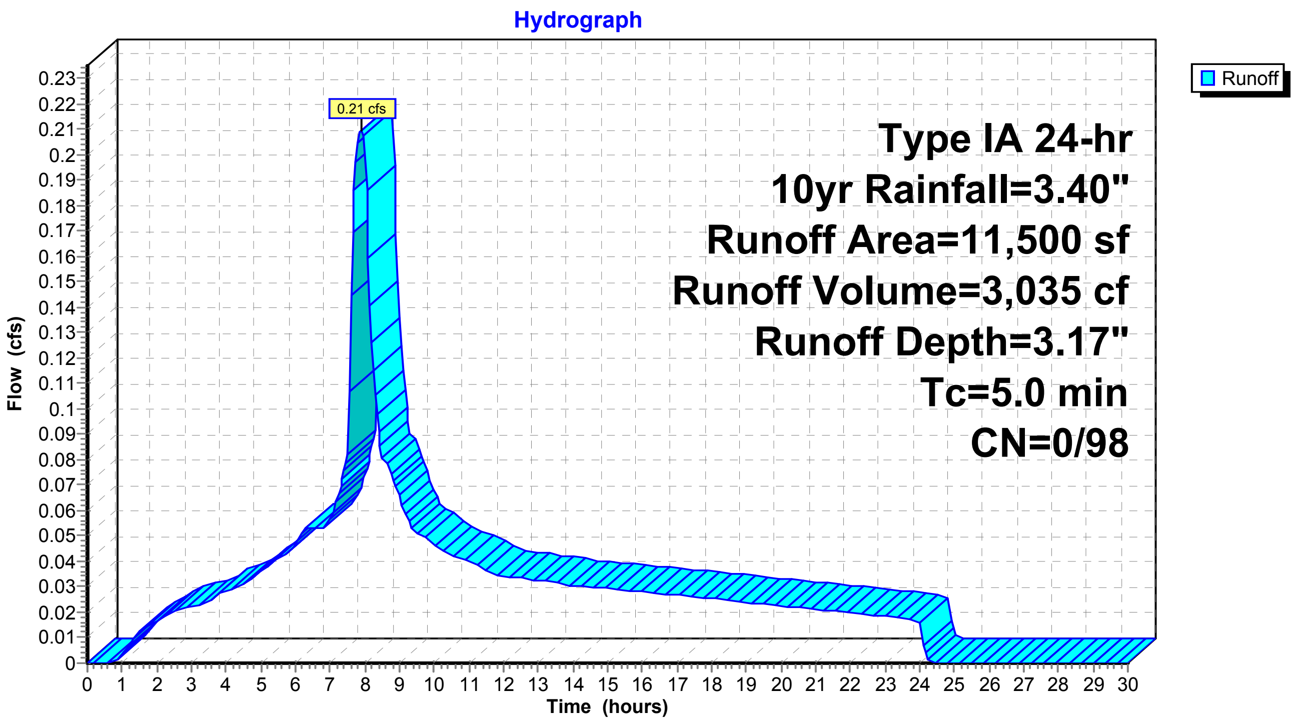
Runoff = 0.21 cfs @ 7.90 hrs, Volume= 3,035 cf, Depth= 3.17"

Runoff by SBUH method, Split Pervious/Imperv., Time Span= 0.00-30.00 hrs, dt= 0.05 hrs
Type IA 24-hr 10yr Rainfall=3.40"

	Area (sf)	CN	Description
*	11,500	98	
	11,500		100.00% Impervious Area

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

Subcatchment Roof: Roof Area



Summary for Pond Drywell: Drywells

Inflow Area = 11,500 sf,100.00% Impervious, Inflow Depth = 3.17" for 10yr event
Inflow = 0.21 cfs @ 7.90 hrs, Volume= 3,035 cf
Outflow = 0.08 cfs @ 8.35 hrs, Volume= 3,035 cf, Atten= 63%, Lag= 27.2 min
Discarded = 0.08 cfs @ 8.35 hrs, Volume= 3,035 cf

Routing by Stor-Ind method, Time Span= 0.00-30.00 hrs, dt= 0.05 hrs / 2
Peak Elev= 115.45' @ 8.72 hrs Surf.Area= 77 sf Storage= 622 cf

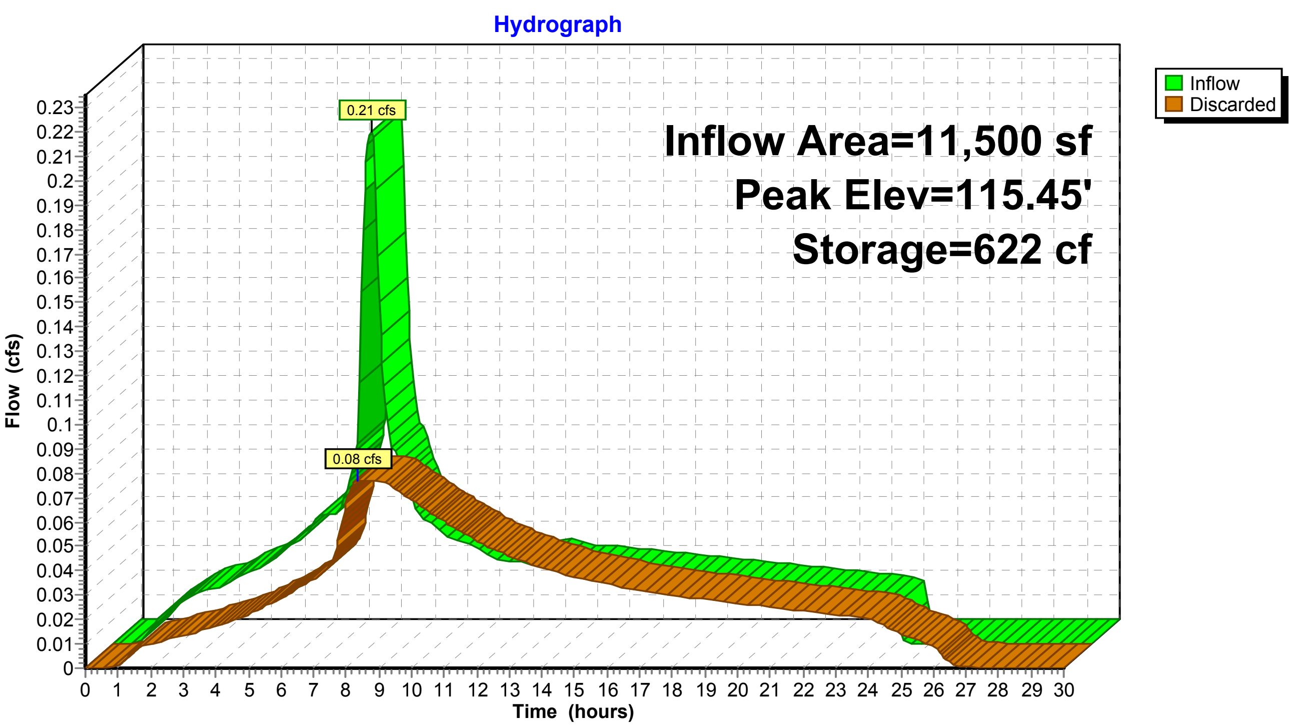
Plug-Flow detention time= 112.7 min calculated for 3,030 cf (100% of inflow)
Center-of-Mass det. time= 112.8 min (777.6 - 664.8)

