ENGINEERING • LLC

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STORMWATER SITE PLAN

FOR

OVERLOOK APARTMENTS PHASE 2

PORT ORCHARD, WASHINGTON

FEBRUARY 2020

Prepared For:
OLAA, LLC
15234 SE 366th Place
Auburn, WA 98092

Prepared By:
Kyle Rose, P.E., Design Engineer

Approved By:
Jeremy Haug, P.E., Project Engineer

JEREMY FRAIN HAUG
STATE OF WASHINGTON
46951
REGISTERED
PROFESSIONAL ENGINEER
2.13.20

Project # 16-300

*I hereby state that this Stormwater Site Plan for **Overlook Apartments Phase 2** has been prepared by me or under my supervision and meets the standard of care and expertise that is usual and customary in this community of professional engineers. I understand that City of Port Orchard does not and will not assume liability for the sufficiency, suitability or performance of drainage facilities prepared by Contour Engineering LLC. This analysis is based on data and records either supplied to, or obtained by, Contour Engineering, LLC. These documents are referenced within the text of the analysis. The analysis has been prepared utilizing procedures and practices within the standard accepted practices of the industry.*

PW20-014

TABLE OF CONTENTS

1.0 PROJECT OVERVIEW

Purpose and Scope

This Stormwater Site Plan accompanies the site development plans for a newly proposed multifamily apartment complex consisting of 98 apartment units. The site is comprised of multiple Kitsap County parcels (see Section 3.0 – Existing Conditions for a list of parcel and parcel areas). The site is located in the City of Port Orchard, WA. North of Mile Hill Drive and East of Highway 16, in a portion of Section 30, Township 24 North, Range 2 East, W.M.

The *2012 Storm Water Management Manual for Western Washington amended in 2014* (2014 DOE SWMM) and the requirements of the City of Port Orchard will establish the methodology and design criteria used for this project.

Project Description

This project will propose the construction of six residential buildings and a community center consisting of a total of 107 dwelling units along with the required infrastructure such as roads, sewer, water, and stormwater facilities. There does not appear to be any significant utilities that run through the subject site, and no apparent evidence of a previous residence(s).

This project exceeds the thresholds within Volume I Section 2.4 of the SWMM, therefor Minimum requirements #1 through #10 will apply to this project.

2.0 STORMWATER SYSTEM SUMMARY

The proposed stormwater system has been designed to meet the minimum requirements of 2014 DOE SWMM, City of Port Orchard, and the Department of Ecology (DOE).

The drainage basin consists of the project site with some run-on from adjacent sites. Stormwater runoff will be directed to the infiltration systems located under the pavement where 100% of stormwater runoff will be infiltrated. On-site grades are shown to direct all stormwater to the middle of the roadway where possible and collected by the proposed conveyance system.

3.0 EXISTING CONDITIONS SUMMARY

Pre-Developed Site Conditions

The project consists of multiple vacant Kitsap County parcels. The site appears to be a vacant parcel with no obvious signs of residence in the past. The site is comprised mostly of small vegetation with some trees in the southeast corner of the site.

Parcel #:	4598-005-028-0007	19,602 SF (0.45 Ac)
	4598-005-022-0003:	23,522 SF (0.54 AC)
	4598-005-017-0109:	23,522 SF (0.54 AC)
	4598-006-001-0303:	31,363 SF (0.72 AC)
	4598-006-004-0003:	23,522 SF (0.54 AC)
	4598-006-007-0000:	23,522 SF (0.54 AC)
	4598-006-010-0104:	7,841 SF (0.18 AC)
Governing Jurisdiction:	City of Port Orchard	
Zoning:	Residential – 12 units/acre	

Topography, Ground Cover and Native Soils

The site has significant grade drop across the parcels, approximately 80 feet of vertical relief. Stormwater runoff generated from the site generally flows west onto nearby developed parcels.

The majority of the site appears to be covered with light vegetation consisting mostly of grass, shrubs, and bushes. The south east and north west corners of the site have a group of trees. Native soils appear have roughly six to 18 inches of forest duff/topsoil underlain by iron stained sand or sand with gravel.

For additional soils information see Appendix C for a geotechnical report.

Adjacent Land Uses

The site is adjacent to the following:

North – ROW (Orlando Street)
East –Singly family dwellings
South – Single Family Dwellings
West – Multi Family Development

Drainage Patterns

Due to the soil conditions of the site, it appears a large portion of the runoff from the site is infiltrated. Any runoff which is not infiltrated will sheet flow down the hill onto the neighboring properties to the west.

Critical and Sensitive Areas

No wetlands or wetland buffers are located on or adjacent to the site. Due to the steep slopes located on site, there is a building setback required from the top of slopes. The building setback from top of slope shall be 10′ minimum which may be reduced through the use of extended foundations. See Geotechnical report for additional details.

4.0 DOWNSTREAM ANALYSIS (QUALITATIVE ANALYSIS)

Runoff appears to be mostly infiltrated within the site. Any runoff which does not infiltrate flows to the west onto developed parcels. Stormwater leaving the site will sheet flow to the parking area located on parcel 302402-3-059-2004 where it would be collected and infiltrated on site.

No erosional or flooding concerns were encountered during the downstream analysis.

5.0 STORMWATER CONTROLS

Developed Site Hydrology

The runoff from developed areas of the site will be collected and conveyed to the infiltration systems on site.

Project Basin

Impervious	= 3.35 Acres
Lawn/Landscape	= 1.42 Acres
Total	= 4.77 Acres

Flow Control Facilities

As the site is proposing more than 10,000 square feet of impervious, Flow Control is required. The site will meet flow control requirements through infiltrating 100% of onsite runoff. The infiltration systems have been designed to collect the entire sites runoff and infiltrate 100% of all runoff. The site has been split into three systems and are described below.

Offsite Basin

The offsite basin consists of the improvements made to SE Lovell Street and Wendell Ave leading into the site. Stormwater will be collected in a series of catch basins before discharging to the underground Infiltration Tank located within Wendell Ave right of way.

South Basin

The South Basin consists of the onsite improvements from the south and east portions of the site, including runoff the community center buildings, Building B, Building C and the east half of Building A. Runoff from these areas will be collected and conveyed to the infiltration system south of Building C in a CMP infiltration .

North Basin
The North Basin consists of the onsite improvements from the north and west portions of the site, including runoff from Building D, Building E, Building F, and the east half of Building A. The north basin has been sized to include additional runoff from future development of Phase 3. Runoff from these areas will be collected and conveyed to the infiltration system located between Building A and Building C in a CMP infiltration tank.

Flow Control Basins Table			
	Offsite Basin	South Basin	North Basin
Impervious (Ac)	0.55	1.47	1.90
Pervious (Ac)	0.11	0.74	0.42
Total (Ac)	0.66	2.21	2.32

Infiltration Design Table			
	Offsite Basin	South Basin	North Basin
Infiltration Rate	4"/hr	10.0"/hr	4.25"/hr
Tank Diameter (LF)	6	6	6
Tank Length (Total)	140	108	324
Bottom of Bed Size (Sq Ft)	1,440	1,120	2,910
Tank Storage (CF)	3,958	3,053	9,161
Gravel Storage (35% Void)	5,116	14,601	13,404
Total Storage Proposed (CF)	9,074	17,654	22,564
Total Storage Required (CF)	5,227		19,730
Facility Bottom Elevation	303.0	286.0	281.50

Detailed calculations for each flow control system can be found within Appendix D. Basin Maps outlining the boundary for each basin can be found in Appendix A. A geotechnical report detailing the infiltration testing and rates can be found in Appendix C.

Water Quality System

Since this project will add more than 5,000 square feet of pollution-generating hard surface (PGHS), water quality treatment is required. Water Quality Treatment will be provided through a filter system placed prior to each of the infiltration systems. A Pretreatment Device will also be installed prior to the filter cartridge system for each basin. Roof runoff from each building will be collected and conveyed through a separate "clean" line which does not need to be treated and will bypass the water quality treatment device. Below is a summary of the treatment basins and treatment rates.

Water Quality Basins Table			
	Offsite Basin	South Basin	North Basin
Impervious (Ac)	0.55	1.09	1.34
Pervious (Ac)	0.11	0.74	0.42
Total (Ac)	0.66	1.83	1.76

Treatment Table			
	Offsite Basin	South Basin	North Basin
Treatment Required (CFS)	0.0897	0.1759	0.2115
Treatment Provided Per Cartridge (CFS)	0.0418	0.0418	0.0418
Number of Cartridges Proposed	3 27"	5 27"	6 27"
Total Treatment Provided (CFS)	0.1254	0.2090	0.2508

See Appendix D for detailed water quality calculations.

Conveyance System

On site storm has been analyzed to ensure all conveyance pipes can pass the 100 year storm event. A Summary of the various conveyance systems is provided below. See Appendix E for full sizing calculations.

6" Conveyance Line (Roof)
Maximum Discharge Proposed = 0.56 CFS
Maximum Discharge Available = 0.57 CFS
Percent Full = 89.6%

12" Conveyance Line (Onsite)
Maximum Discharge Proposed = 2.16 CFS
Maximum Discharge Available = 2.94 CFS
Percent Full = 67.1%

12" Conveyance Line (Offsite)
Maximum Discharge Proposed = 0.82 CFS
Maximum Discharge Available = 2.94 CFS
Percent Full = 37.6%

6.0 Discussion of Minimum Requirements

The Minimum requirements for this project are set forth by the SWMM. Minimum requirements #1 through #10 apply to this project and are discussed below:

#1 - Preparation of Stormwater Site Plans

This storm water site plan satisfies this requirement

#2 - Construction Stormwater Pollution Prevention Plan (SWPPP)

A Construction Stormwater Pollution Prevention Plan has been included with this submittal.

#3 - Source Control of Pollution

Applicable Source Control BMPs will be employed as needed. A comprehensive list of BMPs can be found in the 2014 Stormwater Management Manual for Western Washington, Volume IV as well as within the Operation and Maintenance (O&M) Manual. Construction BMPs will also be employed as needed and are located within the Construction SWPPP for the project.

#4 - Preservation of Natural Drainage Systems and Outfalls

The natural drainage path will be maintained to the maximum extent feasible. All stormwater from the site will either be infiltrated.

#5 - On-site Stormwater Management

The project site will meet the LID Standards. LID Standards will be met by infiltrating 100% of stormwater runoff.

#6 – Runoff Treatment

Water Quality Treatment will be provided through filter cartridges placed upstream of each infiltration system. See section 5.0 for additional details.

#7 – Flow Control

Flow Control will be provided through the use of infiltration. See Section 5.0 for additional details.

#8 – Wetlands Protection

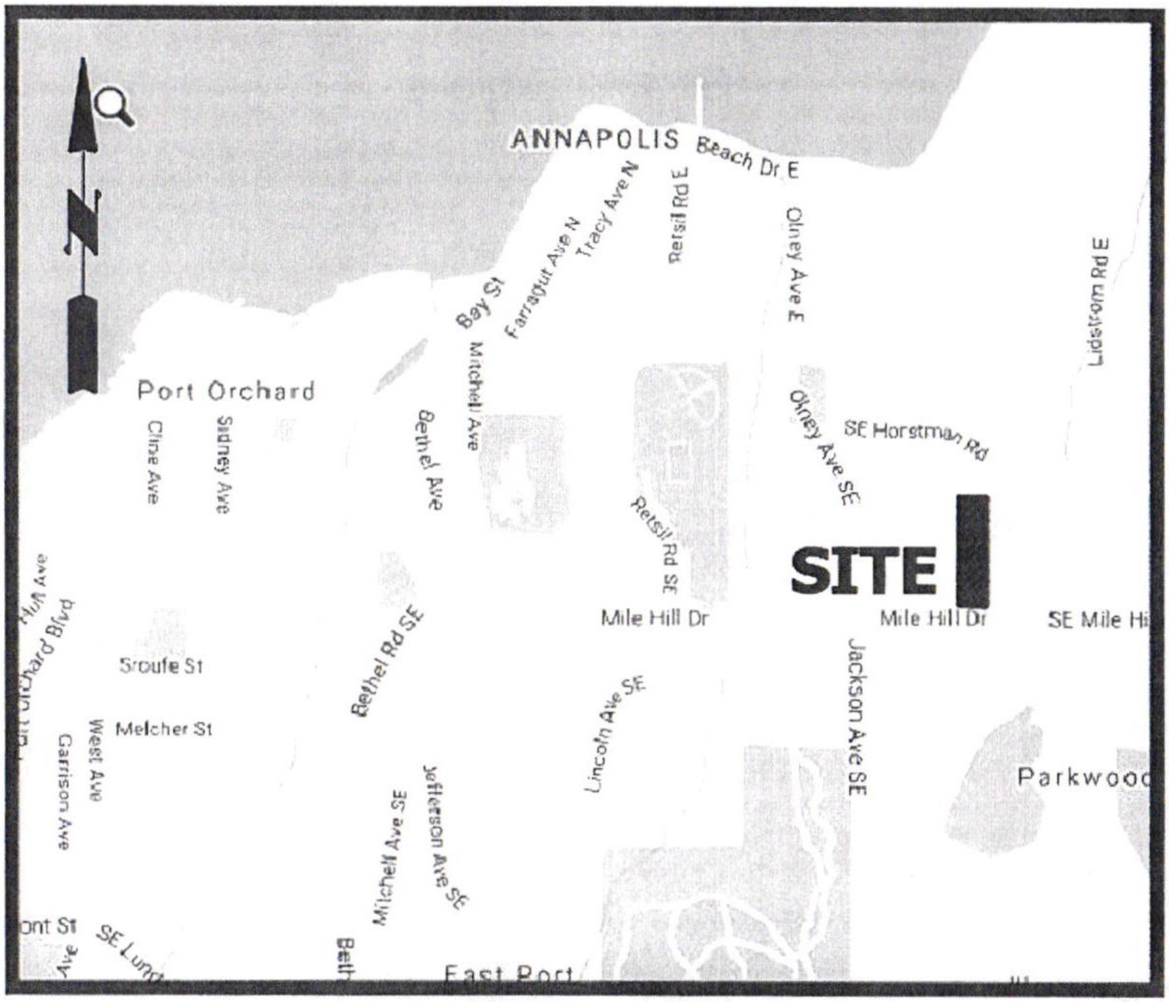
No wetlands or wetland buffers are located on or adjacent to the site.

#9 – Operations and Maintenance

An Operations and Maintenance (O&M) Manual has been included with this submittal package.

