PRELIMINARY STORM DRAINAGE REPORT

SAGERT FARM SUBDIVISION
TUALATIN, OR

June 4, 2015

Prepared For:

Lennar Northwest, Inc
2103 NE 129th Street, Suite 100
Vancouver, WA 98686

REGISTERED PROFESSIONAL ENGINEER
74498 PE
OREGON
Jan. 8, 2009
KATHLEEN E. FREEMAN

Kathleen Freeman 6/4/15

EXPIRES: 12/31/15

Prepared By:
3J Consulting, Inc.
5075 SW Griffith Drive, Suite 150
Beaverton, Oregon 97005
Project No: 13159
KEF

TABLE OF CONTENTS

EXECUTIVE SUMMARY....1
EXISTING CONDITIONS....2
Site....2
Site Geology....2
Existing Hydrology....2
Geotechnical Report....2
Existing Basin Areas....2
POST-DEVELOPED CONDITIONS....2
Site....2
Post-Developed Basin Areas....3
HYDROLOGIC ANALYSIS DESIGN GUIDELINES....3
Design Guidelines....3
Hydrograph Method....3
Design Storm....3
RUNOFF PARAMETERS....4
Basin Runoff....4
HYDRAULIC ANALYSIS AND DESIGN CHARACTERISTICS....4
System Characteristics....4
WATER QUALITY....4
Water Quality Guidelines....4
Water Quality Calculations....5
Water Quality Swale....5
Water Quality Manholes....5
SUMMARY....6
TECHNICAL APPENDIX....A
REFERENCES....A

LIST OF TABLES

Table 1 - Soil Characteristics....2
Table 2 – Existing Onsite Basin Area....2
Table 3 – Post-Developed Onsite Basin Areas....3
Table 4 - Design Storms....3
Table 5 - Basin Runoff Rates....4

EXECUTIVE SUMMARY

The project site consists of a total of 20.90 acres. The proposed development site is located east of SW 65th Avenue, south of SW Borland Road, and north of Saum Creek and the I-205 corridor (See Technical Appendix: Exhibits – Figures 1 & 2). The site is bounded to the east by the Sequoia Ridge subdivision. The site's northern boundary is formed by two separate professional medical office buildings, a PGE substation, and SW Borland Road. The site is bounded by Saum Creek and Interstate 205 to the south. There currently sits a single-family detached home with a wooden barn near the center of the property.

The site slopes downward towards the south. A substantial area in the southern portion of the site is designated with a Significant Natural Resource Overlay and will be preserved in a tract.

The intent of this subdivision is to provide seventy-nine (79) buildable lots, for development with single-family homes, a use permitted outright in the RL zone. The proposed residential subdivision includes the extension of SW Sagert Street (east of SW 65th Avenue).

Runoff from the proposed impervious area will be treated using vegetated swales designed following the current Clean Water Services Design and Construction Standards which uses a rainfall of 0.36" over a 4-hour period with a return period of 96-hours as outlined in section 4.05.06 of *the Design and Construction Standards for Sanitary Sewer and Surface Water Management* issued in June of 2007 and updated in 2009. After treatment, stormwater will be conveyed towards Saum Creek.

Per the City of Tualatin, the downstream system (Saum Creek) has sufficient capacity to convey the added runoff to the Tualatin River without requiring detention (or a downstream analysis).

The purpose of this report is to describe the treatment facilities being proposed and to show that the design followed Clean Water Services Design and Construction Standards.

EXISTING CONDITIONS

Site
The existing site contains a single family residence, a barn and several small to large outbuildings. The surrounding fields have historically been mowed for hay. All existing structures will be demolished for the proposed subdivision.

Site Geology
The soil types as classified by the United States Department of Agriculture Soil Survey of both Washington and Clackamas Counties are identified in Table 1 (See Technical Appendix: Exhibits – Hydrologic Soil Group-Washington County, Oregon).

Soil Type	Hydrologic Group
Quatama Loam	C
Wapato Silty Clay Loam	C/D

Table 1 - Soil Characteristics

The majority of the soils on the site are categorized as hydrologic soil group C which have a slow infiltration rate when thoroughly wet. Group C soils were used to determine the runoff curve numbers.

Existing Hydrology
Runoff from the site generally sheet flows towards the south into Saum Creek. The average slope across the site is approximately 6.5%.

Geotechnical Report
A geotechnical investigation by GeoPacific Engineering, Inc was completed in December of 2013 (See Technical Appendix: Geotechnical Report). Three test pits were explored for infiltration rates at depths 4.5, 8.5 and 11 feet below ground surface. The respective infiltration rates were 0.1, 0.3 and 6.0 in/hr.

Existing Basin Areas
Table 2 shows the current impervious and pervious areas for the property (See Technical Appendix: Exhibits – Existing Site Conditions).

Existing Onsite Basin Area	ft^2	Acres
Impervious Area	30,361	0.70
Pervious Area	879,897	20.20
Total Area	**910,258**	**20.90**

Table 2 – Existing Onsite Basin Area

POST-DEVELOPED CONDITIONS

Site
The proposed site will consist of a 79 lot subdivision with internal streets and sidewalks. Sagert Street will be constructed through the site connecting SW 65th Avenue and the subdivision to the east of the site. Two separate storm systems will be constructed effectively dividing the site at the natural high point in the center of the site. Each storm system will convey runoff to water quality manholes for pretreatment followed by vegetated swales for water quality treatment prior to releasing into Saum Creek.

Post-Developed Basin Areas

Table 3 shows the proposed impervious and pervious areas for the property (See Technical Appendix: Exhibits – Post-Developed Site Conditions). An impervious area of 2,640 ft^2 per lot was assumed following Clean Water Services guidelines for post-developed conditions. The post-developed site will be approximately 44 percent impervious.

Basin	ft^2	Acres
Post-Developed Total Basin Area on Westside	***352,697***	***8.10***
Impervious Area (Includes SW 65th Ave Improvements)	194,710	4.47
Pervious Area	157,987	3.63
Post-Developed Total Basin Area on Eastside	***400,904***	***9.20***
Impervious Area (Includes SW Borland Rd)	212,364	4.88
Pervious Area	188,540	4.32
Pervious Area Not Impacted by Development	***169,475***	***3.89***
[1]Total Post-Developed Area	**923,076**	**21.20**

[1]Includes New Impervious Area from SW Borland Rd and SW 65th Ave Improvements

Table 3 – Post-Developed Onsite Basin Areas

HYDROLOGIC ANALYSIS DESIGN GUIDELINES

Design Guidelines

The site is located within the jurisdiction of the City of Tualatin and Clean Water Services. The guidelines used for the design of this project reflect current Clean Water Services Design and Construction Standards, issued in June of 2007 and updated in 2009.

Hydrograph Method

Naturally occurring rainstorms dissipate over long periods of time. An effective way of estimating storm rainfall is by using the hydrograph method. The Santa Barbara Urban Hydrograph (SBUH) method was used to develop runoff rates. The computer software XPSTORM was used in modeling the hydrology during the existing and post-developed storm events to determine the increase in runoff after the development. Additionally, XPSTORM will be used to size the proposed conveyance systems and rip-rap outfall protection in the finally design phase.

Design Storm

The rainfall distribution to be used for this area is the design storm of 24-hour duration based on the standard Type 1A rainfall distribution. Table 4 shows total precipitation depths for the two storm events used in the analysis, which were used as multipliers for the Type 1A 24-hour rainfall distribution.

Recurrence Interval (Years)	Total Precipitation Depth (inches)
25	3.90
100	4.50

Table 4 - Design Storms

RUNOFF PARAMETERS

Curve Number
The major factors for determining the CN values are hydrologic soil group, cover type, treatment, hydrologic condition, and antecedent runoff condition. The curve number represents runoff potential from the ground. Tables 2-2a and 2-2c from the TR55 Urban Hydrology for Small Watersheds was used to determine the appropriate curve numbers (See Technical Appendix: Exhibits – Tables 2-2a and 2-2c Runoff Curve Numbers).

The existing site was given a curve number of 71 for pervious area, which corresponds to Meadow-continuous grass, protected from grazing and generally mowed for hay with C soils. The post-developed site was given a curve number of 86 for the disturbed pervious area, which corresponds to open space with less than 50% covered in grass (71 was used for the remaining undisturbed area). A curve number of 98 was used for all impervious area.

Time of Concentration
Two pathways for the time of concentration were calculated for the existing site using the TR-55 Method, the existing contours and assuming the site was dense grass. The calculated time of concentration of 26 minutes was used for the existing site (See Technical Appendix: Calculations – Time of Concentration). A time of concentration for the post-developed site and all other offsite basins was assumed to be 5 minutes.

Basin Runoff
The existing and post-developed runoff rates for the site are shown in Table 5 (See Technical Appendix: Hydrographs).

Recurrence Interval (Years)	Existing Runoff Rate (cfs)	[1]Post-Developed Runoff Rate (cfs)
25	4.79	14.62
100	6.71	17.61

[1]Incudes Runoff from Proposed Road Improvements Draining to Site

Table 5 - Basin Runoff Rates

HYDRAULIC ANALYSIS AND DESIGN CHARACTERISTICS

System Characteristics
The stormwater conveyance system will be sized in the final design phase of the project to convey all storm events up to and including the 100-year storm event without any out of system flooding.

WATER QUALITY

Water Quality Guidelines
Per Clean Water Services guidelines, water quality treatment facilities are required to be designed to treat the rainfall of 0.36" over a 4-hour period with a return period of 96-hours. The following shows the calculated treatment flow rate for the design the water quality treatment facilities.

$$\text{Water Quality Volume (WQV)} = \frac{\text{Impervious Area (ft}^2\text{) X 0.36 (in)}}{\text{12 (in/ft)}}$$

$$\text{Water Quality Flow (WQF)} = \frac{\text{WQV}}{\text{14,400 seconds}}$$

Water Quality Calculations

Two vegetated swales will be constructed to treat the stormwater runoff from the post-developed impervious areas. The water quality flow calculations for each are shown below:

Westside Swale (35 Lots):

- Impervious Area (on the lots)= 35 lots X 2,640 sf = 92,400 sf
- Impervious Area (Roads/sidewalks) = 91,130 sf
- Improved Portion of SW 65th Ave Draining to Westside = 11,181 sf
- **Total Impervious Area = 194,710 sf**

$$\text{Water Quality Volume (WQV)} = \frac{194{,}711\ ft^2 \times 0.36\ (in)}{12\ (in/ft)} = 5{,}841\ ft^3$$

$$\text{Water Quality Flow (WQF)} = \frac{5{,}841\ ft^3}{14{,}400\ \text{seconds}} = \mathbf{0.41\ cfs}$$

Eastside Swale (44 Lots):

- Impervious Area (on the lots)= 44 lots X 2,640 sf = 116,160 sf
- Impervious Area (Roads/sidewalks) = 94,570 sf
- Improved Portion of SW Borland Draining to Eastside = 1,634 sf
- **Total Impervious Area = 212,364 sf**

$$\text{Water Quality Volume (WQV)} = \frac{212{,}364\ ft^2 \times 0.36\ (in)}{12\ (in/ft)} = 6{,}371\ ft^3$$

$$\text{Water Quality Flow (WQF)} = \frac{6{,}371\ ft^3}{14{,}400\ \text{seconds}} = \mathbf{0.44\ cfs}$$

Water Quality Swale

Per Sections 3-5-280 and 3-5-430 of the City of Tualatin's Municipal Code, water quality facilities are to be located outside of the defined wetland area of existing or created wetlands. Therefore, both swales will be located outside of the existing delineated wetlands. Each vegetated swale will be designed and constructed to follow Section 4.06.2 of CWS Design and Construction Standards with the minimum dimensions:

- Design Flow: Water Quality Flow
- Hydraulic Residence Time: 9 minutes
- Maximum Water Design Depth: 0.5 feet
- Minimum Freeboard: 1 foot (facilities will not be protected from high flows)
- Manning 'n' value: 0.24
- Minimum Length = 100 feet
- Bottom Width = 2 feet
- Side Slopes = 3:1 in treatment area
- Minimum Channel Slope = 0.5%

Preliminary vegetated swales have been included on the site plans; however, final design calculations have not been included in this report. Final design for the swales will be presented in the Final Storm Drainage Report.

Water Quality Manholes

All stormwater runoff that is conveyed to the proposed conveyance systems will be pretreated in water quality manholes (except for the area from SW 65th Ave conveyed directly to the westside swale). Per

Section 4.06.1 of CWS Design and Construction Standards, the pretreatment volume available for a water quality manhole is $20ft^3$/1.0 cfs, up to the 25-year flow (See Technical Appendix: Hydrographs – 25-Year Post-Developed Runoff (East and West Systems). Each water quality manhole will be sized in the final design phase of the project for the 25-year storm event.