Volume	Invert	Avail.Storage	Storage Description
#1	100.00'	377 cf	**4.00'D x 15.00'H Vertical Cone/Cylinder**x 2 Inside #2
#2	100.00'	233 cf	**7.00'D x 15.00'H Vertical Cone/Cylinder**x 2
			1,155 cf Overall - 377 cf Embedded = 778 cf x 30.0% Voids
#3	115.00'	251 cf	**4.00'D x 10.00'H Vertical Cone/Cylinder**x 2 -Impervious
		862 cf	Total Available Storage

Device	Routing	Invert	Outlet Devices
#1	Discarded	100.00'	**4.500 in/hr Exfiltration over Wetted area**

Discarded OutFlow Max=0.08 cfs @ 8.35 hrs HW=115.13' (Free Discharge)
└─**1=Exfiltration** (Exfiltration Controls 0.08 cfs)

Pond Drywell: Drywells



Appendix C

Operations & Maintenance Plan

Stormwater Management Facilities

Private Operations & Maintenance Plan

4540 SE Milwaukie Apartments

Prepared By:

Humber Design Group, Inc.

117 SE Taylor St. Suite 001
Portland, OR 97214
(P) 503 946 6690

Date: January 23, 2017

Site O&M Responsible Party

This facility is to be maintained by property owner Yoshida Group. Yoshida Group contact is Matthew A. Wand, , 503-284-1114.

Onsite Stormwater System Description

All stormwater runoff generated on-site is managed with a Drywell and Sedimentation Manhole system. The Sedimentation Manhole is a large manhole that allows pollutants to settle out as stormwater collects in the large sump. Stormwater then flows out of an elbowed pipe to the Drywell. The Drywell is a large perforated manhole where stormwater infiltrates through washed, crushed stone or gravel wrapped in filter fabric.

Table 1 - Facility Description Table

Facility Name	Type	Facility Size (sf)	Source	Impervious Area Managed (sf)	Discharge Point
Drywell	**Drywell**	(2) 48" dia, x 25'	Roof / landscape	11,500	Infiltration

Inspection & Maintenance Schedule

First two years: Quarterly
Thereafter: Twice a year
After major rainfall events: Within 48 hours of major rainfall events (more than 1 inch of rain over a 24-hour period)

Inspection & Stormwater Maintenance Prodecures

The following items shall be inspected and maintained as stated.

Drywell and Soakage Trenches

Clean gutters, rain drains, and silt traps twice a year.
Repair/seal cracks. Replace when repair is insufficient.
Prevent large root systems from damaging subsurfaced structural components.
Remove sediment and debris from all accessible components to prevent ponding.
Ponding/lack of infiltration may require decommissioning and replacement. Consult with the City prior to subgrade work.

Overflow Drains, Area Drains, and Piped Storm System

Sediment shall be removed biannually.
Debris shall be removed from inlets and outlets quarterly.
Quarterly inspection for clogging shall be performed.
Grates shall be tamper-proof.
Repair/seal cracks. Replace when repair is insufficient.

Sediment shall be removed biannually.
Debris shall be removed from inlets and outlets quarterly.
Quarterly inspection for clogging shall be performed.
Grates shall be tamper-proof.
Repair/seal cracks. Replace when repair is insufficient.

Vectors

Stormwater facilities shall not harbor mosquito larvae or rats that pose a threat to public health or that undermine the facility structure. Monitor standing water for small wiggling sticks perpandicular to the water's surface. Note holes/burrows in and around facilities. Call Multnomah County Vector Control at 503-988-3464 for immediate assistance to eradicate fectors. Record the time/date, weatehr, and site conditions when vector activity is observed.

Best Management Practices (BMPs)

BMPs prevent pollutants from mixing with stormwater. Typical nonstructural control measures include raking and removing leaves, street sweeping, vacuum sweeping, and limited and controlled application of pesticides, herbicides, and fertilizers.

Spill Prevention

Spill prevention measures shall be exercised when handling substances that can contaminate sormwater. Virtually all sites, including residential and commercial, present dangers from spills. It is important to exercise caution when handling substances that can contaminate stormwater. Activities that pose the chance of hazardous material spills shall not take place near collection facilities.

- The proper authority and the property owner shall be contacted immediately if a spill is observed.
- A spill kit shall be kept near spill-prone operations and refreshed annually.
- Employees shall be trained on spill control measures.
- Shut-off valves shall be tested quarterly.
- Releases of pollutants shall be corrected within 12 hours.

Access Access shall be maintained for all facilities so O&M can be performed as regularly scheduled.

Inspection & Maintenance Logs

The facility owner shall keep a log to record all inspection and maintenance activities (see Sample Log). Record date, description, and contractor (if applicable) for all inspections and any maintenance or repairs performed. Keep work orders and invoices on file and make available upon request of the city inspector.

Inspection Log	Record the date and the personnel who conducted the site inspection. Record the infiltration rate if greater than 48 hours, a description of any and all spills and vector issues, sediment & oil depth, the percentage of vegetation coverage (deseriable and undesirable), and the condition of the system components every quarter for the first 2 years of operation and twice a year after a major storm even thereafter.
Pollution Prevention	All sites shall implement BMPs to prevent hazardous wastes, litter, or excessive oil and sediment from contaminating stormwater. Contact Spill Prevention & Citizen Response at 503-823-7180 for immediate assistance with responding to spills. Record time/date, weather, and site conditions if site activities are found to contaminate stormwater.
Vectors (mosquitoes and rodents)	Stormwater facilities shall not harbor mosquito larvae or rats that pose a threat topublic health or that undermine the facility structure. Monitor standing water for small wiggling sticks perpendicular to the water's surface. Note holes/burrows in and around facilities. Call Multnomah County Vector Control at 503-988-3464 for immediate assistance with eradicating vectors. Record time/date, weather, and site conditions when vector activity is observed.
Depth of Sediment & Oil	Take and record measurement at catch basins, conveyance systems, inlets, outlets and within the facility itself. Compare to capacity thresholds defined in the *Stormwater Management Manual* Section 3.2.4, Summary of Thresholds for Maintenance, or the site-specific O&M Plan.
Percent Vegetation Coverage	Record percent cover of desireable, dead, and invasive vegetation.
Condition of Structural Components	Record type and size of missing or broken components (i.e. width of cracks and/or extent of settling.)
Maintenance	Record date, description, and contractor (if applicable) for all structural repairs, landscape maintenance, and facility cleanout activities.