APPENDIX A

General Exhibits



VICINITY MAP

NOT TO SCALE

OVERLOOK APARTMENTS PHASE II

A PORTION OF THE SE 1/4 OF THE SW 1/4 OF SECTION 30, TOWNSHIP 24 NORTH, RANGE 2 EAST W.M.
CITY OF PORT ORCHARD, KITSAP COUNTY, WASHINGTON

GRAPHIC SCALE
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1 INCH = 30 FEET (22"x34")
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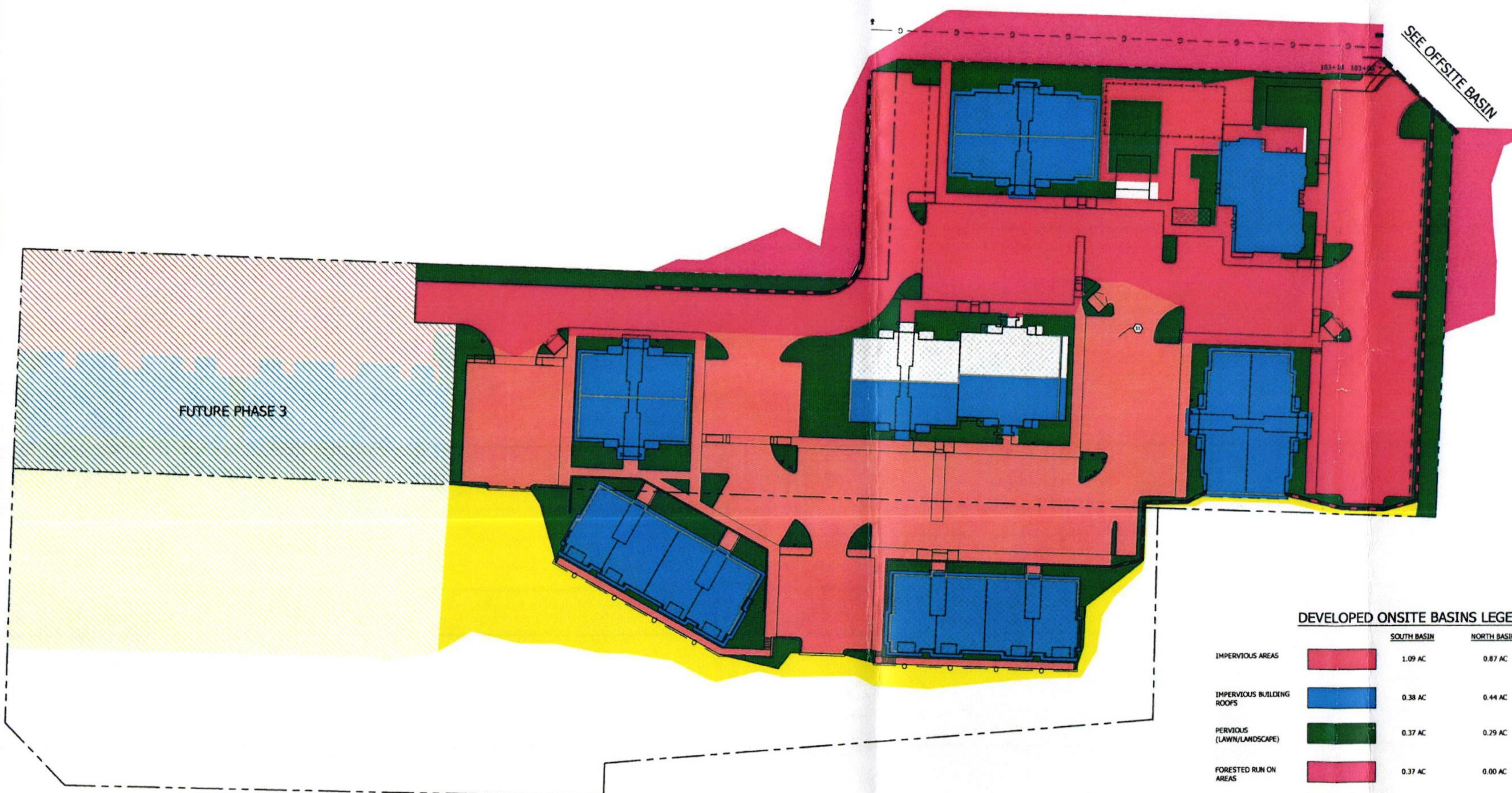

DEVELOPED ONSITE BASINS LEGEND

	SOUTH BASIN	NORTH BASIN	ONSITE TOTAL
IMPERVIOUS AREAS	1.09 AC	0.87 AC	1.953 AC
IMPERVIOUS BUILDING ROOFS	0.38 AC	0.44 AC	0.819 AC
PERVIOUS (LAWN/LANDSCAPE)	0.37 AC	0.29 AC	0.645 AC
FORESTED RUN ON AREAS	0.37 AC	0.00 AC	0.300 AC
UNCONTROLLED DRAINAGE TO OFFSITE	0.011 AC	0.408 AC	0.419 AC
TOTAL	2.21 AC	2.32 AC	4.53 AC

REVISION	DESCRIPTION	DATE	BY

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SHEET TITLE: DEVELOPED ONSITE BASIN MAP

OVERLOOK APARTMENTS PHASE II

CLIENT: OLAA, LLC
15234 SE 366TH PLACE
AUBURN, WA 98092

CONTACT: MARK NORDLIE PHONE: (253) 255-7624

DESIGNER: K. ROSE
ENGINEER: J. HAUG
DRAWN: B. MORRIS
S 30 T 24N R 2E WM
DATE: 2020.02.14
REVISED:

PROJECT: 16-300
DWG NAME: 16-300-C

SHEET	REV.
OF 27	0

OVERLOOK APARTMENTS PHASE II

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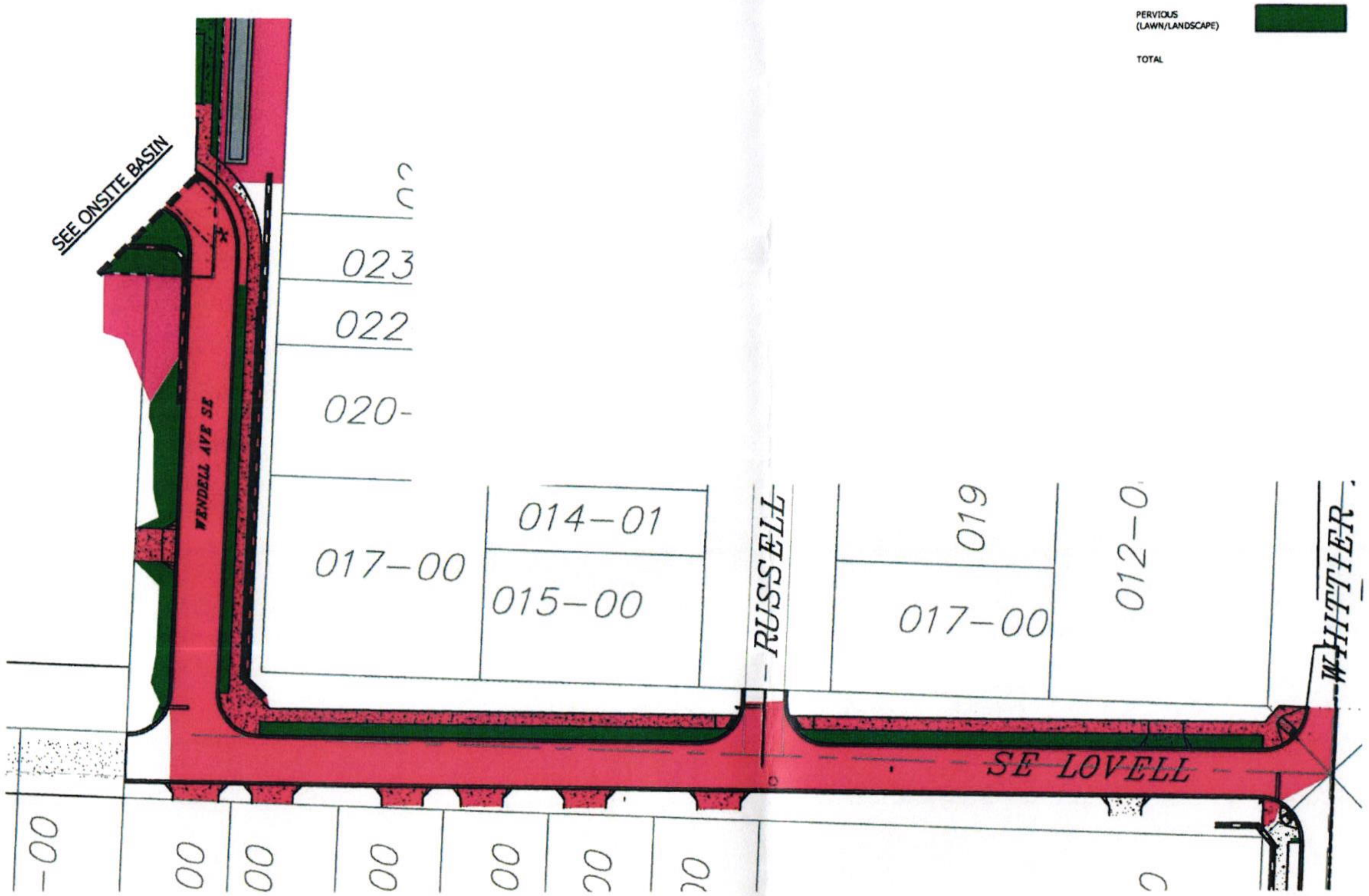

DEVELOPED OFFSITE BASINS LEGEND

	OFFSITE BASIN #1
IMPERVIOUS AREAS	0.55 AC
PERVIOUS (LAWN/LANDSCAPE)	0.11 AC
TOTAL	0.66 AC

REVISION	DESCRIPTION	DATE	BY

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APPENDIX B

Plan Exhibits

GRAPHIC SCALE
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1 INCH = 20 FEET (24"x36")

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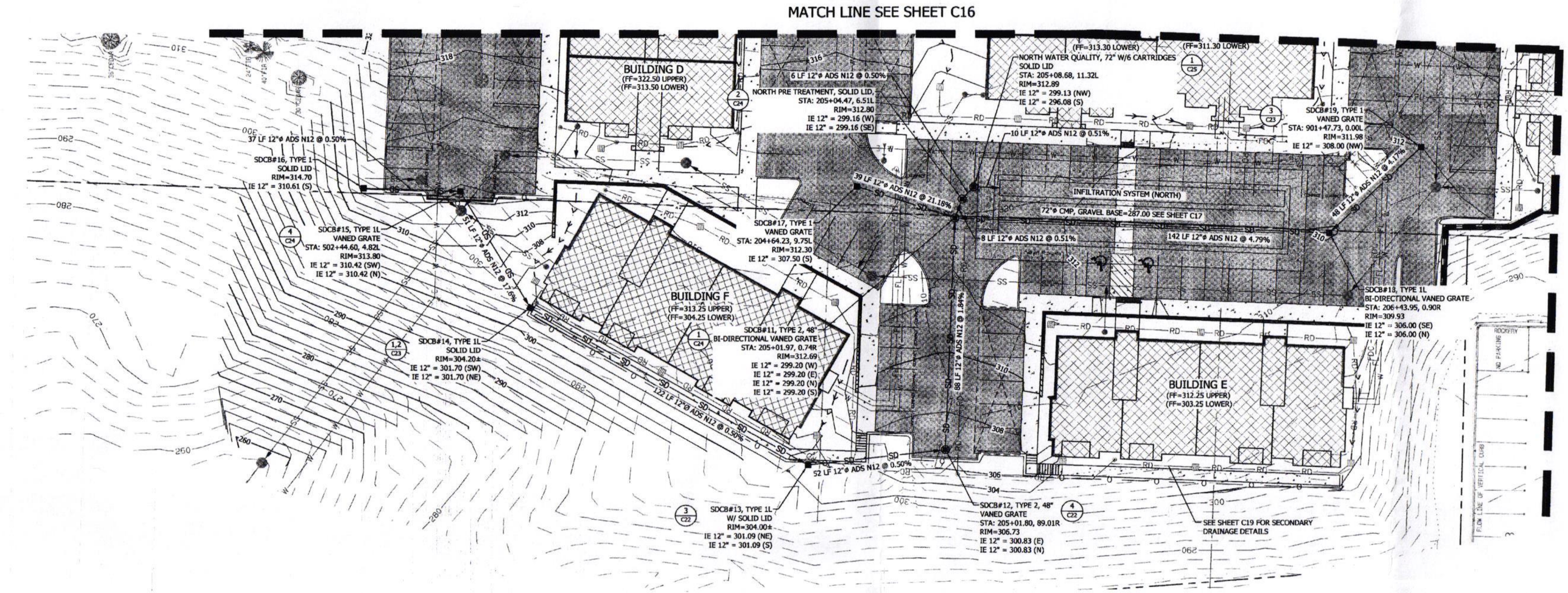

CITY OF PORT ORCHARD STANDARD DRAINAGE NOTES

1. ALL STORM PIPE AND APPURTENANCES SHALL BE LAID IN ACCORDANCE WITH CITY OF PORT ORCHARD DESIGN AND CONSTRUCTION STANDARDS. THIS SHALL INCLUDE LEVELING AND COMPACTING THE TRENCH BOTTOM, THE TOP OF THE FOUNDATION MATERIAL AND ANY REQUIRED BEDDING TO A UNIFORM GRADE SO THAT THE ENTIRE DRAINAGE FACILITY IS SUPPORTED BY A UNIFORMLY DENSE UNYIELDING BASE.
2. ALL STORM PIPE SHALL BE SUBJECT TO A LOW PRESSURE AIR TEST IN ACCORDANCE WITH WSDOT STANDARD SPECIFICATION 7-04.3(1)F AND A VIDEO INSPECTION IN ACCORDANCE WITH THE PORT ORCHARD DESIGN STANDARDS.
3. STORM PIPE COVER, MEASURED FROM THE FINISHED GRADE ELEVATION TO THE TOP OF THE OUTSIDE SURFACE OF THE PIPE, SHALL BE 2 FEET MINIMUM (3 FEET FOR PVC), UNLESS AUTHORIZED BY THE CITY OF PORT ORCHARD CITY ENGINEER UNDER THE FOLLOWING CIRCUMSTANCES: A. UNDER DRAINAGE EASEMENTS, DRIVEWAYS, PARKING STALLS, OR OTHER AREAS SUBJECT TO LIGHT VEHICULAR LOADING, THE PIPE COVER MAY BE REDUCED TO 1 FOOT MINIMUM IF THE COVER IS CONSISTENT WITH THE MANUFACTURER'S RECOMMENDATIONS. B. IN AREAS NOT SUBJECT TO VEHICULAR LOADS, SUCH AS LANDSCAPE PLANTERS AND YARDS, THE PIPE COVER MAY BE REDUCED TO 1 FOOT MINIMUM. C. IF DUCTILE IRON PIPE IS USED, THE PIPE COVER MAY BE REDUCED TO 1 FOOT MINIMUM.
4. STEEL PIPE SHALL BE GALVANIZED AND HAVE ASPHALT TREATMENT #1 OR BETTER INSIDE AND OUT (WSDOT STANDARD SPECIFICATION 9-05.4(3)).
5. ANY DRAINAGE STRUCTURE, SUCH AS A CATCH BASIN OR A MANHOLE, NOT RECEIVING SURFACE RUNOFF AND NOT LOCATED WITHIN A TRAVELED ROADWAY OR SIDEWALK SHALL HAVE A SOLID LOCKING LID. ANY DRAINAGE STRUCTURE ASSOCIATED WITH A PERMANENT RETENTION/DETENTION FACILITY, NOT RECEIVING SURFACE RUNOFF, SHALL HAVE A SOLID LOCKING LID.
6. ALL CATCH BASIN GRATES SHALL CONFORM TO THE 2005 DEPARTMENT OF ECOLOGY STORMWATER MANAGEMENT MANUAL FOR WESTERN WASHINGTON AND THE WSDOT STANDARD PLANS WHEN LOCATED WITHIN THE RIGHT-OF-WAY, AND SHALL INCLUDE A COMBINATION INLET FRAME (OPEN CURB FACE FRAME), WHEN LOCATED IN A SUMP CONDITION OR BEFORE AN INTERSECTION WITH A 4% GRADE OR ABOVE. A HERRINGBONE GRATE MAY BE USED OUTSIDE THE RIGHT-OF-WAY. ALL CATCH BASINS WITHIN THE GUTTER LINE SHALL BE INSTALLED IN ACCORDANCE WITH STANDARD DETAILS AS APPLICABLE. MAXIMUM CATCH BASIN HEIGHT FROM FINISHED GRADE TO PIPE INVERT SHALL BE PER THE APPLICABLE DETAIL.
7. FOR ANY CURB GRADE LESS THAN 0.8% (0.0080 FT/FT), INCLUDING CURB RETURNS, A PROFESSIONAL LAND SURVEYOR, CURRENTLY LICENSED IN THE STATE OF WASHINGTON, SHALL VERIFY THAT THE CURB FORMS OR STRING LINES ARE AT THE GRADES NOTED ON THE APPROVED PLANS PRIOR TO PLACEMENT OF CONCRETE. THE CONTRACTOR IS RESPONSIBLE FOR SURVEY COORDINATION AND COSTS.
8. FOR ANY DRAINAGE PIPE GRADE LESS THAN 0.5% (0.0050 FT/FT), A PROFESSIONAL LAND SURVEYOR, CURRENTLY LICENSED IN THE STATE OF WASHINGTON, SHALL VERIFY THAT THE AS-BUILT PIPE MATCHES THE GRADES NOTED ON THE APPROVED PLANS PRIOR TO COMPLETION OF SUBGRADE. THE CONTRACTOR IS RESPONSIBLE FOR SURVEY COORDINATION AND COSTS.
9. ALL DRIVEWAY CULVERTS LOCATED WITHIN THE CITY OF PORT ORCHARD RIGHT-OF-WAY SHALL BE OF SUFFICIENT LENGTH TO PROVIDE A MINIMUM 3:1 SLOPE FROM THE EDGE OF THE DRIVEWAY TO THE BOTTOM OF THE DITCH. CULVERTS SHALL HAVE BEVELED END SECTIONS TO MATCH THE SIDE SLOPE.
10. ROCK FOR EROSION PROTECTION OF DITCHES, WHERE REQUIRED, MUST BE OF SOUND QUARRY ROCK, PLACED TO A DEPTH OF ONE FOOT (1'), AND MUST MEET THE FOLLOWING SPECIFICATIONS: 100% MUST PASS THE 8" SIEVE, 40% MAXIMUM CAN PASS THE 3" SIEVE AND 10% MAXIMUM CAN PASS THE 3/4" SIEVE.
11. DRAINAGE OUTLETS (STUB-OUTS) SHALL BE PROVIDED FOR EACH INDIVIDUAL LOT, EXCEPT FOR THOSE LOTS APPROVED FOR INFILTRATION BY THE CITY OF PORT ORCHARD. STUB-OUTS SHALL CONFORM TO THE FOLLOWING:
    a) EACH OUTLET SHALL BE SUITABLY LOCATED AT THE LOWEST ELEVATION ON THE LOT TO SERVICE ALL FUTURE ROOF DOWNSPOUTS AND FOOTING DRAINS, DRIVEWAYS, YARD DRAINS, AND ANY OTHER SURFACE OR SUBSURFACE DRAINS NECESSARY TO RENDER THE LOTS SUITABLE FOR THEIR INTENDED USE. EACH OUTLET SHALL HAVE FREE-FLOWING, POSITIVE DRAINAGE TO AN APPROVED STORMWATER CONVEYANCE SYSTEM OR TO AN APPROVED OUTFALL LOCATION.
    b) OUTLETS ON EACH LOT SHALL BE LOCATED WITH A FIVE-FOOT-HIGH, 2" X 4" STAKE MARKED "STORM" OR "DRAIN". THE STUB-OUT SHALL EXTEND ABOVE SURFACE LEVEL, BE VISIBLE, AND BE SECURED TO THE STAKE.
    c) PIPE MATERIAL SHALL BE IN ACCORDANCE WITH PORT ORCHARD DESIGN STANDARDS. IF NONMETALLIC, THE PIPE SHALL CONTAIN A WIRE OR USE OTHER ACCEPTABLE MEANS OF DETECTION.
    d) DRAINAGE EASEMENTS ARE REQUIRED FOR DRAINAGE SYSTEMS DESIGNED TO CONVEY FLOWS THROUGH INDIVIDUAL LOTS.
    e) THE APPLICANT/CONTRACTOR IS RESPONSIBLE FOR COORDINATING THE LOCATIONS OF ALL STUB-OUT CONVEYANCE LINES WITH RESPECT TO OTHER UTILITIES (E.G., POWER, GAS, TELEPHONE, TELEVISION, ETC.).
    f) ALL INDIVIDUAL STUB-OUTS SHALL BE PRIVATELY OWNED AND MAINTAINED BY THE LOT HOMEOWNER.