SUMMARY

The proposed stormwater management system for Sagert Farm Subdivision includes two vegetated swales separated by topography. Each have been shown on the plans and will be finalized in the final design phase of the project. The stormwater conveyance system for the project will consist of storm pipe, catch basin/inlets and manholes which will convey stormwater runoff to each of the new swales. The conveyance system will be sized in the final design phase of the project. The proposed stormwater management system will meet the requirements of the City of Tualatin and Clean Water Services.

TECHNICAL APPENDIX

Exhibits

- Figure 1 – Vicinity Map
- Figure 2 – Site Location
- Hydrologic Soil Group-Washington County
- Tables 2-2a and 2-2c Runoff Curve Numbers
- Existing Site Conditions (Exhibit 1a and 1b)
- Post-Developed Site Conditions (Exhibits 2a and 2b)
- Water Quality Manhole – Drawing No. 240

Drawings

- Sheet C100 – Overall Existing Conditions
- Sheets C121-C124 – Phase 2 Grading & ESC Plans
- Sheet C200 – Overall Site Plan
- Sheets C211-C214 – Street & Storm Plan
- Sheet C300 – Overall Composite Utility Plan

Calculations

- Time of Concentration

Hydrographs

- Existing Runoff Hydrograph
- Post-Developed Runoff Hydrograph (Entire Site Including Borland Rd and SW 65th Ave)
- Pretreatment Hydrographs (Excluding Non-Impacted Pervious Area)

XPSTORM OUTPUT – Not Included in the Preliminary Storm Drainage Report

Geotechnical Report

- Geotechnical Engineering Report, GeoPacific Engineering, Inc, December 11, 2013

REFERENCES

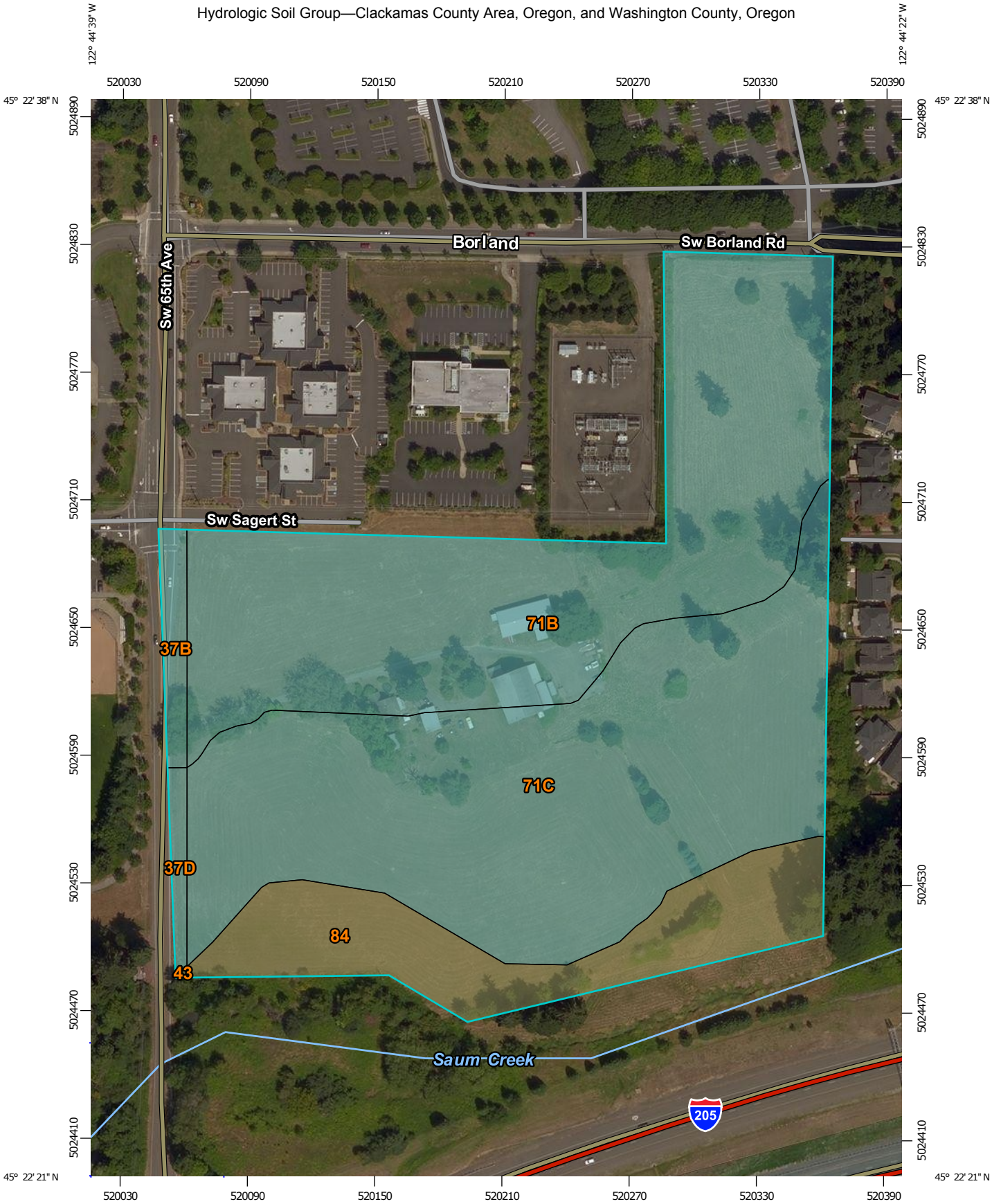
1. Design and Construction Standards for Sanitary Sewer and Surface Water Management Issued June 2007 – Clean Water Services

EXHIBITS

Figure 1 - Vicinity Map

Figure 2 - Site Location

Hydrologic Soil Group—Clackamas County Area, Oregon, and Washington County, Oregon



Map Scale: 1:2,460 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0 35 70 140 210

Feet: 0 100 200 400 600

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Lines

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Soil Rating Points

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Water Features

Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Clackamas County Area, Oregon
Survey Area Data: Version 9, Sep 19, 2014

Soil Survey Area: Washington County, Oregon
Survey Area Data: Version 12, Sep 19, 2014

Your area of interest (AOI) includes more than one soil survey area. These survey areas may have been mapped at different scales, with a different land use in mind, at different times, or at different levels of detail. This may result in map unit symbols, soil properties, and interpretations that do not completely agree across soil survey area boundaries.

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Aug 3, 2014—Aug 23, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Hydrologic Soil Group— Summary by Map Unit — Clackamas County Area, Oregon (OR610)				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
71B	Quatama loam, 3 to 8 percent slopes	C	7.4	39.7%
71C	Quatama loam, 8 to 15 percent slopes	C	8.2	44.0%
84	Wapato silty clay loam	C/D	2.6	13.7%
Subtotals for Soil Survey Area			**18.2**	**97.4%**
Totals for Area of Interest			**18.7**	**100.0%**

Hydrologic Soil Group— Summary by Map Unit — Washington County, Oregon (OR067)				
Map unit symbol	**Map unit name**	**Rating**	**Acres in AOI**	**Percent of AOI**
37B	Quatama loam, 3 to 7 percent slopes	C	0.3	1.7%
37D	Quatama loam, 12 to 20 percent slopes	C	0.2	0.9%
43	Wapato silty clay loam	C/D	0.0	0.0%
Subtotals for Soil Survey Area			**0.5**	**2.6%**
Totals for Area of Interest			**18.7**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: *None Specified*

Tie-break Rule: Higher

Table 2-2a Runoff curve numbers for urban areas [1]

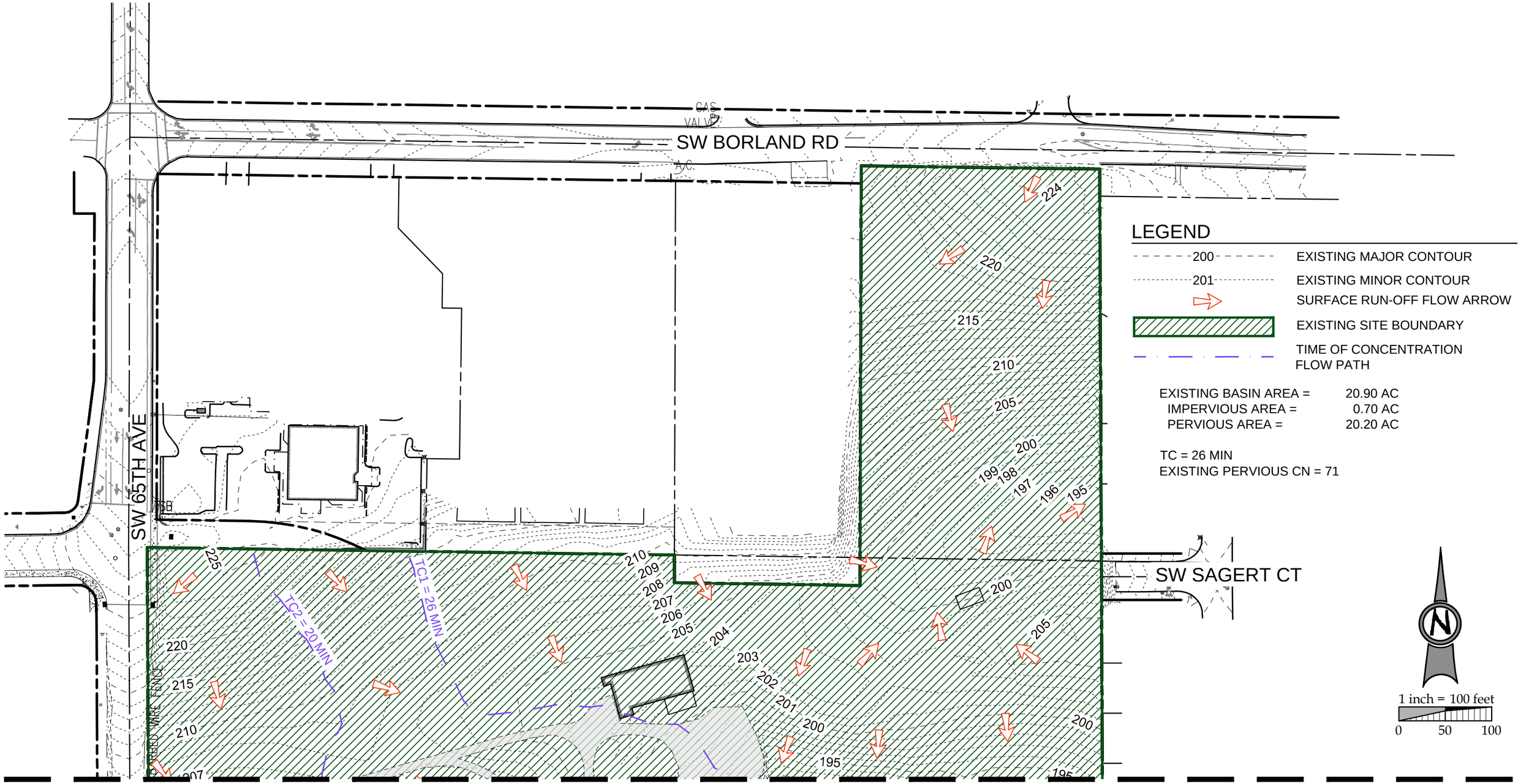
Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2c Runoff curve numbers for other agricultural lands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.
2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.
3 *Poor*: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
4 Actual curve number is less than 30; use CN = 30 for runoff computations.
5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.