Sample Log Form

Date: ______________________ Time: ______________ Initial: __________
Work performed by: ____
Work performed: ____
Details: ____
____
____

Date: ______________________ Time: ______________ Initial: __________
Work performed by: ____
Work performed: ____
Details: ____
____
____

Date: ______________________ Time: ______________ Initial: __________
Work performed by: ____
Work performed: ____
Details: ____
____
____

Date: ______________________ Time: ______________ Initial: __________
Work performed by: ____
Work performed: ____
Details: ____
____
____

OPERATIONS & MAINTENANCE FORM

PRIVATE STORMWATER MANAGEMENT FACILITIES

CITY OF PORTLAND
Stormwater Management Manual

FORM 2

☐ This O&M Form supercedes document number ______

(for official county use only)

PROJECT NAME 4540 SE Milwaukie Avenue

PERMIT INFORMATION

Permit # ______

Permit Submittal Date 01/23/2017

SITE INFORMATION *(include all parcels)*

R# (6 Digits) R143025, R143024

Site Address 4540 SE Milwaukie Ave

City / State / Zip Portland, OR 97214

Preparation Date: 01/19/2017

OWNER INFORMATION (ALL LEGAL OWNERS)

Name (1) Yoshida Group

Name (2) ______

Address *(Mailing)* 8440 NE Alderwood Rd, Suite A

City / State / Zip Portland, OR 97220

O&M PREPARER INFORMATION

Name Alex Wesolovski, Humber Design Group, Inc.

Address *(Mailing)* 117 SE Taylor St, Suite 001

City / State / Zip Portland, OR 97214

Phone *(area code required)* 503-946-6690

Email alex.wesolovski@hdgpdx.com

Site Legal Description:

BURNELL'S ADD, BLOCK 281, LOT5&6

Responsible Party for Maintenance *(check one)*

- ☐ Homeowners Association
- ☒ Property Owner
- ☐ Property Management Company
- ☐ Tenant
- ☐ Other (describe) ______ *(not Contractor or Consultant)*

Contact Information for Responsible Party

Contact Name Matthew A. Wand

Contact Organization Yoshida Group

Phone *(area code required)* 503-284-1114

Email: matt.wand@yoshida.com

Maintenance Practices and Schedule

These operation and maintenance practices are required in accordance with Portland City Code, Chapter 17.38.

The requirements are based on the current version of the *City of Portland Stormwater Management Manual* on the date of permit submittal.

For the **Simplified Approach,** please attach the current O&M Specifications for each facility type from the *Stormwater Management Manual*, Chapter 3.3.1.

For the **Presumptive and Performance Approaches,** please attach the approved, site specific O&M Plan per the *Stormwater Management Manual,* Chapter 3.3.2.

SITE PLAN

Provide a site plan sketch in the area provided below, or attach a scaled site plan to this submittal that includes all of the information required as shown in Appendix D6 on page D.6-1, in Operations & Maintenance Form Instructions, Site Plan.

STEP 1 – COMPLETE THE FOLLOWING TABLE

Stormwater Facility Type (Chapter 2)	Stormwater Facility Size (sf)	Drainage is from Roof or Lot?	Impervious Area Treated (sf)	Discharge Point
Drywell	(2) 48" dia. X 25' deep	Roof	9,530	Infiltration
Drywell	(2) 48" dia. X 25' deep	Lot	1,970	
Totals	2		11,500	

Maintaining the stormwater management facility or facilities listed above shown on the following (or attached) site plan is a required condition of building permit approval for the identified property. Property owners are required to operate and maintain facilities in accordance with the O&M plan on file with the City of Portland. This requirement is binding on all current and future owners of the property. Failure to comply with the O&M plan can trigger an enforcement action, including penalties. The O&M plan may be modified by written consent of current owners and written approval of the Bureau of Environmental Services.

STEP 2 – REQUIRED SITE PLAN

(insert or draw here, or attach separate sheet)

☒ I Have Attached a Site Plan

SIGNATURE AND ACKNOWLEDGEMENT

By signing below, the owner accepts and agrees to the terms and conditions contained in this O&M Form and in any document executed by filer and recorded with it. The owner further acknowledges that this documentation has been prepared on their behalf and that they are responsible for the quality and completeness of the O&M Plan. Any failure to comply with the terms of these plans may result in enforcement actions by BES requiring the property owner to restore the stormwater facilities to a functional state as approved under original requirements.

The owner also accepts that the City requires property owners to submit and record, with the County, complete and accurate O&Ms enforceable under City Code 17.38 and that substantial changes to the O&M require City approval prior to County recording. A revised O&M must state that it supersedes a previous O&M (with cited county document number; See Page 1).

THIS PAGE MUST BE SIGNED IN THE PRESENCE OF A NOTARY.

____
Property Owner or Authorized Representative (1) Signature

____
Property Owner or Authorized Representative (2) Signature

NOTARY SIGNATURE AND STAMP

☐ INDIVIDUAL Acknowledgement

This acknowledgement is intended for property owned by individuals or trusts.

STATE of OREGON county of: ____________________

This instrument was acknowledged before me on: *(date)* ____________________

By: *(owner 1)* ____________________

By: *(owner 2)* ____________________

Notary Signature ____________________

My Commission Expires ____________________

Notary Seal:

OR

☐ CORPORATE Acknowledgement

This acknowledgement is intended for corporation, government agencies, school districts, or other formal entities