STORM DRAINAGE NOTES

1. STORM DRAINAGE CONVEYANCE SHALL BE INSTALLED PER CITY STANDARDS AND PER CITY SPECIFICATIONS.
2. ALL STORM DRAIN MAINS SHALL BE A MINIMUM 12"Ø HAVING A MINIMUM SLOPE OF 0.50% UNLESS OTHERWISE SPECIFIED ON PLANS OR APPROVED BY THE CITY.
3. PIPE SHALL HAVE A MINIMUM COVER OF 1.0' IN NON-TRAFFIC BEARING AREAS AND 2.0' IN TRAFFIC BEARING AREAS UNLESS OTHERWISE SPECIFIED ON PLANS.
4. PIPE SHALL BE ADS N-12 OR APPROVED EQUIVALENT UNLESS COVER IS LESS THAN 2.0' IN TRAFFIC BEARING AREAS. IN THIS CASE, DUCTILE IRON PIPE OR APPROVED EQUIVALENT SHALL BE USED PER ALLOWED CITY STANDARDS.
5. SEE DETAIL 5 ON SHEET C22 FOR PIPE BEDDING REQUIREMENTS.

VERIFICATION NOTE

ALL EXISTING UTILITIES IN THE CONSTRUCTION AREA SHALL BE IDENTIFIED AND VERIFIED FOR DEPTH AND LOCATION PRIOR TO ANY CONSTRUCTION ACTIVITIES SO TO IDENTIFY ANY POTENTIAL CONFLICTS WITH PROPOSED CONSTRUCTION. CONTACT PROJECT ENGINEER IMMEDIATELY IF ANY CONFLICTS ARE IDENTIFIED.

PRIOR TO ANY CONSTRUCTION ACTIVITIES, VERIFY EXISTING TOPOGRAPHY IS CONSISTENT WITH WHAT IS SHOWN ON PLANS AND IF THERE ARE ANY POTENTIAL CONFLICTS WITH PROPOSED CONSTRUCTION ACTIVITIES. CONTACT PROJECT ENGINEER IMMEDIATELY IF ANY CONFLICTS ARE IDENTIFIED.

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JEREMY F. HAUG
STATE OF WASHINGTON
PROFESSIONAL ENGINEER

SHEET TITLE: DRAINAGE PLAN 1 (NORTH)

OVERLOOK APARTMENTS PHASE II

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S 30 T 24N R 2E WM
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OVERLOOK APARTMENTS PHASE II

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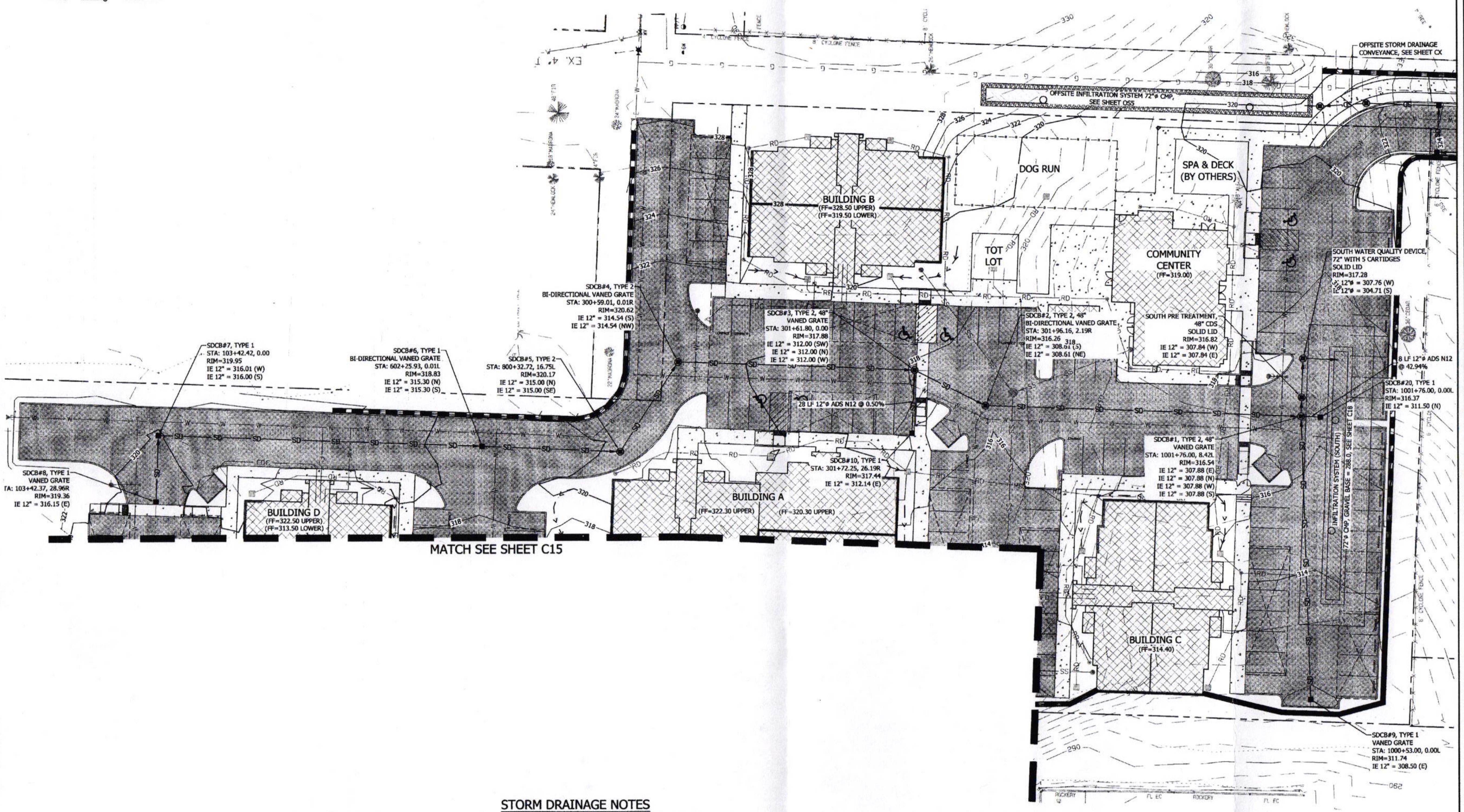

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JEREMY F. HAUG, STATE OF WASHINGTON, REGISTERED PROFESSIONAL ENGINEER

SHEET TITLE: DRAINAGE PLAN 2 (SOUTH)

OVERLOOK APARTMENTS PHASE II

CLIENT: OLAA, LLC
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OVERLOOK APARTMENTS PHASE II

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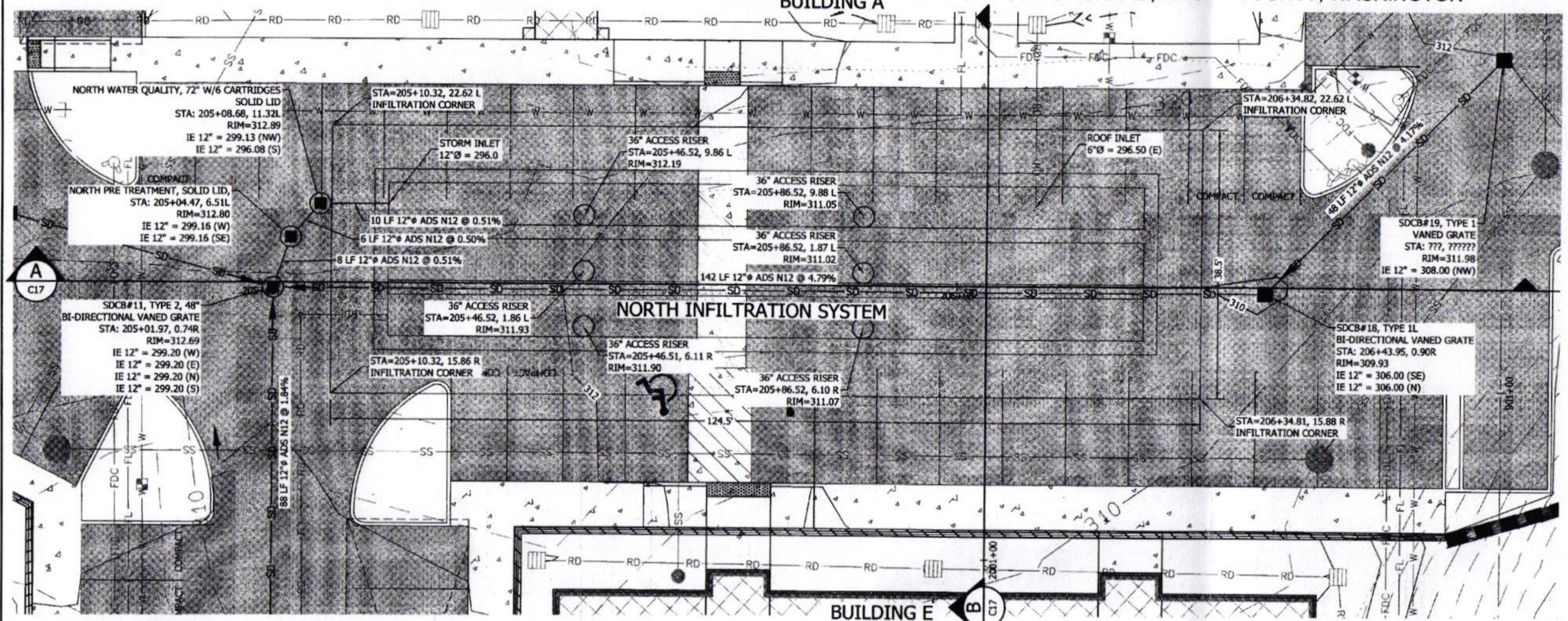
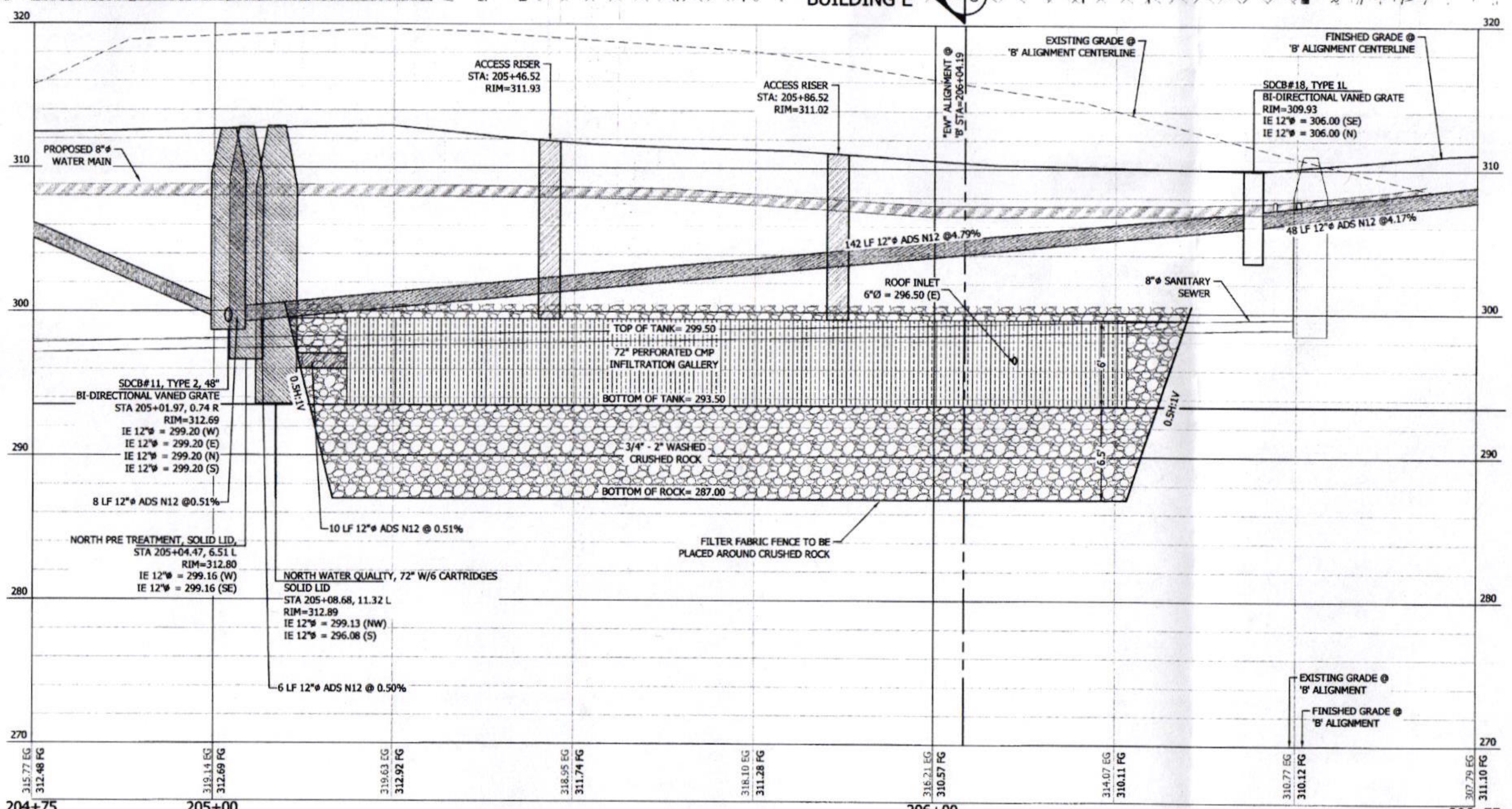
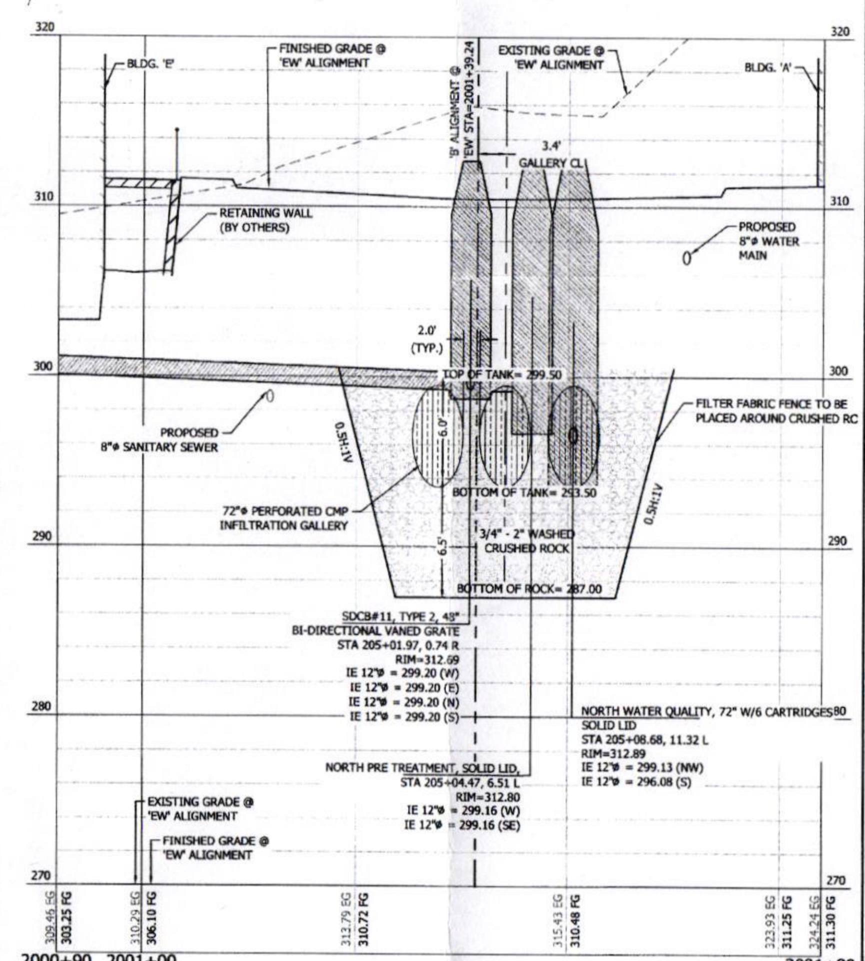
GRAPHIC SCALE
1 INCH = 10 FEET (24"x36")





(A) INFILTRATION SYSTEM 1 PROFILE
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=5'



(B) INFILTRATION SYSTEM 1 SECTION
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=5'

VERIFICATION NOTE

ALL EXISTING UTILITIES IN THE CONSTRUCTION AREA SHALL BE IDENTIFIED AND VERIFIED FOR DEPTH AND LOCATION PRIOR TO ANY CONSTRUCTION ACTIVITIES SO TO IDENTIFY ANY POTENTIAL CONFLICTS WITH PROPOSED CONSTRUCTION. CONTACT PROJECT ENGINEER IMMEDIATELY IF ANY CONFLICTS ARE IDENTIFIED.