SEE SHEET 1b FOR CONTINUATION

3J CONSULTING, INC

CIVIL ENGINEERING
WATER RESOURCES
LAND USE PLANNING

EXISTING SITE CONDITIONS
SAGERT FARMS SUBDIVISION

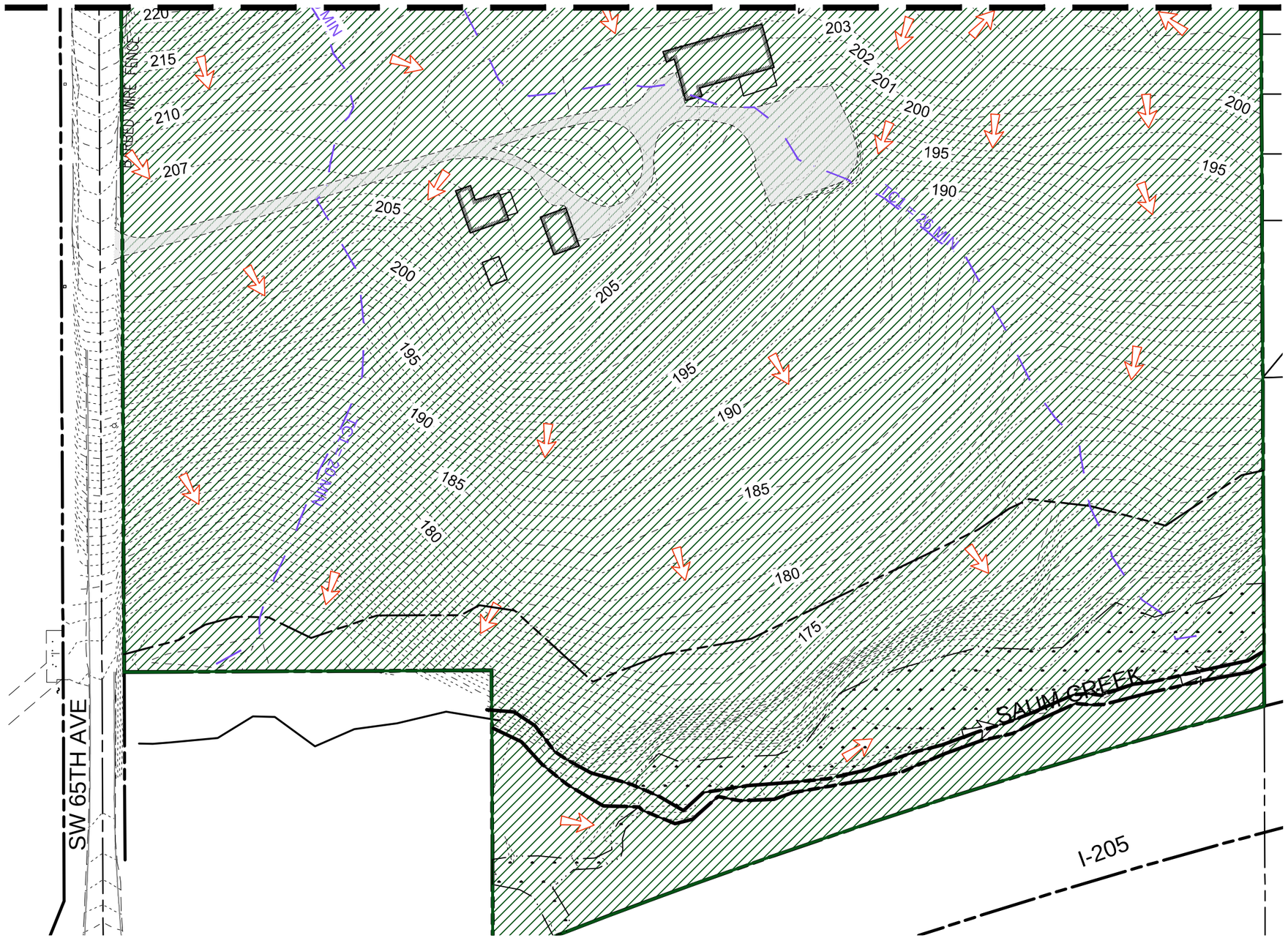
Drainage Report

Exhibit 1a

Date: 06/02/15

By: KEF

SEE SHEET 1a FOR CONTINUATION



LEGEND

200	EXISTING MAJOR CONTOUR
201	EXISTING MINOR CONTOUR
	SURFACE RUN-OFF FLOW ARROW
	EXISTING SITE BOUNDARY
	EXISTING GRAVEL
	TIME OF CONCENTRATION FLOW PATH
	EXISTING WETLAND
	TOP OF SAUM CREEK

EXISTING BASIN AREA = 20.90 AC
IMPERVIOUS AREA = 0.70 AC
PERVIOUS AREA = 20.20 AC

TC = 26 MIN
EXISTING PERVIOUS CN = 71

N

1 inch = 100 feet
0 50 100

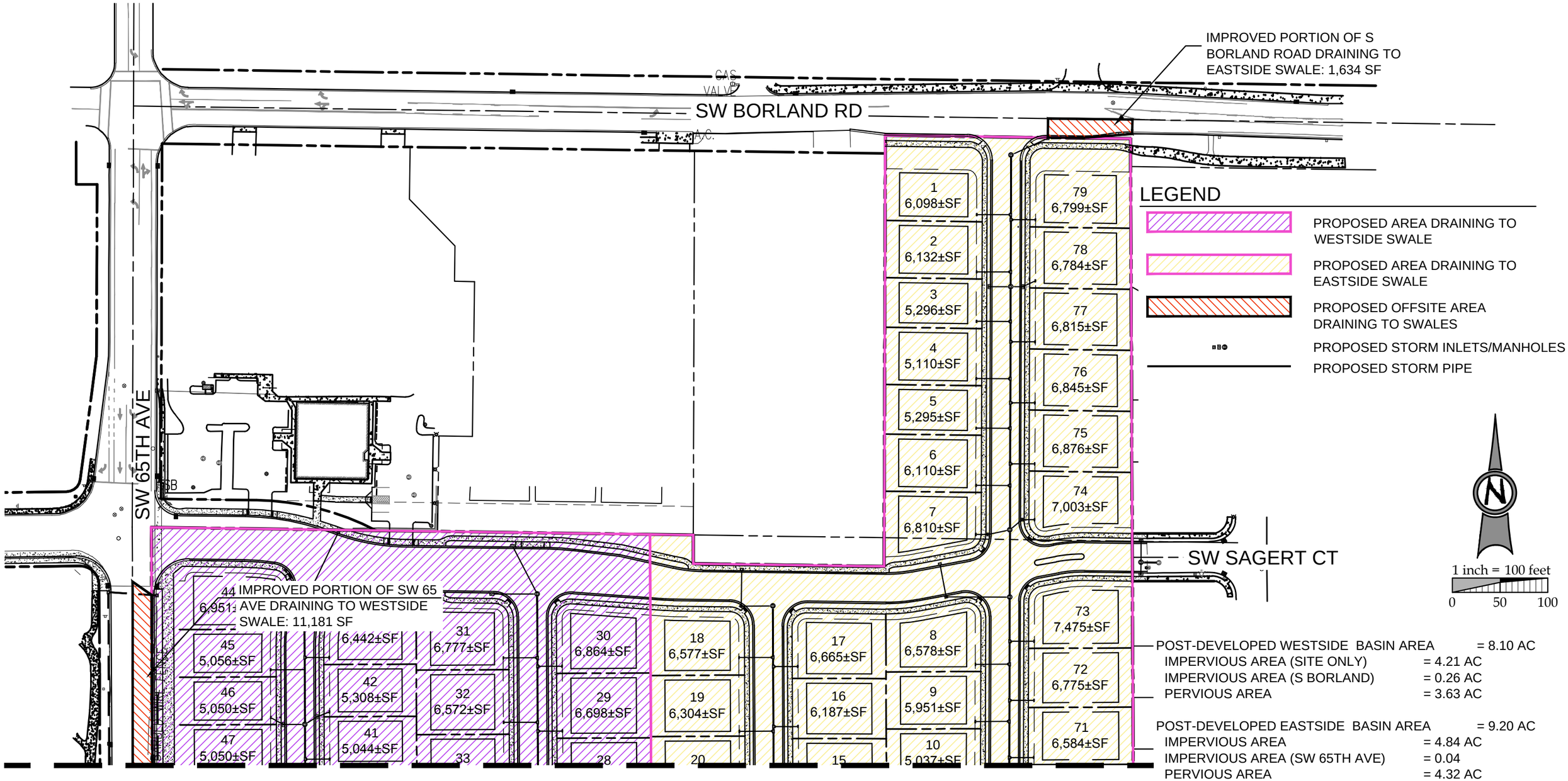
3J CONSULTING, INC
CIVIL ENGINEERING
WATER RESOURCES
LAND USE PLANNING

EXISTING SITE CONDITIONS
SAGERT FARMS SUBDIVISION

Drainage Report

Exhibit 1b

Date: 06/04/15 By: KEF



SEE SHEET 2b FOR CONTINUATION

3J CONSULTING, INC

CIVIL ENGINEERING
WATER RESOURCES
LAND USE PLANNING

POST-DEVELOPED SITE CONDITIONS
SAGERT FARMS SUBDIVISION

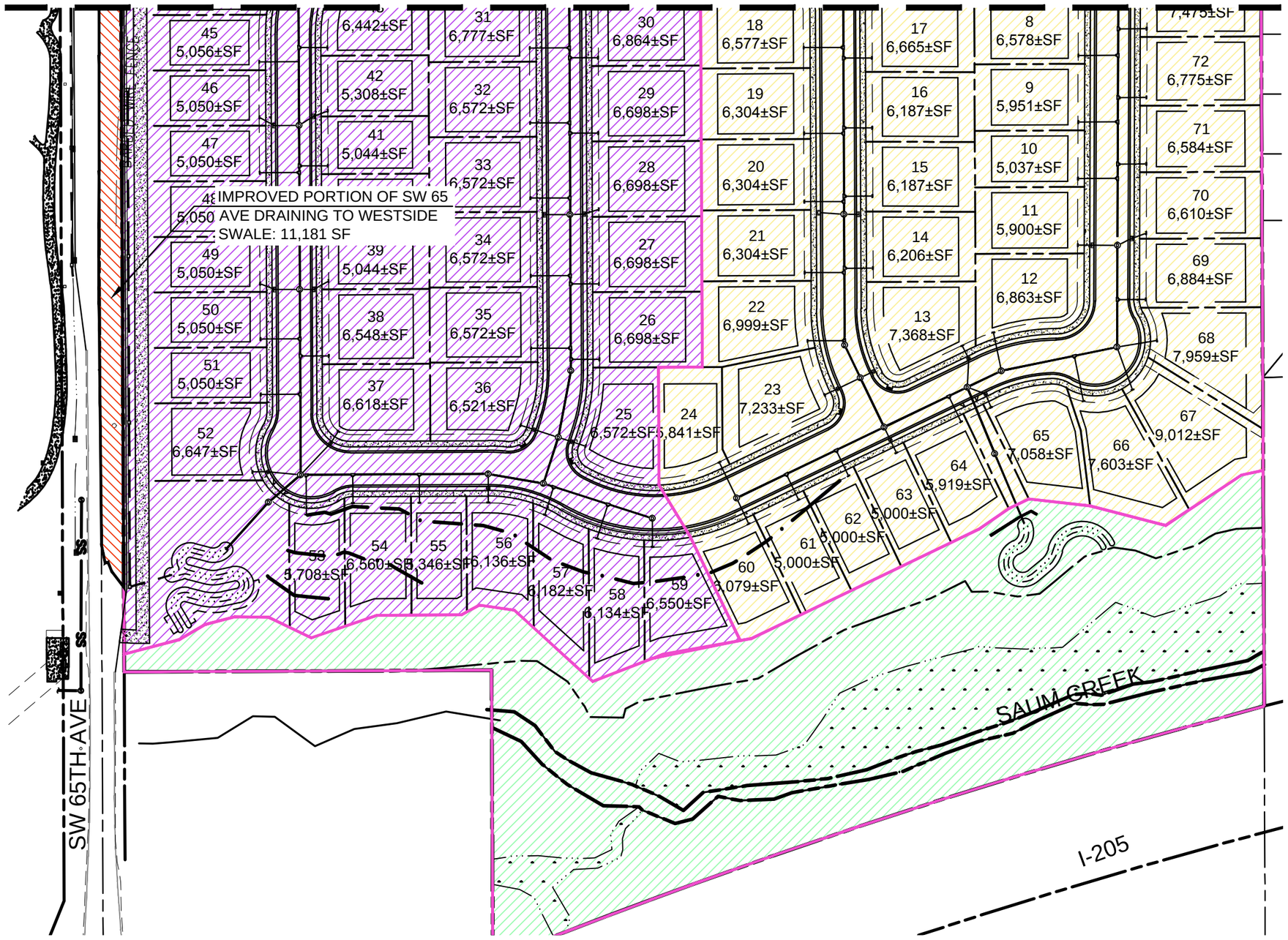
Drainage Report

Exhibit 2a

Date: 06/02/15

By: KEF

SEE SHEET 2a FOR CONTINUATION



LEGEND

- EXISTING WETLAND
- TOP OF SAUM CREEK
- PROPOSED AREA DRAINING TO WESTSIDE SWALE
- PROPOSED AREA DRAINING TO EASTSIDE SWALE
- PROPOSED OFFSITE AREA DRAINING TO SWALES
- AREA DRAINING TO DIRECTLY TO SAUM CREEK
- PROPOSED STORM INLETS/MANHOLES
- PROPOSED STORM PIPE
- PROPOSED VEGETATED SWALES

POST-DEVELOPED WESTSIDE BASIN AREA = 8.10 AC
- IMPERVIOUS AREA (SITE ONLY) = 4.21 AC
- IMPERVIOUS AREA (S BORLAND) = 0.26 AC
- PERVIOUS AREA = 3.63 AC

POST-DEVELOPED EASTSIDE BASIN AREA = 9.20 AC
- IMPERVIOUS AREA = 4.84 AC
- IMPERVIOUS AREA (SW 65TH AVE) = 0.04
- PERVIOUS AREA = 4.32 AC

PERVIOUS AREA NOT IMPACTED BY DEVELOPMENT = 3.89 AC

N

1 inch = 100 feet

0 50 100

3J CONSULTING, INC

CIVIL ENGINEERING
WATER RESOURCES
LAND USE PLANNING

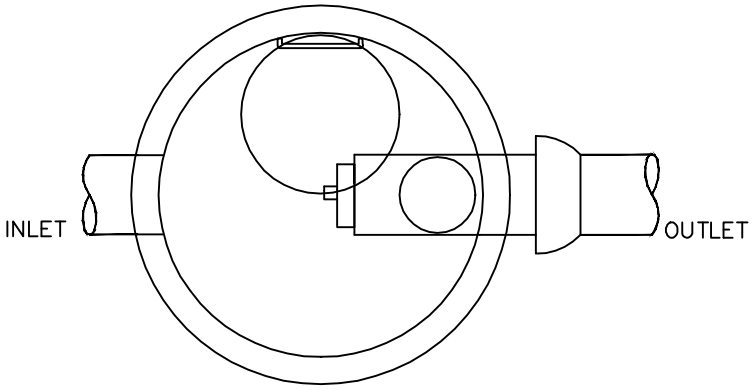
POST-DEVELOPED SITE CONDITIONS
SAGERT FARMS SUBDIVISION

Drainage Report

Exhibit 2b

Date: 06/02/15

By: KEF



PLAN

NOTES:
1. ALL MANHOLE SECTIONS SHALL CONFORM TO THE REQUIREMENTS OF ASTM C-478 AND APPLICABLE PROVISIONS OF STD. MANHOLE DRAWING NO. 010
2. INLET AND OUTLET PIPE NOT TO EXCEED 18"DIA.
3. PROVIDE SPECIAL DETAIL FOR OUTLET FLOW CONTROL EXCEEDING 18" DIA.
4. ALL OUTLETS SHALL HAVE FLOW CONTROL DEVICE.