STATE of OREGON county of: ____________________

This instrument was acknowledged before me on: *(date)* ____________________

By: *(representative)* ____________________

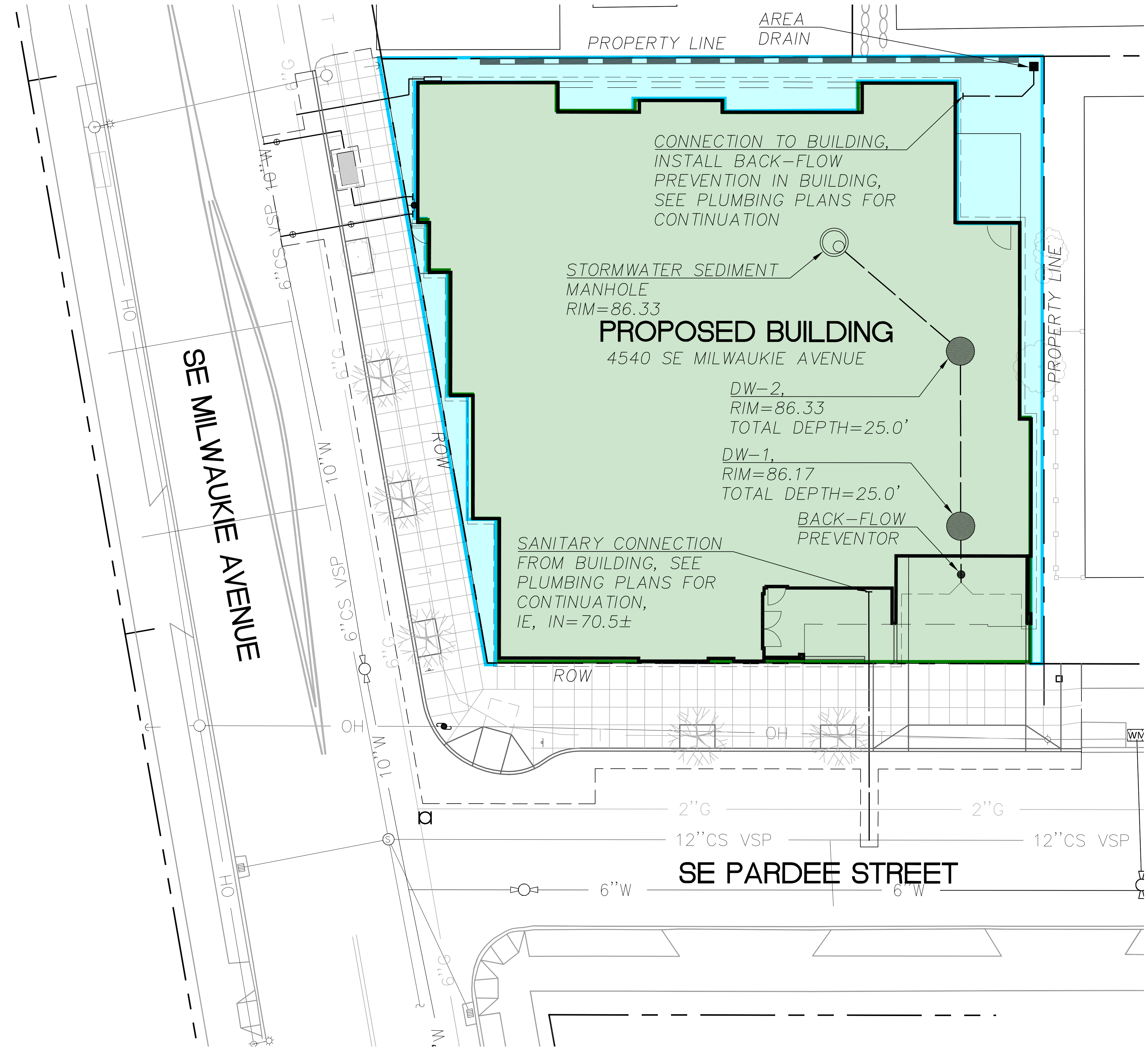
As: *(Title)* ____________________

Of: *(Corporation)* ____________________

Notary Signature ____________________

My Commission Expires ____________________

Notary Seal:



O AND M MAP

SCALE: 1"=20'

GRAPHIC SCALE

0 20 40

1 inch = 20 ft.

4540 SE MILWAUKIE AVE

O AND M MAP

PROJECT NO.:	SMI001
DRAWN BY:	AFW
DESIGN BY:	AFW
REVIEWED BY:	MSW
DATE:	01/23/2017

Humber Design Group, Inc.
Portland, OR • 503.946.6690 • hdgpdx.com

Appendix D

Additional Forms & Associated Reports

July 18, 2016
HGSI Project No. 16-2049

Marty Treece
Treece and Lambert, LLC
2905 SW 1st Avenue
Portland, Oregon 97201

Via email (pdf format); hard copies provided on request

Subject: **GEOTECHNICAL ENGINEERING REPORT**
4524 AND 4540 SE MILWAUKIE AVENUE
PORTLAND, OREGON

Hardman Geotechnical Services Inc. (HGSI) performed a geotechnical engineering study for the proposed new residential project at 4524 and 4540 SE Milwaukie Avenue in the City of Portland, Oregon (see Vicinity Map, Figure 1). The purpose of the geotechnical study was to explore and evaluate the surface and subsurface conditions at the site, and to provide geotechnical recommendations for foundation design and site development.

PROJECT DESCRIPTION

The site is located northeast of the intersection of SE Milwaukie Avenue and SE Pardee Street in Portland, Oregon. Comprised of two tax lots, the subject site totals about 0.25 acres (11,065 square feet). The address of the northernmost property is 4524 SE Milwaukie Avenue, an existing home on that property was reportedly constructed in 1923. The southern property, 4540 SE Milwaukie Avenue, is occupied by a metal commercial/shop building reportedly constructed in 1954. The back of the property at 4540 SE Milwaukie Avenue has stored materials including building supplies, a travel trailer, etc. The number of and location of exploratory borings was limited by the presence of existing buildings and stored material.

Topography at the site slopes gently to moderately down to the east. The portlandmaps website indicates portions of the site have slopes steeper than 25%, primarily along the eastern margin of the property. There is an existing concrete wall along the east side of 4540 SE Milwaukie Avenue that elevates the storage yard area of that property up to about 4 feet above the neighboring property to the east.

We understand the site is to be developed as a residential apartment building or buildings, up to 5 stories in height. The ground floor will be at grade, with no substantial basement excavations planned. Some minor excavations or fills may be needed on site due to the sloping topography. We anticipate the existing wall along the east side of the site will be replaced, but at this time the replacement wall type, height and other features are not defined.

The project will also include underground utilities and stormwater disposal facilities, with on-site infiltration if feasible. Design of stormwater facilities will be performed by others. The site and grading plans have not yet been finalized. At present, specific wall and column locations, and structural loading, are being determined. HGSI should review the structural configurations and loads during the design process, and update the recommendations of this report as necessary based on specific design details.

SCOPE OF WORK AND AUTHORIZATION

Our scope of work for the project consisted of site reconnaissance, exploratory drilling, infiltration testing, geotechnical analyses, and preparation of this report. This geotechnical study was performed in accordance with HGSI Proposal No. 16-621, dated June 21, 2016, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

REGIONAL GEOLOGY AND SEISMIC SETTING

The subject site lies within the Portland Basin, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. The Portland Basin is a northwest-southwest trending structural basin produced by broad regional downwarping of the area. The Portland Basin is approximately 20 miles wide and 45 miles long and is filled with consolidated and unconsolidated sedimentary rocks of late Miocene, Pliocene and Pleistocene age.

The subject site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley, the last of which occurred about 10,000 years ago (Madin, 1990). Underlying the project site, these deposits consist of horizontally layered, micaceous, silt to fine sand.

At least three major fault zones capable of generating damaging earthquakes are known to exist in the region. These include the Portland Hills Fault Zone, Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone. These potential earthquake source zones are included in the determination of seismic design values for structures, as presented in the *Seismic Design* section. None of the known faults extend beneath the site.

FIELD EXPLORATION

EXPLORATORY BORINGS

The site-specific exploration for this study consisted of exploratory borings. On July 5, 2016, two borings, designated B-1 and B-2, were drilled to depths of approximately 21.5 feet at approximate locations shown on Figure 2. It should be noted that exploration locations were determined in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

The boreholes were drilled using a trailer mounted drill rig and solid stem auger methods. At each boring location, SPT (Standard Penetration Test) sampling was performed in general accordance with ASTM D1586 using a 2-inch outside diameter split-spoon sampler and a 140-pound hammer equipped with a rope and cathead mechanism. During the test, a sample is obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows for each 6 inches of penetration is recorded. The Standard Penetration Resistance ("N-value") of the soil is calculated as the number of blows required for the final 12 inches of penetration. If 50 or more blows are recorded within a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the number of inches driven. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. At the completion of the borings, the holes were backfilled with bentonite.

Explorations were conducted under the full-time observation of HGSI personnel. Soil samples were classified in the field and representative portions were placed in relatively air-tight plastic bags. These soil samples were then returned to the laboratory for further examination and laboratory testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in general accordance with the Unified Soil Classification System.

Summary boring logs are attached. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types. The actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

INFILTRATION TESTING

On July 5, 2016, HGSI performed open-hole falling head infiltration tests in borings B-1. The test borings were pre-saturated a minimum of 2 hours prior to testing due to the coarse grained nature of site soils. Following the soil saturation, infiltration tests were conducted. The water level was measured to the nearest 0.1 inch from a fixed point. The change in water level was recorded at intervals for a total period of at least 2 hours. Table 1 presents the results of the falling head infiltration tests.