PRIOR TO ANY CONSTRUCTION ACTIVITIES, VERIFY EXISTING TOPOGRAPHY IS CONSISTENT WITH WHAT IS SHOWN ON PLANS AND IF THERE ARE ANY POTENTIAL CONFLICTS WITH PROPOSED CONSTRUCTION ACTIVITIES. CONTACT PROJECT ENGINEER IMMEDIATELY IF ANY CONFLICTS ARE IDENTIFIED.

CALL 811 AT LEAST 48 HOURS BEFORE YOU DIG

DRAINAGE NOTE

BEFORE INSTALLATION OF THE INFILTRATION SYSTEM, THE GEOTECHNICAL ENGINEER SHALL INSPECT THE UNDERLYING SOILS AND DETERMINE THE FINAL DEPTH OF THE GRAVEL BED. IF SOILS HAVE DIFFERING DESIGN CRITERIA FROM THOSE FOUND DURING SOIL INVESTIGATIONS NOTIFY PROJECT ENGINEER.

INFILTRATION SYSTEM NOTE

PIPE DIAMETER:	6.0'
PIPE LENGTH:	108 LF x3
GRAVEL BASE AREA:	2,910 SF
GRAVEL TOP AREA:	4,792 SF
TANK STORAGE:	9,161 CF
GRAVEL STORAGE:	13,404 CF
TOTAL STORAGE:	22,564 CF (0.518 AC-FT)
POROSITY:	35%
INFILTRATION RATE:	4.25"/HR
BOTTOM OF GRAVEL ELEV:	287.0
BOTTOM OF TANK ELEV:	281.5

REVISION	DESCRIPTION	DATE	BY

CONTOUR ENGINEERING · LLC
CIVIL ENGINEERS~SURVEYORS~LAND PLANNERS
3309 56th Street NW, Suite 106 Gig Harbor, WA 98335
Phone: 253-857-5454 ~ Fax: 253-509-0044 ~ info@contourpllc.com

JEREMY F. HAUG — STATE OF WASHINGTON — REGISTERED PROFESSIONAL ENGINEER — 2.14.20

SHEET TITLE: NORTH INFILTRATION SYSTEM PLAN AND PROFILE
OVERLOOK APARTMENTS PHASE II

CLIENT: OLAA, LLC
15234 SE 366TH PLACE
AUBURN, WA 98092
CONTACT: MARK NORDLIE
PHONE: (253) 255-7624

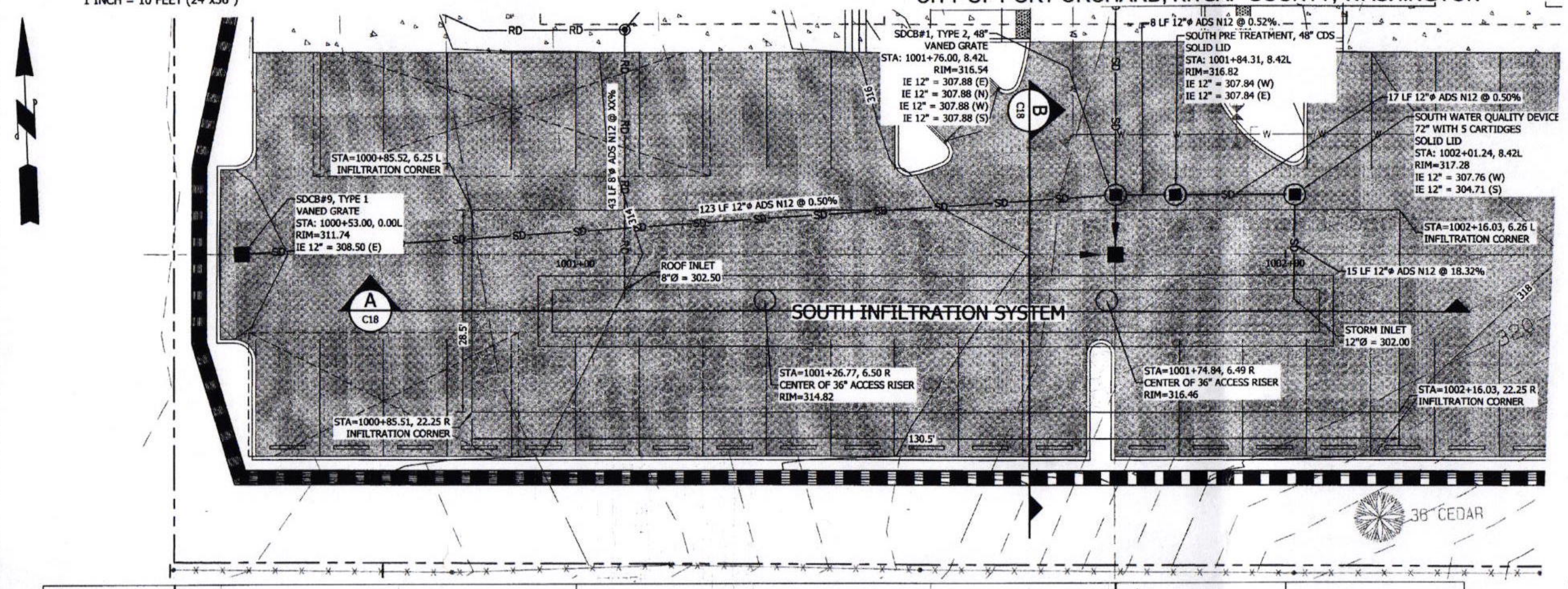
DESIGNER: K. ROSE
ENGINEER: J. HAUG
DRAWN: B. MORRIS
S 30 T 24N R 2E WM
DATE: 2020.02.14
REVISED:
PROJECT: 16-300
DWG NAME: 16-300-C

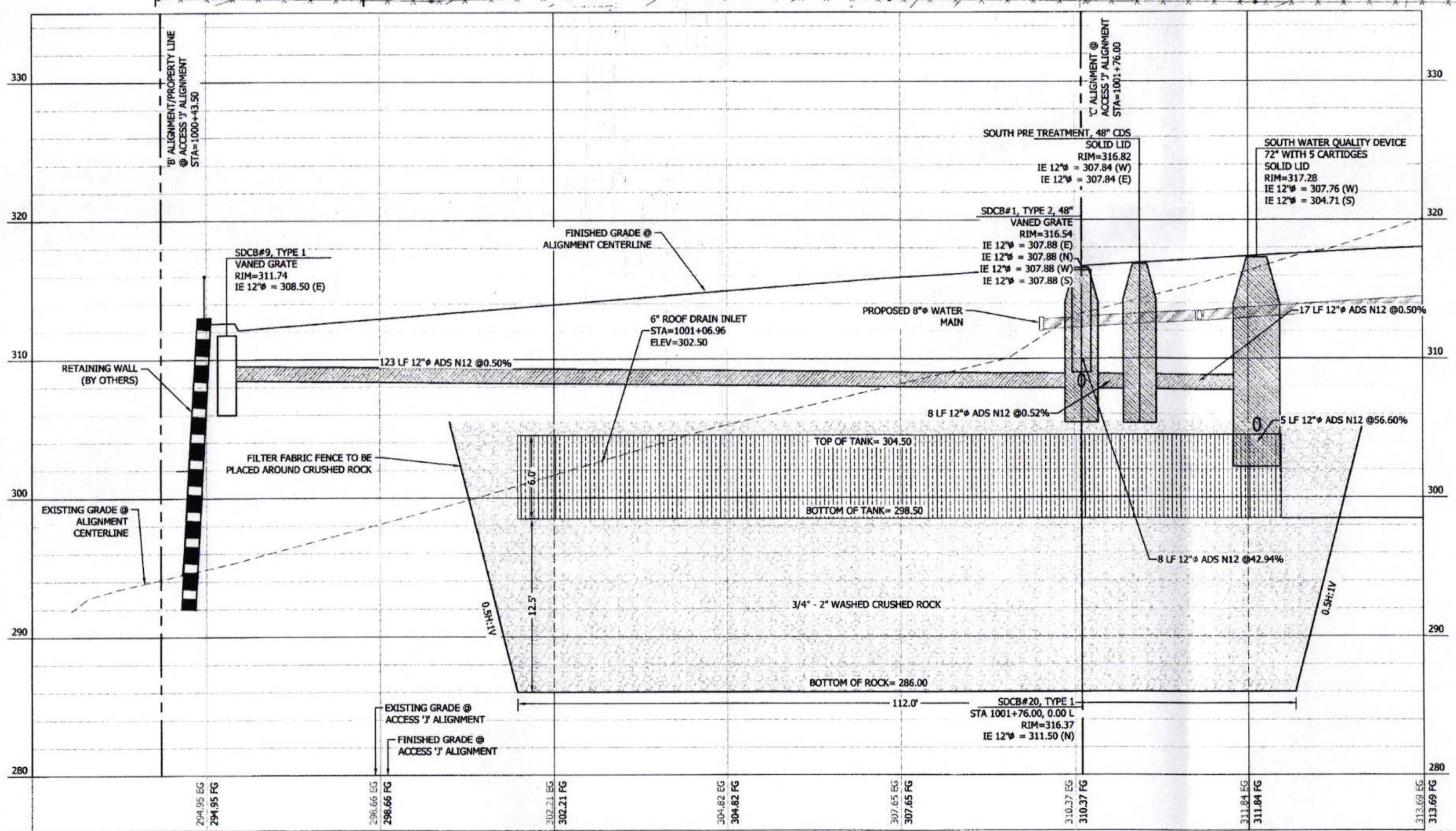
SHEET	REV.
C17	0

OVERLOOK APARTMENTS PHASE II

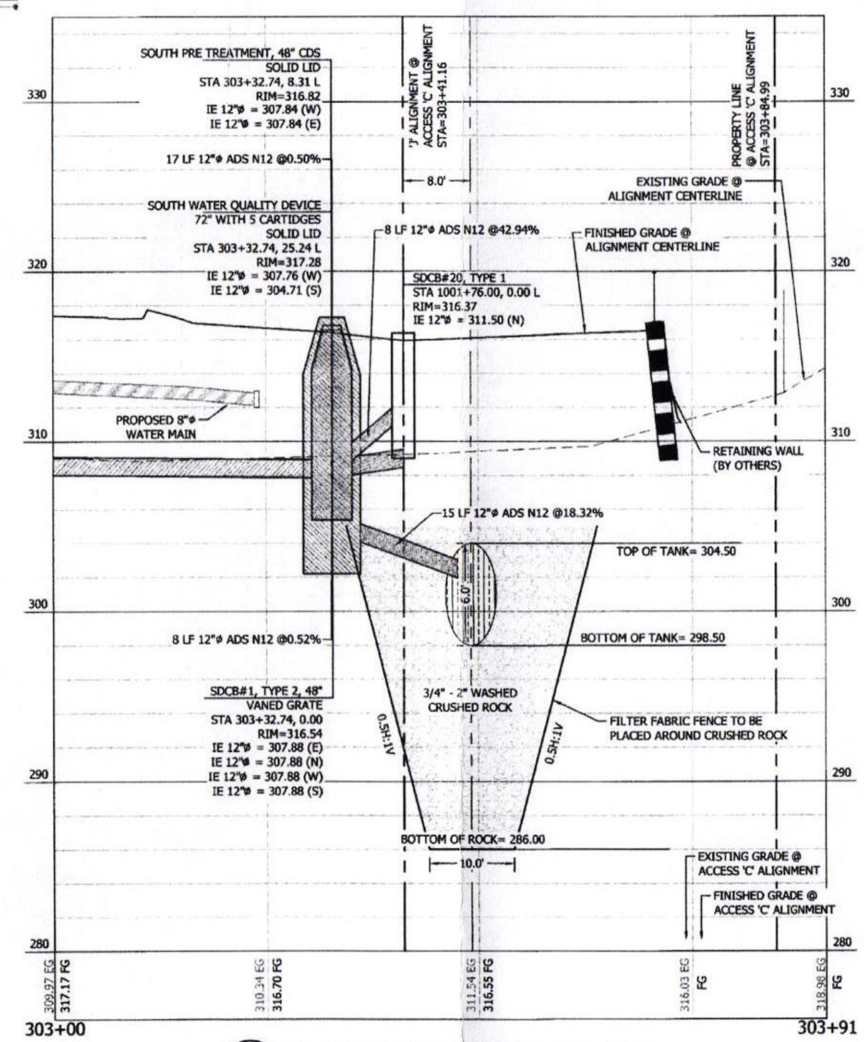
A PORTION OF THE SE 1/4 OF THE SW 1/4 OF SECTION 30, TOWNSHIP 24 NORTH, RANGE 2 EAST W.M.
CITY OF PORT ORCHARD, KITSAP COUNTY, WASHINGTON

GRAPHIC SCALE
0 10 20
1 INCH = 10 FEET (24"x36")





A INFILTRATION SYSTEM 2 PROFILE
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=5'



B INFILTRATION SYSTEM 2 SECTION
HORIZONTAL SCALE: 1"=10'
VERTICAL SCALE: 1"=5'

VERIFICATION NOTE

ALL EXISTING UTILITIES IN THE CONSTRUCTION AREA SHALL BE IDENTIFIED AND VERIFIED FOR DEPTH AND LOCATION PRIOR TO ANY CONSTRUCTION ACTIVITIES SO TO IDENTIFY ANY POTENTIAL CONFLICTS WITH PROPOSED CONSTRUCTION. CONTACT PROJECT ENGINEER IMMEDIATELY IF ANY CONFLICTS ARE IDENTIFIED.

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DRAINAGE NOTE

BEFORE INSTALLATION OF THE INFILTRATION SYSTEM, THE GEOTECHNICAL ENGINEER SHALL INSPECT THE UNDERLYING SOILS AND DETERMINE THE FINAL DEPTH OF THE GRAVEL BED. IF SOILS HAVE DIFFERING DESIGN CRITERIA FROM THOSE FOUND DURING SOIL INVESTIGATIONS NOTIFY PROJECT ENGINEER.