SUMP VOLUME AVAILABLE

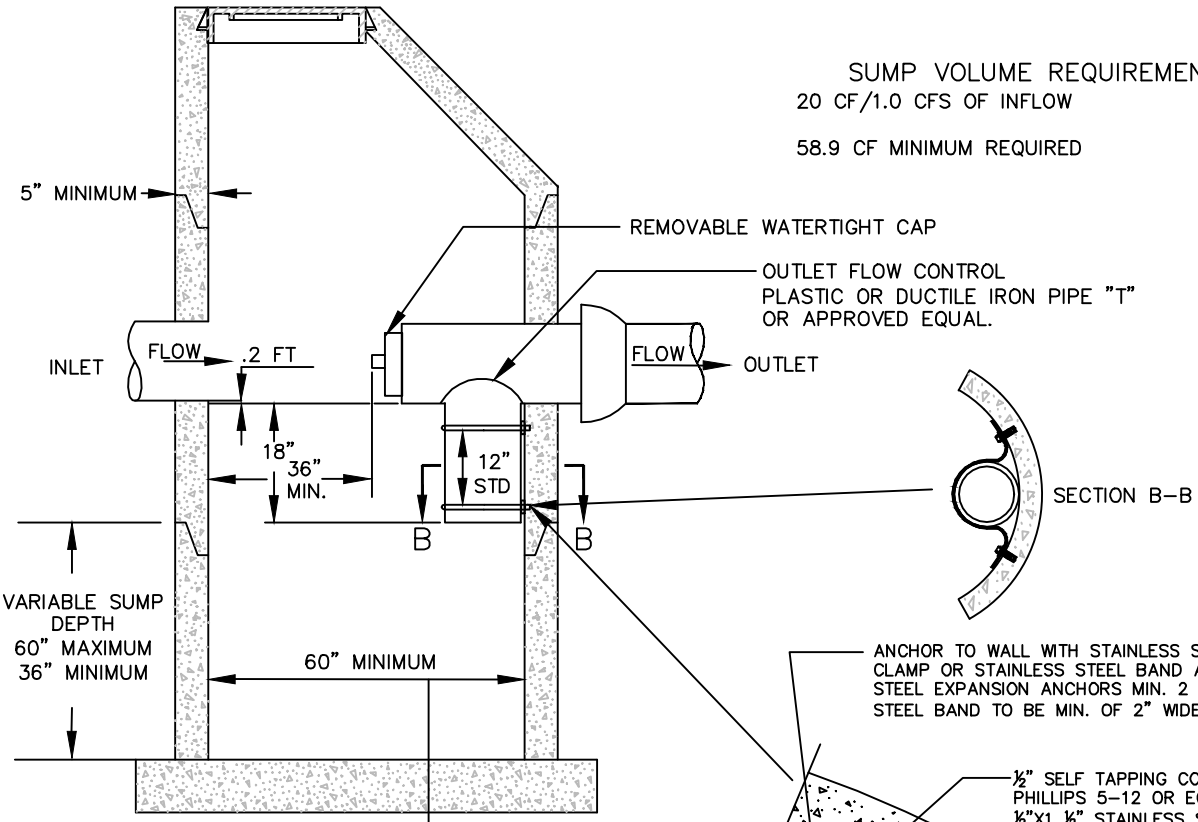
	MINIMUM	MAXIMUM
60" M.H.=	58.9 CF	98.1 CF
72" M.H.=	84.8 CF	141.3 CF
84" M.H.=	115.4 CF	192.3 CF

PROVIDE SPECIAL DETAIL FOR VOLUME REQUIREMENTS EXCEEDING 192.3 CF

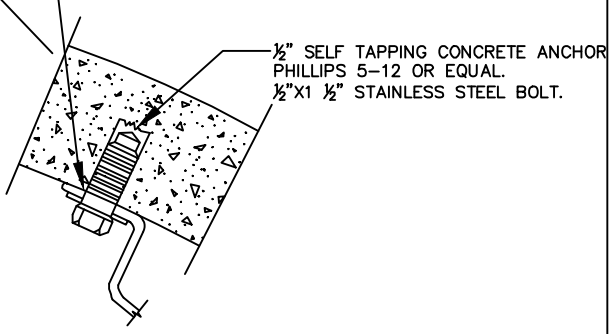
SUMP VOLUME REQUIREMENTS

20 CF/1.0 CFS OF INFLOW

58.9 CF MINIMUM REQUIRED



MANHOLE DIAMETER TO BE DETERMINED BY:
1. SUMP VOLUME REQUIREMENTS.
2 NUMBER AND SIZE OF INLETS & OUTLETS.



CLAMP DETAIL
(SECTION A-A)
N.T.S.

WATER QUALITY MANHOLE (MECHANICAL)

DRAWING NO. 240

REVISED 12-06

Clean Water Services
Our commitment is clear.