Table 1. Summary of Infiltration Test Results

Boring	Depth (feet)	Soil Type	Infiltration Rate(in/hr)	Hydraulic Head Range (inches)
B-1	21	Silty fine Sand	9	86 - 82

SUBSURFACE CONDITIONS

The following discussion is a summary of subsurface conditions encountered in our explorations. For more detailed information regarding subsurface conditions at specific exploration locations, refer to the attached boring logs. Also, please note that subsurface conditions can vary between exploration locations, as discussed in the *Uncertainty and Limitations* section below.

SOIL

Results of the exploration program indicate that the site is underlain by silts and sands belonging to the Willamette Formation. The observed conditions and soil properties are summarized below.

> ***Silt:*** Beneath the 2 inches of gravel in borings B-1 and B-2 we encountered loose to medium stiff, moist silt. This silt was interpreted as belonging to the Willamette Formation and extended to roughly 5 to 8 feet in borings B-1 and B-2 respectively.
>
> ***Silty fine sand to sandy silt:*** Beneath the silt in boring B-1 and B-2, silty fine sand to sandy wilt was encountered. Consistency was generally medium dense / medium stiff to stiff, and slightly moist to moist. Soils appeared to coarsen with depth and extended to the termination of the borings at 21.5 feet. This material is interpreted as belonging to the Willamette Formation.

GROUNDWATER

At the time of our explorations, groundwater was not encountered beneath the site. Regional geologic mapping (Snyder, 2008) indicates that static groundwater is present at a depth of about 40 feet below the existing ground surface at the site. In our experience, it is not uncommon to encounter thin perched groundwater zones within the Willamette Formation in this area, particularly during the wet season.

The groundwater conditions reported above are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. Furthermore, it is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in land use and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Results of this study indicate that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. Spread footings are acceptable for use on this project. Additional discussion and recommendations are presented below regarding site preparation and undocumented fill removal, engineered fill, wet weather earthwork, spread footing foundations, concrete slabs-on-grade, perimeter footing drains, below-grade walls, seismic design, storm water infiltration systems, temporary excavations, utility trench backfill, and erosion control.

SITE PREPARATION AND UNDOCUMENTED FILL REMOVAL

Proposed structure and other areas of proposed improvements should be cleared of debris. Where encountered, undocumented fill within the proposed building footprint, beneath pavements or other settlement-sensitive improvements, should be completely removed and replaced with engineered fill.

Following removal of surficial debris and undocumented fill, the exposed subgrade should be evaluated by HGSI. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded dump truck. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with engineered fill, as described below. The depth of overexcavation, if required, should be evaluated by HGSI at the time of construction.

Based on the site location, it is possible that one or more old dry wells, septic systems or other below-grade structures may be present on site. In the event that old drywell(s) are encountered during site development, the following recommendations are made. Deeper portions of dry wells should be backfilled with controlled density fill (CDF), which is essentially a lean mix concrete consisting of water, sand and cement. We recommend use of “excavatable” CDF so that future excavations can be made through the dry well backfill if any new utilities or other excavations are needed in the affected areas. Above a depth of about 8 feet, at the contractor’s option, backfill may consist of granular soils such as “reject rock,” recycled concrete or similar material approved by HGSI. The granular backfill should be placed in lifts no thicker than about 18 inches and compacted with a “hoe-pac” excavator attachment to a minimum of 90 percent of Modified Proctor (ASTM D-1557). This backfill specification should also be used for any basements or other depressions that require fill during the demolition process.

ENGINEERED FILL

On-site native soils are considered suitable for use as engineered fill in dry weather conditions, provided they are relatively free of organics and are properly moisture conditioned for compaction. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 2 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill and crushed rock backfill soils should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. On-site soils may be wet of optimum; therefore, we anticipate that aeration of native soil will be necessary for compaction operations performed during late spring to early summer.

Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Field density testing should conform to ASTM D2922 and D3017, or D1556. Engineered fill should be periodically observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing.

WET WEATHER EARTHWORK

The on-site soils are moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact fill to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, HGSI should be contacted for additional recommendations.

Under wet weather, the construction area will unavoidably become wet and the condition of exposed fill and native soils will degrade. To limit the impacts of wet weather on the finished building pad surface, consideration may be given to placement of a crushed aggregate pad. Where used, we recommend the working pad be constructed using 1½"–0 crushed aggregate, and should have minimum thickness of at least 12 inches. This thickness is considered adequate to support light construction traffic, but will not be sufficient to support heavy traffic such as loaded dump trucks or other heavy rubber-tired equipment.

SPREAD FOOTING FOUNDATIONS

Spread footing foundations are acceptable for use on this project. Due to the presence of soft/medium stiff soils at anticipated foundation grades, we recommend placement of a minimum of 12 inches of crushed rock beneath structural foundations. With 12 inches of compacted crushed rock beneath footings, we recommend a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for use in design. The recommended maximum allowable bearing pressure may be increased by a factor of 1.33 for short term transient conditions such as wind and seismic loading.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of spread foundations of less than about 1 inch and differential settlement between two adjacent load-bearing components supported on competent soil of less than about ½ inch. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0.5 may be assumed along the interface between the base of the footing and subgrade soils. Passive earth pressure for buried portions of structures may be calculated using an equivalent fluid weight of 390 pounds per cubic foot (pcf), assuming footings are cast against dense, natural soils or engineered fill. The recommended coefficient of friction and passive earth pressure values do not include a safety factor. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. Loose, wet or otherwise softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. HGSI should observe foundation excavations prior to placing crushed rock, to verify that adequate bearing soils have been reached. HGSI should monitor crushed rock placement beneath foundations and perform density tests to verify compliance with the engineered fill density specification.

CONCRETE SLABS-ON-GRADE

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation* section. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content, and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 200 kcf (115 pci) should be assumed for the soils anticipated at subgrade depth. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of crushed rock of 8 inches beneath the slab.

Interior slab-on-grade floors should be provided with an adequate moisture break. The capillary break material should consist of ODOT open graded aggregate per ODOT Standard Specifications 02630-2. The minimum recommended thickness of capillary break materials on re-compacted soil subgrade is 8 inches. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction, and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 90% of its maximum dry density as determined by ASTM D1557 or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. With this type of system, an approximately 2-inch thick layer of sand is often placed over the vapor barrier to protect it from damage, to aid in curing of the concrete, and also to help prevent cement from bleeding down into the underlying capillary break materials. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside HGSI's area of expertise.

PERIMETER FOOTING DRAINS

To minimize soil moisture fluctuations adjacent to the building, we recommend the outside edge of perimeter footings be provided with a drainage system consisting of 3-inch minimum diameter perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, crushed drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Water collected from the footing drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. The footing drains should include clean-outs to allow periodic maintenance and inspection.

Down spouts and roof drains should collect roof water in a system separate from the footing drains in order to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

PERMANENT BELOW-GRADE WALLS

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-reset equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 5.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 390 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and HGSI should be contacted for additional recommendations.