INFILTRATION SYSTEM NOTE

PIPE DIAMETER:	6.0'
PIPE LENGTH:	108 LF
GRAVEL BASE AREA:	1,120 SF
GRAVEL TOP AREA:	3,720 SF
TANK STORAGE:	3,053 CF
GRAVEL STORAGE:	14,601 CF
TOTAL STORAGE:	17,654 CF (0.405 AC-FT)
POROSITY:	35%
INFILTRATION RATE:	10"/HR
BOTTOM OF GRAVEL ELEV:	286.0
BOTTOM OF TANK ELEV:	298.5

REVISION	DESCRIPTION	DATE	BY

CONTOUR ENGINEERING · LLC
CIVIL ENGINEERS~SURVEYORS~LAND PLANNERS
3309 56th Street NW, Suite 106 Gig Harbor, WA 98335
Phone: 253-857-5454 ~ Fax: 253-509-0044 ~ info@contourpllc.com

JEREMY F. HAUG STATE OF WASHINGTON REGISTERED PROFESSIONAL ENGINEER 2.14.20

SHEET TITLE: SOUTH INFILTRATION SYSTEM PLAN AND PROFILE
OVERLOOK APARTMENTS PHASE II

CLIENT: OLAA, LLC
15234 SE 366TH PLACE
AUBURN, WA 98092
CONTACT: MARK NORDLIE PHONE: (253) 255-7624

DESIGNER: K. ROSE
ENGINEER: J. HAUG
DRAWN: B. MORRIS
S 30 T 24N R 2E WM
DATE: 2020.02.14
REVISED:
PROJECT: 16-300
DWG NAME: 16-300-C

OVERLOOK APARTMENTS PHASE II

A PORTION OF THE SE 1/4 OF THE SW 1/4 OF SECTION 30, TOWNSHIP 24 NORTH, RANGE 2 EAST W.M.
CITY OF PORT ORCHARD, KITSAP COUNTY, WASHINGTON

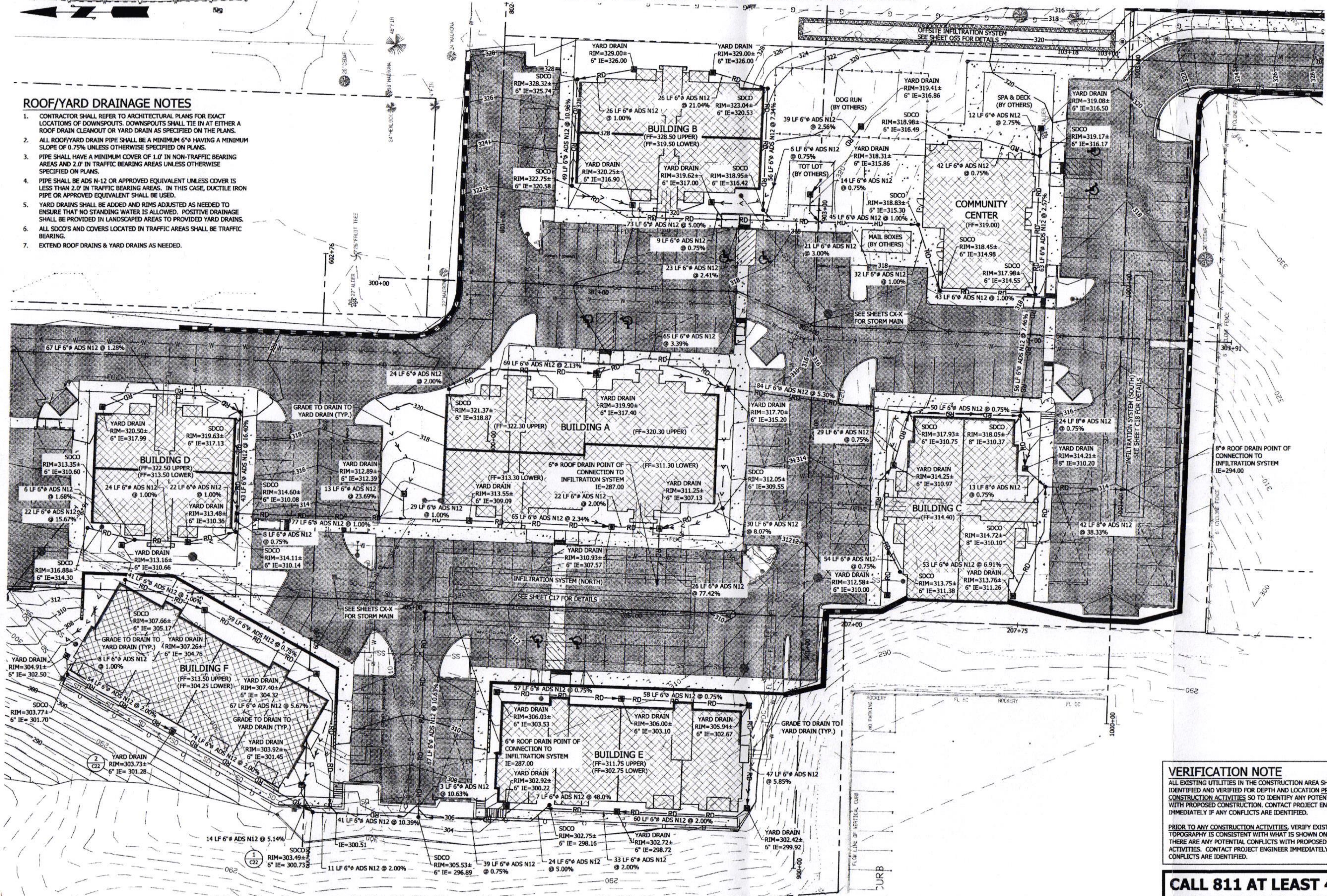
GRAPHIC SCALE

0 20 40

1 INCH = 20 FEET (24"x36")

ROOF/YARD DRAINAGE NOTES

1. CONTRACTOR SHALL REFER TO ARCHITECTURAL PLANS FOR EXACT LOCATIONS OF DOWNSPOUTS. DOWNSPOUTS SHALL TIE IN AT EITHER A ROOF DRAIN CLEANOUT OR YARD DRAIN AS SPECIFIED ON THE PLANS.
2. ALL ROOF/YARD DRAIN PIPE SHALL BE A MINIMUM 6"ø HAVING A MINIMUM SLOPE OF 0.75% UNLESS OTHERWISE SPECIFIED ON PLANS.
3. PIPE SHALL HAVE A MINIMUM COVER OF 1.0' IN NON-TRAFFIC BEARING AREAS AND 2.0' IN TRAFFIC BEARING AREAS UNLESS OTHERWISE SPECIFIED ON PLANS.
4. PIPE SHALL BE ADS N-12 OR APPROVED EQUIVALENT UNLESS COVER IS LESS THAN 2.0' IN TRAFFIC BEARING AREAS. IN THIS CASE, DUCTILE IRON PIPE OR APPROVED EQUIVALENT SHALL BE USED.
5. YARD DRAINS SHALL BE ADDED AND RIMS ADJUSTED AS NEEDED TO ENSURE THAT NO STANDING WATER IS ALLOWED. POSITIVE DRAINAGE SHALL BE PROVIDED IN LANDSCAPED AREAS TO PROVIDED YARD DRAINS.
6. ALL SDCO'S AND COVERS LOCATED IN TRAFFIC AREAS SHALL BE TRAFFIC BEARING.
7. EXTEND ROOF DRAINS & YARD DRAINS AS NEEDED.



VERIFICATION NOTE

ALL EXISTING UTILITIES IN THE CONSTRUCTION AREA SHALL BE IDENTIFIED AND VERIFIED FOR DEPTH AND LOCATION PRIOR TO ANY CONSTRUCTION ACTIVITIES SO TO IDENTIFY ANY POTENTIAL CONFLICTS WITH PROPOSED CONSTRUCTION. CONTACT PROJECT ENGINEER IMMEDIATELY IF ANY CONFLICTS ARE IDENTIFIED.

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CALL 811 AT LEAST 48 HOURS BEFORE YOU DIG

REVISION	DESCRIPTION	DATE	BY

CONTOUR ENGINEERING • LLC
CIVIL ENGINEERS~SURVEYORS~LAND PLANNERS
3309 56th Street NW, Suite 106 Gig Harbor, WA 98335
Phone: 253-857-5454 ~ Fax: 253-509-0044 ~ info@contourpllc.com

JEREMY F. HAUG
STATE OF WASHINGTON
REGISTERED PROFESSIONAL ENGINEER

SHEET TITLE: SECONDARY DRAINAGE PLAN

OVERLOOK APARTMENTS PHASE II

CLIENT: OLAA, LLC
15234 SE 366TH PLACE
AUBURN, WA 98092
CONTACT: MARK NORDLIE
PHONE: (253) 255-7624

DESIGNER: K. ROSE
ENGINEER: J. HAUG
DRAWN: B. MORRIS
S 30 T 24N R 2E WM
DATE: 2020.02.14
REVISED:

PROJECT: 16-300
DWG NAME: 16-300-C

SHEET	REV.
C19	0

APPENDIX C

Geotechnical Report

January 21, 2020

OVAH II LLC
15234 SE 366th Place
Auburn, Washington 98092

Attn: Mr. Scott Fitzsimmons
(253) 606-3102

Geotechnical Engineering Report
Overlook Apartments
xxx – Orlando Street
Port Orchard, Washington
PN: 302402-3-063-2008, 4598-005-028007,
-022-0003, -017-0109, 4598-006-001-
0303, -004-0003, -007-0000, -010-0104
Doc ID: OVAHIILLC.OrlandoSt.PhaseII.RG.REV02

INTRODUCTION

This geotechnical report summarizes our site observations, subsurface explorations, laboratory testing and engineering analyses. This report also provides geotechnical recommendations and design criteria for the proposed multi-family residential development to be constructed at xxx – Orlando Street in Port Orchard, Washington. The site consists of eight contiguous tax parcels and is currently undeveloped. The general location of the site is shown on the attached Site Location Map, Figure 1.

Our understanding of the project is based on correspondence with you, our review of the preliminary site plan prepared by Contour Engineering dated January 29, 2018, our February 15, 2018, November 20, and 26, 2019 site visits, our understanding of the City of Port Orchard Municipal Code (POMC), and our past experience in the project area. We understand that you propose to develop the eight contiguous tax parcels listed above. We understand that the development currently consists of 6 apartment buildings with a total of 106 units, a recreational building, paved access roads, a combination of parking stalls, garages and car ports, and associated utilities, as shown on the Conceptual Site Plan, Figure 2. We anticipate that the proposed apartments will be 3-story, wood-framed structures founded on a combination of conventional shallow deepened foundations.

We previously prepared a geotechnical engineering report dated September 20, 2016 for the Overlook Apartments currently under construction west of the subject site, and also a draft geotechnical report for this site dated March 13, 2018. The City approved our report as part of the building permit applications. Because of the proximity of the proposed development to the steep slopes on the site, Port Orchard is requiring a Geotechnical Report in order to address the geologic hazards at the site per the Critical Areas Ordinance, and provide geotechnical recommendation and design criteria. The City is also requiring a soils report to address the feasibility of stormwater infiltration at the site.

SCOPE

The purpose of our services is to evaluate the surface and subsurface conditions across the site as a basis for providing geotechnical recommendations and design criteria for the proposed development. Specifically, the scope of services for this project included the following:

1. Reviewing the available geologic, hydrogeologic, and geotechnical data for the site area;
2. Exploring subsurface conditions across the site by advancing 6 hollow-stem auger borings and completing one of the borings as a groundwater monitoring well, and also excavating a series of 10 test pits at select locations across the site;
3. Describing surface and subsurface conditions, including soil type, depth to groundwater, and an estimate of seasonal high groundwater levels;
4. Addressing the City of Port Orchard Critical Areas Ordinance for geologic hazards, including recommended buffers and setbacks, as appropriate;
5. Providing geotechnical conclusions and recommendations regarding site grading activities, including site preparation, subgrade preparation, fill placement criteria, suitability of on-site soils for use as structural fill, and temporary and permanent cut and fill slopes;
6. Providing conclusions regarding shallow and deepened foundations and floor slab support and design criteria, including bearing capacity and subgrade modulus, if appropriate;
7. Providing our opinion about the feasibility of onsite infiltration, including a preliminary design infiltration rate based on grain size data, if applicable;
8. Providing recommendations for erosion and sediment control during wet weather grading and construction;
9. Preparing this *Geotechnical Engineering Report* summarizing our site observations and conclusions, and our geotechnical recommendations and design criteria, along with the supporting data.

The above scope of work was completed in accordance with our *Proposal for Geotechnical Engineering Services* dated November 1, 2019.

SITE CONDITIONS

Surface Conditions

The subject site is located at xxx – Orlando Street in the western portion of the Port Orchard glacial upland area in an area of existing residential and commercial development in Port Orchard, Washington. The site consists of eight contiguous tax parcels, that when combined measure approximately 260 to 400 feet wide (east to west) by 320 to 840 feet deep (north to south), and encompass approximately 6.04 acres. As shown on the Site Vicinity Map, Figure 2, the site is bounded by existing residential development and the ongoing Overlook Apartments development to the east and west, by existing commercial development to the south, and by Orlando Street to the north.

According to topographic information obtained from the Kitsap County GIS website, the eastern portion of the site is generally flat to gently sloping down to the west at about 5 to 15 percent, before sloping more steeply down to the west at about 40 to 65 percent. Slopes in this portion of the site have about 30 to 55 feet of vertical relief in the north portion of the site, and 24 to 40 feet of vertical relief in the west central portion of the site. The steep slopes are concentrated primarily on the 302402-3-063-2008 parcel. These slopes transition into inclination of approximately 25 percent at the bottom of the site in the northwest corner. The lower, southern portion of the site slopes down into a localized drainage at about 40 percent with approximately 10 to 20 feet of vertical relief. The drainage trends from east to west and gently slopes down to the west at about 5 to 10 percent. Total topographic relief across the site is on the order of 80 feet. The existing site topography and configuration is shown on the Site and Exploration Map, Figure 3.

Vegetation across the site at the time of our latest site visit typically consists of a mixture of dense brambles, scotch broom, and grasses with scattered madrona and other deciduous trees. The more steeply sloping portion of the site to the north and west is vegetated with mature fir and cedar trees with a moderate understory of ferns, salal, scotch broom, and brambles. No evidence of slope instability or soil movement was observed at the site at the time of our site visit. Most of the toe of the slope has been retained by concrete walls. No evidence of standing water, seeps, or springs was observed on the site at the time of our site visit.

Site Soils

The USDA Natural Resources Conservation Service (NRCS) Web Soil Survey maps the site as being underlain by Alderwood gravelly sandy loam (2 and 14), Indianola loamy sand (20), Indianola-Kitsap complex (21), and Ragnar fine sandy loam (44) soils. The Alderwood soils are derived from glacial till and form on slopes of 8 to 15 percent and 15 to 30 percent, respectively. These soils have a "slight" to "moderate" erosion hazard when exposed and are included in hydrologic soils group B. The Indianola soils are derived from sandy glacial outwash and form on slopes 15 to 30 percent. These soils have a "moderate" erosion hazard and are included in hydrologic soils group A. The Indianola-Kitsap soils are derived from glacial outwash, form on slopes of 45 to 70 percent, have a "severe" erosion hazard, and are also included in hydrologic soils group A. The Ragnar soils are derived from glacial outwash with ash and form on slopes of 0 to 6 percent. These soils have a "moderate" erosion hazard and are included in hydrologic soils group A. A copy of the referenced NRCS Soils Map is included as Figure 4.

Site Geology

The *Washington State Department of Natural Resources Division of Geology and Earth Resources Open File Report 2005-3, 1:100,000-scale* (December 2005) indicates that the site is underlain by glacial till (Qgt) and advance outwash (Qga) deposits. These glacial soils were deposited during the Vashon Stade of the Fraser Glaciation, approximately 12,000 to 15,000 years ago. The glacial till consists of a heterogeneous mixture of clay, silt, sand and gravel that was deposited at the base of the prehistoric continental glacial ice mass and was subsequently over-ridden. The advance outwash soils consist of poorly sorted, lightly stratified mixture of sand and gravel that may contain localized deposits of clay and silt that were deposited by meltwater streams emanating from the advancing ice mass. The glacial till and advance outwash are considered over-consolidated and exhibit high strength and low compressibility characteristics. No evidence of deep seated erosion or other active landslide activity was observed at the time of our site visit. No areas of landslide deposits or mass wasting are noted on the referenced map within the immediate vicinity of the site. An excerpt of the above referenced map is included as Figure 5.

We also reviewed the Department of Natural Resources Natural Hazards Map (Geologic Information Portal). The Map indicates that the site is situated about 1.3 miles south of the Seattle Fault zone. No evidence of surficial fault rupture was observed at the site at the time of our site visit. A copy of the referenced DNR Natural Hazards Map is included as Figure 6.