DRAWINGS

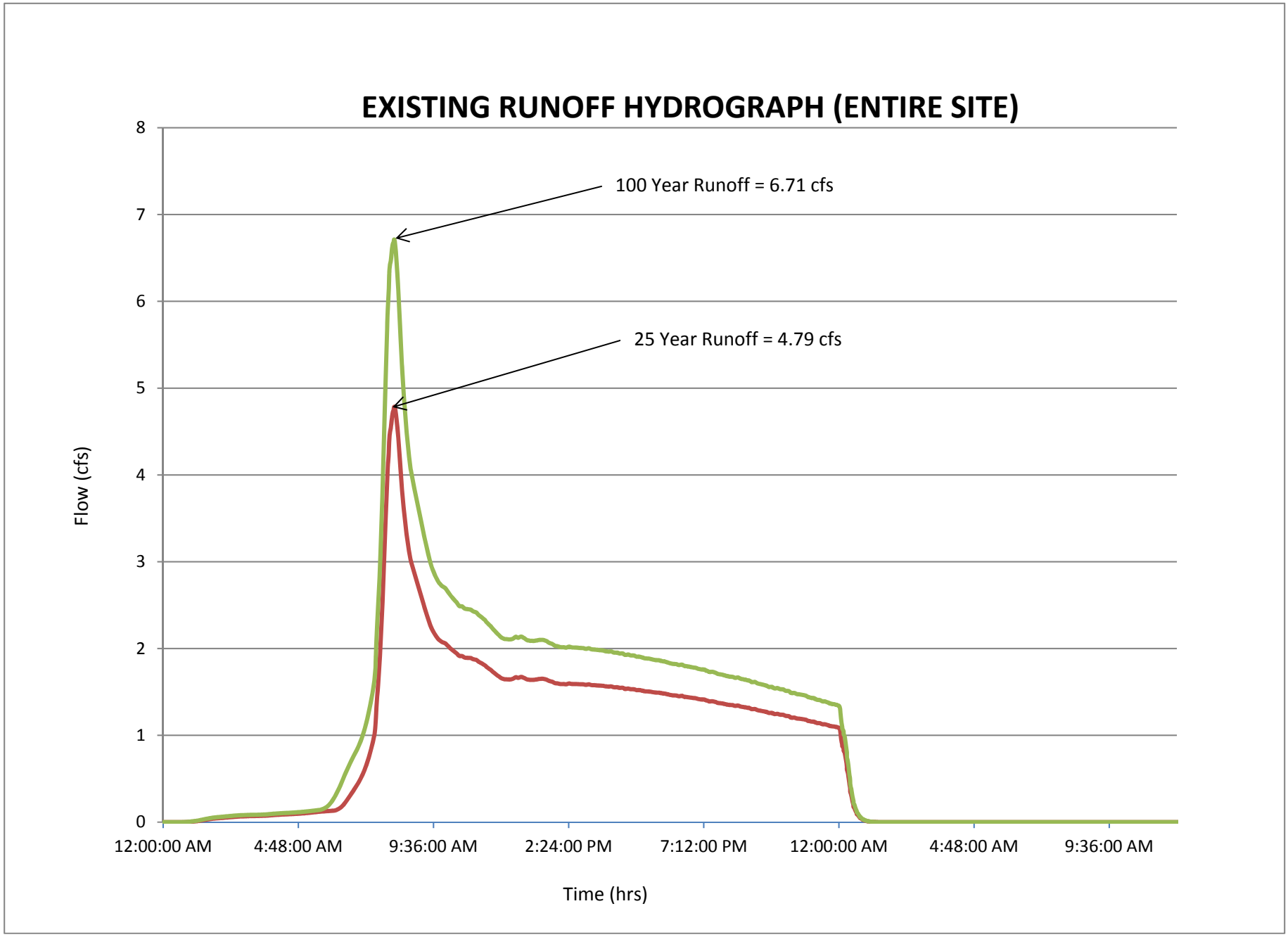
CALCULATIONS

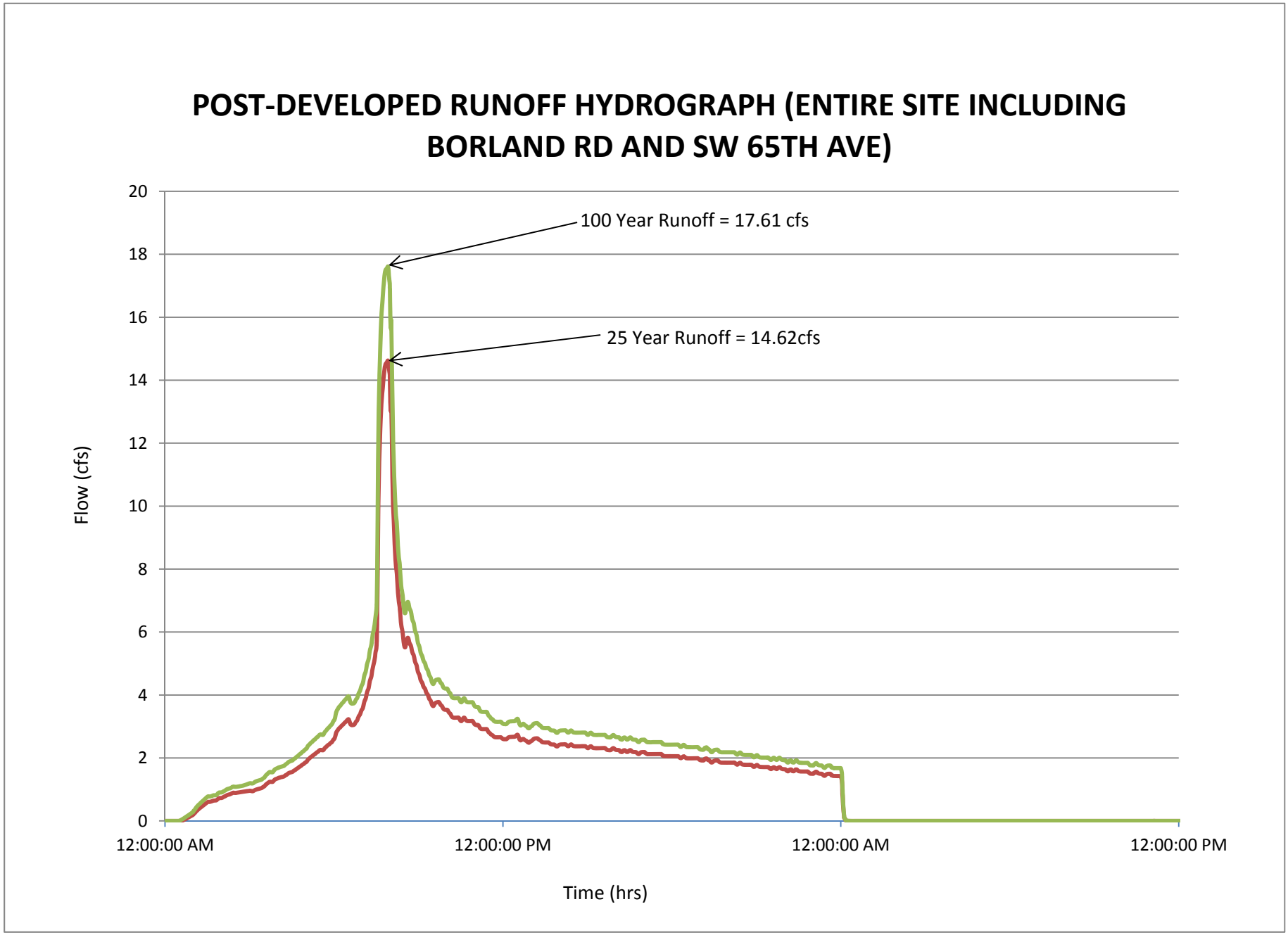
Time of Concentration

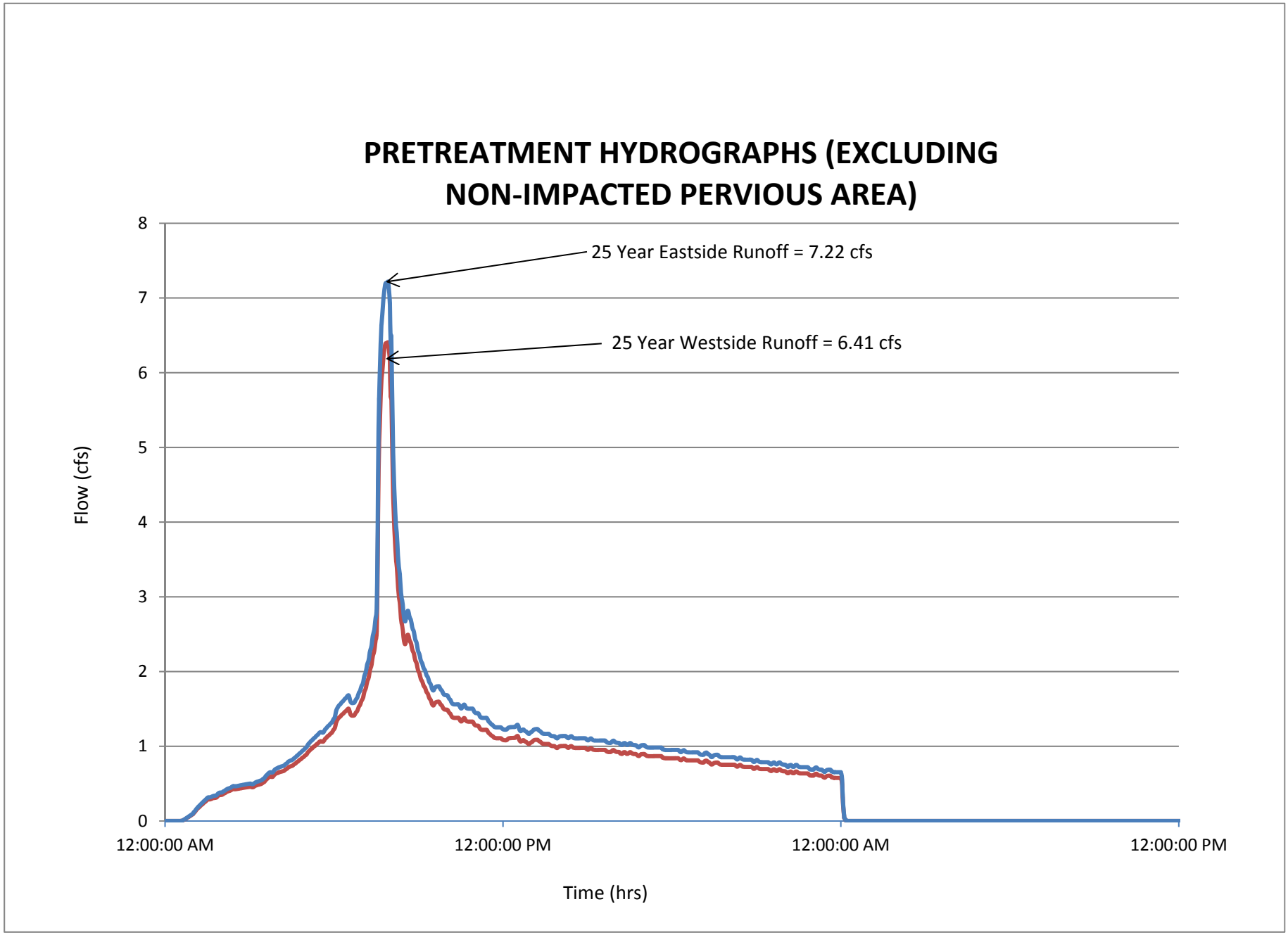
PROJECT NAME:	Sagert Farms Subdivision		
PROJECT NO.	13159	**BY** KEF	**DATE** 5/28/2015

	TC1	TC2	
SHEET FLOW			
INPUT	**VALUE**	**VALUE**	**VALUE**
Surface Description	**Type 4 Cultivated (residue > 20%)**	**Type 4 Cultivated (residue > 20%)**	**Type 5 Grass (short prairie)**
Manning's "n"	**0.17**	**0.17**	**0.15**
Flow Length, L	**300** ft	**300** ft	**0** ft
2-Yr 24 Hour Rainfall, P_2	**2.5** in	**2.5** in	**2.5** in
Land Slope, s	**0.03711** ft/ft	**0.06206** ft/ft	**0.0025** ft/ft
OUTPUT			
Travel Time	0.38 hr	0.31 hr	0.00 hr
SHALLOW CONCENTRATED FLOW			
INPUT	**VALUE**	**VALUE**	**VALUE**
Surface Description	**Unpaved**	**Unpaved**	**Unpaved**
Flow Length, L	**768** ft	**476** ft	**0** ft
Watercourse Slope*, s	**0.0627** ft/ft	**0.09828** ft/ft	**0.027** ft/ft
OUTPUT			
Average Velocity, V	4.04 ft/s	5.06 ft/s	2.65 ft/s
Travel Time	0.053 hr	0.026 hr	0.000 hr
CHANNEL FLOW			
INPUT	**VALUE**	**VALUE**	**VALUE**
Cross Sectional Flow Area, a	**7.5** ft^2	**7.5** ft^2	**15.05** ft^2
Wetted Perimeter, P_w	**11.28** ft	**11.28** ft	**7.69** ft
Channel Slope, s	**0.003** ft/ft	**0.003** ft/ft	**0.00** ft/ft
Manning's "n"	**0.24**	**0.24**	**0.24**
Flow Length, L	**0** ft	**0** ft	**0** ft
OUTPUT			
Average Velocity	0.26 ft/s	0.26 ft/s	0.53 ft/s
Hydraulic Radius, r = a / P_w	0.66 ft	0.66 ft	1.96 ft
Travel Time	0.00 hr	0.00 hr	0.00 hr
Watershed or Subarea T_c =	**0.44 hr**	**0.34 hr**	**0.00 hr**
Watershed or Subarea T_c =	**26 minutes**	**20 minutes**	**0 minutes**

HYDROGRAPHS


EXISTING RUNOFF HYDROGRAPH (ENTIRE SITE)
100 Year Runoff = 6.71 cfs
25 Year Runoff = 4.79 cfs
Flow (cfs)
0
1
2
3
4
5
6
7
8
12:00:00 AM
4:48:00 AM
9:36:00 AM
2:24:00 PM
7:12:00 PM
12:00:00 AM
4:48:00 AM
9:36:00 AM
Time (hrs)


POST-DEVELOPED RUNOFF HYDROGRAPH (ENTIRE SITE INCLUDING BORLAND RD AND SW 65TH AVE)
100 Year Runoff = 17.61 cfs
25 Year Runoff = 14.62cfs
Flow (cfs)
20
18
16
14
12
10
8
6
4
2
0
12:00:00 AM
12:00:00 PM
12:00:00 AM
12:00:00 PM
Time (hrs)


PRETREATMENT HYDROGRAPHS (EXCLUDING NON-IMPACTED PERVIOUS AREA)
25 Year Eastside Runoff = 7.22 cfs
25 Year Westside Runoff = 6.41 cfs
Flow (cfs)
0
1
2
3
4
5
6
7
8
12:00:00 AM
12:00:00 PM
12:00:00 AM
12:00:00 PM
Time (hrs)

XPSTORM OUTPUT
Not Included

GEOTECHNICAL REPORT

GeoPacific Engineering, Inc.

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

December 11, 2013
GeoPacific Project No. 13-3204

Mike Loomis
Lennar Northwest, Inc.
11807 NE 99th Street, Suite 1170
Vancouver, Washington 98682

Jesse Nemec
J.T. Smith Companies
5282 Meadows Road, Suite 171
Lake Oswego, Oregon 97035

Via e-mail with hard copies mailed

Subject: **GEOTECHNICAL ENGINEERING REPORT**
SAGERT PROPERTY
20130 SW 65TH AVENUE
TUALATIN, OREGON

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above referenced project. The purpose of this study was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific proposal P-4637, dated October 30, 2013, and your subsequent authorization of our agreement and *General Conditions for Geotechnical Services.*

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site is located on the east side of SW 65th Avenue in Tualatin, Oregon (Figure 1). The area of the planned development totals approximately 21.1 acres. A single family residence, a barn, and several small to large outbuildings are present in the center of the site. The topography on the site is generally moderately sloping down to the south at grades of 10 to 15 percent. Topography in the northeast portion of the site slopes down to a low lying area at the eastern property boundary, located approximately 350 feet south of the northeast corner of the property. Slopes increase in steepness near the southern property boundary, adjacent to Saum Creek, to grades of approximately 20 to 25 percent. Vegetation on the site consists primarily of grass, and sparse medium to large trees.

It is our understanding that the proposed development includes grading the site to support lots for new single-family homes, approximately 4,000 feet of new public streets, stormwater management facilities, and associated underground utilities. The current site plan (Figure 2) shows a total of 90 lots and one water quality tract. We understand that the existing residence and outbuildings are to be demolished or otherwise removed from the site. A grading plan has not been developed for the site; however, we anticipate moderate cuts and fills will be required and possibly retaining walls.

REGIONAL GEOLOGIC SETTING

Regionally, the subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

The site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley (Yeats et al., 1996; Evarts, 2004). The last of these outburst floods occurred about 10,000 years ago. These deposits typically consist of horizontally layered, micaceous, silt to coarse sand forming poorly-defined to distinct beds less than 3 feet thick.

Underlying the Willamette Formation is the Columbia River Basalt Formation (Beeson el al., 1989). The Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalts are a thick sequence of lava flows. The basalts are composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. Individual basalt flow units typically range from 25 to 125 feet thick and interflow zones are typically vesicular, scoriaceous, brecciated, and sometimes include sedimentary rocks.

At least three major source zones capable of generating damaging earthquakes are thought to exist in the vicinity of the subject site. These include the Gales Creek-Newberg-Mt. Angel Structural Zone, the Portland Hills Fault Zone, and the Cascadia Subduction Zone.

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies about 13 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A recent geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek or Newberg Faults (the faults closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills, and is about 6.7 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills, and is about 4.8 miles northeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000). No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies roughly along the Oregon Coast at depths of 20 and 40 kilometers below the ocean surface.

FIELD EXPLORATION

Subsurface conditions were explored on November 15, 2013 by excavating 14 test pits to depths of 4.5 to 12 feet below the ground surface, using a John Deer 310E backhoe with a 2-foot-wide toothed bucket. The approximate test pit locations are shown on the attached site plan (Figure 2). It should be noted that exploration locations were determined in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

During excavation of the test pits, a GeoPacific engineer observed and recorded soil information such as color, stratigraphy, strength, and soil moisture. Soils were classified in general accordance with the Unified Soil Classification System (USCS).

At the completion of each test pit, the excavation was backfilled using the excavated soils, and tamped with the excavator bucket. This backfill should not be expected to behave as engineered fill and some settling and/or erosion of the ground surface may occur.

SUBSURFACE CONDITIONS

Soil

The following report sections summarize subsurface conditions anticipated at the site, based on our exploration program. On-site soils consist of topsoil, disturbed native material, undocumented fill, and Willamette Formation materials, as described below.

Topsoil: In all test pits, the ground surface was directly underlain by topsoil consisting of dark brown, moderately organic SILT (OL-ML) with fine roots throughout. Topsoil thickness in test pits ranged from about 3 to 12 inches. There is the potential for some tree roots or thicker topsoil zones in forested areas on site.