A coefficient of friction of 0.5 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a minimum 12-inch wide zone of sand and gravel containing less than 5 percent fines against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

HGSI should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

SEISMIC DESIGN

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2012 International Building Code (IBC) with applicable 2014 Oregon Structural Specialty Code (OSSC) revisions. We recommend Site Class D be used for design per the OSSC, which references ASCE 7-10, Chapter 20, Table 20.3-1. Design values determined for the site using the USGS (United States Geological Survey) *Earthquake Ground Motion Parameters* utility are summarized on Table 2.

Table 2. Recommended Earthquake Ground Motion Parameters (2012 IBC / 2014 OSSC)

Parameter	Value
Location (Lat, Long), degrees	45.4899, -122.6510
Mapped Spectral Acceleration Values (MCE, Site Class B):	
Short Period, S_s	0.983 g
1.0 Sec Period, S_1	0.421 g
Soil Factors for Site Class D:	
F_a	1.107
F_v	1.579
$SD_s = 2/3 \times F_a \times S_s$	0.726 g
$SD_1 = 2/3 \times F_v \times S_1$	0.443 g

Potential seismic impacts also include secondary effects such as soil liquefaction, fault rupture potential, and other hazards as discussed below:

- **Soil Liquefaction Potential –** Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. The permanent ground water table is approximately 40 feet below the site (Snyder, 2008). Therefore, soils under the project site are not considered susceptible to liquefaction. It is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.
- **Fault Rupture Potential** – Based on our review of available geologic literature, we are not aware of any mapped active (demonstrating movement in the last 10,000 years) faults on the site. During our field investigation, we did not observe any evidence of surface rupture or recent faulting. Therefore, we conclude that the potential for fault rupture on site is low.
- **Seismic Induced Landslide** – Topography in the vicinity of the subject site is generally flat to gently sloping. The potential for slope instability and seismic induced landslide on site is considered very low.
- **Effects of Local Geology and Topography** – In our opinion, no additional seismic hazard will occur due to local geology or topography. The site is expected to have no greater seismic hazard than surrounding properties and the Portland area in general.

STORMWATER INFILTRATION SYSTEMS

Based on the results of the infiltration testing, deep soils on site exhibit moderate infiltration rates. Groundwater was not encountered in borings advanced to a maximum depth of 21.5 feet. No indications of seasonal high groundwater were observed in the borings. Based on the USGS mapping we anticipated seasonal high groundwater to be about 40 feet below the ground surface (Snyder, 2008).

In-situ infiltration tests were conducted to assess the infiltration capacity of deep soils on site for the use of a drywell. Design of stormwater infiltration facilities will be performed by others. The approximate location of the test is shown on Figure 2, and the test methodology is discussed above in the *Infiltration Testing* section, above. Table 1 summarizes results of the infiltration testing.

Near-surface soils were not tested due to the clients need for a drywell and the shallow depth to sand. At a depth of 21 feet in B-1, the infiltration test result was 9 inches/hour. A value of 9 in/hr may be used for design of deep stormwater facilities, such as dry wells, that extend to depths of at least 13 feet below ground surface. The infiltration rates presented herein do not incorporate a factor of safety. For the design infiltration rate, the system designer should incorporate an appropriate factor of safety against slowing of the rate over time due to biological and sediment clogging.

In the event that dry wells are located near the building's exterior wall, manholes should be designed and constructed in a manner which will limit disturbance of surrounding soils, and prevent the loss of soil from the surrounding area into the manhole. We recommend use of geotextile fabric between the manhole and any surrounding drain rock, and the native soils. Annular space between the manhole and surrounding soil should be backfilled tightly with compacted crushed rock or drain rock.

Dry wells should be located at least 5 feet from any structural foundations. The potential for soil saturation has been accounted for in development of the allowable soil bearing pressures used in design. Therefore, soil saturation from dry well operation is not anticipated to cause significant adverse impacts on foundation support.

Infiltration test methods and procedures attempt to simulate the as-built conditions of the planned disposal system. However, due to natural variations in soil properties, actual infiltration rates may vary from the measured and/or recommended design rates. All systems should be constructed such that potential overflow is discharged in a controlled manner away from structures, and all systems should include an adequate factor of safety. Infiltration rates presented in this report should not be applied to inappropriate or complex hydrological models such as a closed basin without extensive further studies.

TEMPORARY EXCAVATIONS

Site soils consist of medium stiff to stiff native silt, to fine sandy silt and silty fine sand, as described above. At this time the location and finish floor elevations of the new structure are not finalized. Depending on final structural configuration, it may be necessary to provide mechanical shoring system(s) along portions of the north, and east property lines to protect neighboring structures and properties. HGSI should be consulted to provide specific shoring recommendations if needed once the design details are better known.

Where space is available we recommend temporary excavation slopes of 1H:1V (Horizontal:Vertical). Where necessary, temporary excavations up to 1H:2V may be made, or the temporary excavation may consist of a 1H:1V slope with a maximum 4-foot-high vertical cut at the toe.

The temporary excavation slopes recommended herein are anticipated to have an adequate factor of safety considering overall (gross) failure, during the anticipate time span the temporary cut will be open, about 3 to 4 weeks. Some surficial erosion or sloughing may occur on the slope face. Surface water should not be allowed to pond above the temporary cut, nor should surface water be allowed to flow down the slope face. Consideration should be given to covering the temporary cut face with plastic sheeting in the event of rainy weather in the forecast. It is our opinion that there is a low potential for the planned excavation to impact or damage the existing driveway and home on the adjacent property.

HGSI's responsibility for temporary excavation stability includes only the evaluation summarized herein. HGSI is not responsible for any aspect of jobsite safety. The contractor is responsible to designate a "responsible person" for monitoring of temporary excavations on site. We suggest that the tops of temporary excavation slopes be observed at least once daily for indications of movement such as ground cracks. If cracking or other indications of slope movement are observed, work should be halted and HGSI contacted immediately for evaluation of the situation.

UTILITY TRENCH EXCAVATION AND BACKFILL

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that structural trench backfill be compacted to at least 90 percent of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a 3/4”-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

EROSION CONTROL CONSIDERATIONS

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw, bio-bags, silt fences, or other appropriate technology. Where used, erosion control devices should be in place and remain in place throughout site preparation and construction. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HGSI should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, HGSI executed these services in accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

HARDMAN GEOTECHNICAL SERVICES INC.

REGISTERED PROFESSIONAL ENGINEER
64,422PE
OREGON
SEPTEMBER 26, 2000
SCOTT L. HARDMAN

7-18-16

EXPIRES: 06-30-2017

Scott L. Hardman, P.E., G.E.
Geotechnical Engineer

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site Plan
Logs of Borings B-1 and B-2

REFERENCES

Beeson, M.H., Tolan, T.L., and Madin, I.P., 1991, Geologic map of the Portland Quadrangle, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries Geological Map Series GMS-75, scale 1:24,000.