Subsurface Explorations

On February 15, 2018, a field representative from GeoResources, LLC (GeoResources) visited the site and monitored the drilling of 2 borings to depths of 31½ and 36½ feet and the excavation of 10 test pits to depths of about 7¾ to 13 feet below the existing ground surface. The test pits were excavated by a rubber track mounted excavator operated by a licensed earthwork contractor. On November 20 and 26, 2019, we returned to the site to advance an additional 4 borings, two in each proposed infiltration facility to same depths as our previous borings. On December 17, 2019, a representative from GeoResources arrived onsite and advanced four hand auger explorations to depths of about 4 to 8 feet below the existing ground surface in the proposed roadway. The borings were drilled by a licensed driller operating a small track drill rig. A groundwater monitoring well was installed in boring at the lowest elevation at the site. Below, Table 1 summarizes the approximate functional locations, surface elevations, and termination depths of our explorations.

TABLE 1:
APPROXIMATE LOCATIONS, ELEVATIONS, AND DEPTHS OF EXPLORATIONS

Exploration Number	Functional Location	Surface Elevation (feet)	Termination Depth (feet)	Termination Elevation (feet)
B-1	Upper E central, top of slope	305	36½	268½
B-2	Upper E central, top of slope	305	31½	273½
B-101	W end of E/W infiltration facility	310	36½	273½
B-102	E end of E/W infiltration facility	312	31½	280½
B-103	S end of N/S infiltration facility	298	31½	266½
B-104	N end of N/S infiltration facility	306	31½	274½
TP-1	Upper NE portion of site	316	7¾	308¼
TP-2	Upper E central portion of site	322	10	312
TP-3	Lower S portion of site	304	12	292
TP-4	Lower S portion of site	299	9½	289½
TP-5	Lower SE portion of site	312	13	299
TP-6	Lower SE portion of site	314	11	303
TP-7	Upper E central portion of site	327	10½	316½
TP-8	Upper central portion of site	325	11	314
TP-9	Lower W central portion of site	310	11	299
TP-10	Lower S portion of site	310	10+	310+
HA-1	Gravel cut area	345	8	337
HA-2	Russel Ave SE & SE Lovell St	345	4	341
HA-3	Whittier Ave SE & SE Lovell St	350	8	342
HA-4	Near top of slope, Wendell Ave	335	7½	327½
Notes: 1 = Elevation datum: Kitsap County GIS data				

The specific number, locations, and depths of our explorations were selected by GeoResources based on the proposed site configurations and the proposed site development, with considerations of site access limitations and underground utilities. Our field representative continuously monitored the explorations, maintained logs of the subsurface conditions encountered, obtained representative soil samples, and observed pertinent site features. Representative soil samples obtained from the explorations were placed in sealed plastic bags and taken to a laboratory for further examination and testing as deemed necessary. Each boring was then backfilled with bentonite chips and abandoned, and each test pit and hand auger was backfilled with the excavated soils and bucket and tamped in place, but not otherwise compacted.

During drilling, soil samples were obtained at 2½- and 5-foot depth intervals in accordance with Standard Penetration Test (SPT) as per the test method outlined by ASTM: D-1586. The SPT method consists of driving a standard 2-inch-diameter split-spoon sampler 18-inches into the soil with a 140-pound hammer. The number of blows required to drive the sampler through each 6-inch interval is counted, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "SPT blow count". The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The subsurface explorations excavated as part of this evaluation indicate the subsurface conditions at specific locations only, as actual subsurface conditions can vary across the site. Furthermore, the nature and extent of such variation would not become evident until additional explorations are performed or until construction activities have begun. Based on our experience in the area and a review of published geologic literature, it is our opinion that the soils encountered in the explorations are generally representative of the soils at the site. The soils encountered were visually classified in accordance with the Unified Soil Classification System (USCS) and ASTM D: 2488. The USCS is included in Appendix A as Figure A-1. The approximate locations of our explorations are indicated on the attached Site and Exploration Map, Figure 3, while the descriptive logs of our borings and test pits are included in Appendix A, Figures A-2 through A-7.

Subsurface Conditions

Subsurface conditions were evaluated by several different methods, including test pits, borings, and hand-auger exploration. The type and method of exploration was determined by the depth of the exploration and access limitation. The various exploratory methods are described below.

Test Pits

Our test pits encountered fairly uniform subsurface conditions that generally confirmed the mapped stratigraphy across the site. In general, our test pits encountered about ½ to 1½ feet of brown to dark brown sandy topsoil mantling grey brown to tan sand/silty sand with variable amounts of gravel in a loose to medium dense and moist condition to the full depth explored. We interpret these soils to be recessional outwash sands. In test pits TP-3, TP-4, TP-5, and TP-10, we encountered about 3¼ to 11½ feet of brown to grey silty sand and sandy silt with construction debris in a loose to medium dense and moist to damp condition. We interpret these soils to be undocumented fill. These soils were encountered to the full depth explored in TP-3 and TP-5. Underlying the fill in TP-4 and TP-10, we encountered tan to grey brown sand to silty sand in a loose to medium dense condition to the full depth explored. We interpret these soils to be recessional outwash sands.

Borings

Our borings generally encountered interbedded sands and silts with occasional lenses of silty sands. Layers of clean sands were at varying depths of each boring. Boring B-101 encountered soils throughout the boring that were in a loose to medium dense, moist condition to the full 31.5-foot depth explored. We interpret the soils in the vicinity of this boring to be recessional outwash. Borings B-102 to B-104 encountered deeper soils that were in a dense to very dense condition. We interpret these soils to be consistent with advance outwash. Groundwater was observed in boring B-103 at 22.5 feet below the existing ground surface, and dense sandy and silty gravel in a wet condition was observed from 25 to 31.5 feet below the existing ground surface.

Hand Augers

We generally observed medium dense to dense sands with variable silt contents and sand with gravel in our hand auger explorations. In hand auger HA-2, we observed about 3 feet of dense brown gravelly sand with a reworked texture before hitting undisturbed soils. The reworked soils are likely from the utilities that had been installed in the area. We interpret these soils to be advance outwash. Table 2 summarizes the approximate thicknesses, depths, and elevations of selected soil layers.

TABLE 2:
APPROXIMATE THICKNESS, DEPTHS, AND ELEVATION OF SOIL TYPES ENCOUNTERED IN EXPLORATIONS

Exploration Number	Thickness of Topsoil (feet)	Thickness of Undocumented fill (feet)	Depth to Top of Outwash (feet)	Elevation of Top of Outwash (feet)
B-1	½	NE	½	304½
B-2	½	NE	½	304½
B-101	½	NE	½	309½
B-102	½	NE	½	311½
B-103	½	15	15	283
B-104	½	20	20	286
TP-1	½	NE	½	315½
TP-2	¾	NE	¾	321¼
TP-3	½	11½ +	NE	NE
TP-4	1¼	3¼	4½	294½
TP-5	¾	12¼+	NE	NE
TP-6	1	NE	1	313
TP-7	¾	NE	¾	326¼
TP-8	1	NE	1	324
TP-9	1½	NE	1½	308½
TP-10	1	7	8	302
HA-1	1	NE	1	344
HA-2	½	2.5	3	341
HA-3	1	NE	1	349
HA-4	1	NE	1	334

Notes: 1 = Elevation datum: BHC Consultants Survey Data, datum unavailable
NE: Not encountered

Laboratory Testing

Geotechnical laboratory tests were performed on select samples retrieved from the explorations to determine soil index and engineering properties encountered. Laboratory testing included visual soil classification per ASTM D: 2488, moisture content determinations per ASTM D: 2216, and grain size analyses per ASTM D: 422 standard procedures. The results of the laboratory tests are included in Appendix B, and summarized above in Table 3.

TABLE 3:
LABORATORY TEST RESULTS FOR ON-SITE SOILS

Sample	Soil Type	Lab ID Number	Gravel Content (percent)	Sand Content (percent)	Silt/Clay Content (percent)	D10 Ratio (mm)
TP-2, S-2, ¾ -2½'	Weathered outwash	093766	0.7	85.7	13.6	<0.075
TP-7, S-1, 7-10'	Outwash	093770	4.4	89.1	6.5	0.0970
TP-9, S-1, 11-11½'	Outwash	093772	15.4	63.9	20.7	<0.075
B-101, S-6, 20'	Outwash	098897	0.3	58.7	41.0	<0.075
B-102, S-4, 15'	Outwash	098898	0.1	25.3	74.6	<0.075
B-104, S-5, 25'	Outwash	098899	0.6	95.2	4.2	0.1702
B-103, S-3, 15'	Outwash	099063	0.0	82.9	17.1	<0.075
B-103, S-4, 17.5'	Outwash	099064	0.2	85.5	14.3	<0.075
B-103, S-6, 22.5'	Outwash	099065	0.2	91.4	8.4	0.0983
HA-1, S-2, 3'	Weathered outwash	098900	0.0	92.0	8.0	0.1181
HA-3, S-1, 1'	Weathered outwash	098901	15.6	75	9.4	0.0794
HA-4, S-5, 7'	Outwash	098902	0.1	96.2	3.7	0.1332

Cation exchange capacity (CEC) and organic content testing was also performed by an independent laboratory to evaluate the treatment capacity of the shallow on-site soils for LID methods.

Groundwater Conditions

Groundwater was observed in test pit TP-5 at about Elevation 299 feet (February 2015), in boring B-1 at about Elevation 269 feet (February 2018), and in boring B-103 at about Elevation 276 feet (November 2019). Boring B-103 was completed as a monitoring well, groundwater level readings were taken at the time of drilling (ATD), and on the dates indicated in Table 4. A complete summary of the wet season groundwater monitoring will be provided in an addendum letter to be issued at the end of the wet season, late April or early May. Groundwater seepage was not observed at the other exploration locations at the time of exploration.

Evidence of mottling was observed in TP-1, TP-4, TP-5, and TP-8 from about ¾ to 8 feet below the existing ground surface, as well as in B-103 at an elevation of 276 feet. The orange staining and mottling are generally indicative of seasonal perched groundwater. Perched groundwater typically develops when the vertical infiltration of precipitation through a more permeable soil is slowed at depth by a deeper, less permeable soil type. We anticipate fluctuations in the local groundwater levels will occur in response to precipitation patterns, off-site construction activities, and site utilization.

TABLE 4:
APPROXIMATE DEPTHS, AND ELEVATION OF GROUNDWATER ENCOUNTERED IN EXPLORATIONS

Exploration Number	Depth to Groundwater (feet)	Elevation of Groundwater (feet)	Dated Measured
B-103	22.0	276.0	ATD (11/26/2019)
	24.3	273.7	01/06/2020
	24.2	273.8	01/17/2020

Notes:
[1]Elevations estimated by interpolating between contours on Site Survey provided by AP Consulting Engineers.
N/E: Not encountered **ATD**: At time of drilling/digging

CONCLUSIONS

Based on our site observations and data review, subsurface explorations and our engineering analysis, it is our opinion that the proposed multi-family residential development is feasible from a geotechnical standpoint. Our subsurface explorations generally encountered silty sand to sand with silt and variable amounts of gravel. We also encountered undocumented fill in the lower southeast portion of the site. Based on the encountered subsurface soils, it is our opinion that infiltration is feasible in the native outwash soils provided all vertical and horizontal setbacks can be met per the 2012 Stormwater Management Manual for Western Washington (SWMMWW). Pertinent conclusions and geotechnical recommendations regarding the design and construction of the proposed development are presented below.

Geologically Hazardous Areas POMC 18.08.020

The POMC Title 20.162.076 defines geologically hazardous areas based on the following indicators:

Geologically Hazardous Areas:

a) Areas with slopes greater than 30 percent and mapped by the Coastal Zone Atlas or Quaternary Geology and Stratigraphy of Kitsap County as unstable (U), unstable old landslides (UOS) or unstable recent slides (URS).
b) Areas with slopes greater than 30 percent in grade and deemed by a qualified geologist or geotechnical engineer to meet the criteria of U, UOS, or URS.

Areas of Geologic Concern:

a) Areas designated U, UOS, or URS in the Coastal Zone Atlas or Quaternary Geology and Stratigraphy of Kitsap County, with slopes less than 30 percent; or areas found by a qualified geologist to meet the criteria for U, URS, and UOS with slopes less than 30 percent; or

b) Slopes identified as intermediate (I) in the Coastal Zone Atlas or Quaternary Geology and Stratigraphy of Kitsap County, or areas found by a qualified geologist to meet the criteria of I; or
c) Slopes 15 percent or greater, not classified as I, U, UOS, or URS, with soils classified by the Natural Resources Conservation Service as "highly erodible" or "potentially highly erodible"; or
d) Slopes 15 percent or greater with springs or groundwater seepage not identified in subsections (2) (a), (b), or (c) of this section; or
e) Seismic areas subject to liquefaction from earthquakes (seismic hazard areas) such as hydric soils as identified by the Natural Resources Conservation Service, and areas that have been filled to make a site more suitable. Seismic areas may include former wetlands, which have been covered with fill.

The POMC Title 20.162.076 uses the above referenced checklist to define a geologic hazard area. Based on our observations of the site and review of published information, we offer the following comments.

No evidence of active or historic landslide activity, or ongoing erosion, was observed at the site at the time of our site visit. No landslides are mapped on or in the vicinity of the site and the Quaternary Geology and Stratigraphy of Kitsap County indicates that the majority of the site is "stable" and the northwest corner of the site is mapped as "intermediate". Slopes steeper than 30 percent were observed at the site and silt deposits underlying sand were observed, however, these deposits do not appear to be continuous. Groundwater seepage was encountered in some of our explorations, however, it was not observed daylighting on the slope below the site. The NRCS maps the site as being underlain by Alderwood gravelly sandy loam, Indianola loamy sand, Indianola-Kitsap complex, and Ragnar fine sandy loam. As previously stated, these soils have a "slight" to "moderate" to "severe" erosion hazard. The site is underlain by normally consolidated soils and glacially consolidated soils at depth that are in a medium dense to dense condition, making the potential for liquefaction moderate as seismic shaking is not apt to produce a denser configuration. No evidence of recent or ongoing slope instability or erosion was observed at the time of our site visit.

Based on the above, the site does have some of the above hazard indicators for an area of geologic concern per POMC Title 20.162.076. However, no evidence of landslide activity or active landslide hazards were observed at the site at the time of our site visits. Therefore, no prescriptive buffer should be required by the City of Port Orchard.

Slope Stability Analysis

We used the computer program SLIDE version 7.0, from RocScience, 2016, to perform the slope stability analyses. The computer program SLIDE uses a number of methods to estimate the factor of safety (FS) of the stability of a slope by analyzing the shear and normal forces acting on a series of vertical "slices" that comprise a failure surface. Each vertical slice is treated as a rigid body; therefore, the forces and/or moments acting on each slice are assumed to satisfy static equilibrium (i.e., a limit equilibrium analysis). The FS is defined as the ratio of the forces available to resist movement to the forces of the driving mass. An FS of 1.0 means that the driving and resisting forces are equal; an FS less than 1.0 indicates that the driving forces are greater than the resisting forces (indicating failure). We used the Generalized Limit Equilibrium method using the Morgenstern-Price analysis, which satisfies both moment and force equilibrium, to search for the location of the most critical failure surfaces and their corresponding FS. The most critical surfaces are those with the lowest FS for a given loading condition, and are therefore the most likely to move.

We analyzed the site conditions in both a pre-undeveloped and post-developed configuration. The configurations were selected based on the proximity of the proposed structures to the top of the greater-than-30 percent slope, in order to justify a reduction of the prescriptive buffer/setback. Based on our analyses, the FS of the western slope at the site meets the prescribed FS of 1.5 for static but does not meet the FS of 1.1 for seismic conditions in the existing configuration. If a daylight basement configuration is utilized along the slope, the FS is 1.5 and 1.1 for static and seismic conditions. If pin piles are utilized, the FS is 1.5 and 1.2 for static and seismic conditions. Details of the slope stability analyses are included in Appendix C.

Development Standards per POMC 20.162.078

According to POMC Chapter 20.162.078 Section 1(c), a native vegetation buffer should extend from the toe of the slope to 25 feet beyond the top of the slope in geologically hazardous areas or areas of geologic concern.

Any future structures will require a building setback per POMC Chapter 20.162.078 Section 1(d), which states that buildings and impervious surface shall be setback from the top of the slope equal to the height of the slope (1:1 horizontal to vertical) plus the greater of 1/3 the vertical slope height or 25 feet.

Based on our site observations and the results of our slope stability analyses, if the building is properly supported, the site slope does not constitute a geologic hazard. Therefore, no prescriptive setback should be required by the City of Port Orchard.

Recommended Setback

All structures will require a building setback from slopes steeper than 3H:1V (Horizontal: Vertical) or 33 percent with greater than 10 feet of vertical height to satisfy requirements of the 2015 IBC, Section 1808.7. The prescriptive building setback can be evaluated and reduced, and/or a structural setback may be provided, by a licensed geotechnical engineer. The setback distance is calculated based on the vertical height of the slope. The typical IBC setback from the top of the slope equals one third the height of the slope, with a maximum setback of 40 feet from the top of the slope, while a setback from the toe of the slope equals one half the height of the slope, with a maximum setback of 15 feet from the toe of the slope. If the setback from the top of the slope cannot be met, a structural setback may be used. A structural setback consists of deepening the foundation elements so that, when measured horizontally from the front of the footing to the face of the slope, the minimum IBC setback is achieved.