Disturbed Native: Underlying the topsoil in test pits TP-1, TP-2, TP-3, TP-4, TP-5, TP-8, TP-9, TP-10, and TP-13 was disturbed native material consisting of brown, low organic, SILT (ML). The layer of disturbed native material was generally soft and likely resulted from previous agricultural operations on the site. The disturbed native material generally extended to depths of 14 to 24 inches below ground surface (bgs). However, in test pit TP-9 the disturbed native material extended to a depth of 3 feet.

Undocumented Fill: Underlying the topsoil in test pits TP-11 and TP-12 was undocumented fill material. The fill material generally consisted of SILT (ML) with varying amounts of gravel and charred organics. The silt fill material generally had a soft consistency and moisture contents were generally moist. The depth of undocumented fill material in test pits TP-11 and TP-12 was 2.5 and 3 feet bgs, respectively. We anticipate that additional localized fill zones may be present in the vicinity of the existing residence.

Willamette Formation: Underlying disturbed native material in test pits TP-1, TP-2, TP-3, TP-4, TP-5, TP-8, TP-9, TP-10, and TP-13, underlying the topsoil in test pits TP-6, TP-7, and TP-14, and underlying the undocumented fill material in test pits TP-11 and TP-12 was silt belonging to the Willamette Formation. The Willamette Formation material was generally brown in color with increasing amounts of sand content with depth. In test pits TP-1, TP-2, TP-6, TP-7, TP-11, TP-12, TP-13, the Willamette Formation material graded to silty fine sand with a medium dense consistency. Material belonging to the Willamette Formation extended beyond the maximum depths of our explorations (9.5 feet bgs).

Groundwater

On November 15, 2013, groundwater seepage was not encountered in any of the test pits, except TP-9. In test pit TP-9 seepage was encountered at a depth of approximately 1.5 feet and was visually estimated at approximately 3 gallons per minute.

The groundwater conditions reported are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. It is anticipated that groundwater conditions will vary depending on the time of year, rainfall, local subsurface conditions, changes in site utilization, and other factors. During periods of heavy and prolonged precipitation, shallow perched groundwater conditions often occur over fine-grained native deposits such as those beneath the site, particularly during the wet season.

INFILTRATION TESTING

On November 15, 2013, GeoPacific performed three pushed-pipe falling head infiltration tests at the approximate locations shown on Figure 2. The tests were conducted in 6-inch diameter pipes pushed into the native soil at depths of approximately 4.5, 8.5, and 11 feet below the ground surface. The infiltration tests were performed at or near the locations of test pits TP-9, TP-10, and TP-13.

The test holes were pre-saturated for 4 hours prior to performing the tests. During the tests, the water level was measured over 30 minute intervals with approximate head pressures ranging between 4 and 24 inches until three successive measurements showing a consistent infiltration rate were achieved. Approximate test locations are shown in Figure 2. Table 1 presents a summary of our infiltration test measurement results.

Table 1. Results of Infiltration Testing

Test Pit	Depth (feet)	Soil Type	Infiltration Rate(in/hr)	Hydraulic Head Range (inches)
TP-9	4.5	SILT (ML)	0.1	6-18
TP-10	8.5	Sandy SILT (ML)	0.3	6-18
TP-13	11	Silty fine SAND (SM)	6	2-14

The test results indicate very low infiltration rates in the silt and sandy silt materials, and low to moderate infiltration rates in the silty fine sand material. The measured rates reflect vertical flow pathways only. Table 2 summarizes the depths at which silty fine sand was observed in the test pit explorations.

Table 2. Summary of Depths to Silty Fine Sand

Exploration Designation	Depth of Silty Fine SAND (SM) (feet bgs)
TP-1	8
TP-2	6
TP-6	6
TP-7	7
TP-11	9
TP-12	8.5
TP-13	9

PORTABLE DYNAMIC CONE PENETROMETER TESTING

Field tests were also conducted with a Portable Dynamic Cone Penetrometer (PDCP) to determine the strength parameters of the soil for support of pavement. Correlated California Bearing Ratio (CBR) values at each test location are summarized on Table 3, for the depth intervals indicated.

Table 3. PDCP Field Test Results and Correlated CBR Values

Exploration Designation	Material Tested	Depth Interval of Test (feet)	Average Penetration Per Blow (mm)	Correlated CBR
TP-1	SILT (ML)	2 – 4	51	4
TP-3	SILT (ML)	1.5 – 3	43	5
TP-4	SILT (ML)	1. 5 – 3.5	36	6
TP-7	Sandy SILT (ML)	1 – 3	38	6

The test results indicate moderate subgrade soil conditions for support of traffic loading. A low-end CBR value of 4 was used for subgrade soils in our analyses for the proposed access roads, as discussed in a subsequent report section.

CONCLUSIONS AND RECOMMENDATIONS

Results of this study indicate that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The proposed structures may be supported on shallow foundations bearing on competent undisturbed native soils, or engineered fill, designed and constructed as recommended in this report.

Recommendations are presented below for site preparation and undocumented fill removal, engineered fill, wet weather earthwork, seismic design, structural foundations, footing drains, storm water systems, excavation conditions and utility trench backfill, pavement sections, retaining wall design and construction recommendations, general slope stability evaluation, and erosion control considerations. The

recommendations of this report assume the single-family structures will have raised floors and crawlspaces. If structures are planned with basements or concrete slab-on-grade floors, GeoPacific should be contacted for additional recommendations regarding basement retaining wall design and drainage, concrete floor slabs and moisture protection, or other issues.

General Slope Stability Evaluation

Slopes increase in steepness near the southern property boundary, adjacent to Saum Creek, to grades of approximately 25 percent. Slopes in this portion of the site are generally south-facing. The results of our geotechnical investigation indicate that the site is underlain by stiff to very stiff silt and medium dense silty sand belonging to the Willamette Formation. This material is generally considered moderately resistant to deep-seated landsliding on gentle slopes unless adversely impacted by natural processes or human activities. Based on our field reconnaissance observations, slopes near the southern property boundary are generally smooth and uniform, consistent with relatively stable conditions. No evidence of past deep-seated instability (such as scarplets, ground cracks, benches, etc.) was observed in these areas. The steepness of the slopes near the southern property boundary is likely due to gradual erosion from flows in Saum Creek.

The proposed grading plan shows a setback of 15 feet from the top of the bank of Saum Creek. It is our opinion that this setback is sufficient for slope stability and to minimize the risk of developed lots being affected by erosion of the bank by potential flows in Saum Creek.

Site Preparation and Undocumented Fill Removal

Within the areas to receive fill, proposed building footprints, or other settlement-sensitive areas, undocumented fill, disturbed native material, vegetation, and debris should be completely removed and replaced with engineered fill. Debris from clearing should be removed from the site. Undocumented fill was encountered in test pits TP-11 and TP-12 to depths of 2.5 and 3 feet, respectively. Disturbed native material was encountered to depths of 14 to 24 inches in test pits TP-1, TP-2, TP-3, TP-4, TP-5, TP-8, TP-10, and TP-13, and to a depth of 3 feet in test pit TP-9. We anticipate that undocumented fill material and disturbed native material may be re-used as engineered fill during dry weather conditions.

Organic-rich topsoil should be stripped to the relatively inorganic native soils. We anticipate that the depth of stripping will be an average of roughly 6 to 8 inches over most of the site. Deeper stripping will be needed in areas that have been tilled in the past, areas of localized fill deposits, etc. The final depth of stripping removal may vary depending on local subsurface conditions and the contractor's methods, and should be determined on the basis of a site inspection after the initial stripping has been performed.

Stripped organic soil should be stockpiled only in designated areas or removed from the site and stripping operations should be observed and documented by GeoPacific. Any existing subsurface structures (tile drains, old utility lines, septic leach fields, etc.) beneath structures and pavements should be removed and the excavations backfilled with engineered fill.

In construction areas, once stripping is approved, the area should be ripped or tilled to a depth of 12 inches, moisture conditioned, and compacted in-place prior to the placement of engineered fill or crushed aggregate base for pavement (dry weather conditions). Exposed subgrade soils should be evaluated by GeoPacific. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded scraper or dump truck. For smaller areas where access is restricted, and during wet weather, the subgrade should be evaluated by probing the soil with a steel probe.

Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with engineered fill, as described below. The depth of overexcavation, if required, should be evaluated by GeoPacific at the time of construction.

Engineered Fill

In general, we anticipate that undocumented fill material, disturbed native soils, and native soils from planned cuts and utility trench excavations will be suitable for use as engineered fill during dry weather conditions, provided they are adequately moisture conditioned prior to compacting and are free of highly organic material and debris. Imported fill material should be reviewed by GeoPacific prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 8 inches using conventional compaction equipment. We recommend that engineered fill be compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. On-site soils may be wet or dry of optimum; therefore, we anticipate that moisture conditioning of native soil will be necessary for compaction operations.

Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Field density testing should generally conform to ASTM D2922 and D3017, or D1556. Engineered fill should be periodically observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 cubic yards, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Fill Slope Keying and Benching

We recommend that fill slopes for the project be planned no steeper than 2H:1V and be constructed in accordance with the Fill Slope Detail, Figure 3. For fill slopes constructed at 2H:1V or flatter, and comprised of engineered fill placed and compacted as recommended herein, we anticipate that adequate factors of safety against global failure will be maintained.

Prior to placing compacted fill against natural slopes, loose undocumented fill, topsoil, and soft soils should first be removed. Adequate benching should be maintained where fill is placed on existing slopes steeper than about 6H:1V (Horizontal:Vertical). Fill slope keyways should be constructed with a minimum depth of 2 feet and minimum width of H/2 (10 feet minimum), where H equals the vertical height between the base and top of the fill slope (Figure 4). Both benches and keyways should be roughly horizontal in the downslope direction. A subdrain should be incorporated in the fill slope keyway, and GeoPacific should observe the keyway excavations prior to the placement of fill.

Measures should be taken to prevent surficial instability and/or erosion of embankment material. This can be accomplished by conscientious compaction of the embankment fills all the way out to the slope face, by maintaining adequate drainage, and planting the slope face as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to design finish grade. In our experience, compaction of slope faces by "track-walking" is generally ineffective and is therefore not recommended.

Wet Weather Earthwork

The on-site soils are moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact fill to the

recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent fines. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Bales of straw and/or geotextile silt fences should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

Structural Foundations

The proposed residential structures may be supported on shallow foundations bearing on competent undisturbed, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. We recommend a maximum allowable bearing pressure of 2,000 pounds per square foot (psf) for designing footings on native soil near existing grade. The recommended maximum allowable bearing pressure may be increased by a factor of 1.33 for short term transient conditions such as wind and seismic loading. Exterior footings should be founded at least 18 inches below the lowest adjacent finished grade. Minimum footing widths should be determined by the project engineer/architect in accordance with applicable design codes.

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

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

safety factor. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. Loose, wet or otherwise softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. GeoPacific should observe foundation excavations prior to placing formwork and reinforcing steel, to verify that adequate bearing soils have been reached.

The above foundation recommendations are for dry weather conditions. Due to the high moisture sensitivity of on-site soils, construction during wet weather may require overexcavation of footings and backfill with compacted, crushed aggregate.

Footing and Roof Drains

If the proposed structures will have a raised floor, and no concrete slab-on-grade floors are used, perimeter footing drains would not be required based on soil conditions encountered at the site and experience with standard local construction practices. Where it is desired to reduce the potential for moist crawl spaces, footing drains may be installed.

Where used, perimeter footing drains should consist of 3- or 4-inch diameter, perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Water collected from the footing drains should be directed to the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. The footing drains should include clean-outs to allow periodic maintenance and inspection. In our opinion, footing drains may outlet at the curb, or on the back sides of lots where sufficient fall is not available to allow drainage to the street.

Construction should include typical measures for controlling subsurface water beneath the homes, including positive crawlspace drainage to an adequate low-point drain exiting the foundation, visqueen covering the exposed ground in the crawlspace, and crawlspace ventilation (foundation vents). The homebuyers should be informed and educated that some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the home given these other design elements incorporated into its construction. Appropriate design professionals should be consulted regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

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

Retaining Wall Design and Construction Recommendations

Based on the sloping topography on site, we anticipate that site retaining walls will be incorporated into the project grading plan. Retaining wall locations and types have not yet been determined, although we anticipate the walls will consist most likely of Lock and Load, Keystone, or other geogrid-reinforced systems. GeoPacific should be consulted to provide retaining wall design and construction recommendations when the wall type, locations and elevations are finalized.

Seismic Design

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2009 International Residential Code (IRC) for One- and Two-Family Dwellings, with applicable Oregon Structural Specialty Code (OSSC) revisions. We recommend Site Class D be used for design per the OSSC, Table 1613.5.2. Design values determined for the site using the USGS (United States Geological Survey) *Earthquake Ground Motion Parameters* utility are summarized below, based on a location near the center of the site.