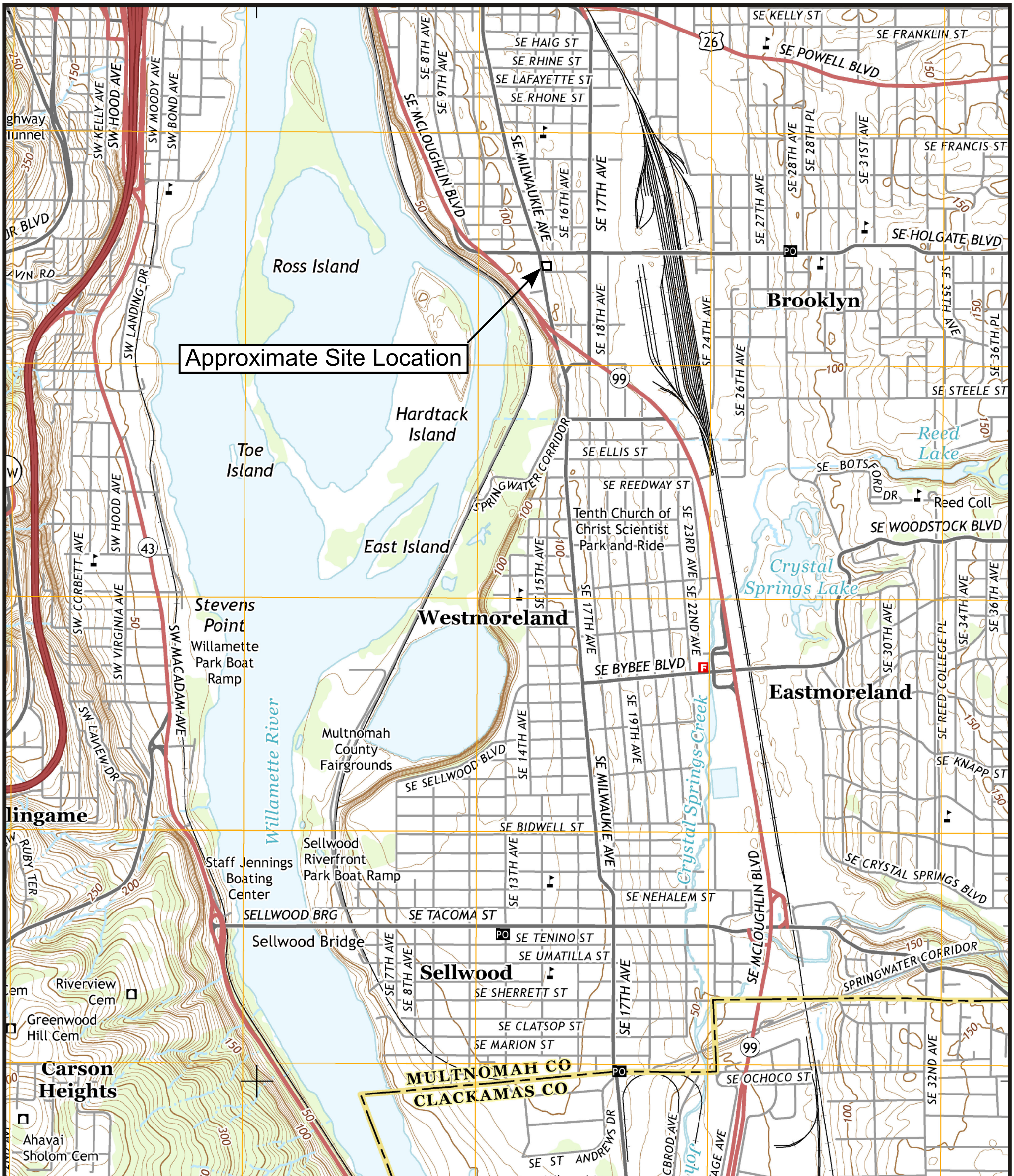
Madin, I.P., 1990, Earthquake hazard geology maps of the Portland metropolitan area, Oregon: Oregon Department of Geology and Mineral Industries Open-File Report 0-90-2, scale 1:24,000, 22 p.

Snyder, D.T., 2008, Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon Area: U.S. Geological Survey Scientific Investigations Report 2008–5059, 41 p., 3 plates.

HARDMAN GEOTECHNICAL SERVICES INC.

Practical, Cost-Effective Geotechnical Solutions

VICINITY MAP



Project: SE Milwaukie Ave Portland, Oregon	Project No. 16-2049	FIGURE 1

HGSI HARDMAN GEOTECHNICAL SERVICES INC.
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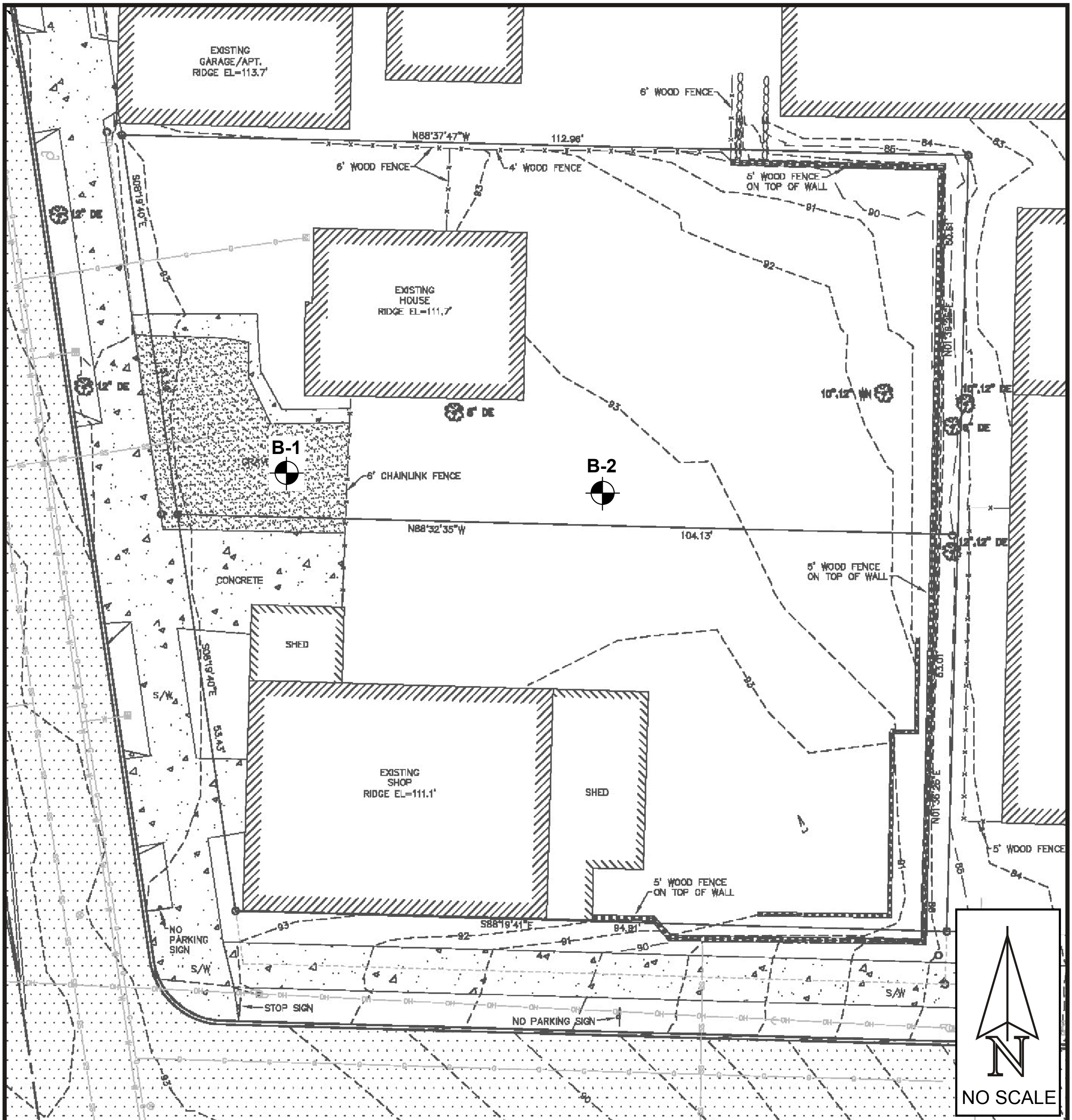
SITE AND EXPLORATION PLAN