As stated above, the steep slope at the site has a vertical height of about 24 to 40 feet. Per the 2015 IBC, in its current configuration, the slope area should have a minimum setback of 8 to 14 feet from the top of the slope. If the setback from the top of the slope cannot be achieved, the IBC allows for the use of a "structural setback", as shown on Figure 7. The structural setback includes deepening foundations and measuring from the bottom of the foundation to the face of the slope, at the corresponding elevation. A daylight basement configuration could be used to meet this setback and should provide additional housing units and higher density.

Seismic Design

Based on our observations and the subsurface units mapped at the site, we interpret the structural site conditions to correspond to a seismic Site Class "D" in accordance with the 2015 IBC (International Building Code) documents and ASCE 7-Chapter 20 Table 20.3-1. This is based on the range of SPT (Standard Penetration Test) blow counts for the soils encountered in our borings. These conditions were assumed to be representative for the subsurface conditions for the site in general.

For design of seismic structures using the IBC 2015, mapped short-period and 1-second period spectral accelerations, SS and S1, respectively, are required. The U.S. Geological Survey (USGS) completed probabilistic seismic hazard analyses (PSHA) for the entire country in November 1996, which were updated and republished in 2002 and 2008. The PSHA ground motion results can be obtained from the USGS website. The results of the updated USGS PSHA were referenced to determine SS and S1 for this site. The results are summarized in the following table (Table 5) with the relevant parameters necessary for IBC 2015 design.

TABLE 5:
2015 IBC Parameters for Design of Seismic Structures

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period	1 Second Period
Mapped SRA	$S_s = 1.581$	$S_1 = 0.606$
Site Coefficients (Site Class D)	$F_a = 1.0$	$F_v = 1.50$
Maximum Considered Earthquake SRA	$S_{MS} = 1.581$	$S_{M1} = 0.909$
Design SRA	$S_{DS} = 1.054$	$S_{D1} = 0.606$

Earthquake-induced geologic hazards may include liquefaction, lateral spreading, slope instability, and ground surface fault rupture. In our opinion, the potential for liquefaction and lateral spreading is not significant because of the depth to groundwater. The ground surface at the project site slopes gently to moderately towards the west; therefore, the potential for earthquake-induced slope instability is also low. According to the Department of Natural Resources Geologic Hazards Map (Geologic Information Portal), the site is located south of the Seattle Fault Zone and southeast of the Gold Creek fault, as shown on Figure 6. In our opinion, the potential for ground surface fault rupture is also low.

Foundation Support

Based on the conceptual site plan, and the recommended setback, it is likely that any structures constructed along the slope to the west will require deepened foundation elements. We recommend using needle pin piles or other deepened foundation elements to support the proposed apartment buildings in the setback area. Outside of the setback area, conventional foundations may be utilized.

Spread Footings

Because of the height and inclination of the site slopes, the potential to use conventional spread footings is likely only limited to the portion of the residence located outside of the recommended setback. Based on the encountered subsurface soil conditions encountered across the site, we recommend that spread footings be founded on the dense glacial till or on structural fill that extends to suitable native soils.

The soil at the base of the footing excavations should be disturbed as little as possible. All loose, soft or unsuitable material should be removed or recompacted per the **Structural Fill** section of this report. If material is overexcavated below a footing, it should be replaced with controlled density fill (CDF) or structural concrete. A rat slab of CDF could be placed after excavation to prevent disturbance of the subgrade. A representative from our firm should observe the foundation excavations to determine if suitable bearing surfaces have been prepared, particularly in the areas where the foundation will be situated on fill material.

The undocumented fill encountered in the southeast portion of the site should be removed from within the building footprints and replaced by structural fill. Over-excavation depths of more than 12¼ feet may be required. Over-excavations should extend laterally out 1-foot for every 1-foot of vertical over-excavation. Any over-excavations should be backfilled with structural fill as described below in the "Structural Fill" section of this report. If Control Density Fill (CDF) is used as backfill, the lateral over-excavation can be limited to 1/3-foot for each 1-foot of over-excavation.

We recommend a minimum width of 24 inches for isolated footings and at least 18 inches for continuous wall footings. All footing elements should be embedded at least 18 inches below grade for frost protection. Footings founded on the glacial till can be designed using an allowable soil bearing capacity of 2,500 psf (pounds per square foot) for combined dead and long-term live loads. The weight of the footing and any overlying backfill may be neglected. The allowable bearing value may be increased by one-third for transient loads such as those induced by seismic events or wind loads.

Lateral loads may be resisted by friction on the base of footings and floor slabs and as passive pressure on the sides of footings. We recommend that an allowable coefficient of friction of 0.35 be used to calculate friction between the concrete and the underlying soil. Passive pressure may be determined using an allowable equivalent fluid density of 300 pcf (pounds per cubic foot). Passive resistance from soil should be ignored in the upper 1 foot. Factors of safety have been applied to these values.

We estimate that settlements of footings designed and constructed as recommended will be less than 1-inch, for the anticipated load conditions, with differential settlements between comparably loaded footings of ½-inch or less. Most of the settlements should occur essentially as loads are being applied. However, disturbance of the foundation subgrade during construction could result in larger settlements than predicted. We recommend that all foundations be provided with footing drains.

Pin Piles

Needle or pin piles consists of small to midsize diameter Schedule-80 steel pipe that are driven into the underlying soils to refusal. Schedule 80 steel is used instead of schedule 40 for corrosion resistance. The steel pipe diameters range from 2 to 6-inches. Individual pipe segments typically range from about 5 to 21 feet long and are successively joined with external threaded couplings, internal slip couplings, or butt welds as pile driving progresses. The large diameter piles use a pneumatic or hydraulic hammer mounted on the arm of a construction vehicle. The pin piles have little to no lateral strength, unless battered. The pin piles must obtain adequate embedment to provide support to the structure. We recommend a minimum embedment of 18 feet below the ground surface at existing grades.

Regardless of diameter or installation method, we recommend that each pin pile be driven to a point of refusal during sustained driving. Because refusal depths are difficult to predict and because soil conditions could vary significantly across the site, we recommend a test pile be installed. The contractor should be prepared for variable pile lengths. Also, it may be necessary to modify pile layouts if rocks or other obstructions are encountered during pile-driving.

When refusal has been achieved, the pin piles can be cut to a predetermined height or elevation. To provide a good bond between the piles and the existing foundation, a steel bracket is typically installed on the foundation element, with an adjustable element to provide a pre-loaded condition. A structural engineer should be responsible for designing the reinforced steel and foundation elements. The minimum pile spacing (center to center) shall be determined by the structural engineer.

For the proposed residence, we recommend that 3 to 4-inch needle piling be utilized. These pilings will need to be installed by a larger, machine-mounted hammer. A properly installed needle pile driven to refusal (defined by the required capacity, installation contractor, and/or accepted construction practice) should provide the following allowable axial capacities.

Design Parameter	Allowable Value		
	3-inch diameter	**4-inch diameter**	**6-inch diameter**
Static Compressive Capacity	12,000 pounds	20,000 pounds	30,000 pounds
Transient Compressive Capacity	16,000 pounds	26,000 pounds	40,000 pounds

We recommend that 3 percent of the piles (up to a maximum of 5 piles) be quick load tested per ASTM: D 1143-81. In areas where the lengths of the pin piles are exposed and not directly incorporated into the foundation grade beams, the area around the pin piles should be backfilled with a well-draining material such as angular quarry spalls. Verification testing should be performed in accordance with the ASTM Quick Test Method (ASTM D1143-81) on 5 percent of the installed piles, or a minimum of 3, whichever is greater.

Floor Slab Support

Slab-on-grade floors, where constructed, should be supported on the medium dense outwash soils or on structural fill prepared as described above. Any areas of old fill material should be evaluated during grading activity for suitability of structural support. Areas of significant organic debris should be removed.

We recommend that floor slabs be directly underlain by a minimum 6-inch thickness capillary break material, such as pea gravel or clean crushed rock. The capillary break material should be placed in one lift and compacted to an unyielding condition.

A synthetic vapor retarder is recommended to control moisture migration through the slabs in enclosed and heated spaces. This is of particular importance where the foundation elements are underlain by the silty glacial till or where moisture migration through the slab is an issue, such as where adhesives are used to anchor carpet or tile to the slab.

Subgrade/Basement Walls

Based on existing topography, we anticipate that the proposed structures may include daylight basement configurations, and that retaining walls and vaults may be required. The lateral pressures acting on subgrade and retaining walls (such as basement walls) will depend upon the nature and density of the soil behind the wall. It is also dependent upon the presence or absence of hydrostatic pressure. If the walls are backfilled with granular well-drained soil, the design active pressure may be taken as 35 pcf (equivalent fluid density). This design value assumes a level backslope and drained conditions as described below. For the condition of a sloping back slope, higher lateral pressures would act on the walls. For a 3H:1V (Horizontal to Vertical) slope above the wall, the active pressure may be taken as 48 pcf; for a 2H:1V back slope condition, a wall design pressure of 55 pcf may be assumed. If basement walls taller than 6 feet are required, a seismic surcharge could be included.

Adequate drainage behind retaining structures is imperative. Positive drainage which controls the development of hydrostatic pressure can be accomplished by placing a zone of drainage behind the walls. Granular drainage material should contain less than 2 percent fines and at least 30% greater than the US No. 4 sieve. Assuming properly compacted structural fill is used to backfill the foundation walls, an allowable active fluid pressure of 35 pcf should be appropriate for design. Typical wall drainage and backfilling details are shown in Figure 8.

A minimum 4-inch diameter perforated or slotted PVC pipe should be placed in the drainage zone along the base and behind the wall to provide an outlet for accumulated water and direct accumulated water to an appropriate discharge location. We recommend that a nonwoven geotextile filter fabric be placed between the soil drainage material and the remaining wall backfill to reduce silt migration into the drainage zone. The infiltration of silt into the drainage zone can, with time, reduce the permeability of the granular material. The filter fabric should be placed such that it fully separates the drainage material and the backfill, and should be extended over the top of the drainage zone.

A geocomposite drain mat may also be used instead of free draining soils, provided it is installed in accordance with the manufacturer's instructions. A soil drainage zone should extend horizontally at least 18 inches from the back of the wall. The drainage zone should also extend from the base of the wall to within 1 foot of the top of the wall. The soil drainage zone should be compacted to approximately 90 percent of the MDD. Over-compaction should be avoided as this can lead to excessive lateral pressures.

Lateral loads may be resisted by friction on the base of footings and as passive pressure on the sides of footings and the buried portion of the wall, as described in the "**Foundation Support**" section. We recommend that an allowable coefficient of friction of 0.35 be used to calculate friction between the concrete and the underlying soil. Passive pressure may be determined using an allowable equivalent fluid density of 350 pcf (pounds per cubic foot). Factors of safety have been applied to these values.

Temporary Excavations

All job site safety issues and precautions are the responsibility of the contractor providing services and work. The following cut and fill slope guidelines are provided for planning purposes only. Temporary cut slopes will likely be necessary during grading operations or during utility installation.

All excavations at the site associated with confined spaces, such as utility trenches and retaining walls, must be completed in accordance with local, state, or federal requirements. Based on current Washington Industrial Safety and Health Act (WISHA, WAC 296-155-66401) regulations, the outwash soils and undocumented fill soils on the site would be classified as Type C soils.

According to WISHA, for temporary excavations of less than 20 feet in depth, the side slopes in Type C soils should be laid back at a slope inclination of 1.5H:1V or flatter from the toe to top of the slope. It should be recognized that slopes of this nature do ravel and require occasional maintenance. All exposed slope faces should be covered with a durable reinforced plastic membrane, jute matting, or other erosion control mats during construction to prevent slope raveling and rutting during periods of precipitation. These guidelines assume that all surface loads are kept at a minimum distance of at least one half the depth of the cut away from the top of the slope and that significant seepage is not present on the slope face. Flatter cut slopes will be necessary where significant raveling or seepage occurs, or if construction materials will be stockpiled along the top of the slope.

Where it is <u>not</u> feasible to slope the site soils back at these inclinations, a retaining structure should be considered. Where retaining structures are greater than 4 feet in height, as measured from the bottom of the footing to the top of the exposed wall face, or have slopes of greater than 15 percent above them, an engineered wall design should be prepared per Washington Administrative Code (WAC 51-16-080 item 5).

This information is provided solely for the benefit of the owner and other design consultants, and should not be construed to imply that GeoResources assumes responsibility for job site safety. It is understood that job site safety is the sole responsibility of the project contractor.

Site Drainage

All ground surfaces, pavements and sidewalks at the site should be sloped away from structures. The site should also be carefully graded to ensure positive drainage away from all structures and property lines. Surface water runoff from the roof area, driveways, perimeter footing drains, and wall drains, should be collected, tightlined, and conveyed to an appropriate discharge point. Stormwater runoff from the proposed roof areas should be collected, tightlined, and routed away from the buildings. Surface water should be collected in catch basins and tightlined with the downspout runoff to an approved discharge point.

We recommend that all foundations be provided with a perimeter footing or foundation drain, per Section 1805.4.2 of the 2015 IBC. The foundation drains should not be connected to the roof drains.

STORMWATER INFILTRATION

The City of Port Orchard uses the 2012 (with 2014 updates) *Stormwater Management Manual for Western Washington* (SWMMWW). Volume III, Section 3.3.7 lists the site suitability criteria (SSC) for infiltration facilities, including minimum setbacks and separation requirements. SSC-5 Depth to Bedrock, Water Table, or Impermeable Layer states that the base of all infiltration basins or trenches shall be 5 feet or more above the seasonal high-water mark, bedrock (or hardpan), or other low permeability layer. The 2014 SWMMWW Glossary defines hardpan as "a cemented or compacted and often clay-like layer of soil that is impenetrable by roots. Also known as glacial till."

Based on our site observations during the time of excavation of our subsurface explorations, evidence of seasonal groundwater is present across portions of the site at 13 to 36 feet below the existing ground surface. As previously stated, we did not observed seepage along the slope below the site, and the groundwater appears discontinuous.

Soil gradation analyses were performed in accordance with ASTM D6913 and Volume III, Section 3.3.6, Method 3 of the 2014 SWMMWW to determine the preliminary infiltration rates in this portion of the site. Based on these analyses, it is our opinion that stormwater infiltration via ponds and trenches is feasible in portions of the site. Permeable pavement is also feasible, should you wish to pursue that option.

We recommend that alternative stormwater management BMPs, such as detention vaults and/or dispersion in accordance with the 2014 SWMMWW, Volume III, Section 3.2.3 be implemented to manage the stormwater runoff generated by the proposed development. In our opinion, dispersing stormwater is appropriate for the proposed development where vegetated flow paths established and maintained in accordance with the 2014 SWMMWW are feasible. The flow paths should remain well vegetated and free of bare spots or obstructions that could concentrate flows.

All minimum setback requirements and design criteria should be considered prior to the selection of any stormwater management facility per the 2014 SWMMWW. All stormwater management facilities should be designed and constructed per the 2014 SWMMWW.

__Test Method__

The 2014 SWMMWW, Volume III, Section 3.3.6 provides three approved methods to estimate the long term design infiltration rate of site soils: 1) Large-Scale Pilot Infiltration Test (PIT), 2) Small-Scale PIT, and 3) soil grain size analysis method. Restrictions do apply to the various methods based on soil conditions and type of infiltration facility. Because our current scope is focused on assessing infiltration feasibility we have used the soil grain size method analysis method.

__Preliminary Design Infiltration Rate__

The design infiltration rate is determined based on the procedure provided in Volume III, Section 3.3.6, Table 3.3.1of the 2014 SWMMWW. Three correction factors are applied to the measured infiltration rate ($I_{measured}$) to determine the design infiltration rate (I_{design}). The design infiltration rate is determined as follows:

$$I_{design} = I_{measured} * CF_v * CF_t * CF_m$$

Where:

I_{design} = Infiltration rate to be used for design of infiltration facility
$I_{measured}$ = Infiltration rate measured in the field or estimated by grain size analysis
CF_v = Accounts for site variability and locations tested (0.33 to 1.0)
CF_t = Test method used (0.4 to 0.75)

Test Method	Correction Factor (CF_t)
Large Scale PIT	0.75
Small Scale PIT	0.50
Double-Ring Infiltrometer	0.50
Grainsize analysis	0.40

CF_m = Degree of influent control to prevent siltation and bio-buildup (0.9)

Based on the definitions and criteria outlined above, we used a value of 0.5 for CF_v, a value of 0.4 for CF_t, and a value of 0.9 for CF_m. Applying these correction factors to the measured infiltration rate, results in a long-term (design) infiltration rate at the specific location. For the purposes of estimating a preliminary infiltration rate and to reflect the early design stages of the project, we selected relatively conservative correction factors. It is possible, that during the design process these values may be increased potentially resulting in higher design infiltration rates.

TABLE 6:
PRELIMINARY DESIGN INFILTRATION RATES BASED ON GRAIN SIZE ANALYSIS

Sample Information			Laboratory Data[2]				Preliminary Design Infiltration rate (in/hr)
Exploration ID	Sample Elevation[1] (ft)	USCS Classification	D_{10} (mm)	D_{60} (mm)	D_{90} (mm)	F_{fines}	
B-101	290.0	SM	0.020	0.117	0.232	0.410	1.00
B-101	280.0	SP-SM	0.118	0.314	0.591	0.061	8.25
B-102	297.0	ML	0.002	0.350	0.246	0.746	0.15
B-102	282.0	SM	0.040	0.315	0.473	0.125	4.25
B-103	278.0	SM	0.050	0.218	0.381	0.171	3.50
B-103	275.5	SM	0.055	0.232	0.405	0.143	4.25
B-103	270.5	SP-SM	0.098	0.267	0.410	0.084	7.00
B-104	283.5	SP	0.170	0.360	0.570	0.042	11.5
HA-1	342.0	SP-SM	0.118	0.338	0.420	0.080	7.50
HA-3	349.0	SP-SM	0.079	0.431	9.365	0.094	4.50
HA-4	328.0	SP	0.132	0.297	0.415	0.037	10.00
TP-2	320.5	SM	0.650	0.233	0.409	0.136	4.50
TP-7	318.5	SP-SM	0.097	0.336	0.777	0.036	7.50
TP-9	299.0	SM	0.050	0.263	15.385	0.207	2.00

Notes:
[1] Sample elevation based on surface elevation presented in Table 1 and depth of sample (Vertical Datum NAVD88)
[2] Sample data obtained from grain size analysis performed in accordance with ASTM D6913, estimated values presented in red.

We understand that two infiltration galleries, labeled as Infiltration System 1 and Infiltration System 2, are proposed at the approximate locations indicated on Figure 2. The proposed bottom of Infiltration System 1 will be at about Elevation 292 feet, and the proposed bottom of Infiltration System 2 will be at about Elevation 294 feet.

Based on the information obtained from borings B-101, B-102, and B-103, which are near the proposed facilities, infiltration appears feasible with the following constraints. The estimated infiltration rates at approximately Elevation 292 feet are about 1.0 in/hr and 0.15 in/hr at the locations of borings B-101 and B-102, respectively. At the location of boring B-103, undocumented fill is present at Elevation 292 feet and extends to approximately Elevation 283 feet. Significantly higher infiltration rates are estimated at lower elevations at the locations explored. Specifically, at the location of B-101 the estimated preliminary infiltration rate is 8.25 in/hr below about Elevation 285 feet; at of the location B-102 the estimated preliminary infiltration rate is 4.25 in/hr below about Elevation 287 feet; and at of the location B-103 the estimated preliminary infiltration rate is 4.25 in/hr below about Elevation 287 feet.

Infiltration is not permitted through undocumented fill. The existing fill can be removed down to native soils and replaced with free draining soils that can provide a minimum infiltration rate of 8 in/hr. *Gravel Backfill for Drains* or *Gravel Backfill for Drywells* as defined by WSDOT 9-03.12(4) and WSDOT 9-03.12(5) can each provide an infiltration rate of at least 8 in/hr.

Because groundwater monitoring through the wet season has not been completed, we can only offer a preliminary estimate to the seasonal high groundwater elevation. Based on the data collected to date, we anticipate the seasonal high groundwater level will be between about Elevation 274 feet and 276 feet.

Construction Considerations

We recommend that a representative from our firm be onsite at the time of excavation of the proposed infiltration facilities to verify that the soils encountered during construction are consistent with the soils observed in our subsurface explorations. In-situ infiltration testing should be performed at the time of construction to verify the recommended infiltration rate and to determine if a different site specific infiltration rate would be more appropriate for the site.

It should be noted that special care is required during the grading and construction periods to avoid fine sediment contamination. This may be accomplished using an alternative stormwater management location during construction. All contractors, builders, and subcontractors working on the site should be advised to avoid allowing "dirty" stormwater or excess sediment to enter the proposed pervious pavement area during construction and landscaping activities. No concrete trucks should be washed or cleaned onsite.

Suspended solids could clog the underlying soil and reduce the infiltration rate of the facilities. To reduce potential clogging of the infiltration systems, the infiltration system should not be connected to the stormwater runoff system until after construction is complete and the site area is landscaped, paved or otherwise protected. Temporary systems may be utilized throughout construction. Periodic sweeping of paved areas will help extend the life of the infiltration system.

Alternative stormwater management methods, such as permeable pavement may also be considered. LID systems for water quality requires Cation Exchange Capacity (CEC) be at least 5mEq/100g and a minimum organic content of 1 percent in order for soils to be used as a treatment layer beneath a water quality facility, such as permeable pavement. One representative soil sample was tested by Spectra Laboratories. The results of this test indicate that the CEC for the site soils is about 14.5 mEq/100g and that the organic matter is about 3.5 percent, exceeding the required CEC and organic matter content.

Permeable Pavement

Design guidance for permeable pavement is covered in Volume III, Section 3.4 of the 2014 SWMMWW. Minimum vertical separation from bottom of pavement section to bedrock, seasonal high groundwater table, or other impermeable layer shall be greater than 1 foot. We did not observe any evidence of a shallow seasonal groundwater table in our explorations.

The design infiltration rate for permeable pavement is determined based on the procedure provided in Volume III, Section 3.4. The preferred test method is small-scale PIT. Two correction factors are applied to the influent control shape (CF_m). The design infiltration rate is determined as follows:

$$I_{design} = I_{measured} * CF_v * CF_m$$

Where:

I_{design} = Infiltration rate to be used for design of infiltration facility
$I_{measured}$ = Infiltration rate measured in the field or estimated by grain size analysis
CF_v = Accounts for site variability and locations tested (0.33 to 1.0)
CF_m = Quality of pavement aggregate base material (0.9 to 1.0)

TABLE 7:
PRELIMINARY DESIGN INFILTRATION RATES BASED ON GRAIN SIZE ANALYSIS

Sample Information			Laboratory Data[2]				Preliminary Design Infiltration rate (in/hr)
Exploration ID	Sample Elevation[1] (ft)	USCS Classification	D_{10} (mm)	D_{60} (mm)	D_{90} (mm)	F_{fines}	
HA-1	342.0	SP-SM	0.118	0.338	0.420	0.080	12.5
HA-3	349.0	SP-SM	0.079	0.431	9.365	0.094	7.5
HA-4	328.0	SP	0.132	0.297	0.415	0.037	16.5

Notes:
[1] Sample elevation based on surface elevation presented in table 1 and depth of sample (Vertical Datum NAVD88)
[2] Sample data obtained from grain size analysis performed in accordance with ASTM D6913

Based on the definitions and criteria outlined above, we used a value of 0.33 for CF_v, a value of 0.9 for CF_m. Applying these correction factors to the measured infiltration rate, results in a long-term (design) infiltration rate at the specific location. **The infiltration rates presented for permeable pavement are different than those for infiltration ponds/trenches.** For the purposes of estimating a preliminary infiltration rate and to reflect the early design stages of the project, we selected relatively conservative correction factors. It is possible, that during the design process these values may be increased potentially resulting in higher design infiltration rates.

LID systems for water quality requires Cation Exchange Capacity (CEC) be at least 5mEq/100g and a minimum organic content of 1 percent in order for soils to be used as a treatment layer beneath a water quality facility, such as permeable pavement. Two representative soil samples were sent to an outside laboratory for testing, the test results are presented in Table 8, below. The results indicate that the samples tested have CEC values less than 5 mEq/100g. The test results indicate organic content between about 0.84 and 1.1 percent. Based on the indicated test results, the site soils at the locations tested do not meet the required treatment criteria. Accordingly, amended soil will be necessary as part of a permeable pavement design

TABLE 8:
Cation Exchange Capacity and Organic Content of Select Samples

Sample ID	Organic Content[1]	Cation Exchange Capacity[2]
B-1, S-1, D=2.5ft	0.84	3.21
HA-3, S-2, D=3ft	1.1	4.85

Notes:

[1] Organic content presented as percent if dry weight, as determined by ASTM D2974-13

[2] Cation exchange capacity (CEC) presented as miliequivalents per 100 grams mEq/100g as determined by SW846 9081

EARTHWORK RECOMMENDATIONS

Site Preparation

All structural areas on the site to be graded should be stripped of vegetation, organic surficial soils, and other deleterious materials including any existing structures, foundations or abandoned utility lines. Organic topsoil is not suitable for use as structural fill, but may be used for limited depths in non-structural areas. Stripping depths ranging from ½ to 1½ feet should be expected to remove these unsuitable soils. Areas of thicker topsoil or organic debris may be encountered in areas of heavy vegetation, in depressions, or near the wetlands.

In addition to the removal of topsoil, the undocumented fill soils across the site should be removed. Recommendations regarding removal, processing and replacement of the undocumented fill is discussed below in the "**Suitability of On-Site Materials as Fill**" section.

Where placement of fill material is required, the stripped and exposed subgrade areas should be compacted to a firm and unyielding surface prior to fill placement. Excavations for debris removal should be backfilled with structural fill compacted to the densities described in the **"Structural Fill"** section of this report.

We recommend that a member of our staff evaluate the exposed subgrade conditions after removal of vegetation and topsoil stripping is completed and prior to placement of structural fill. The exposed subgrade soil should be proof-rolled with heavy rubber-tired equipment during dry weather or probed with a ½-inch diameter steel T-probe during wet weather conditions.

Any soft, loose or otherwise unsuitable areas delineated during proof-rolling or probing should be recompacted, if practical, or over-excavated and replaced with structural fill. The depth and extent of overexcavation should be evaluated by our field representative at the time of construction.

Structural Fill

All material placed as fill associated with mass grading, utility trench backfill, under building areas, or under roadways should be placed as structural fill. The structural fill should be placed in horizontal lifts of appropriate thickness to allow adequate and uniform compaction of each lift. Structural fill should be compacted to at least 95 percent of maximum dry density (MDD) as determined in accordance with ASTM D-1557.

The appropriate lift thickness will depend on the fill characteristics and compaction equipment used. We recommend that the appropriate lift thickness be evaluated by our field representative during construction. We recommend that our representative be present during site grading activities to observe the work and perform field density tests, as necessary.

The suitability of material for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines (material passing US No. 200 sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. During wet weather, we recommend use of well-graded sand and gravel with less than 5 percent (by weight) passing the US No. 200 sieve, based on that fraction passing the ¾-inch sieve, such as "*Gravel Backfill for Walls*" (WSDOT Standard Specification 9-03.12(2)). If prolonged dry weather prevails during the earthwork and foundation installation phase of construction, higher fines content (up to 10 to 12 percent) will be acceptable.

Material placed for structural fill should be free of debris, organic matter, trash, and cobbles greater than 6-inches in diameter. The moisture content of the fill material should be adjusted as necessary for proper compaction.

Fill placed on slopes that are steeper than 5H:1V (20 percent) should be "keyed" into the undisturbed native soils by cutting a series of horizontal benches per the 2015 IBC, Appendix J. The benches should be 1½ times the width of the equipment used for grading and be a maximum of 3 feet in height. Subsurface drainage may be required in areas where significant seepage is encountered during grading. Collected drainage should be directed to an appropriate discharge point. Surface drainage should be directed away from all slope faces.

Suitability of On-Site Materials as Fill

During dry weather construction, any non-organic on-site soil may be considered for use as structural fill, provided it meets the criteria described above in the "**Structural Fill**" section of this report and can be compacted as recommended. If the soil material is over the optimum moisture content when excavated, it will be necessary to aerate or dry the soil prior to placement as structural fill. We generally did not observe the site soils to be excessively moist at the time of our subsurface exploration program.

The previously placed fill encountered across the site consists of a mixture of sand, silt, and gravel with construction debris and organic material. We not anticipate that these soils will be suitable for use as structural fill because of the presence of construction debris and organic material, unless they are processed. Screening the granular fill soils with a 3-inch sieve to remove organics would be appropriate. Removal and procession of the undocumented fill soils should include excavating down to native soils, and an appropriate level of processing to meet the specification for common borrow WSDOT 9-03.14(3). GeoResources personnel should provide sufficient laboratory testing and monitoring to ensure the above specification is met and the material is replaced as structural fill.

The native weathered glacial outwash glacial outwash soils encountered across the site generally consisted of sand to silty sand with variable amounts of gravel. These soils are generally comparable to "common borrow" material and will be suitable for use as structural fill provided the moisture content is maintained within 2 percent of the optimum moisture content. Because of the variable fines content, these soils may be moisture sensitive and may be difficult to impossible to compact during wet weather conditions, or where seepage occurs.

We recommend that completed graded-areas be restricted from traffic or protected prior to wet weather conditions. The graded areas may be protected by paving, placing asphalt-treated base, placing a layer of free-draining material such as pit run sand and gravel or clean crushed rock material containing less than 5 percent fines, or some combination of the above.

Erosion Control

Weathering, erosion and the resulting surficial sloughing and shallow land sliding are natural processes that affect steep slope areas. As noted, no evidence of surficial raveling or sloughing was observed at the site. To manage and reduce the potential for these natural processes, we recommend the following:

1. No drainage of concentrated surface water or significant sheet flow onto or 50 feet of the top of steep slope areas.
2. No fill should be placed within the buffer or setback zones unless retained by engineered retaining walls or constructed as an engineered fill.
3. Grading should be limited to providing surface grades that promote surface flows away from the top of slope to an appropriate discharge location.

Erosion protection measures will need to be in place prior to grading activity on the site. Erosion hazards can be mitigated by applying BMPs outlined in the 2012 SWMMWW.

Wet Weather Earthwork Considerations

In the Puget Sound area, wet weather generally begins October 1st and continues through April 30th, although rainy periods could occur at any time of year. It is encouraged that earthwork be scheduled during the dry weather months of June through September. Some of the soils at the site contain sufficient fines to produce an unstable mixture when wet. Such soil is highly susceptible to changes in water content and tends to become unstable and impossible to proof-roll and compact if the moisture content exceeds the optimum.

In addition, during wet weather months, the groundwater levels could increase, resulting in seepage into site excavations. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, construction traffic, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following recommendations are provided:

1. The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas and to prevent ponding of water.

2. Work areas or slopes should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
3. Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill could be accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.
4. Fill material should consist of clean, well-graded, sand and gravel, of which not more than 5 percent fines by dry weight passes the US No. 200 sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve. The gravel content should range from between 20 and 50 percent retained on a US No. 4 mesh sieve. The fines should be non-plastic.
5. No exposed soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
6. In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see gradation requirements, above).
7. Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
8. Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

We recommend that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

LIMITATIONS

We have prepared this report for use by OVAH II, LLC, and other members of the design team, for use in the design of a portion of this project. The data used in preparing this report and this report should be provided to prospective contractors for their bidding or estimating purposes only. Our report, conclusions and interpretations are based on our subsurface explorations, data from others and limited site reconnaissance, and should not be construed as a warranty of the subsurface conditions.

Variations in subsurface conditions are possible between the explorations and may also occur with time. A contingency for unanticipated conditions should be included in the budget and schedule. Sufficient monitoring, testing and consultation should be provided by our firm during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork and foundation installation activities comply with contract plans and specifications.

The scope of our services does not include services related to environmental remediation and construction safety precautions. Our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

If there are any changes in the loads, grades, locations, configurations or type of facilities to be constructed, the conclusions and recommendations presented in this report may not be fully applicable. If such changes are made, we should be given the opportunity to review our recommendations and provide written modifications or verifications, as appropriate.

◆ ◆ ◆

We have appreciated the opportunity to be of service to you on this project. If you have any questions or comments, please do not hesitate to call at your earliest convenience.

Respectfully submitted,
GeoResources, LLC

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Attachments:
- Figure 1: Site Location Map
- Figure 2: Site & Exploration Plan
- Figure 3: Site Vicinity Map
- Figure 4: NRCS SCS Soils Map
- Figure 5: USGS Geologic Map
- Figure 6: WA DNR Natural Hazards Map
- Figure 7: Typical Structural Setback
- Figure 8: Typical Wall Drainage and Backfill
- Appendix A - Subsurface Explorations
- Appendix B - Laboratory Test results
- Appendix C – Slope Stability Analysis