Table 4. Recommended Earthquake Ground Motion Parameters (2006 IRC)

Parameter	Value
Location (Lat, Long), degrees	45.3742, -122.7418
Mapped Spectral Acceleration Values (MCE):	
Short Period, S_s	0.92 g
1.0 Sec Period, S_1	0.33 g
Soil Factors for Site Class D:	
F_a	1.13
F_v	1.74
Residential Site Value = 2/3 x F_a x S_s	0.69 g
Residential Seismic Design Category	D_1

Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. Following development, on-site soils will consist predominantly of engineered fill or native fine-grained soils, which are not considered susceptible to liquefaction. Therefore, it is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.

Storm Water Management

We understand that on-site storm water management facilities may include water quality tracts, shallow infiltration facilities, and/or deep infiltration facilities. Infiltration test results indicate that infiltration rates in the silt and sandy silt soils are on the order of 0.1 to 0.3 inches per hour, and that the infiltration rates in the silty fine sand soils are on the order of 6 inches per hour. Table 2 summarizes the depths at which silty fine sand was encountered in the test pits. The designer should select an appropriate infiltration value based on our test results and the location/elevation of the proposed infiltration facility. The infiltration rates do not incorporate a factor of safety. For the design infiltration rate, the system designer should incorporate an appropriate factor of safety against slowing of the rate over time due to biological and sediment clogging.

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

Excavating Conditions and Utility Trench Backfill

We anticipate that on-site soils can be excavated using conventional heavy equipment. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Heath Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only.

Shallow, perched groundwater should be anticipated in excavations and utility trenches. The depth of groundwater will likely be less during the wet weather season and greater during the dry weather season. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that structural trench backfill be compacted to at least 90% of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

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

Pavement Sections - On-Site Public Streets

We understand that asphalt pavements may be incorporated in project design for the construction of new on-site public streets. For the purposes of pavement section design we assumed subgrade soils will be prepared to provide a minimum resilient modulus of 6,000 pci, equivalent to a CBR value of about 4. Using the methodology presented in the 1993 AASHTO (American Association of State Highway and Transportation Officials) pavement design guidelines, Table 5 presents recommended minimum pavement section for new on-site streets, and minor driveways and parking lots, under dry weather construction conditions. Based on our experience, these pavement sections are generally appropriate for minor residential streets and driveways that will not be subjected to significant heavy truck traffic (less than 200 trips per day). Changes in anticipated traffic levels will affect the recommended pavement section. GeoPacific should be contacted for additional recommendations if any paved areas are to be designed for heavy truck traffic.

Table 5. Recommended Minimum Dry Weather Pavement Section for On-Site Streets

Material Layer	Minimum Thickness (inches)	Compaction Standard
Asphaltic Concrete (AC)	3	92% of Rice Density (top lift) 91% of Rice Density (lower lifts) AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	95% of Modified Proctor ASTM D1557
Crushed Aggregate Base 1½"-0	8	95% of Modified Proctor ASTM D1557
Recommended Subgrade	N/A	Approved Native Subgrade or Engineered Fill Compacted to 95% of Standard Proctor ASTM T-99

In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving. If pavement areas are to be constructed during wet weather, GeoPacific should review subgrade at the time of construction so that condition specific recommendations can be provided. Wet weather pavement construction is likely to require soil amendment or geotextile fabric and an increase in base course thickness.

During placement of pavement section materials, density testing should be performed to verify compliance with project specifications. Generally, one subgrade, one base course, and one AC compaction test is performed for every 100 to 200 linear feet of paving.

Erosion Control Considerations

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction, in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of "bio-bags", silt fences, and other appropriate technology. Where used, these erosion control devices should be in place and remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

UNCERTAINTIES AND LIMITATIONS

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

appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

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

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

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

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

12-12-13

EXPIRES: 06-30-2015

SLH - FOR -

Benjamin G. Anderson
Staff Engineer

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

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site and Exploration Plan
Figure 3 – Fill Slope Detail
Test Pit Logs (TP-1 through TP-14)

REFERENCES

Atwater, B.F., 1992, Geologic evidence for earthquakes during the past 2,000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, Vol. 97, p. 1901-1919.

Beeson, M.H., Tolan, T.L., and Anderson, J.L., 1989, The Columbia River Basalt Group in western Oregon; Geologic structures and other factors that controlled flow emplacement patterns: Geological Society of America Special Paper 239, in Volcanism and tectonicism in the Columbia River flood-basalt province published by the Geological Society of America, p. 223-246.

Carver, G.A., 1992, Late Cenozoic tectonics of coastal northern California: American Association of Petroleum Geologists-SEPM Field Trip Guidebook, May, 1992.

Geomatrix Consultants, 1995, Seismic Design Mapping, State of Oregon: unpublished report.

Goldfinger, C., Kulm, L.D., Yeats, R.S., Appelgate, B, MacKay, M.E., and Cochrane, G.R., 1996, Active strike-slip faulting and folding of the Cascadia Subduction-Zone plate boundary and forearc in central and northern Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, P. 223-256.

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

Peterson, C.D., Darioenzo, M.E., Burns, S.F., and Burris, W.K., 1993, Field trip guide to Cascadia paleoseismic evidence along the northern California coast: evidence of subduction zone seismicity in the central Cascadia margin: Oregon Geology, Vol. 55, p. 99-144.

Schlicker, H.G. and Finlayson, C.T., 1963, Geology and Geologic Hazards of the Canby and Oregon City Quadrangle, Oregon: Oregon Department of Geology and Mineral Industries, Bulletin No. 99, scale 1:24,000.

Unruh, J.R., Wong, I.G., Bott, J.D., Silva, W.J., and Lettis, W.R., 1994, Seismotectonic evaluation: Scoggins Dam, Tualatin Project, Northwest Oregon: unpublished report by William Lettis and Associates and Woodward Clyde Federal Services, Oakland, CA, for U. S. Bureau of Reclamation, Denver CO (in Geomatrix Consultants, 1995).

Werner, K.S., Nabelek, J., Yeats, R.S., Malone, S., 1992, The Mount Angel fault: implications of seismic-reflection data and the Woodburn, Oregon, earthquake sequence of August, 1990: Oregon Geology, v. 54, p. 112-117.

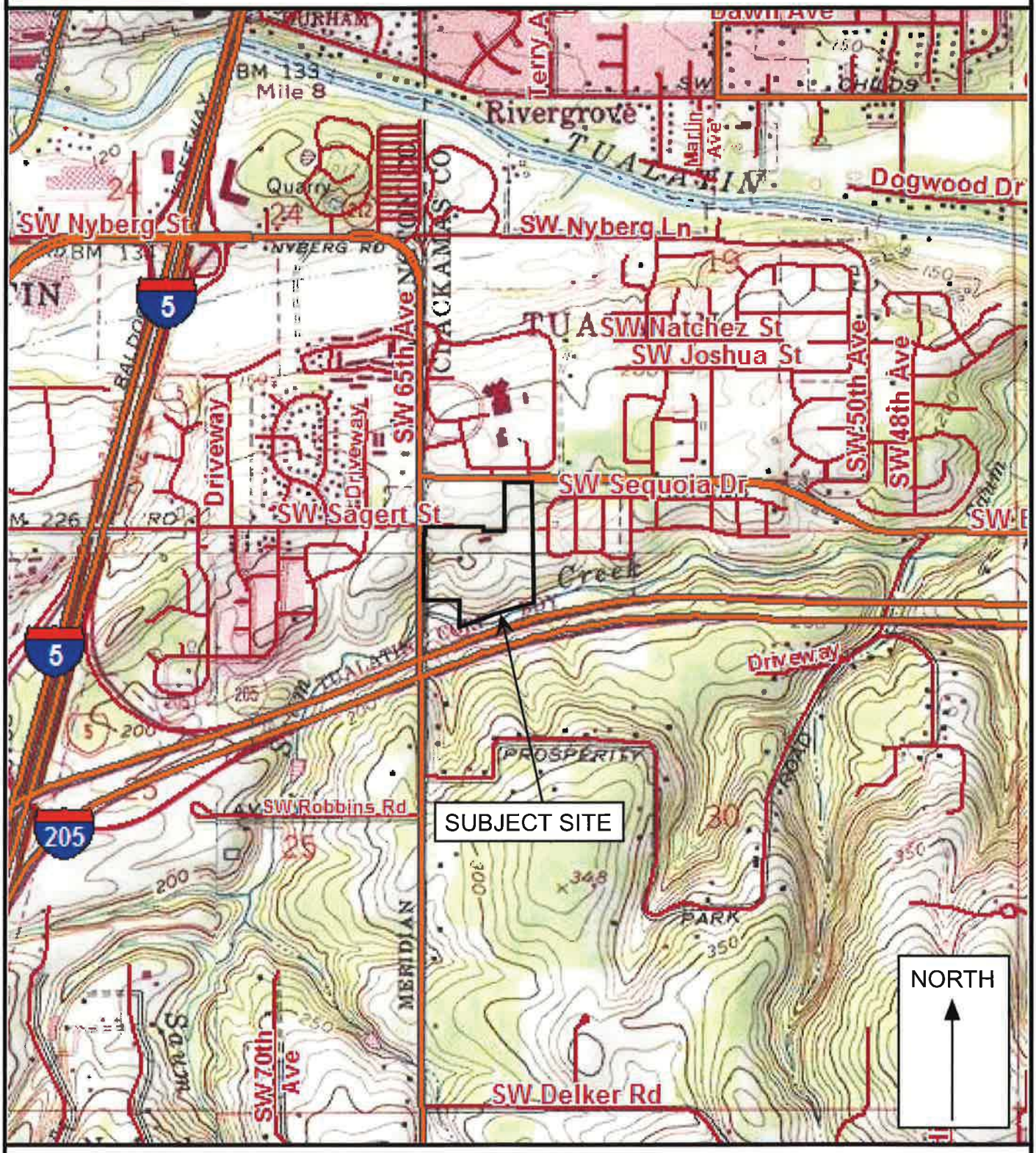
Wong, I. Silva, W., Bott, J., Wright, D., Thomas, P., Gregor, N., Li., S., Mabey, M., Sojourner, A., and Wang, Y., 2000, Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan Area; State of Oregon Department of Geology and Mineral Industries; Interpretative Map Series IMS-16.

Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1996, Tectonics of the Willamette Valley, Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, Vol. 1: U.S. Geological Survey Professional Paper 1560, P. 183-222, 5 plates, scale 1:100,000.

Yelin, T.S., 1992, An earthquake swarm in the north Portland Hills (Oregon): More speculations on the seismotectonics of the Portland Basin: Geological Society of America, Programs with Abstracts, v. 24, no. 5, p. 92.

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

VICINITY MAP



Legend:

Approximate Scale 1 in = 2,000 ft

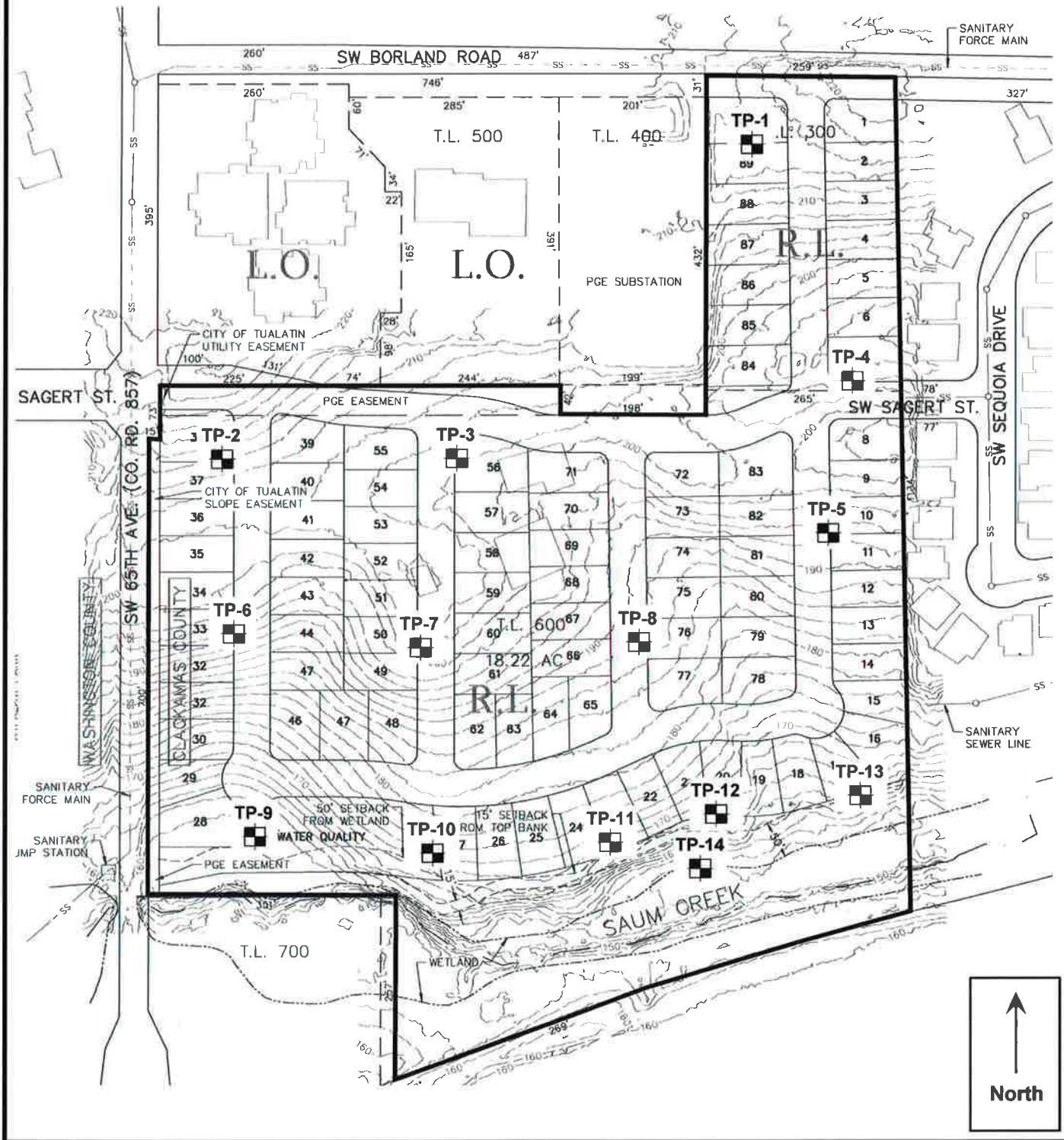
Date: 11/21/13
Drawn by: BGA

Base maps: National Geographic TOPO!, Tele Atlas, Oregon, 1990.