Base map prepared by Summit Land Surveyors

Legend

B-2 Boring Designation and Approximate Location

Project SE Milwaukie Ave Portland, Oregon	Project No. 16-2049	FIGURE 2

BORING LOG

Project: SE Milwaukie Avenue, Portland, Oregon	Project No. 16-2049	Boring No. B-1

Depth (ft)	Sample Interval	SPT N-Value (Blows/foot)	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Groundwater	Material Description
						2 inches of gravel
						Medium stiff, fine sandy Silt, brown, slightly moist
	X	4				
5	X	6				Loose to medium dense/medium stiff to stiff, silty fine sand to sandy silt, gray and brown, slightly moist to moist. Substantial root at 6 feet.
	X	6				
10	X	6				
15	X	7				
20	X	14				
						Boring terminated at 21.5 feet No groundwater or seepage encountered
25						

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10110 SW Nimbus Avenue, Suite B-5
Portland, Oregon 97223
(503) 530-8076

LEGEND

STP Drive Sample

Water Level at Time of Drilling

Date Drilled: 7-5-16

Logged By: IDM

BORING LOG

Project: SE Milwaukie Avenue, Portland, Oregon	Project No. 16-2049	Boring No. B-2

Depth (ft)	Sample Interval	SPT N-Value (Blows/foot)	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Groundwater	Material Description
0–0.2						2 inches of gravel
0.2–8.5	☒	3				Soft to medium stiff, Silt, brown, moist
5	☒	5				
	☒	8				
8.5–21.5						Loose to medium dense/medium stiff to stiff, silty fine Sand to sandy silt, gray and brown, moist.
10	☒	8				
15	☒	7				
20	☒	15				
21.5						Boring terminated at 21.5 feet No groundwater or seepage encountered
25						

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LEGEND

☒ STP Drive Sample

▽ Water Level at Time of Drilling

Date Drilled: 7-5-16

Logged By: IDM

ORIGINAL APPEAL

Development Services

From Concept to Construction

Phone: 503-823-7300 Email: bds@portlandoregon.gov 1900 SW 4th Ave, Portland, OR 97201
More Contact Info (http://www.portlandoregon.gov//bds/article/519984)

APPEAL SUMMARY

Status: Decision Rendered

Appeal ID: 14825	**Project Address:** 4540 SE Milwaukie Ave
Hearing Date: 3/22/17	**Appellant Name:** Matha Williams, PE
Case No.: P-005	**Appellant Phone:** 503-946-6680
Appeal Type: Plumbing	**Plans Examiner/Inspector:** n/a
Project Type: commercial	**Stories:** 4 **Occupancy:** R-2 **Construction Type:** V-A
Building/Business Name:	**Fire Sprinklers:** Yes - not provided
Appeal Involves: Erection of a new structure	**LUR or Permit Application No.:** 17-111682-CO
Plan Submitted Option: pdf [File 1] [File 2]	**Proposed use:** Multi-family apartments complex

APPEAL INFORMATION SHEET

Appeal item 1

Code Section	OPSC 1101.5.3.2, 2016 Portland Stormwater Management Manual - Chapter 2, Drywell Design Requirements, Setbacks Pg 2-118.
Requires	"no drywell shall be located closer than five (5) feet of a property line nor closer than ten (10) feet (3m) to any building unless approved by the Authority Having Jurisdiction."
Proposed Design	The applicant is proposing the use of drywell systems to be installed underneath the building structure for multiple reasons; see 'Reason for Alternate' section for more information. Drywells located under the structure have been taken into account by the Geotechnical engineer and structural engineer per attached documents. The drywell system proposed for the building was sized to infiltrate the 10-year storm. The building roof area would produce 0.21 cfs of runoff during the 10-year storm. The sizing of the drywell system was done using HydroCAD®. A design infiltration rate of 4.5 in/hr was used for calculations. The drywell system will be tested at the time of installation to verify infiltration capacity. Feasibility of on-site infiltration: The feasibility of the drywell system location is based on infiltration testing, maintenance, structural design, and strength of soils. Infiltration testing was performed by Hardman Geotechnical Services Inc. and documented in the "Geotechnical Engineering Report" dated July 18, 2016. This report shows measured infiltration rates of 9 in/hr onsite at a depth of 21 feet. The drywells are deep and will be discharging stormwater 5-15 feet below the bottom of the footings. The infiltration rate of the deep soils will prevent saturation of the shallow soils directly underneath the buildings. See attachments for supporting data on the effectiveness of infiltration for the sites.
Reason for alternative	The applicant proposes the drywell system to be installed underneath the building structures due to space limitations on the site (e.g. the vertical construction consumes the property footprint from zero lot line to zero lot line) preventing location of drywells in accordance with the OPSC. It is

also the applicant's belief that the use of drywells is a superior system to other types of retention and treatment-only facilities as it altogether eliminates discharge of the 25-year and 100-year storm to the combined storm/sanitary sewer line at SE Stark Street. Since the discharge to the drywells is roof area, no mechanical pre-treatment or accompanying maintenance is required. The proposed system will have minimal sediment loads compared to vehicular traffic areas. There is also precedent from the city for approval of this scenario for the above stated reasons.
Mitigation of Maintenance and Overflow Concerns:
All the drywells will have accessible, bolt down manhole rims located in open vehicle drive aisles or loading areas to allow for maintenance as required by Oregon Department of Environmental Quality (ODEO}. Maintenance will be performed in the same manner as if the drywell was located outside the building. The applicant has confirmed with a local company (River City Environmental Inc.) that a vacuum truck is capable of reaching lengths up to 300 feet for drywell maintenance. The drywells will be maintained by a professional management company who will follow the county recorded operations and maintenance plan for the drywells.
Mitigation of Soil Bearing Concerns
The strength of the soils will not be affected by the infiltration of stormwater runoff as explained in the attached memo from Geo Design, Inc. dated November 21, 2016.

APPEAL DECISION

Drywell system located underneath the building: Denied. Proposal does not meet minimum requirements for stormwater disposal.

Appellant may contact Joe Blanco (503-823-2059) for more information.

Pursuant to City Code Chapter 25.07, you may appeal this decision to the Plumbing Code Board of Appeal within 180 calendar days of the date this decision is published. For information on the appeals process and costs, including forms, appeal fee, payment methods and fee waivers, go to www.portlandoregon.gov/bds/appealsinfo, call (503) 823-7300 or come in to the Development Services Center.