Project: Sagert Property Tualatin, Oregon	Project No. 13-3204	FIGURE 1

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

SITE PLAN AND EXPLORATION LOCATIONS



Legend

TP-1
Test Pit Designation and Approximate Location

0 200'
APPROXIMATE SCALE 1"=200'

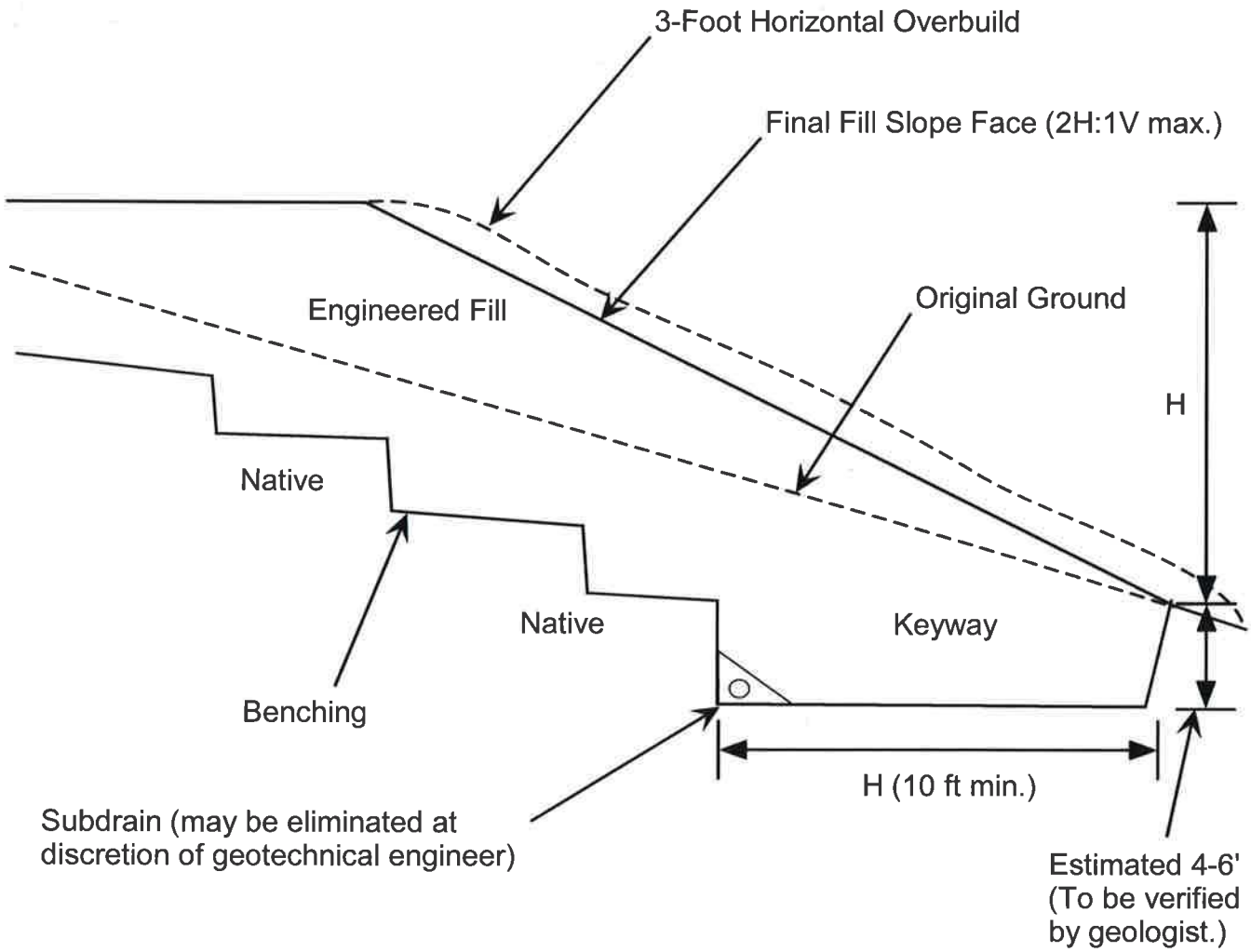
Date: 11/19/13
Drawn by: BGA

Project: Sagert Property Tualatin, Oregon	Project No. 13-3204	FIGURE 2

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

FILL SLOPE DETAIL

TYPICAL KEYWAY, BENCHING & FILL SLOPE DETAIL



Recommended subdrain is minimum 3-inch-diameter ADS Heavy Duty grade (or equivalent), perforated plastic pipe enveloped in a minimum of 3 cubic feet per lineal foot of 2" to 1/2" open-graded gravel drain rock wrapped with geotextile filter fabric (Mirafi 140N or equivalent).

Project: Sagert Property Tualatin, Oregon	Project No. 13-3204	FIGURE 3

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						10" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						10" soft, low organic SILT (ML), brown, moist (Disturbed Native)
2	3.0					Stiff, SILT (ML), brown, moist (Willamette Formation)
3	3.0					
4	3.0					
	4.5					Grades to very stiff
5						
6						
7						
8						
						Medium dense, silty fine SAND (SM), moist (Willamette Formation)
9						
10						Test pit terminated at 10 feet
11						
12						Note: No seepage or groundwater encountered
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						10" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						10" soft, low organic SILT (ML), brown, moist (Disturbed Native)
2	2.0					Stiff, SILT (ML), brown, moist (Willamette Formation)
3	2.5					
4	>4.5					
5						Grades to very stiff and sandy
6						Medium dense, silty fine SAND (SM), moist (Willamette Formation)
7						
8						
9						
10						
11						Test pit terminated at 11 feet
12						
13						Note: No seepage or groundwater encountered
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						6" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						12" soft, low organic SILT (ML), brown, moist (Disturbed Native)
2	2.0					Stiff, SILT (ML), brown, moist (Willamette Formation)
	2.0					
3	>4.5					Grades to very stiff
4						
5						
6						
7						Grades to sandy
8						Test pit terminated at 8 feet
9						
10						Note: No seepage or groundwater encountered
11						
12						
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-4**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						6" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						12" soft, low organic SILT (ML), brown, moist (Disturbed Native)
2	2.0					Stiff, SILT (ML), brown, very moist (Willamette Formation)
	2.0					
3	3.5					
4	>4.5					Grades to very stiff
5						
6						
7						
8						Grades to sandy
9						
10						
11						Test pit terminated at 10 feet
12						Note: No seepage or groundwater encountered
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-5**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						6" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1	0.5					12" soft, low organic SILT (ML), brown, moist (Disturbed Native)
2						Stiff, SILT (ML), brown, moist (Willamette Formation)
	2.0					
3						
	3.5					
4						Grades to very stiff
5						
6						
7						Grades to with trace sand
8						
9						Grades to sandy
10						
11						Test pit terminated at 11 feet
12						
13						Note: No seepage or groundwater encountered
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-6**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
1						12" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
2	1.0					Stiff, sandy SILT (ML), brown, moist (Willamette Formation)
3	1.5					
4	1.5					
5						
6						
7						Medium dense, silty fine SAND (SM), gray brown, damp (Willamette Formation)
8						
9						Test pit terminated at 9 feet
10						
11						Note: No seepage or groundwater encountered
12						
13						
14						
15						
16						
17						

LEGEND

Bag Sample (100 to 1,000 g) | Bucket Sample (5 Gal. Bucket) | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-7**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						10" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						Very stiff, sandy SILT (ML), brown, moist (Willamette Formation)
2	4.0					
3	4.0					
4	4.5					
5						
6						
7						
8						Medium dense, silty fine SAND (SM), gray brown, damp (Willamette Formation)
9						
10						Test pit terminated at 10 feet
11						
12						Note: No seepage or groundwater encountered
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-8**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						3" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						11" soft, low organic SILT (ML), brown, moist (Disturbed Native)
	1.0					Stiff, SILT (ML), brown, moist (Willamette Formation)
2	1.5					
						Grades to with occasional charred organics
3	1.5					
4	>4.5					Grades to very stiff to hard and less moist
5						
6						
7						Grades to sandy and gray brown
8						Test pit terminated at 8 feet
9						
10						Note: No seepage or groundwater encountered
11						
12						
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-9**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
1	0.5					Very soft, moderately organic SILT (OL-ML), dark gray and dark brown, fine roots throughout upper 4 inches, very moist (Disturbed Native) Grades to blue gray and with seepage at 1.5 feet
2						
3						
4	3.5					Stiff, SILT (ML), brown, moist (Willamette Formation)
5						Test pit terminated at 4.5 feet
6						
7						Notes: Seepage encountered at 1.5 feet Discharge visually estimated at approximately 3 gallons per minute Water measured at 2.5 feet after three hours
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-10**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						4" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						20" soft, low organic SILT (ML), brown, moist (Disturbed Native)
	1.0					
2						Stiff, SILT (ML), brown, moist (Willamette Formation)
	3.5					
3	3.5					
4	>4.5					Grades to gray brown and with trace fine sand
5						
6						
7						Grades to sandy
8						
9						Test pit terminated at 8.5 feet
10						
11						Note: No seepage or groundwater encountered
12						
13						
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-11**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						5" very soft to soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						Soft, low organic SILT (ML), brown, moist (Undocumented Fill)
2						
3	3.5					Very stiff, SILT (ML), gray brown, with trace fine sand, damp (Willamette Formation)
4	4.0					Grades to gray brown and with trace fine sand
5						
6						
7						Grades to sandy
8						
9						
10						Medium dense, silty fine SAND (SM), brown, damp (Willamette Formation)
11						
12						Test pit terminated at 12 feet
13						
14						Note: No seepage or groundwater encountered
15						
16						
17						

LEGEND

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-12**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						5" very soft to soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						Soft to medium stiff, low organic SILT (ML), with charred organics and occasional gravel, brown and gray, nonhomogeneous, moist (Undocumented Fill)
2						
3						
	3.5					Very stiff, SILT (ML), brown, moist (Willamette Formation)
4						
	4.5					
5						
6						
						Grades to sandy
7						
8						
9						Medium dense, silty fine SAND (SM), brown, damp (Willamette Formation)
10						
11						
12						
13						Test pit terminated at 12 feet
14						Note: No seepage or groundwater encountered
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-13**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						4" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout, moist (Topsoil)
1						20" soft to medium stiff, low organic SILT (ML), brown, moist (Disturbed Native)
	2.5					
2						Very stiff, SILT (ML), brown, moist (Willamette Formation)
	>4.5					
3						
	>4.5					
4						Grades to gray brown and with trace fine sand
5						
6						
7						Grades to sandy
8						
9						
						Medium dense, silty fine SAND (SM), brown, moist (Willamette Formation)
10						
11						Test pit terminated at 11 feet
12						
13						Note: No seepage or groundwater encountered
14						
15						
16						
17						

LEGEND

Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation:

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Sagert Property
Tualatin, Oregon

Project No. 13-3204

Test Pit No. **TP-14**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
						10" soft, moderately organic SILT (OL-ML), dark brown, fine roots throughout upper 4 inches, moist (Topsoil)
1						Medium stiff, SILT (ML), gray, with orange mottling, moist (Willamette Formation)
	4.0					
2						
	4.0					
3						
	4.5					
4						
5						
6						Grades to brown
7						
8						Test pit terminated at 8 feet
9						
10						Note: No seepage or groundwater encountered
11						
12						
13						
14						
15						
16						
17						

LEGEND

Bag Sample (100 to 1,000 g) | Bucket Sample (5 Gal. Bucket) | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 11/15/13
Logged By: BGA
Surface Elevation: