Willow Ridge

West Linn, Oregon

DRAINAGE ANALYSIS

October 2020

REGISTERED PROFESSIONAL ENGINEER
7120
OREGON
July 16, 1971
BRUCE D. GOLDSON

EXPIRES: 06/30/2021
SIGNATURE DATE: 10/14/2020

Prepared By:

Bruce D. Goldson, PE

Theta, llc

PO Box 1345, Lake Oswego, Oregon 97035

2014-129L

INDEX

PURPOSE:

This is a proposed 6-lot development at the end of Cornwall and Landis Street. This development would connect these two roads together with the extension of Landis Street. The property slopes to the south and currently has one residential house with the remainder of the property being undeveloped. This house is to be removed with this development. This report proposes to demonstrate that a storm water system is feasible to collect storm water from the new impervious surfaces and dispose to a system and not unfavorable impact downhill residents. This report also demonstrates that the storm water system for the Tanner's Stonegate development was designed to accommodate the Willow Ridge project and to provide water quality for the extension of Landis Street into Willow Ridge. Storm water from future lots 2-6 are will not be part of the Landis Street system.

NARRATIVE ASSUMPTIONS

The Tanner's Stonegate project construction drawings show a storm sewer line to the westerly property line of the proposed Willow Ridge project. The plans also show a tentative roadway extension into the Willow Ridge property with a note *"future expansion". The plans also show within the roadway of Tanner's Stonegate 370 lineal feet of 60-inch reinforced concrete detention pipe with a control manhole having orifices to regulate flow. Within the control manhole is a water quality orifice and a flow control orifice. Downstream of the control manhole is a water quality facility for low flows prior to discharge into the natural drainage course. North of the Tanner's Stonegate project the extension of Landis Street is also labeled "future expansion". Inspection* of the construction plans reveal that only the houses on the easterly side of Landis Street, Landis Street, and Stonegate Lane plus to land east of the houses on Landis Street could be collected in this storm system. Detail 7/C3.2, flow control MH illustrates a water quality orifice at 3.5-inches and a flow control orifice at 4.0". Checking in the field the water quality riser is open at the top. This therefore effectively becomes an 8-inch orifice when the volume reaches that elevation. Stains in the control manhole Indicate that the volume has very been significantly above the overflow level of the water quality riser. The Tanner's Stonegate

project provides water quality downstream of the public storm system and is privately maintained. Although providing additional water quality appears redundant a added water quality on the Willow Ridge project will remove pollutants, floatable material and organic materials such as leaves. At this time a planter box is proposed to collect the storm water from the road extension flowing towards Tanner's Stonegate. Catch basins at the boundary between Tanner's Stonegate and Will Ridge will divert the storm water from the street into the planter box. A separate planter for proposed lot 1, which will be sized based on the final house plan during the building permit phase. An overflow for this planter would also be connected to the public system.

The original storm report for Tanner's Stonegate could not be found and therefore this analysis has been undertaken to determine if there is sufficient capacity in the existing detention system to accommodate the proposed Willow Ridge project. Only the new public street area and lot 1 from the proposed Willow Ridge project will be directed to the Tanner's Stonegate facility. The undeveloped topography directs surface and subsurface storm water from a portion of the proposed road extension and most of lot 1 towards Tanner's Stonegate. The proposed road extension has a high point in the profile which directs storm water towards Tanner's Stonegate and Cornwall Street. A smaller portion of the proposed Landis extension will flow towards the intersection with Cornwall. This storm water from both directions will be captured and directed to a storm water planters. Currently there are no storm facilities on Cornwall. Improvements on Cornwall from Landis to Sunset will be a narrow strip of new AC without curbs. A roadside swale is proposed to collect and provide water quality with infiltration. Catch basins are proposed at the intersection with Landis as an overflow. The impervious roof areas downhill from the road extension of Willow Ridge would be directed on-site lined rain gardens or planter boxes with overflow to the drainage way on the easterly side of the property.
Individual rain gardens or planters are proposed for lots 2-6, sized based on the actual impervious area during the building permit process. A preliminary impervious area of 2600 SF was used to illustrate an approximate size. An overflow connections to the public storm will be provided for each lot and directed to a natural drainage way to the south.

Regulatory

2.0013 Minimum Design Criteria

A. Storm Detention Facilities

2. Storms to be evaluated shell include to 2, 5, 10, 25, and 100-year event. Allowable post-development discharge rates for the 2, 5, 10, and 25-year events hall be that of the pre-development rate. An outfall structure such as a "V-North" weir of single of multiple orifice structure shall be designed to control the release rate for the above events. No flow control *orifice smaller than 1 in. shall be allowed. If the maximum release cannot be met with all the site drainage controlled by a single 1 in. orifice, the allowable release rate provided by the 1 in. orifice will be considered adequate as approved by the City Engineer. The detention volume was calculated to be 7265 CF.*

References Regulatory

1. King County Department of Public Works, Surface Water Management Division, Hydrographic Programs, Version 4.21B
2. Tanner's Stonegate construction plans by Otak (8-21-2001)
3. City of Portland Sewer & Drainage Facilities Design Manual, Chart 1
4. City of West Linn Public Works Design Standards (2010) Section two-storm Facilities Design Manual

Summary

Event	Pre flow	Post flow	With Orifices
2-year	1.38 cfs	0.83 cfs	0.64 cfs
5-year	1.83 cfs	1.23 cfs	1.23 cfs
10-year	2.05 cfs	1.43 cfs	1.42 cfs
25-year	2.43 cfs	1.78 cfs	1.78 cfs

Time of concentration

Pre $T= 0.42((nL))^{0.8}/(p)^{.5}(s)^{.4} = 0.42((.24)(167))^{.8}/(2.6)^{.5}(0.08)^{.4} = 13.7$ min.

Post $T_1= 0.42((nL))^{0.8}/(p)^{.5}(s)^{.4} = 0.42((.01)(170))^{.8}/(2.6)^{.5}(0.03)^{.4} = 1.6$ min.

$T_2 = L/60(k)(s)^{.5} = 167/(60)(42)(0.01)^{.5} = 6.6$ min & $T_3 = 233/(60)(42)90.065)^{.5} = 0.1$ min

$T_{post} = 1.6+6.6+0.1 = 8.3$ min

Areas:

The areas used are shown on the storm analysis drawing.

Tanner's Stonegate basin = 105, 995 SF + Willow ridge street = 26,128 SF for total = 132123 SF= 3.03 acres

HYDROGRAPH RESULTS (DETENTION, WATER QUALITY, INFILTRATION)

KING COUNTY DEPARTMENT OF PUBLIC WORKS
Surface Water Management Division
HYDROGRAPH PROGRAMS
Version 4.21B
1 - INFO ON THIS PROGRAM

2 - SBUHYD
3 - MODIFIED SBUHYD
4 - ROUTE
5 - ROUTE2
6 - ADDHYD
7 - BASEFLOW
8 - PLOTHYD
9 - DTATA
10 - REFAC
11 - RETURN TO DOS

ENTER OPTION:
2
SBUH/SCS METHOD FOR COMPUTING RUNOFF HYDROGRAPH
STORM OPTIONS:
1 - S.C.S. TYPE-1A
2 - 7-DAY DESIGN STORM
3 - STORM DATA FILE
SPECIFY STORM OPTION:
1
S.C.S. TYPE - 1A RAINFALL DISTRIBUTION
ENTER; FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
25,24,3.9
Xxxxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXX 25-YEAR 24-HOUR STORM xxxx 3.90 "TOTAL PRECIP Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

---

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
1.44,86,1.62,98,8.3
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	1.4	81.0	1.6	98.0	8.3

PEAK-Q(CFS)	T-PEAK(HRS)	VOL(CU-FT)
2.43	7.83	34383

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:25wr
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S - STOP

C
ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
3.06,86,0.0,98,13.7
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	3.1	86.0	.0	98.0	13.7

PEAK-Q(CFS)	T-PEAK(HRS)	VOL(CU-FT)
1.78	7.83	27233

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S – STOP

N

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1 - S.C.S. TYPE-1A
2 - 7-DAY DESIGN STORM
3 - STORM DATA FILE
SPECIFY STORM OPTION:
1
S.C.S. TYPE - 1A RAINFALL DISTRIBUTION
ENTER; FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
2,24,2.5
Xxxxxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXX 2-YEAR 24-HOUR STORM xxxx 2.50 "TOTAL PRECIP Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

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ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
1.44,86,1.62,98,8.3
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	1.4	81.0	1.6	98.0	8.3
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
1.38	7.83		19848		

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:2wr
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S - STOP

C
ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
3.06,86,0.0,98,13.7
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	3.1	86.0	.0	98.0	13.7
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
0.83	7.83		13785		

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:wr2
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S – STOP
N

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1 - S.C.S. TYPE-1A
2 - 7-DAY DESIGN STORM
3 - STORM DATA FILE
SPECIFY STORM OPTION:
1
S.C.S. TYPE - 1A RAINFALL DISTRIBUTION
ENTER; FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
5,24,3.1

Xxxxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXX 5-YEAR 24-HOUR STORM xxxx 3.10 "TOTAL PRECIP Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
1.44,86,1.62,98,8.3
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	1.4	81.0	1.6	98.0	8.3
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
1.83	7.83		25997		

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:5wr
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S - STOP

C
ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
3.06,86,0.0,98,13.7
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	3.1	86.0	.0	98.0	13.7
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
1.23	7.83		19386		

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:wr5
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S – STOP
N

1 - S.C.S. TYPE-1A
2 - 7-DAY DESIGN STORM
3 - STORM DATA FILE
SPECIFY STORM OPTION:
1
S.C.S. TYPE - 1A RAINFALL DISTRIBUTION
ENTER; FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
10,24,3.4
Xxxxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXX 10-YEAR 24-HOUR STORM xxxx 3.40 "TOTAL PRECIP Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
1.44,86,62,98,8.3
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	1.44	81.0	1.6	98.0	8.3
PEAK-Q(CFS)	T-PEAK(HRS)		VOL(CU-FT)		
2.05	7.83		29122		

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:10wr
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S - STOP

C
ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
3.06,86,0.0,98,13.7
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	
3.1	3.1	86.0	.0	98.0	13.7

PEAK-Q(CFS)	T-PEAK(HRS)	VOL(CU-FT)
1.43	7.83	22288

ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:wr10
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S – STOP
s

DETENTION

KING COUNTY DEPARTMENT OF PUBLIC WORKS
Surface Water Management Division
HYDROGRAPH PROGRAMS
Version 4.21B
1 - INFO ON THIS PROGRAM
2 - SBUHYD
8 - PLOTHYD
9 - DTATA
10 - REFAC
11 - RETURN TO DOS

10
R/D FACILITY DESIGN ROUTINE
SPECIFY TYPE OF R/D FACULTY

1 - POND	4 - INFILTRATION POND
2 - TANK	5 - INFILTRATION TANK
3 -VAULT	6 - GRAVEL TRENCH/BED

2
ENTER: TANK DIAMETER (ft), EFFECTIVE STORAGE DEPTH (ft)
5,5
ENTER [d:][path]filename[.ext] OF PRIMARY DESIGN INFLOW HYDROGRAPH:
C:25post
PRELIMINARY DESIGN INFLOW PEAK = 2.43 CFS
ENTER PRIMARY DESIGN RELEASE RATE(cfs)
1.78
ENTER NUMBER OF INFLOW HYDROGRAPHS TO BE TESTED FOR PERFORMANCE (5 MAXIMUM)
3
ENTER [d:][path] filename[.ext] OF HYDROGRAPH 1:

C:10wr
ENTER TARGET RELEASE RATE (cfs)
1.43
ENTER [d:][path] filename[.ext] OF HYDROGRAPH 2:
C:5wr
ENTER TARGET RELEASE RATE (cfs)
1.23
ENTER [d:][path] filename[.ext] OF HYDROGRAPH 3:
C:2wr
ENTER TARGET RELEASE RATE (cfs)
0.83

ENTER; NUMBER OF ORIFICES, RISER-HEAD (ft), RISER-DIAMETER(in)
2,5,12
RISER OVERFLOW DEPTH FOR PRIMARY PEAK INFLOW = .41FT
SPECIFY ITERATION DISPLAY: Y - YES, N – NO
N
SPECIFY: R - REVIEW/REVISE INPUT, C – CONTINUE
C
INITIAL STORAGE VALUE FOR ITERATION PURPOSES: 11202 CU-FT
BOTTOM ORIFICE : ENTER Q-MAX (cfs)
0.4
DIA. = 2.57 INCHES
TOP ORIFICE ENTER HEIGHT(ft)
3.07
DIA. = 6.05 INCHES

PERFORMANCE:	INFLOW	TARGET-OUTFLOW	ACTUAL-OUTFLOW	PK-STAGE	STORAGE
DESIGN HYD:	2.43	1.78	1.78	4.99	4800
TEST HYD: 1	2.05	1.43	1.42	4.22	4300
TEST HYD: 2	1.83	1.23	1.23	3.87	3980
TEST HYD: 3	1.38	.83	.64	3.28	3330

WATER QUALITY LANDIS STREET

Easterly portion:

Based on the preliminary plans 9580 SF of new impervious surface has been calculated for the extension of Landis Street into the proposed Willow Ridge development. Using the City of Portland Presumptive Approach Calculator and assuming a planter box to be installed at the westerly end of the project a facility having a bottom surface are of 126 SF meets the water quality criteria. A planter box with inside dimensions of 6-feet by 21-feet has been shown of the preliminary plans.

Westerly portion:

From the high point on Landis to the intersection a total of 5531 SF flows towards the Landis/Cornwall intersection. Preliminary sizing using Portland Presumptive Approach Calculator which accounts for both water quality and quantity finds a total of 166 SF is required.

WATER QUALITY FUTURE IMPERVIOUS ROOFS

The final sizing will be determined based on the actual impervious footprint. The proposed lined flow through planter boxes will not be used to infiltrate into the ground because of the steep slope and neighbor's concerns about added runoff. Preliminary sizing using the Portland Presumptive Approach Calculator will provide water quality and quantity. A preliminary size of 78 SF results in a planter of 5X16 or 3X13.

CONCLUSION

To replicate the original report would be impossible with the available information. Based on a field investigation it doesn't appear that the facility as constructed meets the City of West Linn storm water standards with the water quality riser overflow as constructed.
The above calculation indicate that there is excess capacity in the detention system to receive the Willow Ridge development, but the flow could be better controlled by raising the water quality flow riser to the same overflow height as the flow control riser and changing the water quality orifice to 2.87-inches and the quantity orifice to 4.27-inches.

Based on the available information and these calculations the Tanner's Stonegate project has provided sufficient detention volume to accommodate the Willow Ridge development. Although redundant a new water quality facility demonstrates how independent water quality is achieved for the Willow Ridge development.

The southerly part of Landis will be collected in a planter sized for both quantity and quality. Individual storm facilities for the new houses also will provide quantity and quality.

The combined effect of this storm plan is to reduce the surface area receiving storm water and the downstream impacts by over 30,000 SF of impervious surface or 0.70 acres. For the 10-year event this represents more than 8000 CF not impacting the downhill properties.

1 - S.C.S. TYPE-1A
2 - 7-DAY DESIGN STORM
3 - STORM DATA FILE
SPECIFY STORM OPTION:
1
S.C.S. TYPE - 1A RAINFALL DISTRIBUTION
ENTER; FREQ(YEAR), DURATION(HOUR), PRECIP(INCHES)
10,24,3.4
Xxxxxxxxxxxxxxxxxxxxxxxxx S.C.S.TYPE-1A DISTRIBUTION xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx
XXXXXXXXXXXX 10-YEAR 24-HOUR STORM xxxx 3.40 "TOTAL PRECIP Xxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxxx

ENTER: A(PERV),CN(PERV),A(IMPERV),CN(IMPERV),TC FOR BASIN NO. 1
0.0,86,70,98,8.3
DATA PRINT OUT:

AREA(ACRES)	PERVIOUS		IMPERVIOUS		TC(MINUTES)
	A	CN	A	CN	

.7	.0	81.0	.7	98.0	13.7

PEAK-Q(CFS)	T-PEAK(HRS)	VOL(CU-FT)
.53	7.83	8036

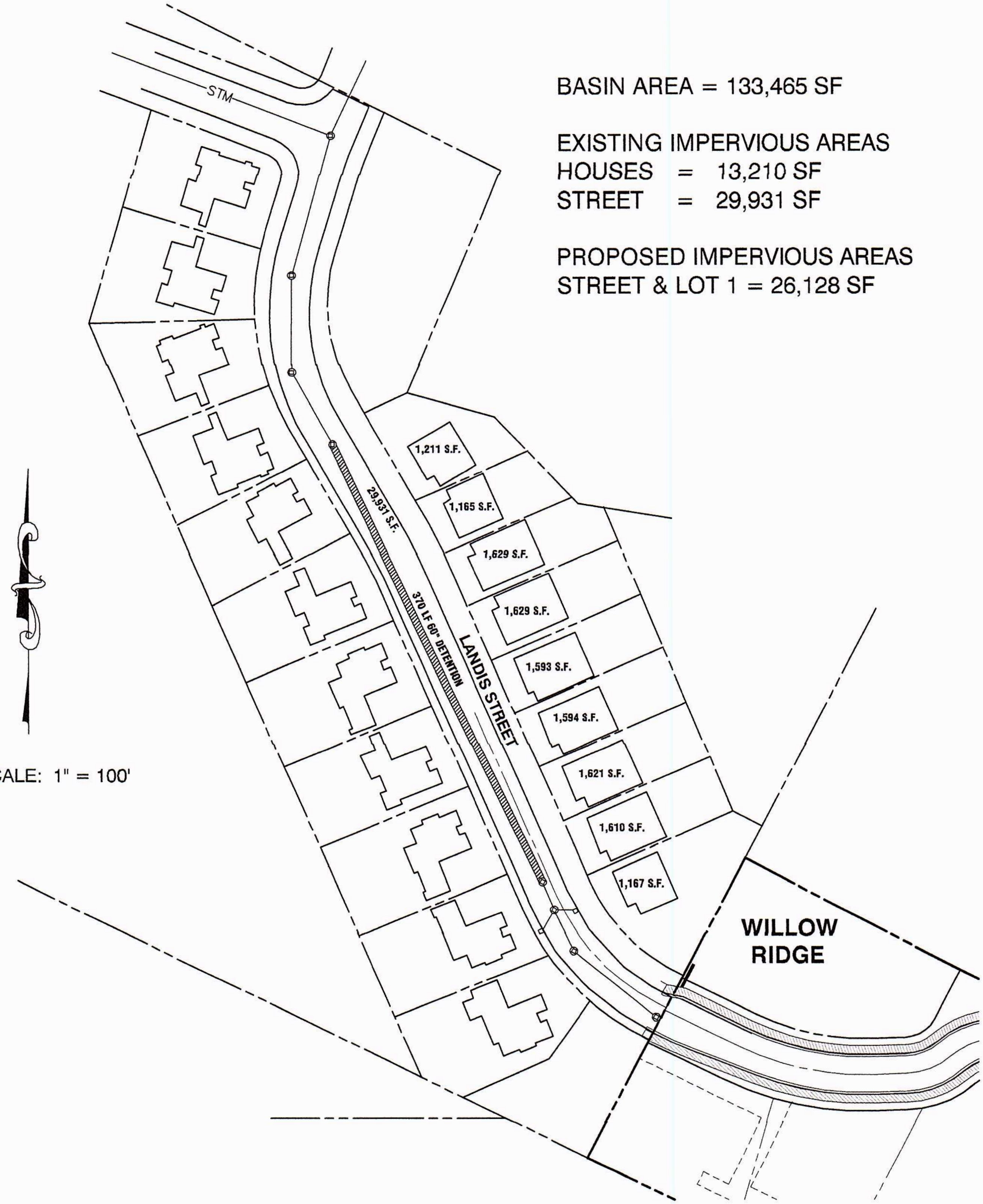
ENTER [dk:][path]filename[.ext] FOR STORAGE OF COMPUTED HYDROGRAPH:
C:save
SPECIFY: C - CONTINUE, N - NEWSTORM, P -PRINT, S – STOP

The proposed storm and sanitary collection system serving lot 2-6 will have a crushed rock pipe zone and will act as a graded French drain to drain subsurface ground water away from the downhill properties.

BASIN AREA = 133,465 SF

EXISTING IMPERVIOUS AREAS
HOUSES = 13,210 SF
STREET = 29,931 SF

PROPOSED IMPERVIOUS AREAS
STREET & LOT 1 = 26,128 SF



SCALE: 1" = 100'

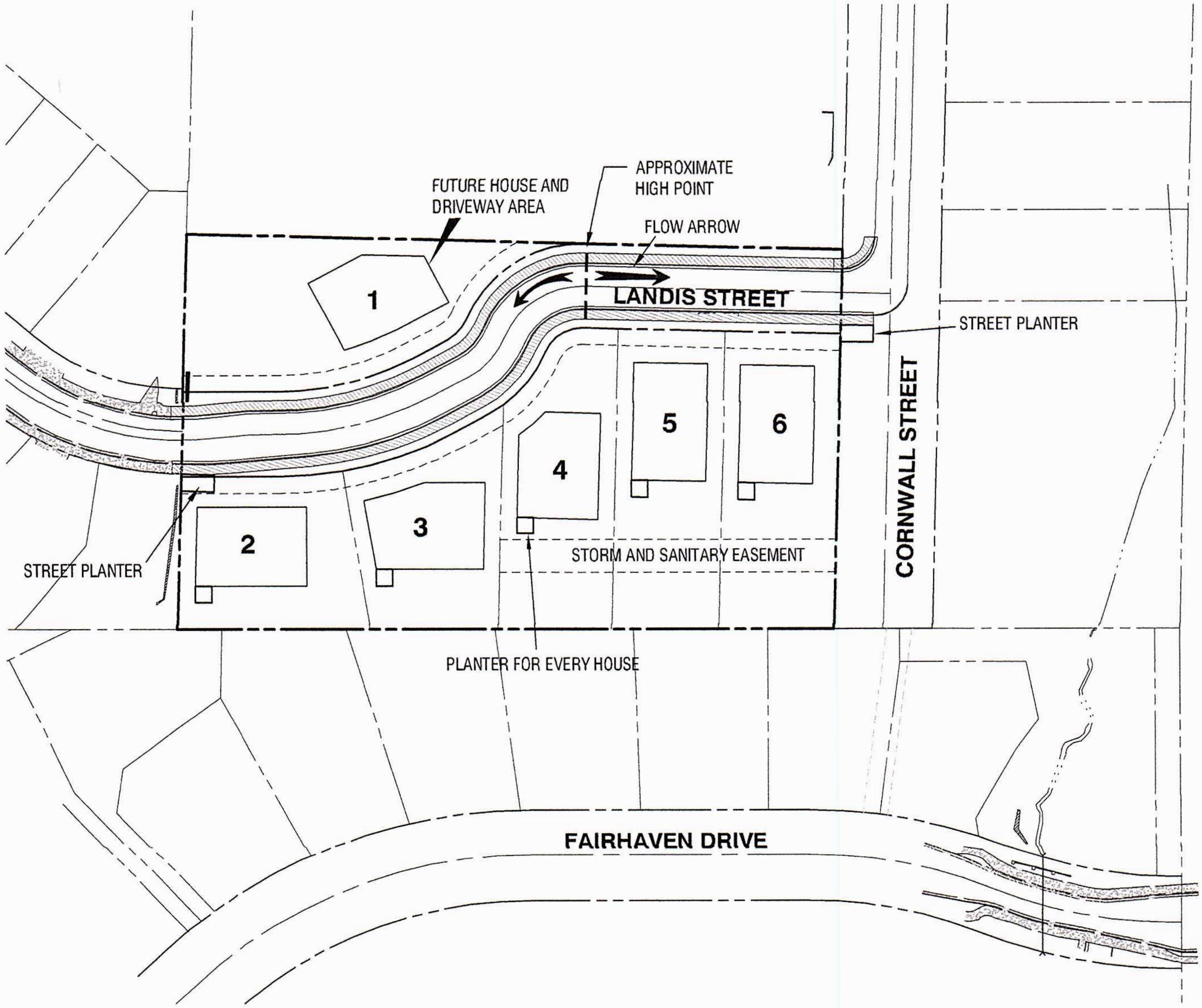
2010-129L

STORM ANALYSIS

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

WILLOW RIDGE

West Linn, Oregon



EXISTING CONDITION:
2.2 ACRES, INCLUDING 4600 S.F. OF IMPERVIOUS AREA
DISCHARGES UNCONTROLLED TO SOUTH

PROPOSED CONDITION:
STORMM WATER FACILITIES FOR STREET AND HOUSES
WITH NO INFILTRATION DIRECTING IMPERVIOUS STORM
WATER TO APPROVED DISPOSAL.

SCALE: 1" = 100'

2010-129L

GRAPHIC REPRESENTATION OF FUTURE IMPERVIOUS AREA

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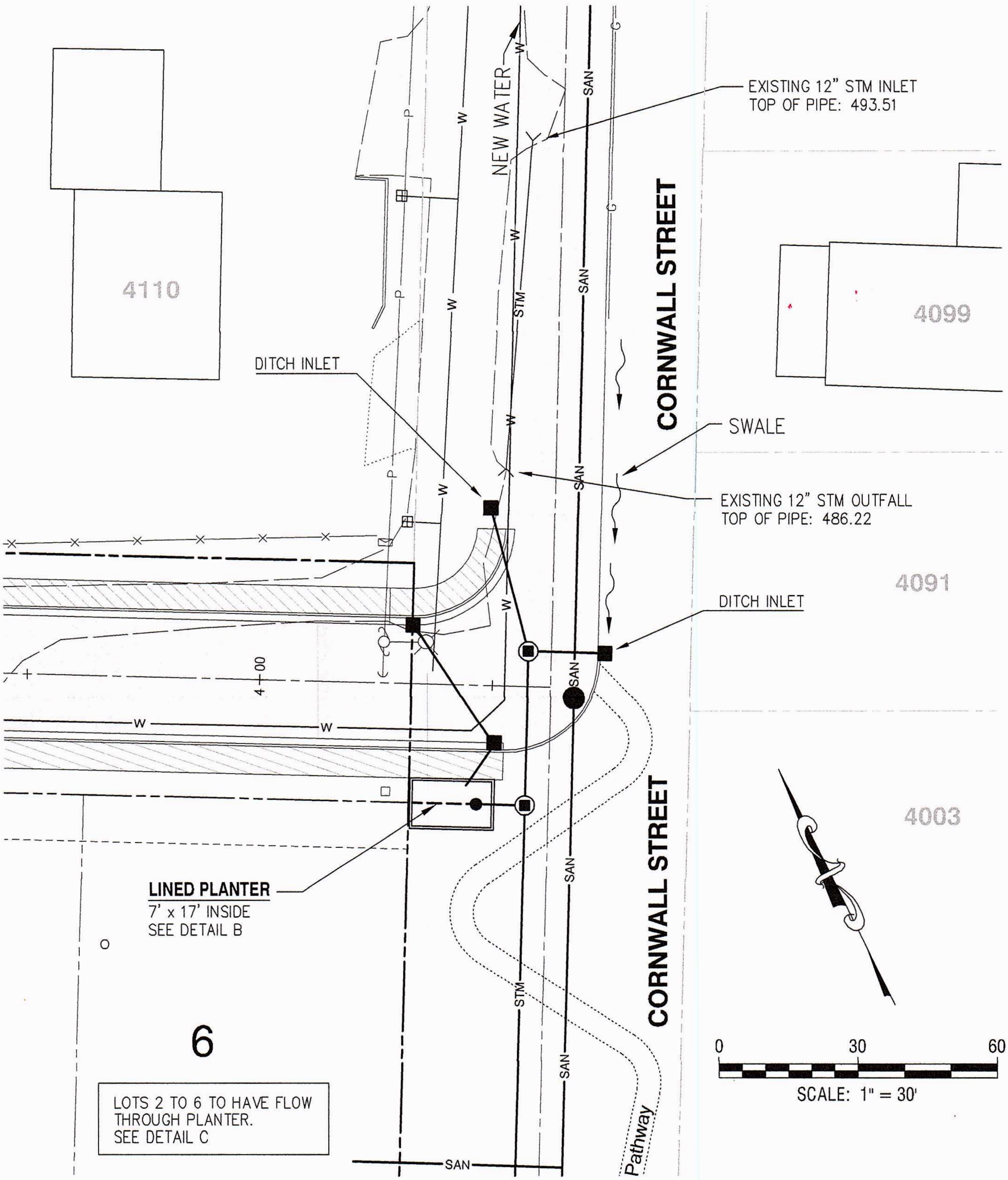
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Willow Ridge

4086 Cornwall Street
West Linn, Oregon



2010-129L **STORM DRAINAGE REPORT**

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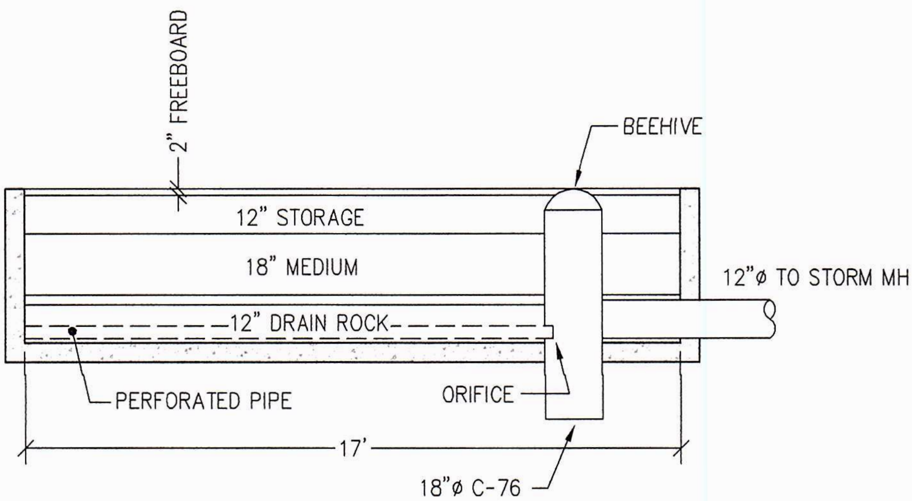
Lake Oswego, Oregon 97035
503-481-8822
email: thetaeng@comcast.net

Willow Ridge
West Linn, Oregon

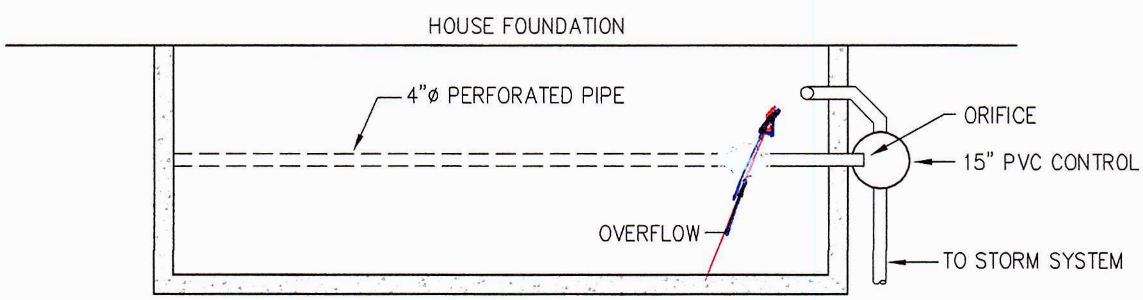
1
2



STREET SECTION WATER QUALITY AND QUANTITY PLANTER

SCALE: 1" = 5' HORIZONTAL
1" = 5' VERTICAL

B
1



RESIDENTIAL WATER QUALITY/ QUANTITY PLANTER LOTS 3–6

SCALE: 1" = 5' HORIZONTAL
1" = 5' VERTICAL

C
1

2010-129L **STORM DRAINAGE REPORT**

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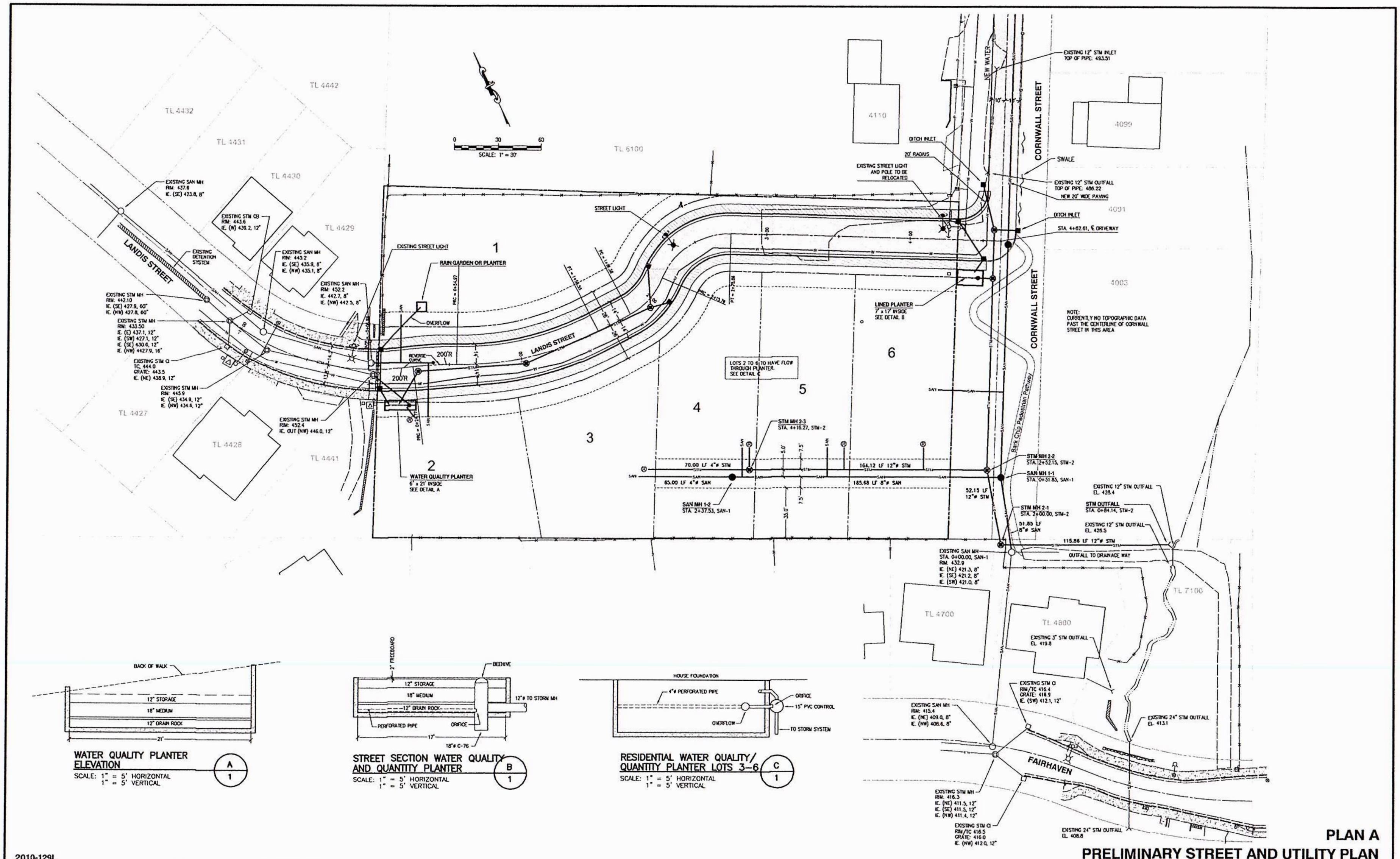
Lake Oswego, Oregon 97035
503-481-8822
email: thetaeng@comcast.net

Willow Ridge
West Linn, Oregon

2/2



WATER QUALITY PLANTER ELEVATION (A/1)
SCALE: 1" = 5' HORIZONTAL
1" = 5' VERTICAL

STREET SECTION WATER QUALITY AND QUANTITY PLANTER (B/1)
SCALE: 1" = 5' HORIZONTAL
1" = 5' VERTICAL

RESIDENTIAL WATER QUALITY/ QUANTITY PLANTER LOTS 3–6 (C/1)
SCALE: 1" = 5' HORIZONTAL
1" = 5' VERTICAL

PLAN A
PRELIMINARY STREET AND UTILITY PLAN

2010-129L

DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	1" = 30'			
DATE:	July, 2017	11/02/2017	1	APPLICATION
FILE:	Cornwall Street Prelim11	DATE	NO.	REVISION

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Icon Construction & Development, LLC
1980 Willamette Falls Drive, Suite 200
West Linn, Oregon 97068
Phone: (503) 657-0406

Willow Ridge
West Linn, Oregon

SHEET:
1/4

Presumptive Approach Calculator ver. 1.2

Catchment Data

Catchment ID: A
Date: 04/16/20
Permit Number: 0
Run Time 4/16/2020 5:36:05 PM

Project Name: cornwall LANDIS WQ
Project Address: 4069 Cornwell
west linn, Oregon
Designer: goldson
Company: theta

Drainage Catchment Information

Catchment ID	A	
	Catchment Area	
Impervious Area	9,480	SF
Impervious Area	0.22	ac
Impervious Area Curve Number, CN_{imp}	98	
Time of Concentration, Tc, minutes	5	min.

Site Soils & Infiltration Testing Data

Infiltration Testing Procedure:	Open Pit Falling Head	
Native Soil Field Tested Infiltration Rate (I_{test}):	1	in/hr
Bottom of Facility Meets Required Separation From High Groundwater Per BES SWMM Section 1.4:	Yes	

Correction Factor Component

CF_{test} (ranges from 1 to 3)	2	

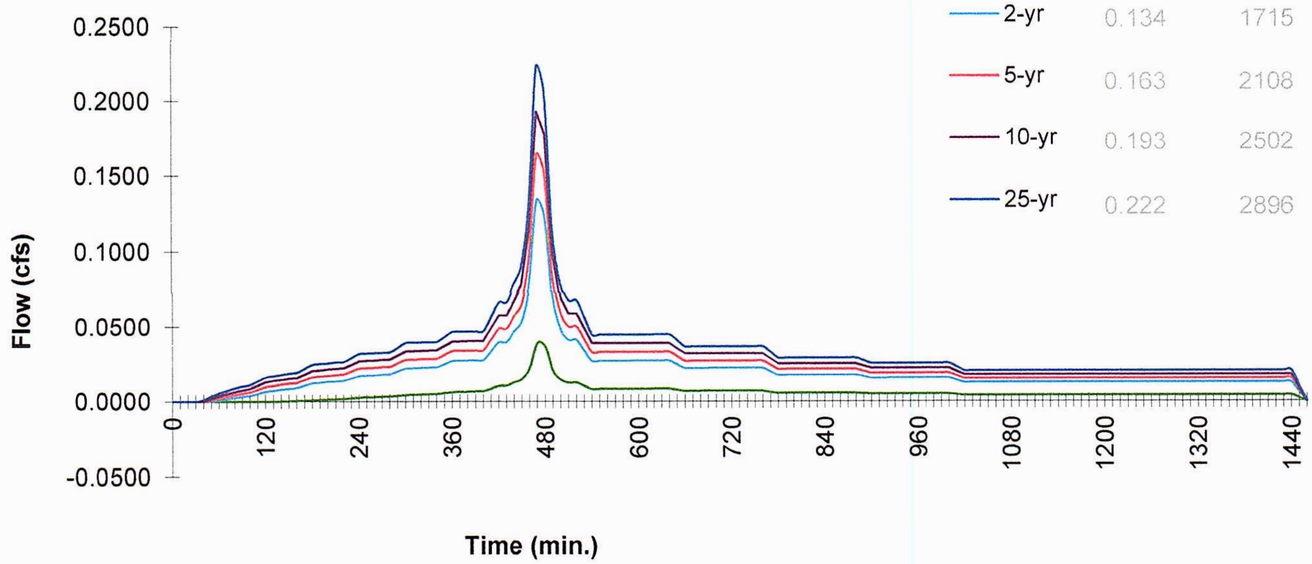
Design Infiltration Rates

I_{dsgn} for Native (I_{test} / CF_{test}):	0.50	in/hr
I_{dsgn} for Imported Growing Medium:	2.00	in/hr

Execute SBUH

SBUH Results

	Peak Rate (cfs)	Volume (cf)
PR	0.039	495
2-yr	0.134	1715
5-yr	0.163	2108
10-yr	0.193	2502
25-yr	0.222	2896



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Presumptive Approach Calculator ver. 1.2

Catchment ID: A

Run Time 4/16/2020 5:36:05 PM

Project Name: cornwall **Catchment ID:** A **Date:** 4/16/2020

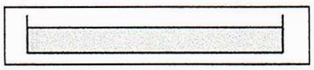
Instructions:
1. Identify which Stormwater Hierarchy Category the facility.
2. Select Facility Type.
3. Identify facility shape of surface facility to more accurately estimate surface volume, except for Swales and sloped planters that use the PAC Sloped Facility Worksheet to enter data.
4. Select type of facility configuration.
5. Complete data entry for all highlighted cells.

Catchment facility will meet Hierarchy Category: 3

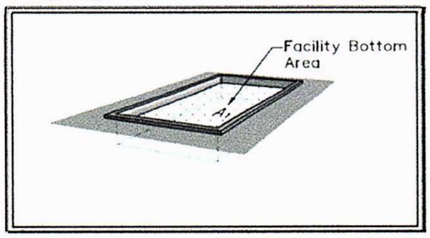
Goal Summary:

Hierarchy Category	SWMM Requirement	RESULTS box below needs to display…	
		Pollution Reduction as a	10-yr (aka disposal) as a
3	Off-site flow to drainageway, river, or storm-only pipe system.	PASS	N/A

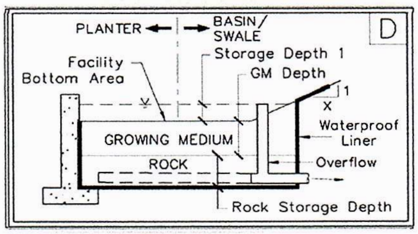
Facility Type = Planter (Flat)



Facility Shape: Rectangle/Square



Facility Configuration: D



Calculation Guide
Max. Rock Stor. Bottom Area
126 SF

DATA FOR ABOVE GRADE STORAGE COMPONENT

Facility Bottom Area =	126	sf
Bottom Width =	6.0	ft
Facility Side Slope =	0	to 1
Storage Depth 1 =	12	in
Growing Medium Depth =	18	in
Freeboard Depth =	N/A	in
Surface Capacity at Depth 1 =	126	cf
GM Design Infiltration Rate =	2.00	in/hr
Infiltration Capacity =	0.006	cfs

BELOW GRADE STORAGE

Rock Storage Capacity =		cf
Native Design Infiltration Rate =		in/hr
Infiltration Capacity =		cfs

RESULTS

Pollution Reduction: **PASS**

Overflow Volume: 0 CF

98% Surf. Cap. Used

Run PAC

Output File

	2-yr	5-yr	10-yr	25-yr
Peak cfs	0.134	0.163	0.193	0.222

FACILITY FACTS

Total Facility Area Including Freeboard = **126 SF**

Sizing Ratio (Total Facility Area / Catchment Area) = **0.013**

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Catchment Data

Presumptive Approach Calculator ver. 1.2

Project Name:	**Willow Ridge** (LOT Rain Garden)	**Catchment ID:**	**A**
Project Address:	**4086 Cornwall St**	**Date:**	**12/18/19**
	West Linn	**Permit Number:**	
Designer:	**Goldson**	Run Time	12/18/2019 7:34:29 PM
Company:	**Theta**		

Drainage Catchment Information

Catchment ID	A	
	Catchment Area	
Impervious Area	2,600	SF
Impervious Area	0.06	ac
Impervious Area Curve Number, CN_{imp}	98	
Time of Concentration, Tc, minutes	5	min.

Site Soils & Infiltration Testing Data

Infiltration Testing Procedure:	Open Pit Falling Head	
Native Soil Field Tested Infiltration Rate (I_{test}):	1	in/hr
Bottom of Facility Meets Required Separation From High Groundwater Per BES SWMM Section 1.4:	Yes	

Correction Factor Component

CF_{test} (ranges from 1 to 3)	2	

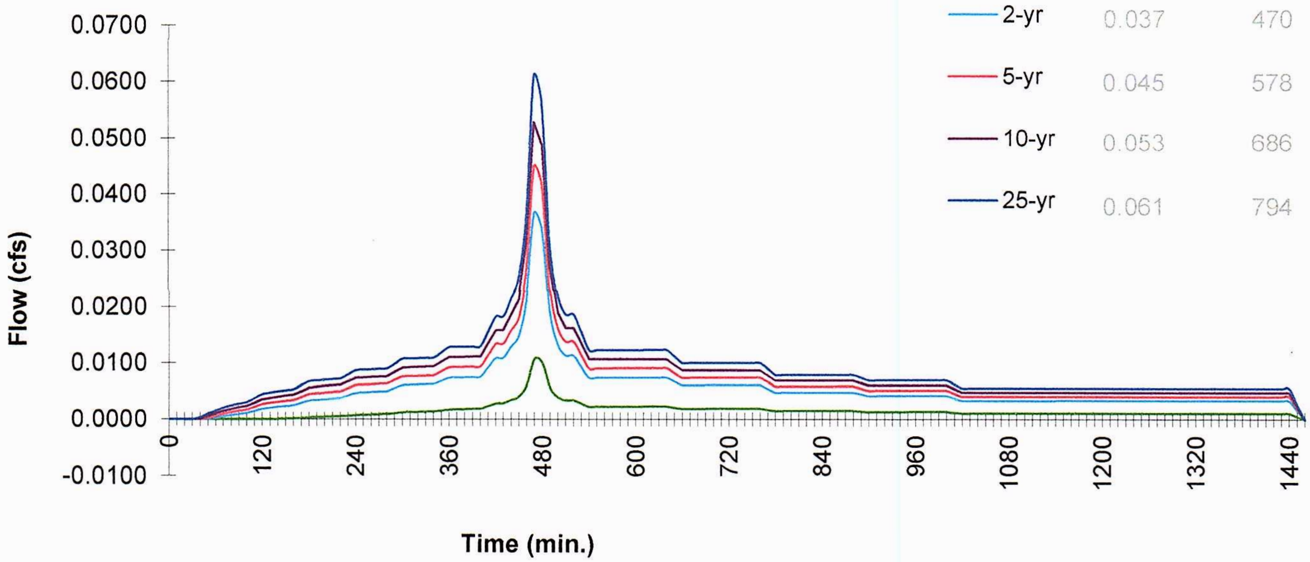
Design Infiltration Rates

I_{dsgn} for Native (I_{test} / CF_{test}):	0.50	in/hr
I_{dsgn} for Imported Growing Medium:	2.00	in/hr

Execute SBUH

SBUH Results

	Peak Rate (cfs)	Volume (cf)
PR	0.011	136
2-yr	0.037	470
5-yr	0.045	578
10-yr	0.053	686
25-yr	0.061	794



Printed: 12/18/2019 7:37 PM

Presumptive Approach Calculator ver. 1.2

Catchment ID: A

Run Time 12/18/2019 7:34:29 PM

Project Name: Willow Ridge **Catchment ID:** A **Date:** 12/18/2019

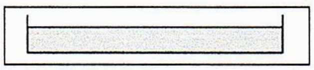
Instructions:
1. Identify which Stormwater Hierarchy Category the facility.
2. Select Facility Type.
3. Identify facility shape of surface facility to more accurately estimate surface volume, except for Swales and sloped planters that use the PAC Sloped Facility Worksheet to enter data.
4. Select type of facility configuration.
5. Complete data entry for all highlighted cells.

Catchment facility will meet Hierarchy Category: 4

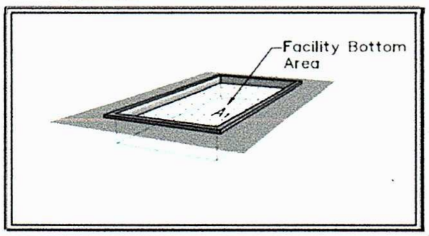
Goal Summary:

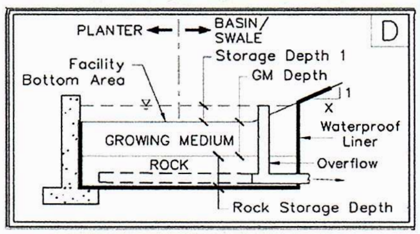
Hierarchy Category	SWMM Requirement	RESULTS box below needs to display…	
		Pollution Reduction as a	10-yr (aka disposal) as a
4	Off-site flow to a combined sewer.	PASS	N/A

Facility Type = Planter (Flat)



Facility Shape: Rectangle/Square **Facility Configuration:** D





Calculation Guide
Max. Rock Stor. Bottom Area
60 SF

DATA FOR ABOVE GRADE STORAGE COMPONENT

Facility Bottom Area =	60	sf
Bottom Width =	6.0	ft
Facility Side Slope =	0	to 1
Storage Depth 1 =	12	in
Growing Medium Depth =	18	in
Freeboard Depth =	N/A	in
Surface Capacity at Depth 1 =	60	cf
GM Design Infiltration Rate =	2.00	in/hr
Infiltration Capacity =	0.003	cfs

BELOW GRADE STORAGE

Rock Storage Capacity =		cf
Native Design Infiltration Rate =		in/hr
Infiltration Capacity =		cfs

RESULTS

Pollution Reduction		Overflow Volume	Surf. Cap. Used
	PASS	0 CF	27%

Run PAC

Output File

	2-yr	5-yr	10-yr	25-yr
Peak cfs	0.037	0.045	0.053	0.061

FACILITY FACTS

Total Facility Area Including Freeboard = **60 SF**

Sizing Ratio (Total Facility Area / Catchment Area) = **0.023**

Printed: 12/18/2019 7:37 PM

Presumptive Approach Calculator ver. 1.2

Catchment Data

Catchment ID: A
Date: 10/13/20
Permit Number: 0
Run Time 10/13/2020 6:41:33 PM

Project Name: willon ridge Landis east
Project Address: 4086 cornwald
West LINN
Designer: Goldson
Company: Theta

Drainage Catchment Information

Catchment ID	A	
	Catchment Area	
Impervious Area	5,531	SF
Impervious Area	0.13	ac
Impervious Area Curve Number, CN_{imp}	98	
Time of Concentration, Tc, minutes	5	min.

Site Soils & Infiltration Testing Data

Infiltration Testing Procedure:	Open Pit Falling Head	
Native Soil Field Tested Infiltration Rate (I_{test}):	1	in/hr
Bottom of Facility Meets Required Separation From High Groundwater Per BES SWMM Section 1.4:	Yes	

Correction Factor Component

CF_{test} (ranges from 1 to 3)	2	

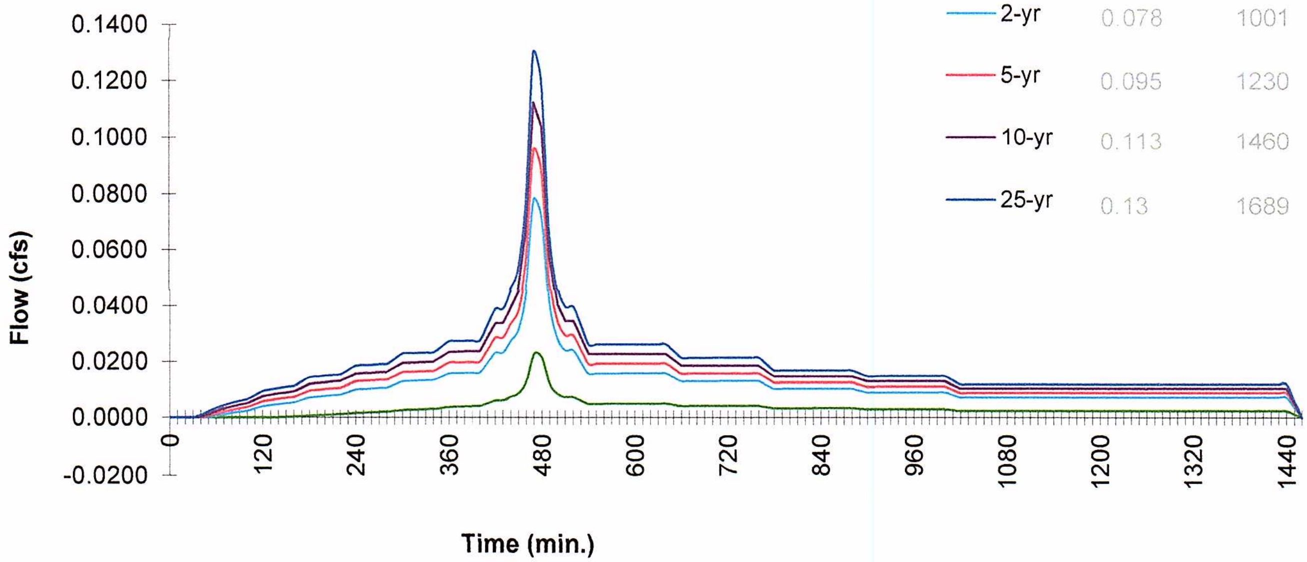
Design Infiltration Rates

I_{dsgn} for Native (I_{test} / CF_{test}):	0.50	in/hr
I_{dsgn} for Imported Growing Medium:	2.00	in/hr

Execute SBUH

SBUH Results

	Peak Rate (cfs)	Volume (cf)
PR	0.023	289
2-yr	0.078	1001
5-yr	0.095	1230
10-yr	0.113	1460
25-yr	0.13	1689



Printed: 10/13/2020 6:44 PM

Presumptive Approach Calculator ver. 1.2

Catchment ID: A

Run Time 10/13/2020 6:41:33 PM

Project Name: willon ridge Landis east **Catchment ID:** A **Date:** 10/13/2020

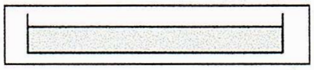
Instructions:
1. Identify which Stormwater Hierarchy Category the facility.
2. Select Facility Type.
3. Identify facility shape of surface facility to more accurately estimate surface volume, except for Swales and sloped planters that use the PAC Sloped Facility Worksheet to enter data.
4. Select type of facility configuration.
5. Complete data entry for all highlighted cells.

Catchment facility will meet Hierarchy Category: 3

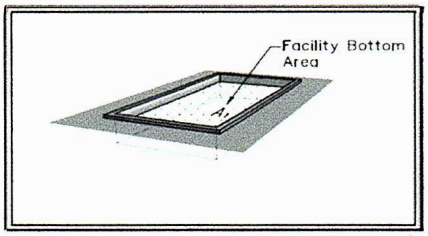
Goal Summary:

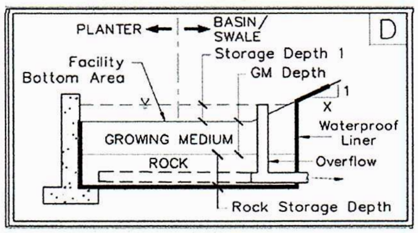
Hierarchy Category	SWMM Requirement	RESULTS box below needs to display…	
		Pollution Reduction as a	10-yr (aka disposal) as a
3	Off-site flow to drainageway, river, or storm-only pipe system.	PASS	N/A

Facility Type = Planter (Flat)



Facility Shape: Rectangle/Square **Facility Configuration:** D





Calculation Guide
Max. Rock Stor. Bottom Area
166 SF

DATA FOR ABOVE GRADE STORAGE COMPONENT

Facility Bottom Area =	166	sf
Bottom Width =	7.0	ft
Facility Side Slope =	0	to 1
Storage Depth 1 =	12	in
Growing Medium Depth =	18	in
Freeboard Depth =	N/A	in
Surface Capacity at Depth 1 =	166	cf
GM Design Infiltration Rate =	2.00	in/hr
Infiltration Capacity =	0.008	cfs

BELOW GRADE STORAGE

Rock Storage Capacity =		cf
Native Design Infiltration Rate =		in/hr
Infiltration Capacity =		cfs

RESULTS

Pollution Reduction: **PASS** Overflow Volume: 0 CF 15% Surf. Cap. Used

Run PAC

Output File

	2-yr	5-yr	10-yr	25-yr
Peak cfs	0.078	0.095	0.113	0.130

FACILITY FACTS

Total Facility Area Including Freeboard = **166 SF**

Sizing Ratio (Total Facility Area / Catchment Area) = **0.030**

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

October 28, 2020
Project No. 19-5378

Icon Construction
1980 Willamette Falls Drive, #200
West Linn, OR 97068
Phone 503-657-0406
Email: darren@iconconstructino.net; rickgivens@gmail.com

**SUBJECT: REBUTTAL TO PUBLIC TESTIMONY OF WILLIAM HOWARD
WILLOW RIDGE ESTATES AKA CORNWALL STREET SUBDIVISION
4096 CORNWALL STREET
WEST LINN, OREGON**

References:

1. William House Public Testimony: Willow Ridge Geologic and Hydrologic Risk Parameters, 4096 Cornwall St., West Linn, OR, Tax Lot 6300, October 7, 2020.

2. Carlson Geotechnical, Report of Geotechnical Investigation, Cornwall Street Subdivision, 4096 Cornwall Street, West Linn, Oregon, January 7, 2016.

3. GeoPacific Engineering, Inc., Geotechnical Report, Willow Ridge Subdivision, 4096 Cornwall Street, West Linn, Oregon, October 23, 2020.

4. RNSA Inc., Technical Memorandum, Groundwater Characteristics, Willow Ridge Project Site, Willow Ridge Project Site, 4096 Cornwall Street, West Linn, Oregon October 28, 2020.

This letter is in response to the written public testimony of William House. In summary, Mr. House's two key findings state that the previous geotechnical studies 1) do not recognize the presence of a perched water table at Willow Ridge and 2) as a result do not adequately address the geologic risk from shallow landslides and "flooding". Verbal public testimony by Pam Yokubaitis concluded that the Carlson Geotechnical report of January 7, 2016 had expired and cannot be relied upon for a land use submittal.

In response to these concerns, the applicant has engaged a hydrogeologist for responding to the groundwater source and flow rate issues, and GeoPacific Engineering, Inc. to update the geotechnical report and study the validity of the technical claims relating to geotechnical engineering and engineering geology regarding assessment of the potential presence of groundwater on the proposed development and how that affects slope stability and "flooding" (better identified as groundwater intrusion) issues.

The geotechnical studies in the record, including this letter and a letter to be prepared by Roger N. Smith Associates, Inc., fulfill the requirements of Community Development Code (CDC) 85.200E(5). This standard requires a report by an engineering geologist on Type 1 lands (Slopes over 35% grade) and a geologic hazard report on Type I and Type II lands (slopes 25-35% grade). The subject property contains areas with slopes that exceed these slope gradient guidelines.

After performing our study, which included site-specific explorations conducted by GeoPacific's Certified Engineering Geologist, Beth Rapp, and also observed by Roger Smith (hydrogeologist), we conclude that the aquifer theorized to outcrop on the site is not present. Neither springs, loose basalt, nor significant quantities of water were encountered in subsurface explorations at or below the elevations theorized by Mr. House. Furthermore, with typical proper subdrainage precautions, the street and home construction will not create or be subject to increased groundwater concerns or slope stability hazards. It is our conclusion that the relatively light groundwater seepage encountered in our explorations can be easily managed by installing conventional subdrains during grading and footing drains during home construction.

In comparison to the over 150 projects GeoPacific has completed in the City of West Linn over the last 20 years and approximately 7,000 projects in the Metro region, we conclude that the groundwater conditions would fall near the average range of perched groundwater given the minor amount of seepage we encountered in our site-specific explorations. Due to the presence of basalt rock that is characterized by a moderate to high resistance to slope instability, we would consider the proposed construction to present a very low slope stability hazard compared to similarly sloping projects. Finish grades on the site after grading will be no steeper than 2H:1V (50 percent). GeoPacific has evaluated slope stability for the proposed grading and it is our opinion that the proposed slopes will have adequate factors of safety for stability.

The subsurface data utilized by Mr. House in formulating his theories was obtained from regional geologic mapping and from a well log a few hundred feet away from the subject site. Soil descriptions on well logs are not considered equivalent to soil borings drilled for geologic or geotechnical studies, and are not logged by licensed professionals, or their representatives. For example, the loose basalt referenced by Mr. House from the well log may or may not mean loose in the geotechnical sense since there is no record of Standard Penetration Test (SPT) blow counts in the well log. It is our opinion that soil descriptions from well logs should not be solely relied upon to develop geologic cross sections and also should not be extrapolated a few hundred feet away from the well location.

Due to the potential for differing site conditions, regional mapping is not to be relied upon or used to refute site-specific studies, especially for slope hazard assessment or subsurface conditions at a specific site. Proper assessment of geologic risk on sloping sites, requires that the site be studied, on a site-specific basis, by either an Oregon licensed professional civil engineer with relevant experience, a licensed geotechnical engineer, or an Oregon licensed engineering geologist.

Due to the importance of both local geologic experience and construction and engineering knowledge, the Oregon State Board of Geologist Examiners (OSBGE) and The Oregon State Board Examiners for Engineering and Land Surveying (OSBEELS) require licensing for individuals to perform geotechnical or engineering geologic studies. These boards do not recognize individuals that have only been previously licensed in other states or whose previous license was not specifically focused on the three types of licenses listed above as qualified to perform geologic/geotechnical assessments on proposed construction projects.

Community Development Code (CDC) 85.200.E(6) states:

> "6. Per the submittals required by CDC 85.170(C)(3), the applicant must demonstrate that the proposed methods of rendering known or potential hazard sites safe for development, including proposed geotechnical remediation, are feasible and adequate to prevent landslides or other damage to property and safety. The review authority may impose conditions, including limits on type or intensity of land use, which it determines are necessary to mitigate known risks of landslides or property damage."

Based upon our analysis and the information presented in the reports presently in the record and the additional information contained in the new letters, the applicant has met their burden under this standard. The development of the site is geotechnically feasible and, with implementation of measures presented in these studies, will adequately avert slope instability, landslides, and other damage to properties relating to geotechnical issues caused by development of the site.

Development of the site, with adequate surface and subsurface drainage, should result in drains that outfall in a controlled manner as designed by the project civil engineer, Theta Engineering, LLC. Furthermore, development measures should not significantly alter existing groundwater flow other than possibly a slight reduction. In our experience, existing nearby properties with insufficient drainage measures are most likely due to home construction defects and are very unlikely caused by insurmountable regional groundwater issues.

In a stance of solidarity to the confidence we have in our conclusions regarding the Willow Ridge site, we have stamped this letter with one engineer's stamp, one geotechnical engineer's stamp and three certified engineering geologist's stamps.

Willow Ridge
Project No. 19-5378

Sincerely,

GEOPACIFIC ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
14743PE
OREGON
JAN. 23, 1990
JAMES D. IMBRIE
EXPIRES: 06/30/2021

CERTIFIED
OREGON
JAMES D. IMBRIE
E1734
ENGINEERING GEOLOGIST
Exp 5-30-2

James D. Imbrie, P.E., G.E., R.G., C.E.G.
Principal Geotechnical Engineer and Engineering Geologist

CERTIFIED
OREGON
ELIZABETH K. RAPP
No. E2190
ENGINEERING GEOLOGIST
exp 2/1/21

Beth K. Rapp, R.G., C.E.G.
Associate Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
90013PE
OREGON
MAY 22, 2019
BENJAMIN G. ANDERSON
Expires: 12/31/21

Benjamin G. Anderson, P.E.
Associate Engineer

CERTIFIED
OREGON
Benjamin L Cook
E2245
ENGINEERING GEOLOGIST
EXP 1/1/21

Benjamin L. Cook, R.G., C.E.G.
Associate Engineering Geologist

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

October 27, 2020
Project No. 19-5378

Darren Gusdorf
ICON Construction & Development, LLC
1969 Willamette Falls Drive, Suite 260
West Linn, Oregon 97068
Via email: darren@iconconstruction.net

SUBJECT: GEOTECHNICAL REPORT
WILLOW RIDGE SUBDIVISION
4096 CORNWALL STREET
WEST LINN, OREGON

Reference: *Report of Geotechnical Investigation, Cornwall Street Subdivision, 4096 Cornwall Street, West Linn, Oregon,* Carlson Geotechnical report dated January 7, 2016.

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site development.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site is located to the west of the southern terminus of Cornwall Street in the City of West Linn, Clackamas County, Oregon (Figure 1). The property is approximately 2.2 acres in size and topography is moderately sloping to the southeast to southwest at grades of approximately 15 to 55 percent (Figures 2 & 3). The site is currently occupied by one home, barn, and outbuilding. Vegetation consists primarily of short grasses, brambles, and sparse trees.

It is our understanding that the proposed development includes 6 lots for single family homes, construction of new street, and associated underground utilities (Figure 2). The existing structures will be removed. The grading plan provided for our review indicates maximum cuts and fills will be on the order of 8 feet or less and will be limited to the vicinity of the proposed street.

REGIONAL AND LOCAL GEOLOGIC SETTING

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 located on a southwest facing slope at elevations of approximately 430 to 495 feet above sea level. The subject site is underlain by the Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalt Formation, which are a thick sequence of lava flows which form the crystalline basement of the Tualatin Valley (Schlicker and Finlayson, 1979; Beeson et al., 1989; Gannett and Caldwell, 1998). 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.

REGIONAL SEISMIC SETTING

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

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 approximately 3.8 miles northeast of the site. The East Bank Fault is oriented roughly parallel to the Portland Hills Fault, on the east bank of the Willamette River, and is located approximately 10.9 miles north of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is approximately 2.8 miles northeast of the site. The Oatfield Fault is considered to be potentially seismogenic (Wong, et al., 2000). Madin and Mabey (1996) indicate the Portland Hills Fault Zone has experienced Late Quaternary (last 780,000 years) fault movement; however, movement has not been detected in the last 20,000 years. 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).

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 approximately 16.4 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 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 Fault or Newberg Fault (the fault 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).

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 between 20 and 40 miles.

SUBSURFACE CONDITIONS

Our site-specific exploration for this report was conducted on October 14, 2018. A total of 7 exploratory test pits were excavated with a medium sized trackhoe to depths of 8 to 15 feet at the approximate locations presented on Figure 3. It should be noted that test pit locations were located 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.

A GeoPacific Engineering Geologist continuously monitored the field exploration program and logged the test pits. Soils observed in the explorations were classified in general accordance with the Unified Soil Classification System (USCS). Rock hardness was classified in accordance with Table 1, modified from the ODOT Rock Hardness Classification Chart. During exploration, our geologist also noted geotechnical conditions such as soil consistency, moisture and groundwater conditions. Logs of test pits are attached to this report. The following report sections are based on the exploration program and summarize subsurface conditions encountered at the site.

Table 1. Rock Hardness Classification Chart

ODOT Rock Hardness Rating	Field Criteria	Unconfined Compressive Strength	Typical Equipment Needed For Excavation
Extremely Soft (R0)	Indented by thumbnail	<100 psi	Small excavator
Very Soft (R1)	Scratched by thumbnail, crumbled by rock hammer	100-1,000 psi	Small excavator
Soft (R2)	Not scratched by thumbnail, indented by rock hammer	1,000-4,000 psi	Medium excavator (slow digging with small excavator)
Medium Hard (R3)	Scratched or fractured by rock hammer	4,000-8,000 psi	Medium to large excavator (slow to very slow digging), typically requires chipping with hydraulic hammer or mass excavation)
Hard (R4)	Scratched or fractured w/ difficulty	8,000-16,000 psi	Slow chipping with hydraulic hammer and/or blasting
Very Hard (R5)	Not scratched or fractured after many blows, hammer rebounds	>16,000 psi	Blasting

Undocumented Fill: Undocumented fill was not encountered in our explorations. Our reconnaissance and the topographic survey indicate that some fill may have been placed to the south of the existing home and in the vicinity of the existing outbuildings. We anticipate other areas of fill may be present adjacent to the Cornwall Street right of way. Fill up to 4.25 feet in thickness was encountered by others in the vicinity of the existing home and outbuildings (Carlson Geotechnical, 2016).

Topsoil Horizon: Directly underlying the ground surface in test pits TP-1 through TP-7 was a topsoil horizon consisting of brown, moderately to highly organic silt (OL-ML) with gray basalt fragments. The topsoil horizon was generally loose, contained many fine roots, and extended to a depth of 8 to 14 inches below the ground surface.

Residual Soil: Underlying the topsoil horizon in test pits TP-1 through TP-7 was clayey silt (ML) to silty clay (CL) residual soil resulting from in-place weathering of the underlying Columbia River Basalt Formation. The light reddish-brown silty clay to clayey silt contained varying quantities of basalt fragments and was generally characterized by a very stiff consistency. In test pits TP-1 and TP-7, the residual soil extended to a depth of 3 to 4 feet.

Columbia River Basalt Formation: The residual soil in test pits TP-1 through TP-7 was underlain by weathered basalt belonging to the Columbia River Basalt Formation. Generally, the gray to brown basalt was extremely soft (R0) to very soft (R1) with trace light reddish brown silty clay to clayey silt matrix. The basalt was vesicular; however, the vesicles were not interconnected and were often filled with yellow clay mineralization. The weathered basalt generally contained harder (R3) basalt boulders. The basalt was excavatable to a depth of 13.5 to 15 feet in test pits TP-1 through TP-3, TP-6, and TP-7. Practical refusal on medium hard (R3) boulders was achieved with a medium sized trackhoe at a depth of 8 feet in test pit TP-4 and 12 feet in test pit TP-5. Table 2 presents the depths at which rock was first encountered in test pits and the depth at which practical refusal was achieved with a medium sized backhoe equipped with rock teeth.

Table 2. Depth of Basalt Bedrock Encountered in Explorations

Test Pit	Depth Rock First Encountered	Depth of Practical Refusal on Medium Hard (R3) Basalt
TP-1	3’	>15’
TP-2	3’	>13.5’
TP-3	3’	>15’
TP-4	4’	8’ (on boulder)
TP-5	3’	12’ (on boulder)
TP-6	3’	>15’
TP-7	4’	>15’

Soil Moisture and Groundwater

On October 14, 2020, perched groundwater seepage was encountered in test pits TP-1 through TP-3 and TP-4 through TP-7at depths of 1.5 feet to 10. Discharge was visually estimated at less than 1/10 gallon per minute to ½ gallon per minute. Generally the seepage was localized to one elevation indicating the seepage was related to surface water/precipitation and not static groundwater. Regional groundwater mapping indicates that static groundwater is present at a depth of approximately 200 to 240 feet below the ground surface (Snyder, 2008). Experience has shown that temporary storm related perched groundwater within the near surface soils often occur over fine-grained native deposits such as those beneath the site during the wet season and particularly in mottled soils such as were identified in the test pits. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors.

Slope Stability

For the purpose of evaluating slope stability, we reviewed 1:24,000 scale topographic mapping by the U.S. Geological Survey (Figure 1), Lidar based high resolution digital elevation maps (Figure 2), reviewed published geologic mapping and the statewide Landslide Database (Figure 2), and 1:360 scale topographic mapping by Theta, LLC. dated November 2, 2017 (Figure 3), performed a field reconnaissance, and explored subsurface conditions at the site with seven exploratory test pits, the locations of which are presented on Figure 3.

Regional geologic mapping (Schlicker and Finlayson, 1979; Burns, 2009) and the statewide landslide database (Dogami Slido, 2020) indicate no mapped landslides are present at the site or within 2,000 feet of the site, as presented on Figure 2. Subsurface exploration indicates that the ground surface is underlain by residual soil and basalt bedrock belonging to the Columbia River Basalt Formation. Pocket penetrometer measurements of the residual soil indicate an approximate unconfined compressive strength of 1.5 to greater than 4.5 tons/ft^2 which correlates to a stiff to very stiff consistency. The residual soil extended to a depth of 3 to 4 feet and was underlain by highly weathered, basalt bedrock in test pits. The weathered basalt extended beyond the maximum depth of exploration (8 to 15 feet). No weak zones such as volcanic ash layers were observed in explorations and contacts between the layers appeared to be gradational. Our explorations indicate that native soils underlying the slope are characterized by moderate to high shear strength and a moderate to high resistance to slope instability.

Topography at the subject site is predominantly moderately sloping to the southeast to southwest with grades up to 55 percent (Figure 3). Our reconnaissance and review of Lidar based high resolution digital elevation maps (Dogami, 2020) indicate slope geomorphology on the site is smooth and uniform. Some slope areas in the vicinity of the existing structures may have been oversteepened by fill placement. No evidence of recent movement (ground cracks, scarps, or hummocky topography) was observed during our reconnaissance. In our opinion, the slope instability hazard at the subject site is very low and there are no off-site slope stability hazards that would affect the proposed development. Existing Columbia River Basalt Formation materials are stiff to hard and no evidence of recent landslide movement was observed.

CONCLUSIONS AND RECOMMENDATIONS

Our investigation indicates that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and sufficient geotechnical monitoring is incorporated into the construction phases of the project. In our opinion, the greatest geotechnical issues for project completion included:

1. The presence of boulders within the weathered basalt. Medium hard (R3) basalt boulders were encountered within the weathered basalt in test pits throughout the site. The weathered basalt was generally excavatable to depths of 13.5 to 15 feet; however, practical refusal was achieved on medium hard (R3) boulders at a depth of 8 feet in test pit TP-4 and at 12 feet in test pit TP-5. A larger excavator should be able to achieve greater depths but difficult excavating conditions should be expected.

2. The potential to encounter fill soils. Our reconnaissance and the topographic survey indicate some fill soils may be present in the vicinity of the existing structures. Test pits by others encountered fill up to 4.25 feet in the northeastern portion of the site (Carlson Geotechnical, 2016).

Stormwater Disposal

It is our understanding that stormwater disposal of street runoff, roof, and foundation water will be routed to either treatment structures onsite that outlet to an existing underground detention system in Landis Street or be routed to lined rain gardens which will outfall to the creek to the east of the site via a storm pipe. While infiltration at the site is poor, it is our opinion that the proposed stormwater disposal plan will not have an adverse effect on slope stability.

Site Preparation

Areas of proposed buildings, new streets, and areas to receive fill should be cleared of vegetation and any organic and inorganic debris. Existing buried structures should be demolished and any cavities structurally backfilled. Inorganic debris and organic materials from clearing should be removed from the site. Existing fill and any organic-rich topsoil should then be stripped from construction areas of the site or where engineered fill is to be placed. Fill was not encountered in our explorations; however, our reconnaissance indicates that fill is likely present in the vicinity of the existing structures and potentially along the Cornwall Street right of way. Up to 4.25 feet of fill was encountered by others near the existing structures in the northeastern portion of the site (Carlson Geotechnical, 2016).

Organic-rich topsoil should then be stripped from native soil areas of the site. The estimated depth range necessary for removal of topsoil in cut and fill areas is approximately 9 to 12 inches, respectively. The final depth of soil removal will be determined on the basis of a site inspection after the stripping/excavation has been performed. Stripped topsoil should preferably be removed from the site due to the high density of the proposed development. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

Any remaining undocumented fills and subsurface structures (tile drains, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be removed and the excavations backfilled with engineered fill.

Once stripping of a particular area is approved, the area must be ripped or tilled to a depth of 12 inches, moisture conditioned, root-picked, and compacted in-place prior to the placement of engineered fill or crushed aggregate base for pavement. Exposed subgrade soils should be evaluated by the geotechnical engineer. 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, 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, over-excavated and replaced with engineered fill (as described below), or stabilized with rock prior to placement of engineered fill. The depth of overexcavation, if required, should be evaluated by the geotechnical engineer at the time of construction.

Engineered Fill

All grading for the proposed development should be performed as engineered grading in accordance with the applicable building code at time of construction with the exceptions and additions noted herein. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 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 standard compaction equipment. We recommend that engineered fill be compacted to at least 95% of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Site earthwork will be impacted by soil moisture and shallow groundwater conditions. Earthwork in wet weather would likely require extensive use of cement or lime treatment, or other special measures, at a considerable additional cost compared to earthwork performed under dry-weather conditions.

Excavating Conditions and Utility Trenches

We anticipate that on-site soils can be excavated using conventional heavy equipment such as scrapers and trackhoes. Weathered basalt bedrock was encountered in test pits throughout the site at depths of 3 to 4 feet. The basalt was excavatable to depths of 13.5 to 15 feet; however, large, medium hard (R3) boulders were encountered within the weathered basalt and practical refusal was achieved on these medium hard (R3) boulders at a depth of 8 feet in test pit TP-4 and 12 feet in test pit TP-5. A larger excavator should be able to achieve greater depths; however, difficult excavating conditions should be expected.

All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soil is classified 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 groundwater seepage zones only. 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.

Saturated soils and groundwater may be encountered in utility trenches, particularly during the wet season. We anticipate that dewatering systems consisting of ditches, sumps and pumps would be adequate for control of perched groundwater. Regardless of the dewatering system used, it should be installed and operated such that in-place soils are prevented from being removed along with the groundwater.

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 trench backfill be compacted to at least 95% of the maximum dry density obtained by Modified Proctor ASTM D1557 or equivalent. Initial backfill lift thickness 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, one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Erosion Control Considerations

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion except in areas of moderately sloping topography. 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 straw wattles and silt fences. If 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.

Wet Weather Earthwork

Soils underlying the site are likely to be 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
- Geotextile silt fences, straw wattles, and fiber rolls 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.

Pavement Design

For design purposes, we used an estimated resilient modulus of 9,000 for compacted native soil. Table 3 presents our recommended minimum pavement section for dry weather construction.

Table 3. Recommended Minimum Dry-Weather Pavement Section

Material Layer	Light-duty Public Streets	Private Driveways	Compaction Standard
Asphaltic Concrete (AC)	3 in.	2.5 in.	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾”-0 (leveling course)	2 in.	2 in.	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1½”-0	8 in.	6 in.	95% of Modified Proctor AASHTO T-180
Subgrade	12 in.	12 in.	95% of Standard Proctor AASHTO T-99 or equivalent

Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill (see *Site Preparation* Section). 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, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition-specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project.

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 asphalt compaction test is performed for every 100 to 200 linear feet of paving.

Spread 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. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. The recommended minimum widths for continuous footings supporting wood-framed walls without masonry are 12 inches for single-story, 15 inches for two-story, and 18 inches for three-story structures. Minimum foundation reinforcement should consist of a No. 4 bar at the tops of stem walls, and a No. 4 bar at the bottom of footings. Concrete slab-on-grade reinforcement should consist of No. 4 bars placed on 24-inch centers in a grid pattern.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. A maximum chimney and column load of 30 kips is recommended for the site. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For heavier loads, the geotechnical engineer should be consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil and any loose soil to competent subgrade that is suitable for bearing support. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require overexcavation of footings and backfill with compacted, crushed aggregate.

Our recommendations are for house construction incorporating raised wood floors and conventional spread footing foundations. If living space of the structures will incorporate basements, a geotechnical engineer should be consulted to make additional recommendations for retaining walls, water-proofing, underslab drainage and wall subdrains. After site development, a Final Soil Engineer's Report should either confirm or modify the above recommendations.

Permanent Below-Grade Walls

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

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

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the

Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 6.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 320 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.42 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

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

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

Wall drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade. An adequate grade to a low point outlet drain in the crawlspace is required by code. Underslab drains are sometimes added beneath the slab when placed over soils of low permeability and shallow, perched groundwater.

Water collected from the wall drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the wall drains in order to reduce the potential for clogging. The drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building.

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

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

Seismic Design

The Oregon Department of Geology and Mineral Industries (Dogami), Oregon HazVu: 2020 Statewide GeoHazards Viewer indicates that the site is in an area where *very strong* ground shaking is anticipated during an earthquake. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2015 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2014). We recommend Site Class C be used for design per the OSSC, Table 1613.5.2 and as defined in ASCE 7, Chapter 20, Table 20.3-1. Design values determined for the site using the ATC (Applied Technology Council) *ASCE7-16 Hazards by Location online Tool* website are summarized in Table 4 and are based upon existing soil conditions.

Table 4. Recommended Earthquake Ground Motion Parameters (ATC 2020)

Parameter	Value
Location (Lat, Long), degrees	45.357, -122.634
Mapped Spectral Acceleration Values (MCE):	
Peak Ground Acceleration PGA_M	0.453
Short Period, S_s	0.838 g
1.0 Sec Period, S_1	0.377 g
Soil Factors for Site Class C:	
F_a	1.2
F_v	1.5
Residential Site Value = 2/3 x F_a x S_s	0.671 g
Residential Seismic Design Category	D

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. According to the Oregon HazVu: Statewide Geohazards Viewer, the subject site is regionally characterized as not having a risk of soil liquefaction (DOGAMI:HazVu, 2020). Our explorations indicate on site soils are not susceptible to liquefaction.

Footing and Roof Drains

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 to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point and storm system well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

If the proposed structures will have a raised floor, and no concrete slab-on-grade floors in living spaces 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. If concrete slab-on-grade floors are used, perimeter footing drains should be installed as recommended below.

Where necessary, 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. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. 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 meet the street.

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. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. 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 attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology 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.

Beth K. Rapp, C.E.G.
Senior Engineering Geologist

James D. Imbrie, P.E., G.E.
Principal Geotechnical Engineer

Attachments: References
Checklist of Recommended Geotechnical Testing and Observation
Figure 1 – Vicinity Map
Figure 2 – Lidar Based Vicinity Map-With Mapped Landslides
Figure 3 – Site Grading Plan and Exploration Locations
Test Pit Logs (TP-1 – TP-7)

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CHECKLIST OF RECOMMENDED GEOTECHNICAL TESTING AND OBSERVATION

Item No.	Procedure	Timing	By Whom	Done
1	Preconstruction meeting	Prior to beginning site work	Contractor, Developer, Civil and Geotechnical Engineers	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Soil Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Soil Technician	
4	Compaction testing of engineered fill (90% of Modified Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Compaction testing of trench backfill (95% of Standard Proctor)	During backfilling, tested every 4 vertical feet for every 200 lineal feet	Soil Technician	
6	Street Subgrade Compaction (95% of Standard Proctor)	Prior to placing base course	Soil Technician	
7	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 lineal feet	Soil Technician	
8	AC Compaction (92% (bottom lift) / 92% (top lift) of Rice)	During paving, tested every 200 lineal feet	Soil Technician	
9	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

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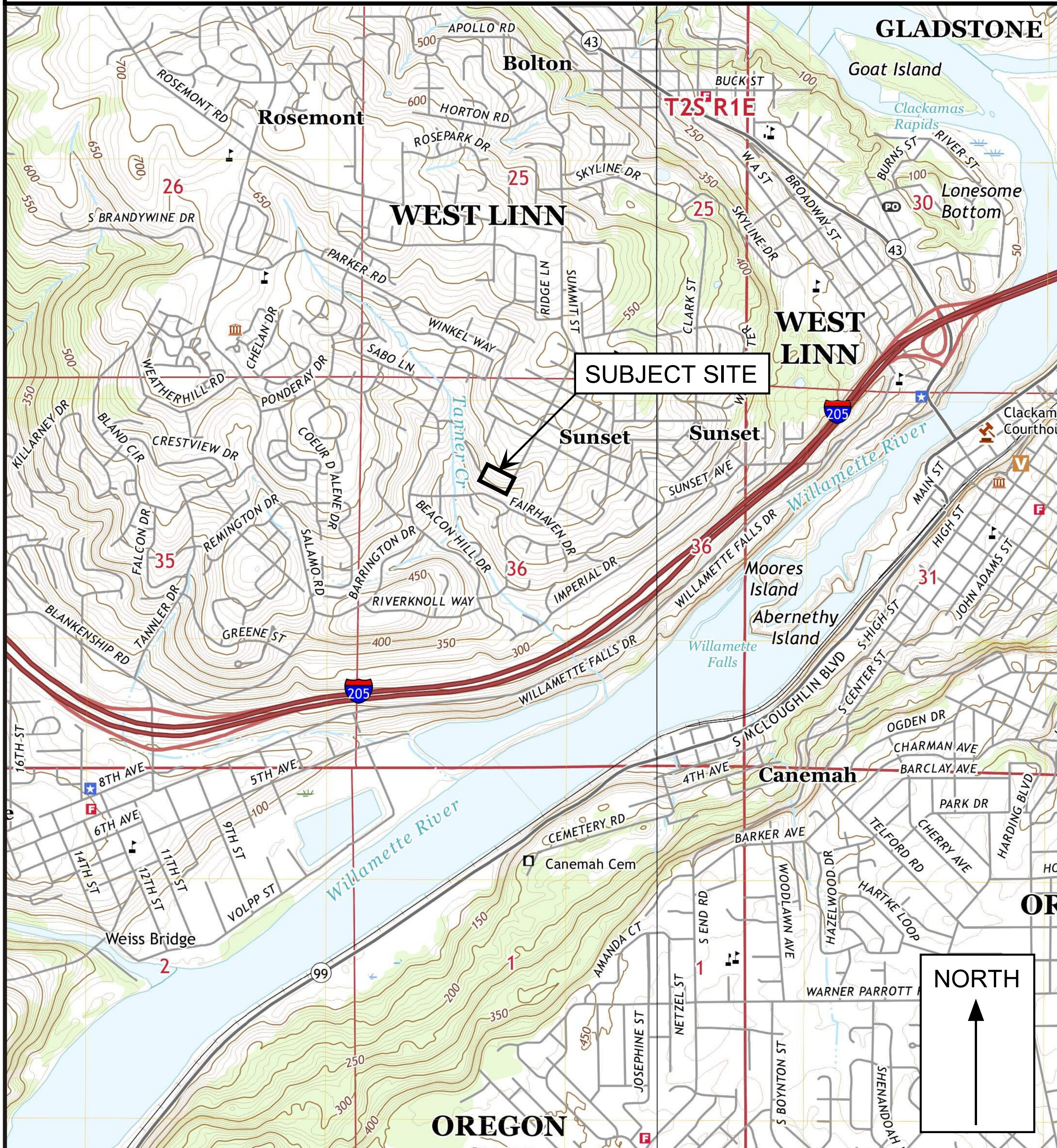
VICINITY MAP



Legend

Approximate Scale 1 in = 2,000 feet

Date: 10/27/2020
Drawn by: EKR

Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Canby, Oregon Quadrangle, 2020 and Oregon City, Oregon Quadrangle, 2020.

Project: Willow Ridge West Linn, Oregon	Project No. 19-5378	FIGURE 1

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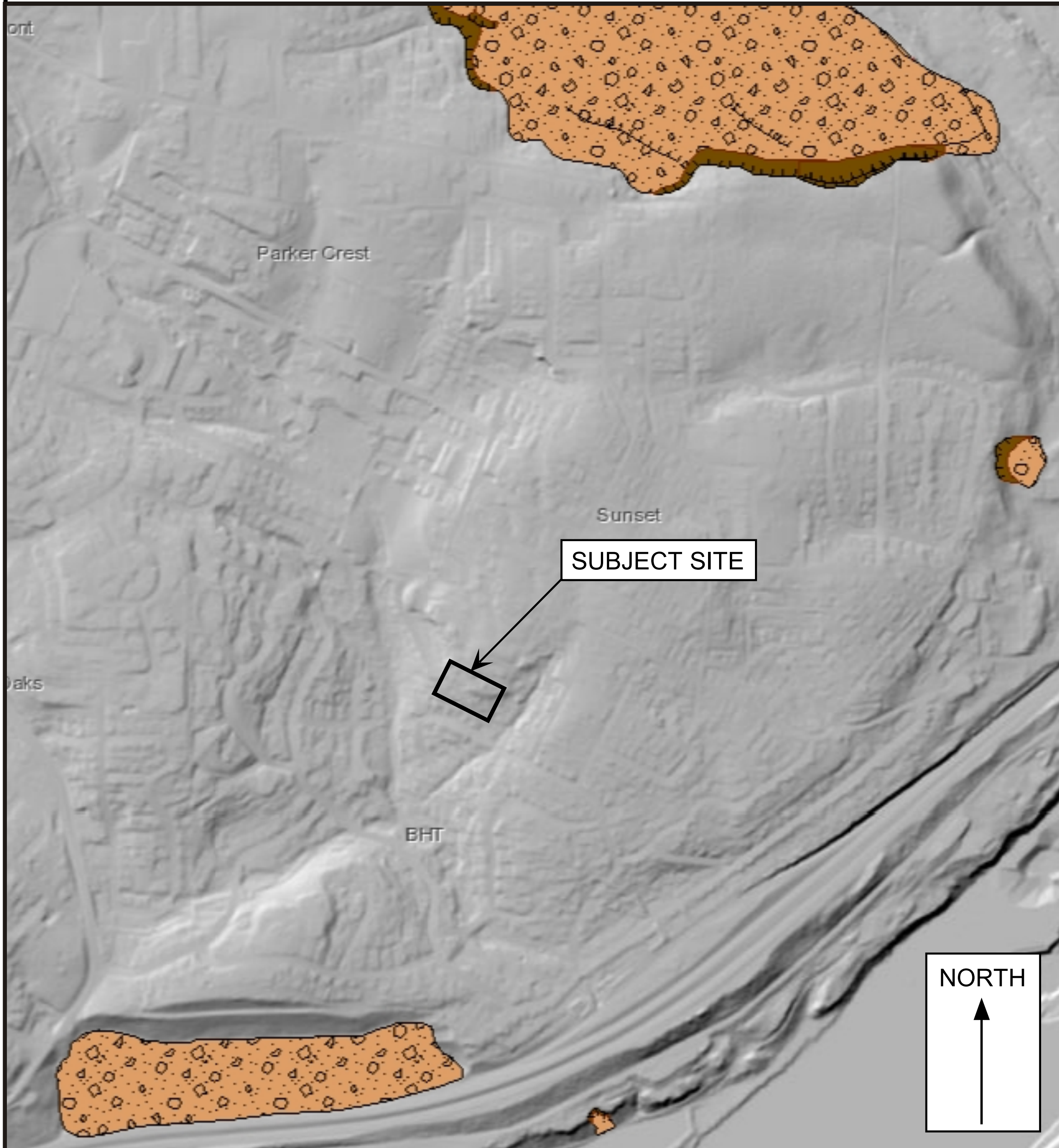
LIDAR BASED VICINITY MAP - WITH MAPPED LANDSLIDES



Legend

Approximate Scale 1 in = 1000 ft

Date: 10/26/2020
Drawn by: EKR

Base map: Oregon Department of Geology and Mineral Industries, 2020, Statewide Landslide Information Database for Oregon (SLIDO): https://gis.dogami.oregon.gov/slido/

Project: Willow Ridge West Linn, Oregon	Project No. 19-5378	FIGURE 2

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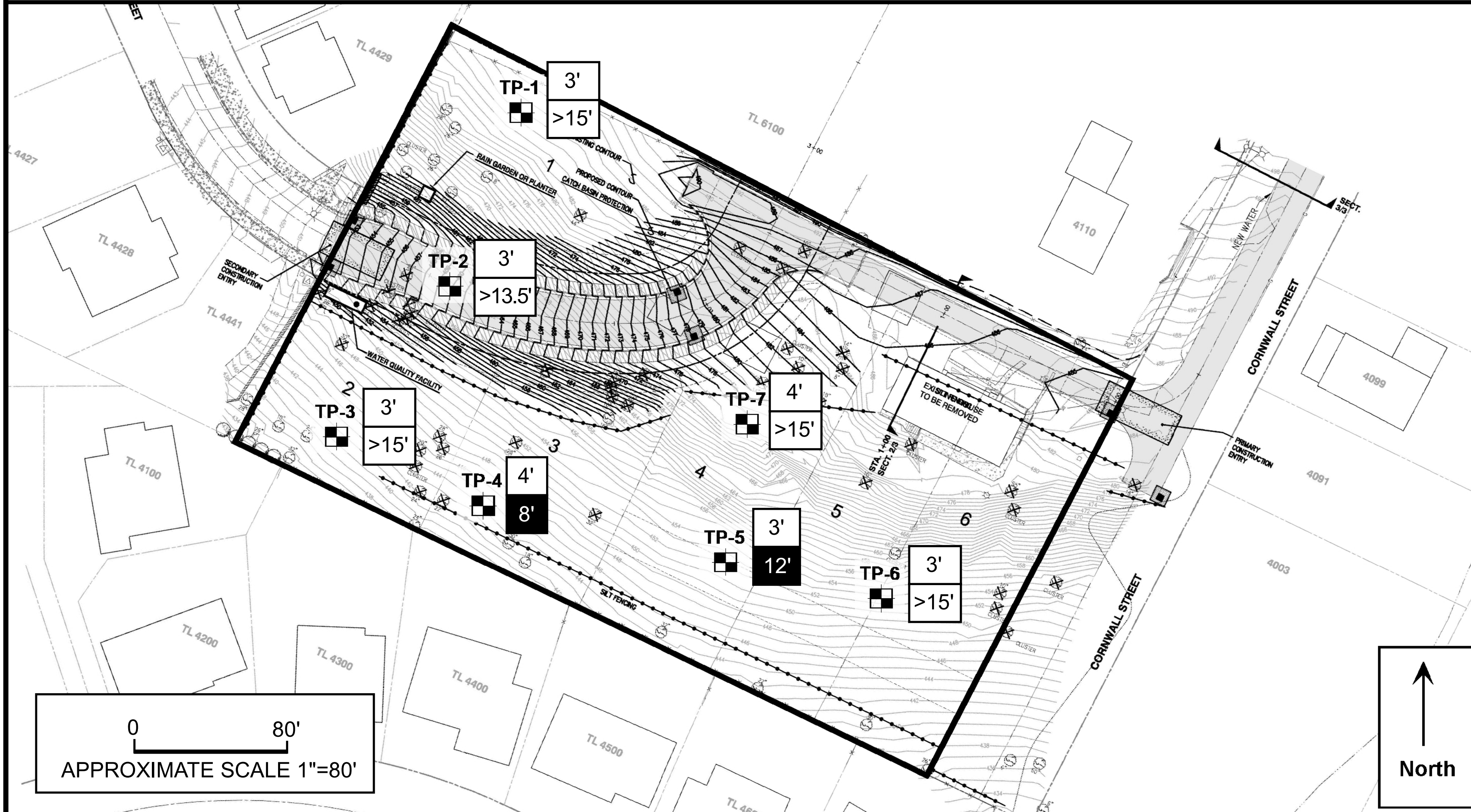
SITE GRADING PLAN AND EXPLORATION LOCATIONS



Legend

TP-1	Test Pit Designation and Approximate Location
3.5'	3.5' = Depth at Which Rock is First Encountered
15.5'	15.5' = Depth of Practical Refusal on Rock
	>19'= Depth is Beyond Maximum Exploration Depth
5'	5' = Depth of Practical Refusal on Boulder

Project: Willow Ridge West Linn, Oregon	Project No. 19-5378	Date: 10/27/2020 Drawn by: EKR	FIGURE 3

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TEST PIT LOG

Project: Willow Ridge West Linn, Oregon	Project No. 19-5378	Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1						Moderately to highly organic SILT (OL-ML), trace gravel, dark brown, fine roots throughout, loose, moist (Topsoil Horizon)
1	2.0					
1–3						Stiff to very stiff, silty CLAY (CL), with gravel, light reddish brown, gravel is subrounded to subangular gray basalt, trace black staining, moist (Residual Soil)
2	2.0					
2.5	4.5					
3	4.5					
3–9						Stiff to very stiff, very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace soft (R2) to medium hard (R3) boulders below 7 feet, black staining, moist (Columbia River Basalt Formation)
4	4.5					
9–15						Stiff, extremely soft (R0) to very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, dark gray, trace soft (R2) to medium hard (R3) boulders, black staining, moist to wet in seepage zone (Columbia River Basalt Formation)
15–17						Test Pit Terminated at 15 Feet. Note: Groundwater seepage encountered at 10 feet. Discharge visually estimated at ~1/2 gallon per minute.

LEGEND

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

Date Excavated: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 485'

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Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Willow Ridge, West Linn, Oregon	Project No. 19-5378	Test Pit No. TP-2

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.1	1.0					Moderately organic SILT (OL-ML), trace gravel (gray basalt), dark brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1.1–3	1.5 4.5				Seepage (3 ft)	Stiff to very stiff, silty CLAY (CL), trace gravel, light reddish brown, gravel is subrounded to subangular gray basalt, trace black staining, moist (Residual Soil)
3–13.5	4.5 4.5					Stiff to very stiff, very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace soft (R2) to medium hard (R3) boulders up to 12 inches diameter below 7 feet, vesicular, yellow secondary mineralization, black staining, moist (Columbia River Basalt Formation)

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

Test Pit Terminated at 13.5 Feet.

Note: Groundwater seepage encountered at 3 feet.
Discharge visually estimated at less than 1/10 gallon per minute.

LEGEND

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

Date Excavated: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 460'

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TEST PIT LOG

Project: Willow Ridge
West Linn, Oregon

Project No. 19-5378

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**
						Moderately organic SILT (OL-ML), trace gravel (gray basalt), dark brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1	1.0					
	4.0					Stiff to very stiff, silty CLAY (CL), trace gravel, light reddish brown, gravel is subrounded to subangular gray basalt, trace black staining, moist (Residual Soil)
2	4.5					
3	4.5					
4	4.5					
5						Stiff to very stiff, very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace soft (R2) to medium hard (R3) boulders up to 9 inches, vesicular, black staining, moist to wet in seepage zone (Columbia River Basalt Formation)
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						Test Pit Terminated at 15 Feet.
16						Note: Groundwater seepage encountered at 6 feet. Discharge visually estimated at less than 1/10 gallon per minute.
17						

LEGEND

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

Date Excavated: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 440'

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TEST PIT LOG

Project: Willow Ridge West Linn, Oregon	Project No. 19-5378	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
0–0.8						Moderately organic SILT (OL-ML), trace gravel (gray basalt), dark brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1	4.5					Stiff to very stiff, silty CLAY (CL) to clayey SILT (ML), trace gravel, light reddish brown, gravel is subrounded to subangular gray basalt, trace black staining, damp (Residual Soil)
2	4.5					
3	4.5					
4	4.5					Stiff to very stiff, very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace medium hard (R3) boulders up to 36 inches diameter below 7 feet, vesicular, black staining, moist (Columbia River Basalt Formation)
5						
6						
7						
8						Practical Refusal on Medium Hard (R3) Boulder at 8 Feet. Note: No groundwater or seepage encountered.
9						
10						
11						
12						
13						
14						
15						
16						
17						

LEGEND

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

Date Excavated: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 444'

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Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Willow Ridge, West Linn, Oregon	Project No. 19-5378	Test Pit No. TP-5

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.8						Moderately organic SILT (OL-ML), trace gravel (gray basalt), dark brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1	2.0				Seepage	Stiff to very stiff, silty CLAY (CL) to clayey SILT (ML), with gravel, light reddish brown, gravel is subrounded to subangular gray basalt, trace black staining, moist to wet in seepage zone (Residual Soil)
2	2.0					
3	4.5					Stiff to very stiff, very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace soft (R2) to medium hard (R3) boulders greater than 36 inches diameter below 8 feet, vesicular, yellow secondary mineralization, black staining, moist (Columbia River Basalt Formation)
4	4.5					
5						
6						
7						
8						
9						
10						
11						
12						Practical Refusal on Medium Hard (R3) Boulder at 12 Feet.
13						Note: Groundwater seepage encountered at 1.5 and 2 feet. Discharge visually estimated at approximately 1/2 gallon per minute.
14						
15						
16						
17						

LEGEND

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

Date Excavated: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 452'

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TEST PIT LOG

Project: Willow Ridge West Linn, Oregon	Project No. 19-5378	Test Pit No. **TP-6**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	**Material Description**
0–1.0						Moderately organic SILT (OL-ML), trace gravel (gray basalt), dark brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1	2.0					Stiff to very stiff, silty CLAY (CL) to clayey SILT (ML), trace gravel, light reddish brown, gravel is subrounded to subangular gray basalt, trace black staining, moist (Residual Soil)
2	4.0					
3	4.5				Seepage	Stiff to very stiff, very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace medium hard (R3) boulders up to 12 inches diameter below 4 feet, black staining, moist to wet in seepage zone (Columbia River Basalt Formation)
4	4.5					
5–15						
15						Test Pit Terminated at 15 Feet.
16–17						Note: Groundwater seepage encountered at 3.5 feet. Discharge visually estimated at less than 1/10 gallon per minute.

LEGEND

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

Date Excavated: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 452'

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TEST PIT LOG

Project: Willow Ridge, West Linn, Oregon | Project No. 19-5378 | Test Pit No. **TP-7**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.0						Moderately organic SILT (OL-ML), trace gravel (gray basalt), dark brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1	2.0					
1.0–3.9						Stiff to very stiff, silty CLAY (CL) to clayey SILT (ML), trace gravel, light reddish brown, gravel is subrounded to subangular gray basalt up to 24 inches diameter, trace black staining, moist to wet in seepage zone (Residual Soil)
2	2.0					
3	3.0				Seepage	
4	4.5					
3.9–15						Stiff to very stiff, extremely soft (R0) to very soft (R1), weathered BASALT, trace reddish brown silty clay matrix, red brown to gray, trace soft (R2) to medium hard (R3) boulders up to 18 inches diameter below 14 feet, vesicular, yellow secondary mineralization, black staining, moist (Columbia River Basalt Formation)

Test Pit Terminated at 15 Feet.

Note: Groundwater seepage encountered at 4 feet.
Discharge visually estimated at 1/10 gallon per minute.

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: 10/14/2020
Logged By: B. Rapp
Surface Elevation: 476'

October 28, 2020

REGISTERED PROFESSIONAL
OREGON
ROGER N. SMITH
No. G838
GEOLOGIST
Expires 8-31-21

Icon Construction & Development, LLC
Darren Gusdorf, General Manager
1969 Willamette Falls Dr., Suite 260
West Linn, OR 97068

TECHNICAL MEMORADUM

GROUNDWATER CHARACTERISTICS
WILLOW RIDGE PROJECT SITE
4096 Cornwall St., West Linn, OR

The following memorandum was developed in response to public comments regarding proposed development of a 2.17-acre lot (TL 6300) at 4096 Cornwall Street, West Linn (see Figure 1). This technical memorandum specifically addresses groundwater conditions observed and documented on the project property.

Objective

The objective of this document is to expand the understanding of groundwater movement onto and through the Willow Ridge site. This is a more detailed review than earlier testimony and is based on observations made and data collected recently from the Willow Ridge parcel. This new information is used to determine how groundwater will best be managed during the development of the site.

Approach

On October 14, 2020 seven test pits were excavated by Icon Construction across the site (see Figure 2). The test pits, most to a depth of 15 feet, were logged by a GeoPacific senior engineering geologist (Beth Rapp). Soil and groundwater conditions were also assessed by Roger Smith, a senior geologist/hydrogeologist from RNSA, Inc. while the pits were open. Notes and logs collected were combined with an earlier geotechnical investigation at the site by Carlson Geotechnical (January 2016) when seven pits between 8 and 10 feet deep were also excavated across the project site.

From this data a hydrogeologic cross-section was developed and is attached (see Figure 3). This cross-section illustrates our present understanding of groundwater movement through the site.

Willow Ridge Soil and Hydrogeology

A hydrogeologic description of shallow soil is summarized as follows:

0-1 feet (depths varies) comprised of organic **Silt**. Considered to have low to moderate vertical and lateral infiltration capacity, depending on the slope and vegetation cover.

1-4 feet (depths varies) **Clay**, documented with gravels and sand in some areas. Considered to have a low intrinsic permeability, however, secondary permeability from root and animal borrows increases capacity for groundwater movement. Seeps were observed from this unit into all test pits on the property except in TP-4 which had no groundwater and TP-1 which had a deeper groundwater seep (discussed below).
4-15 feet (depths varies) **Weathered Basalt**, with varying degrees of basalt decomposition. Clay being a major decomposition product, permeability is considered low to very low. This unit was found across the property in every test pit. No groundwater seeps were found in this zone with the exception of TP-1 where a seep was noted at 10 feet.

Local Infiltration and Groundwater Flow
The NRCS (National Resources Conservation Services) classified the soil overlying the project site Saum **silt loam**, with subcategories related to slope grade. The NRCS categorizes the Saum as a soil Group C. This is a soil identified as having 'slow infiltration when thoroughly wet and having an underlying unit impeding infiltration'.

Although no infiltration rates were measured or known for the project site, infiltration of precipitation through Columbia River Basalts has been reported to be less than 6% of annual precipitation (NGS, 1997). The slope of the property (average 22% grade) and the shallow top soil overlying a high clay content weathered basalt would likely lower this estimate.

The expected low infiltration rates were reflected in the observations of only minor seeps in the shallow soils of the site test pits along with dry soil conditions below a depth of 3 to 4 feet. These conditions were observed even though 1.5 inches of rainfall had occurred in the four days preceding site excavations (Portland Hydra Station #4, Sylvania CC). An exception to the observed shallow infiltration seeps was test pit TP-1 located near the upper portion of the site. This pit had no water seepage in the shallow top soil or root zone but did have seepage beginning at approximately 10 feet emerging from a soft to extremely soft weathered basalt zone. The other exception was test pit TP-4 located near the bottom of the project property. This pit had no groundwater seeps in shallow soils or at deeper levels where excavation met refusal by rock at 8 feet.

Test pit groundwater seepage observed in five of the pits located on the lower (and steeper) portion of the site all originated from the shallow organic silt loam and/or clay unit in the upper 3 to 4 feet of the soil profile. This zone was observed to contain roots and animal burrows. The only groundwater movement in this area of the property was through this shallow 0 to 4-foot deep zone. From 4 to 15 feet deep, no seeps were observed through the zone logged as weathered basalt.

Test pit TP-1was excavated in the upper northwest quadrant of the property (see Figure 2). Groundwater conditions in this pit were different from the other pit seeps. Groundwater in TP-1 entered the excavation at a depth of 10+ feet and at a higher rate. Visual estimates and water volume accumulation calculations set the infiltration rate at between 0.5 and 1.0 gpm. No groundwater seeped from the upper 4 feet of the soil profile. The apparent higher permeability in the weathered basalt below 10 feet in the area of the test pit is related to the reported 'very soft to extremely soft' nature of the weathered basalt. The soft condition of the weathered basalt was noted in TP-7 (approximately 200 feet to the southeast) but without any groundwater seepage except in the upper 3 feet (see hydrogeologic cross-section in Figure 3). Test pit TP-2 (directly down gradient of TP-1 approximately 100 feet), also had a 'soft' zone but also no groundwater seep in that zone. Seepage in both these adjacent test pits occurred only in the upper 3 feet of the soil profile. Migration of groundwater from TP-1 is interpreted as flowing downgradient toward the lower and steeper portion of the site on top of the low permeable high-clay content soft unit of weathered basalt. The groundwater appears to be moving laterally rather than vertically until it intercepts the permeable top soil and root permeable zone 3 to 4 feet thick then flowing down-slope within this shallow zone (see dotted lines on the cross-section). The transition of the deeper groundwater in the upper portion of the site to the shallow 3-4 feet deep permeable zone is considered to occur near the break in the slope of the property. The groundwater then flows into this shallow zone and down through the property. This is illustrated in the cross section of Figure 3.

Northwest, upgradient and off the project property, there is an open undeveloped area with a moderate slope (see Figure 1). This is considered to be a very localized portion of the full recharge area but is considered to directly contribute to groundwater observed in TP-1. This local recharge area is covered with vegetation which would assist infiltration resulting in the off-site surface water enter the groundwater found at 10 feet in TP-1.

Therefore, the proposed model of groundwater flow through the project property is from a recharge area upgradient of the site, along a moderate slope onto the property at an approximate depth of 10 feet, then along the top of the low permeable clay-rich weathered basalt to the more permeable 3 to 4-foot top soil and root and animal affected zone. Groundwater then flows down across the property through the zone in the upper portion of the soil column. No emerging groundwater at depth in the test pit was observed during the test pit work.

Conclusion

Most of the groundwater at Willow Ridge appears to originate from recharge areas upgradient (north and northwest) off the project site rather than from direct infiltration on the property. Test pits indicate that groundwater flows onto the property above a soft clay-rich weathered basalt unit. The weathered basalt was logged to a depth of 15 feet in all pits except TP-4 where rock stopped excavation. Groundwater flow occurs along a moderate slope and at a depth of approximately 10 feet in the upper part of the property and intercepts a shallow (3 to 4 foot deep) permeable zone of top soil (approximately 1-foot thick and a 2 to 3-foot thick layer of clay with roots and animal burrows) where it migrates from the upper portion of the property down across the lower portion of the property. None of the six pits across the lower portion of the property were noted as having groundwater seeps below 4 feet.

Shallow groundwater is present on the site; however, it appears to be confined to flowing through the property on top of the underlying clay-rich weathered basalt. No springs were seen on the property during site work and there does not appear to be any groundwater flowing horizontally through an underlying basalt interflow zone as suggested by William House's cross-section (Public Testimony October 7, 2020).

Managing the shallow groundwater on the project site will need to be included in the civil engineering design of the development and is not part of this memorandum. However, intercepting groundwater along the upper area boundary of the property would reduce migration across the lower portion.

Construction of impermeable surfaces such as driveways and roofs during development will increase area runoff rates. Capturing this excess for disposal to appropriate outlets will reduce risk of increased groundwater or surface water runoff rates to properties down slope. Surface disposal or infiltration of captured water is not recommended.

Uncertainties

Earlier public testimony by W. House proposed a 20-foot thick zone above the Gingko basalt unit and beneath the Sandy Hollow unit of the Frenchman Springs member of Wanapum Basalt would deliver groundwater to the center portion of the Willow Ridge project site. Based on our recent site investigation, it is our understanding the primary groundwater flow in the project area is at or near the top of the weathered basalt and not through an interflow zone of the two basalt units.

No seepage of groundwater below 4 feet in test pits was observed in the low portion of the property during the pit excavation work on October 14th, 2020. These pits extended to a depth of 15 feet across the property. It is unclear whether groundwater, to any significant flow rate, exists at any depth below 15 feet.

References

1. Carlson Geotechnical, 1-7-2016, CGT Project Number G1504283, Report of Geotechnical Investigation, Cornwall Street Subdivision, 4096 Cornwall Street, West Linn Oregon.
2. GeoPacific Engineers, Inc, 10-15-20, Project 19-5378, Willow Ridge Site Grading Plan and Exploration Locations, with 7 geotechnical test pit logs.
3. Northwest Geological Services, Inc (NGS), 5-1997, Geologic and Hydrogeologic Study of the Residential Acreage-Zoned Areas of Marion County Underlain by the Columbia River Basalt and Older Rocks

Attachments

Figure 1 - Topographic Map with ¼ Mile Study Area
Figure 2 - GeoPacific Engineers Site Grading Plan,
Figure 3 - NW- SE Hydrogeologic Cross-Section A-A'

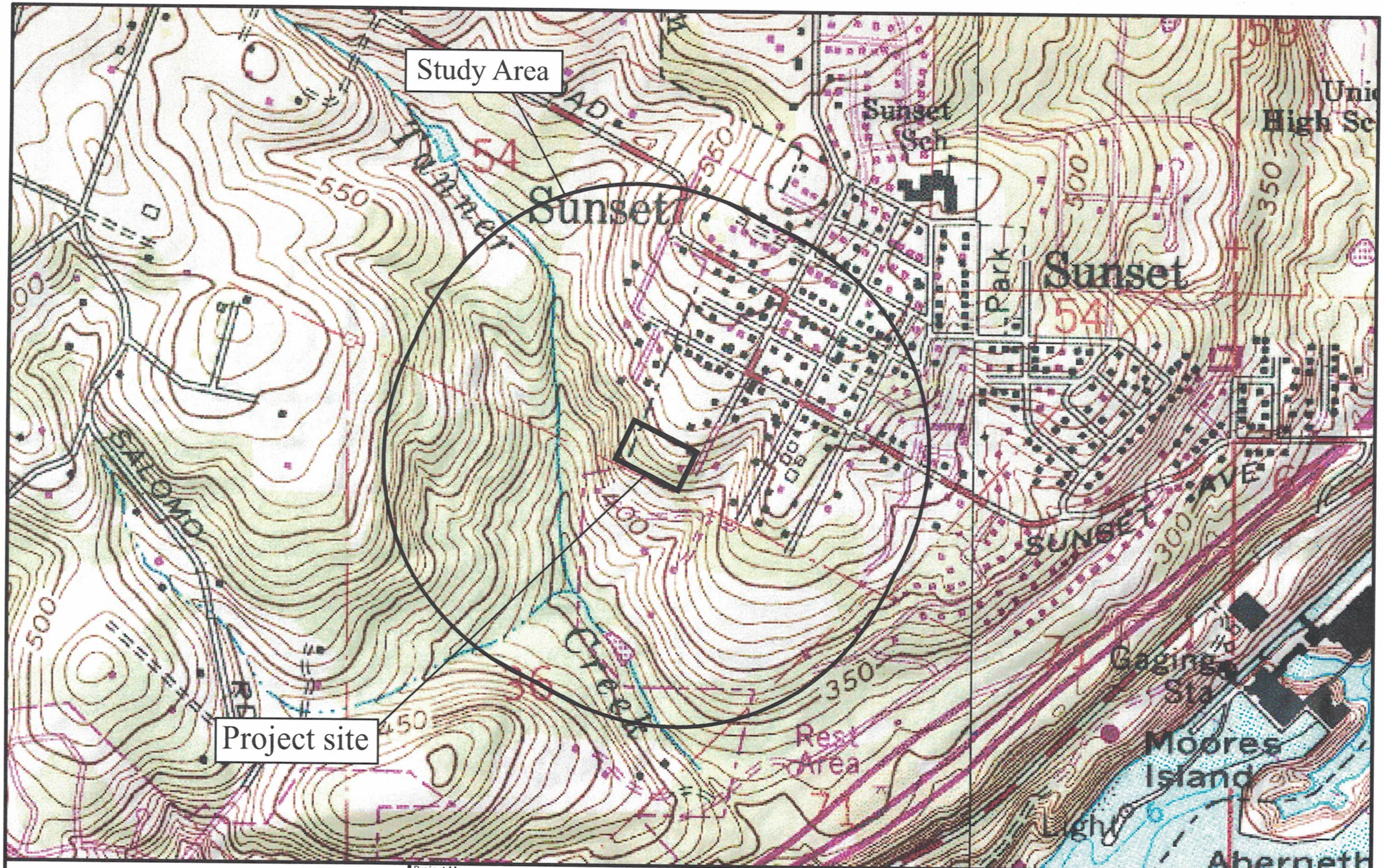
Bio -RNSA, Inc Principal

ROGER N. SMITH
Principal, RNSA, Inc. (since 1986)
EDUCATION
MSc Geology, 1980
BSc Geological Engineering, 1971

REGISTRATIONS/CERTIFICATIONS
Hydrogeologist - Washington
Geologist - Washington
Geologist - Oregon, Idaho
Water Rights Examiner, Oregon
Cert. Monitoring Well Constructor, Oregon
Hazardous Materials Supervisor (HAZMAT)

PROFESSIONAL EXPERIENCE
For the past 38 years Mr. Smith has been practicing in the geological profession. He has worked on projects in Australia, Africa and the United States. Roger N. Smith Associates, Inc. (RNSA) groundwater and environmental consulting firm was founded 34 years ago in Portland. Mr. Smith's professional work over his career has included extensive experience in:
1.) Groundwater resource and sustainability studies
2.) Environmental investigations and remediation
3.) Geothermal resource development
4.) Mineral resource development (2nd generation mining professional in the Coeur d'Alene silver mining area).
1986-2020. In 1986 Mr. Smith founded RNSA, Inc. a groundwater and environmental consulting firms. His firm has completed more than 1,100 projects in the northwest related to aquifer studies, geothermal resources, slope stability as well as general geologic studies completed for private, State and Federal entities. Mr. Smith has completed more than 50 hydrogeologic assessments (mostly in Clackamas County) since 2010.

Figure 1 - Topographic Map with 1/4 Mile Study Area



Project Manager:	Roger Smith
Adapted From:	TOPOZONE by Jimmy Holcomb
Project Number:	1166-20
Date Last Modified:	10/27/2020

RNSA, INC.
Groundwater and Environmental Consultants

Topographic Map with 1/4 Mile Study Area

Project Address: 4096 Cornwall St., West Linn OR, Taxlot: 6300

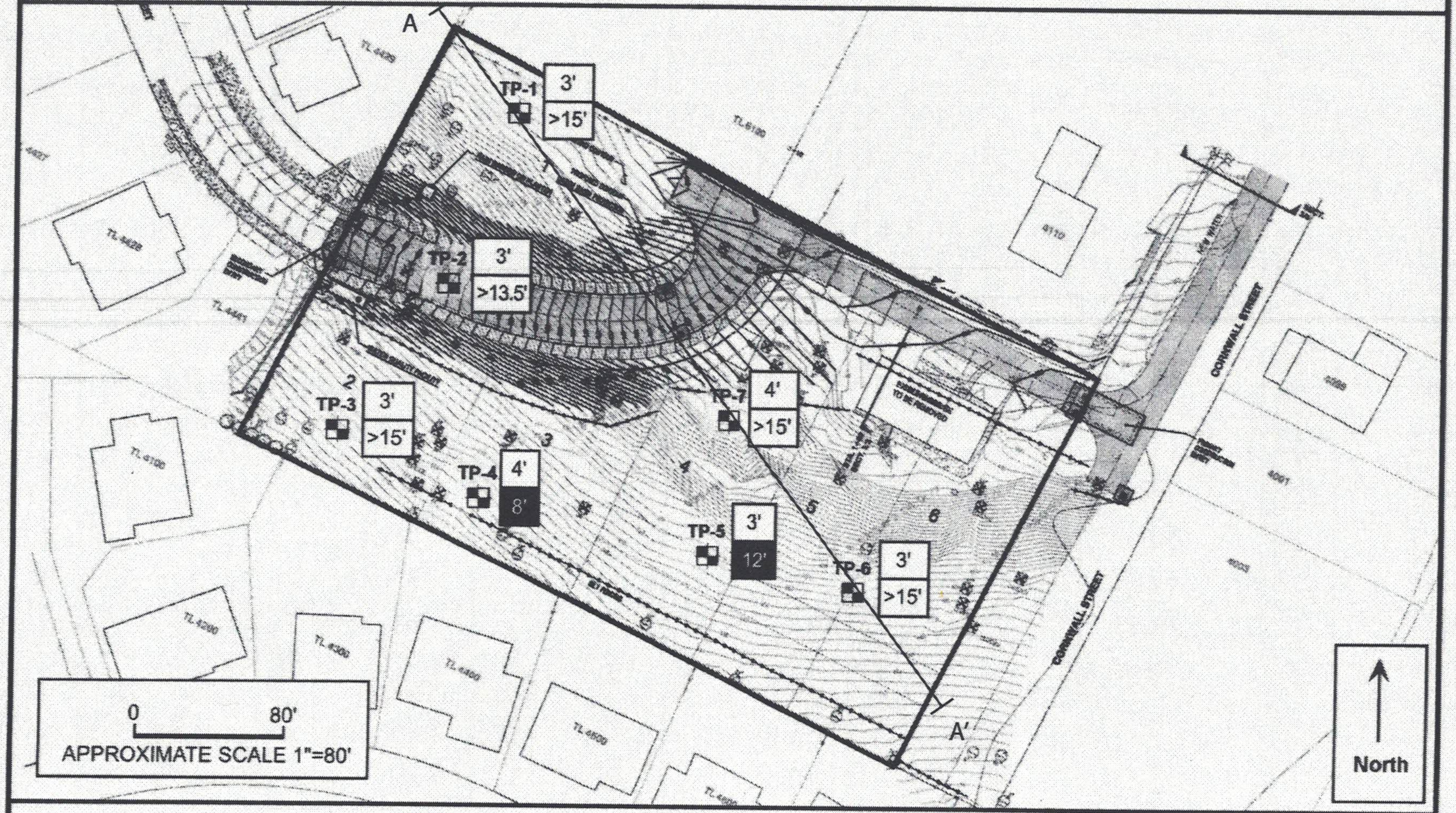
Approximately 1/4 Mile

NORTH

Figure 1

Figure 2 - GeoPacific Engineers Site Grading Plan

SITE GRADING PLAN AND EXPLORATION LOCATIONS WITH CROSS-SECTION TRACE A -A'



Legend

TP-1 Test Pit Designation and Approximate Location

3.5' = Depth at Which Rock is First Encountered

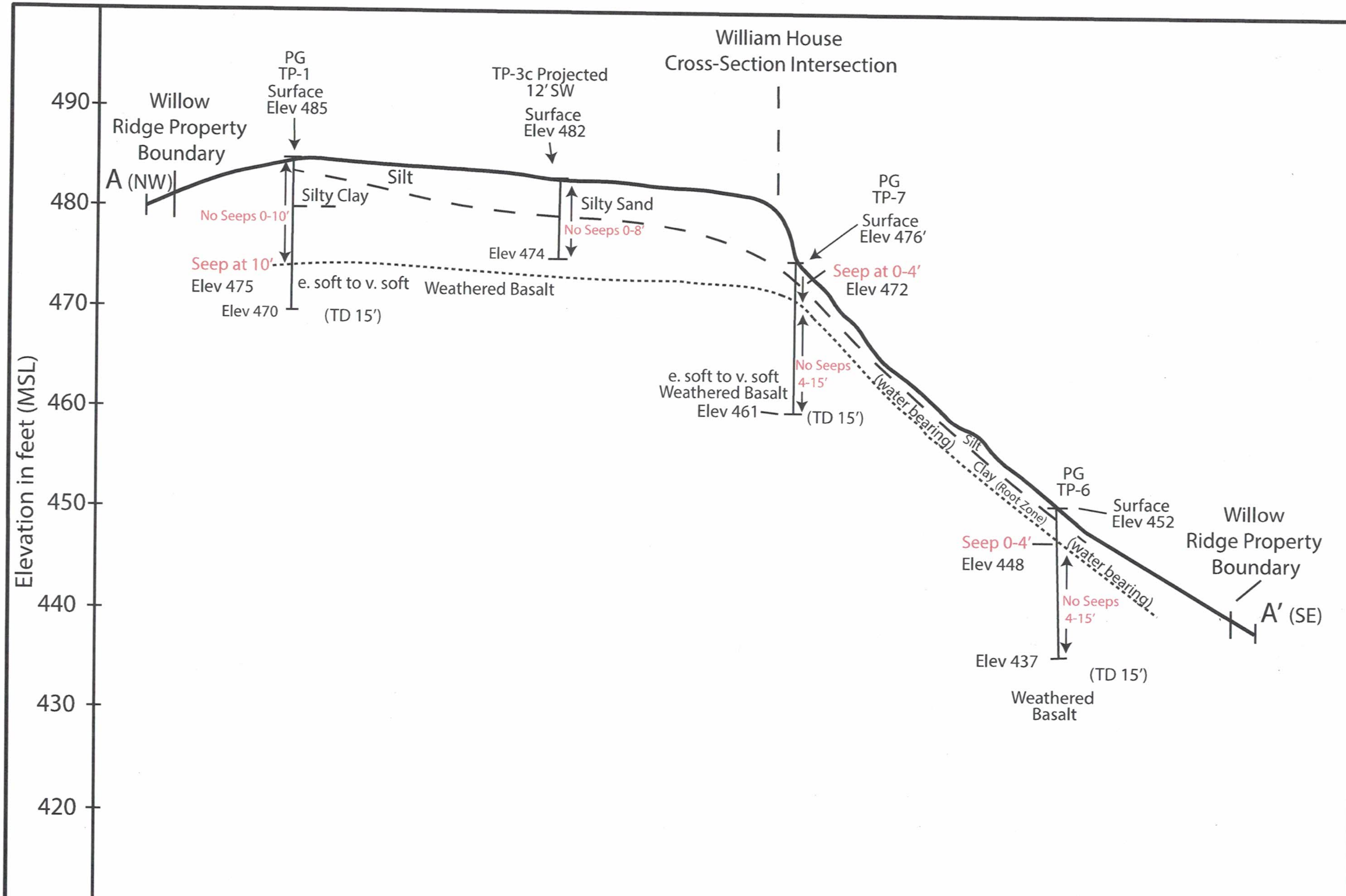
15.5' = Depth of Practical Refusal on Rock

>19'= Depth is Beyond Maximum Exploration Depth

5' = Depth of Practical Refusal on Boulder

Project: Willow Ridge West Linn, Oregon	Adapted from GeoPacific	RNSA Project:1166-20	Date: 10/27/20 Adapted by : JRH	Figure 2

Figure 3 - NW - SE Hydrogeologic Cross-Section A-A'



RNSA, INC. Groundwater and Environmental Consultants	**Legend** TP - 3c Carlton Geotech Test Pit; PG-TP - 1 GeoPac. Test Pit; TD: Total Depth (ft.); Horizontal Scale: 50 ft (approx.) Vertical Scale 1" = 12.5'; H/V = 4	Project Manager: Roger Smith Digitized By: Jimmy Holcomb Project Number: 1166-20 Date Last Modified: 10/28/2020	Willow Ridge, West Linn NW-SE Hydrogeologic Cross Section A-A' Project Address: 4096 Cornwall St., West Linn OR Taxlot: 6300 Figure 3

October 26, 2020

Rick Givens
Planning Consultant
18680 Sunblaze Dr.
Oregon City, Oregon 97045

Mr. Charles Mathews, Vice Chairman
City of West Linn Planning Commission
22500 Salamo Rd.
West Linn, OR 97068

RE: SUB-20-01, Willow Ridge

Dear Mr. Matthews:

At the initial public hearing on the Willow Ridge subdivision application questions were raised by yourself and other commissioners as to whether the proposed gated emergency vehicle access is permitted by the West Linn Community Development Code (CDC). As you will recall, the Tentative Plan provides for Lots 5 and 6 to be accessed from Landis Street via a shared flag lot driveway. It is also proposed that the driveway would be extended through to Cornwall Street in order to provide for an emergency vehicle connection. The proposed gate would only serve to prevent traffic other than emergency vehicles or City maintenance crews utilizing the connection to Cornwall Street. Also, please note that the use of locking bollards would be a design option that would be preferable to the applicant instead of a gate. In reviewing the CDC, I find two references to gated accesses:

48.030.I. Gated accessways to residential development other than a single-family home are prohibited.

85.200.A.20 20. Gated streets. Gated streets are prohibited in all residential areas on both public and private streets. A driveway to an individual home may be gated.

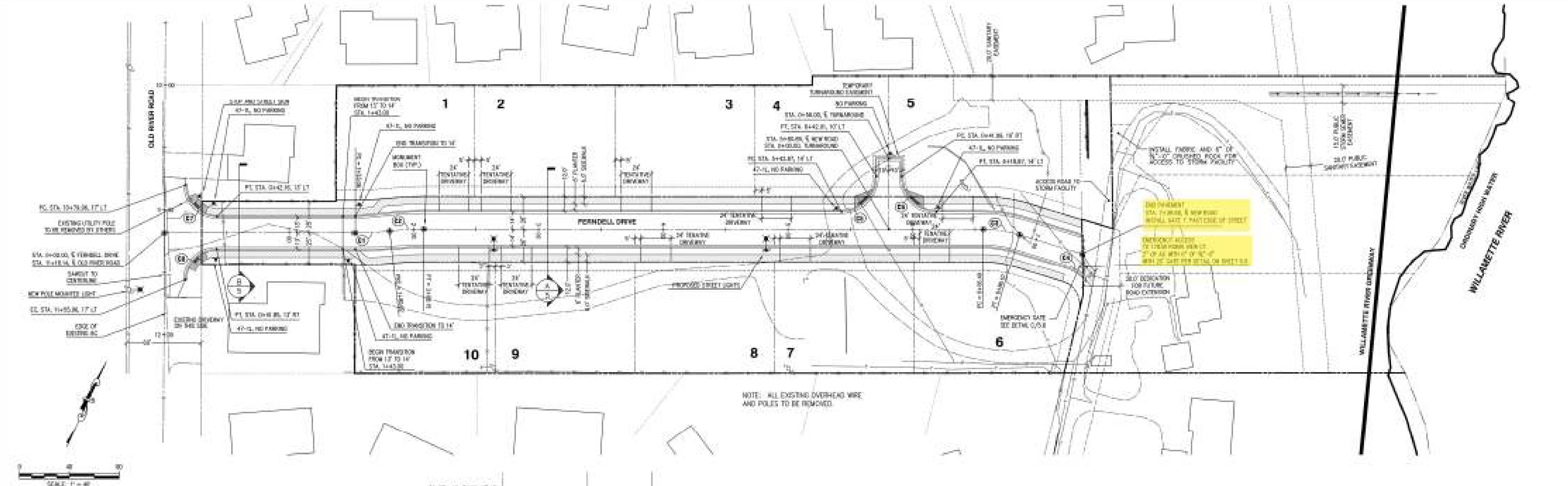
The gated portion of the accessway is located in an easement across the shared flag strip of Lots 5 and 6. It is not located in a public or private street. The gated portion of the drive does not serve to access any lots within the development; full access to all lots will be available from Landis Street. The sole purpose of the gated access is to provide better emergency and maintenance vehicle access to the proposed development and to the existing Landis Street neighborhood in order to improve public safety. There is nothing in either of the above code provisions that prohibits gated access for emergency vehicle use. This has been allowed elsewhere in West Linn. The Fernvilla Estates (formerly Ferndell Estates) subdivision off of Old River Drive provides for a gated emergency vehicle access between Fernvilla Drive and Robin View Court, as shown on the attached construction plans. We ask that Willow Ridge be approved with a gated, or preferably bollard, design to ensure good emergency and maintenance access but avoid cut-through traffic.

Sincerely yours,

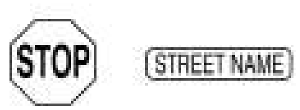
Rick Givens

Rick Givens

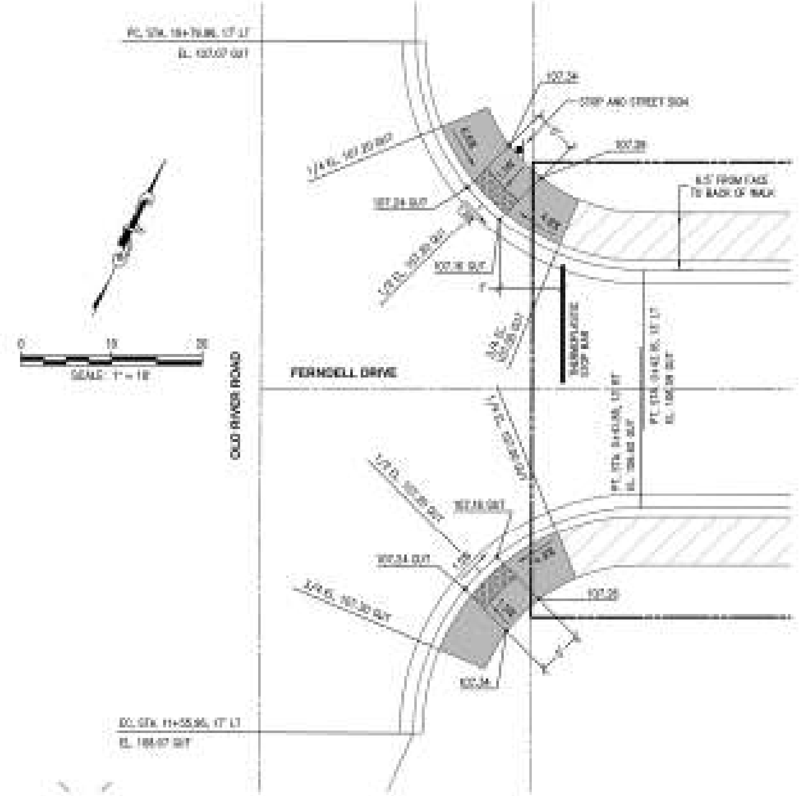
Cc: Mark Handris, Mike Robinson



NOTE: ALL EXISTING OVERHEAD WIRE AND POLES TO BE REMOVED.



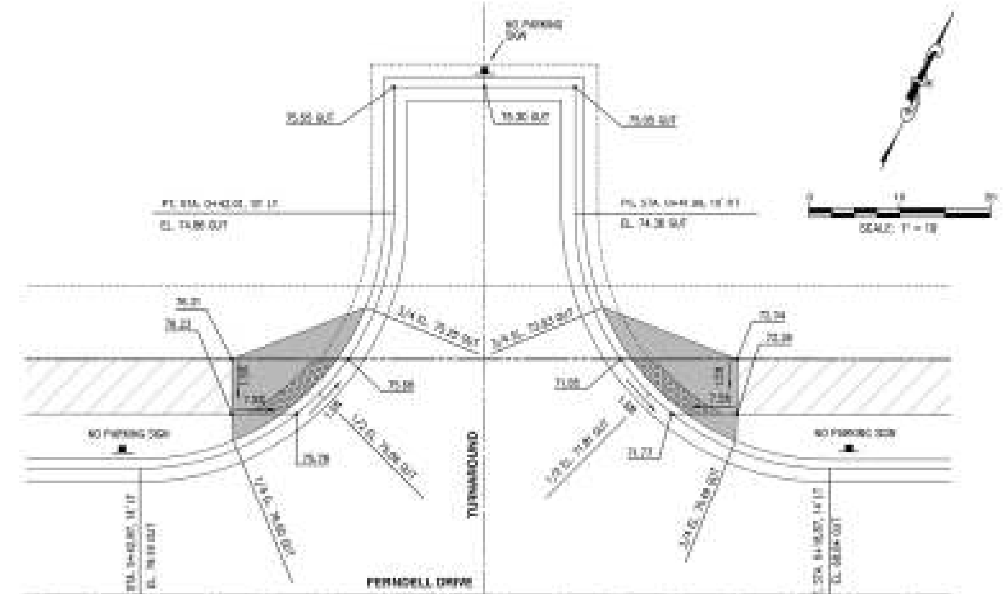
SIGNAGE PER MANUAL OF UNIFORM TRAFFIC CONTROL DEVICES (MUTCD) CITY OF WEST LINN STANDARDS.



FERNDELL DRIVE &
OLD RIVER ROAD
INTERSECTION DETAIL

1
4.0

SCALE: 1" = 10'



FERNDELL DRIVE &
TURNAROUND
INTERSECTION DETAIL

1
4.0

SCALE: 1" = 10'

CURVE TABLE						
CURVE	RADIUS	LENGTH	TANGENT	CHORD	BEARING	DELTA
C1	250.00	27.58	13.80	27.55	N61°59'23"E	6°19'02"
C2	250.00	27.53	13.78	27.51	S61°59'38"W	6°18'32"
C3	100.00	28.62	14.80	28.51	S73°17'30"W	16°58'13"
C4	30.00	15.11	7.76	14.97	N84°21'45"W	27°53'18"
C5	28.00	43.99	28.01	39.59	N50°07'30"E	90°01'09"
C6	28.00	43.97	27.99	39.58	S89°52'10"E	89°58'51"
C7	25.00	29.80	23.51	20.43	S88°44'52"E	68°14'28"
C8	25.00	29.16	24.96	35.30	S20°18'39"W	66°45'22"

2018-250

STREET PLANS

DESIGNED:	BDG			
DRAWN:	BJS			
SCALE:	1" = 40'	06/06/2019	2	REVISED PER CITY REVIEW NO. 2
DATE:	June, 2017	03/28/2019	1	REVISED PER CITY REVIEW NO. 1
FILE:	Ferndell Estates Prelim3	DATE	NO.	REVISION

Theta, llc
ENGINEERING - SURVEYING - PLANNING
PO Box 1345
Lake Oswego, Oregon 97035
503/481-8822
email: thetaeng@comcast.net

Hymark Custom Homes
18811 Trillium Drive
West Linn, Oregon 97068

Ferndell Estates
3300 & 3350 Riverside Court
West Linn, Oregon
Legal: 21E14AD00990

SHEET:
4.0

ARD ENGINEERING
21370 SW Langer Farms Pkwy
Suite 142, Sherwood, OR 97140

REGISTERED PROFESSIONAL ENGINEER
54983PE
OREGON
JULY 10, 2002
MICHAEL T. ARD
EXPIRES: 12/31/2021

Technical Memorandum

To: Mark Handris, Icon Construction

From: Michael Ard, PE

Date: October 28, 2020

Re: Willow Ridge Traffic Impact Analysis - Update

This memorandum is written to provide additional information and clarification regarding the potential traffic impacts associated with the proposed Willow Ridge residential development in West Linn, Oregon.

A prior memo dated June 25, 2020 described a "tentative" development plan and an "alternative" plan. Since that time, the original "alternative" plan became the preferred plan option, resulting in a discrepancy between the traffic analysis and other material in the project application. This update modifies the descriptions of the two options to be consistent with the language in the other application materials. Additionally, it addresses a minor change wherein if a full street connection is not made between Landis Street and Cornwall Street, all lots would take access via Landis Street.

Project Description

The proposed Willow Ridge Subdivision will include six lots for single-family homes located on a 2.17-acre site between the existing eastern terminus of Landis Street and the southern terminus of Cornwall Street. Two potential street connections have been proposed.

Under the tentative site plan, Landis Street would be extended into the site to a hammerhead turn-around, effectively limiting public vehicular access through the site. Lots 1-4 would take individual access to Landis Street, while a flag-lot driveway would provide access for lots 5 and 6. This driveway would provide a continuous paved surface between the hammerhead at the end of Landis Street and the existing south end of Cornwall Street. The driveway would also serve as an emergency vehicle access easement, thereby improving connectivity for emergency vehicles.

Although the tentative plan without a full street connection is preferred by local residents, city staff have expressed the desire for a full public street connection through the site between the existing east end of Landis Street and the existing southern end of Cornwall Street. Accordingly, a second "Alternative Plan" was developed for the site. Under this plan, Landis Street would be extended through the site to connect to the south end of Cornwall Street, with the entire connection accessible to the public.

This analysis will include examination of both the tentative and alternative site plans, along with relevant information regarding traffic volumes, adequacy of street widths, and the requirements of the City of West Linn's Public Works Design Standards.

Existing Conditions

Under existing conditions, Landis Street is a dead-end road serving 20 single-family homes. The street has a paved width of 28 feet, with closely spaced driveways along both sides of the roadway. Continuous curb-tight sidewalks are in place along the west side of the roadway and connecting to existing sidewalks along the south side of Stonegate Lane. Partial sidewalks are also in place along the east side of Landis Street, but are not available toward the north end of the street. Existing partial sidewalks are also in place along the north side of Stonegate Lane.

The width and design of Landis Street is typical of a queuing street, which may not fully accommodate simultaneous two-way travel at all points. Instead, where vehicles are parked along the street drivers may need to pull to one side to allow opposing traffic to pass. This limits the effective capacity of the street to approximately 1,000 vehicles per day.

Cornwall Street is also a dead-end road serving 10 existing homes, including the existing home on the subject property. The street has a paved width of 15 to 20 feet, with no sidewalks on either side of the roadway. The narrower cross-section of Cornwall Street is even more restrictive that Landis Street. Although it can accommodate two-way travel drivers may need to carefully select where to pass to ensure adequate road width is available. Additionally, since there are no sidewalks provided pedestrians and cyclists must share the limited road width with motor vehicles. Since Cornwell Street is a relatively short dead-end roadway (approximately 600 feet) serving a very limited number of homes, travel speeds and traffic volumes would be expected to be very low, allowing pedestrians to safely share the roadway with motor vehicle traffic.

Trip Generation

The subject property is currently developed with one single-family home. Under the proposed plan, a total of 6 homes will be provided within the project site, resulting in an overall net increase of five homes. In order to determine the increase in traffic attributable to the proposed development, a trip generation analysis was prepared using data from the Institute of Transportation Engineer's Trip Generation Manual, 10^{th} Edition. The data used was for land use code 210, *Single Family Detached Housing*, and is based on the number of dwelling units.

Based on the analysis, the proposed development is projected to result in a net increase of 3 trips during the morning peak hour, 5 trips during the evening peak hour, and 46 average daily trips. A summary of the trip generation calculations is provided in Table 1 on the following page. Detailed trip generation worksheets are also provided in the attached technical appendix.

Table 1 - Trip Generation Calculation Summary

	Morning Peak Hour			Evening Peak Hour			Daily Trips		
	In	Out	Total	In	Out	Total	In	Out	Total
6 Single Family Homes	1	3	4	4	2	6	28	28	56
- 1 Existing Home	0	-1	-1	-1	0	-1	-5	-5	-10
Net New Site Trips	**1**	**2**	**3**	**3**	**2**	**5**	**23**	**23**	**46**

Based on the trip generation analysis, the traffic impacts attributable to the proposed homes will be minimal. Per the City of West Linn Public Works Design Standards Section 5.0014, a Traffic Impact Analysis will generally be required when a proposed development will generate 1,000 vehicle trips per weekday or more, or when a development's location, proposed site plan, and traffic characteristics could affect traffic safety, street capacity, or known traffic problems or deficiencies in a development's study area. For tentative subdivision projects, CDC Section 85.170(B)(2) further requires a Traffic Impact Analysis where an increase in site traffic volume generation by 250 average daily trips or more is projected.

The proposed development is projected to result in less than 5 percent of the traffic volume that would trigger the need for a Traffic Impact Analysis per the city's Public Works Design Standards Section 5.0014, and just 20 percent of the traffic volume that would trigger the need for a Traffic Impact Analysis per CDC 85.170(B)(2). However, since a potential street connection could result in other transportation safety and operations impacts additional analysis is appropriate to determine the extent and nature of any traffic operations and safety impacts. For this additional analysis both the tentative plan and the alternative plan were separately considered.

Tentative Plan – Operational and Safety Analysis

Under the tentative site plan, Landis Street would be extended into the site to provide access to lots 1-6 but would not provide a public street connection to Cornwall Street. The driveway serving lots 5 and 6 would extend to Cornwall Street with an easement allowing emergency vehicles through access between Landis Street and Cornwall Street. Notably, this access could also be designed to accommodate through pedestrian and bicycle trips in order to improve local connectivity for non-motorized travel modes while avoiding traffic increases on Cornwell Street which would otherwise result in reduced safety for pedestrians and cyclists where no sidewalks are provided. Since the existing home on the subject property takes access via Cornwall Street, the tentative plan will result in an increase of six new homes taking access via Landis Street and a reduction of one home taking access via Cornwall Street. Landis Street and Stonegate Lane would be projected to experience an increase of approximately 60 trips per day (30 percent of existing traffic volumes), and Cornwall Street would experience a decrease of 10 trips per day (a reduction of 10 percent of existing traffic volumes).

Based on the analysis, the tentative site plan would result in no significant impacts to the existing residential neighborhoods along Landis Street and Cornwall Street. Since an emergency vehicle connection would be maintained between Landis Street and Cornwall Street, it is likely that this limited connection could also accommodate pedestrian and bicycle traffic, thereby improving local-street connectivity for non-motorized travel modes. Due to the minimal transportation impacts of this design plan, no detailed operational and safety analysis would normally be required to address the impacts of the tentative site plan.

Alternative Plan – Operational and Safety Analysis

Under the tentative site plan, Landis Street would be extended through the site, connecting to the southern end of Cornwall Street. This street connection is contemplated in the city's Transportation System Plan as project LSC-16 "Landis Street extension to Cornwall Street" and is indicated as having priority "low".

Several other local street connections are also indicated in the project vicinity, including LSC-15 (Landis Street extension from Stonegate Lane to Winkel Way), LSC-19 (New east-west connection from Reed Street to Cornwall Street), LSC-21 (New north-south connection from the Landis Street extension to the new east-west connection) and LSC-26 (Sabo Lane extension from Beacon Hill to Sunset Avenue). Each of these local street connection projects is intended to increase connectivity for pedestrians, cyclists and motor vehicles within the local street network.

The timing of the local street connection projects may be critical to maintaining safe and efficient operation of the local street network. Since the proposed Willow Ridge development would construct the Landis Street connection to Cornwall Street without the benefit of the several other local street connections anticipated in the city's Transportation System Plan, it is appropriate to examine the potential impacts of making this street connection without the support of the other street connections planned for the future.

In order to determine the likely traffic demands for the new street connection, a fastest-path analysis was conducted. "Break even" points within the existing street network were identified where the new street connection would result in equal travel times taking either the proposed new street connection or an existing travel route. For homes and destinations located closer than this break-even point, existing vehicular trips would be assumed to move to the new street connection. Where existing street connections would provide a faster travel time, traffic would not be expected to divert to the new street.

For homes located to the northeast of the subject property, diversions would be expected to occur from locations where the new street would provide the fastest travel route either to the existing commercial and institutional uses along Salamo Road or to the 10th Street area with its connections to I-205. Based on the analysis, for all locations except those on Cornwall Street south of Sunset Avenue the fastest path to the commercial and institutional uses along Salamo Road will be via Parker Road. For trips to and from 10th Street, the fastest path will be via Sussex Street, Fairhaven Drive, Beacon Hill Drive and Barrington Drive. Accordingly, no diversions of existing traffic from areas northeast of the site are projected except those associated with the 10 existing homes on Cornwall Street.

For homes located to the west of the subject property, diversions would be expected to occur from locations where the new street would provide the fastest travel route to Sunset Avenue and Summit Street, which provide connections to Highway 43 and I-205. Based on the analysis, some existing homes along Landis Street, Beacon Hill Drive, Winkel Way, Sabo Lane and Quail Ridge Court would have a new fastest travel path following completion of the new roadway. Approximately 106 homes are projected to benefit from the new street connection.

Assuming that 30% of trips from these homes travel to and from the east, the projected impact on Cornwall Street would be the addition of approximately 320 daily trips. Adding these to the existing daily trips on Cornwall Street and approximately 15 trips from the proposed Willow Ridge development will result in a total traffic volume of approximately 415 trips per day. Traffic volumes on Stonegate Lane would be projected to increase from approximately 200 trips per day to approximately 490 trips per day. Note that the net increase on Stonegate Lane is slightly lower since the 20 existing homes on Stonegate Lane would add some traffic to Cornwall Street, thereby diverting those trips away from Stonegate Lane.

The projected traffic volumes on Landis Street and Stonegate Lane are within the carrying capacity of a queuing street. However, the adjacent homes would experience a notable increase in through traffic, with traffic volumes more than doubling along the local street.

The added traffic volumes on Cornwall Street are expected to have a more significant impact than on Landis Street. Since Cornwall Street has no sidewalks and the roadway is in many areas significantly less than 20 feet wide, increasing traffic volumes will result in more friction and increased conflicts along this existing 600-foot road segment. City staff previously indicated that in conjunction with completion of the Landis Street connection some funding would be provided to widen the existing cross-section of Cornwall Street to provide a continuous width of 20 feet. This proposed road width is sufficient to accommodate simultaneous two-way travel along the street segment. When there are pedestrians or people riding bicycles within the roadway the low projected traffic volumes in conjunction with the improved 20-foot street width would allow drivers to safely maneuver around vulnerable road users in a manner similar to avoiding vehicle conflicts on a queuing street. However, a dedicated funding source has not yet been identified for improvements to the existing Cornwell Street cross-section and the timing of this improvement is unknown.

A more detailed discussion of the adequacy of street widths is provided in the "Street Width Analysis" section below.

Street Width Analysis

The proposed extension of Landis Street would have a paved width of 28 feet. Under the alternative site plan a public street connection would be provided through the site between Landis Street and Cornwall Street, which has an existing paved width of 15 to 20 feet. The paved widths of all roadways must be capable of supporting the projected traffic loads as well as the needs of emergency vehicles (including fire apparatus).

Oregon's Transportation Planning Rule includes language in OAR 660-012-0045(7) stating "*Local governments shall establish standards for local streets and accessways that minimize pavement width and total right-of-way consistent with the operational needs of the facility. The intent of this requirement is that local governments consider and reduce excessive standards for local streets and accessways in order to reduce the cost of construction, provide for more efficient use of urban land, provide for emergency vehicle* *access while discouraging inappropriate traffic volumes and speeds, and which accommodate convenient pedestrian and bicycle circulation.*" In order to assist local governments with balancing the needs of safety, livability and emergency vehicle access, guidelines were created by stakeholder consensus and published as "Neighborhood Street Design Guidelines, An Oregon Guide to Reducing Street Widths". This guide provides several recommended local street cross sections that effectively minimize paved widths in conformance with the requirements of the Transportation Planning Rule while accommodating the needs of emergency vehicles. The recommended design guidelines were specifically endorsed and supported by the Office of the State Fire Marshal, the Oregon Fire Chiefs Association, the Oregon Fire Marshal's Association, the Oregon Chiefs of Police Association and the Oregon Refuse and Recycling Association, as well as ODOT, several planning associations, the Oregon Building Industry Association, 1000 Friends of Oregon, Oregon's Department of Land Conservation & Development, and Metro.

Notably, the guidelines include three recommended cross-sections for neighborhood streets. These consist of a 28-foot paved width with parking on both sides, a 24-foot paved width with parking on one side, and a 20-foot road width with no parking. The 24-foot and 28-foot cross-sections are described as "queueing streets" since vehicles may need to pull to one side to allow opposing traffic to pass, thereby limiting the effective traffic capacity of these roadways to 1,000 vehicles per day or less. Diagrams showing the recommended street cross-sections are included in the attached technical appendix.

Since the tentative site plan will utilize precisely the paved street width recommended for neighborhood streets, since Landis Street and Stonegate Lane will each carry fewer than 1,000 vehicles per day, and since the tentative site plan will result in a net reduction in traffic on Cornwall Street, no additional mitigations for traffic are necessary or recommended in conjunction with the proposed development.

Under the alternative site plan, traffic volumes would increase on Stonegate Lane, Landis Street and Cornwall Street. The projected traffic volumes on all three street segments are well within levels tolerable for queueing streets, but overall traffic volumes on these local streets would more than double on Stonegate Lane and Landis Street. Traffic volumes on Cornwall Street would increase by more than four times the existing traffic volumes, and absent funding the cross-section would remain narrower than the desired minimum of 20 feet identified in the *Oregon Street Design Guidelines.*

Conclusions

Based on the detailed analysis, the tentative site plan could be implemented while maintaining traffic volumes within acceptable levels for the affected local streets and intersections. The proposed street width for the extension of Landis Street is sufficient to accommodate the traffic volumes on the roadway as well as emergency vehicles, and the emergency vehicle easement at the east side of the driveways serving lots 5 and 6 would provide improved connectivity for emergency vehicles in the site vicinity.

Under the tentative site plan existing traffic patterns in the site vicinity would experience a negligible change in volumes, since the proposed development will generate a net increase of just 6 trips during the highest-volume hour on Stonegate Lane and Landis Street (an increase of one vehicle every ten minutes). Accordingly, implementation of the tentative plan would result in no significant operational or safety impacts to the existing transportation system.

Under the alternative site plan local street connectivity would be improved in the site vicinity, helping balance traffic volumes on the local street network and providing a second point of access for the existing homes along Landis Street and Cornwall Street. Traffic volumes would increase noticeably on Stonegate Lane and Landis Street, with traffic volumes more than doubling. Traffic volumes on Cornwall Street would increase by more than four times, and currently the 15- to 20-foot width of Cornwall Street is less than the desired minimum even for low-volume local residential streets. It is recommended that this street width be improved to at least a continuous 20-foot width in the future as funds become available.

Since dedicated sidewalks are not currently available along Cornwall Street, it is recommended that the city consider providing a connection along the Landis Street alignment that is limited to pedestrians, cyclists and emergency vehicles. This could be accomplished under the tentative site plan in conjunction by using a barrier at the east side of the driveways serving lots 5 and 6 that is accessible to emergency vehicles, pedestrians and cyclists only and which restricts through motor vehicle traffic at the east end of the proposed development. Notably, per city staff such a restriction would not be permissible under the alternative plan, since even a temporary barricade is not permitted on a public street.

It should be noted that the impact of through trips on Landis Street and Cornwall Street will be significantly reduced in the future upon completion of other local-street connections in the site vicinity. Once a new street connection is provided between the east side of Stonegate Lane and Parker Road (using portions of LSC-15 and LSC-26), this street connection will provide a faster, more efficient travel route than the Cornwall Street/Landis Street connection. For this reason, selection of a local street connectivity plan that complies with the city's Transportation System Plan by providing a future connection between the end of Landis Street and Cornwall Street through the undeveloped and underdeveloped properties located to the north of the proposed development would allow deferment of the connection to a time when traffic impacts would be reduced and would maintain the safe and efficient operation of Cornwall Street without creating short-term undesirable impacts to safety and neighborhood livability.

If you have any questions regarding this updated analysis, please feel free to contact me at (503)537-8511 or at mike.ard@gmail.com.

Appendix

Trip Generation Calculation Worksheet

Land Use Description:	Single-Family Detached Housing
ITE Land Use Code:	210
Independent Variable:	Dwelling Units
Quantity:	6 Dwelling Units

Summary of ITE Trip Generation Data

AM Peak Hour of Adjacent Street Traffic

Trip Rate: 0.74 trips per dwelling unit

Directional Distribution: 25% Entering 75% Exiting

PM Peak Hour of Adjacent Street Traffic

Trip Rate: 0.99 trips per dwelling unit

Directional Distribution: 63% Entering 37% Exiting

Total Weekday Traffic

Trip Rate: 9.44 trips per dwelling unit

Directional Distribution: 50% Entering 50% Exiting

Site Trip Generation Calculations

6 Dwelling Units

	Entering	Exiting	Total
AM Peak Hour	1	3	4
PM Peak Hour	4	2	6
Weekday	28	28	56

Data Source: *Trip Generation Manual, 10th Edition* , Institute of Transportation Engineers, 2017

Trip Generation Calculation Worksheet

Land Use Description:	Single-Family Detached Housing
ITE Land Use Code:	210
Independent Variable:	Dwelling Units
Quantity:	1 Dwelling Units

Summary of ITE Trip Generation Data

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Trip Rate: 0.74 trips per dwelling unit

Directional Distribution: 25% Entering 75% Exiting

PM Peak Hour of Adjacent Street Traffic

Trip Rate: 0.99 trips per dwelling unit

Directional Distribution: 63% Entering 37% Exiting

Total Weekday Traffic

Trip Rate: 9.44 trips per dwelling unit

Directional Distribution: 50% Entering 50% Exiting

Site Trip Generation Calculations

1 Dwelling Units

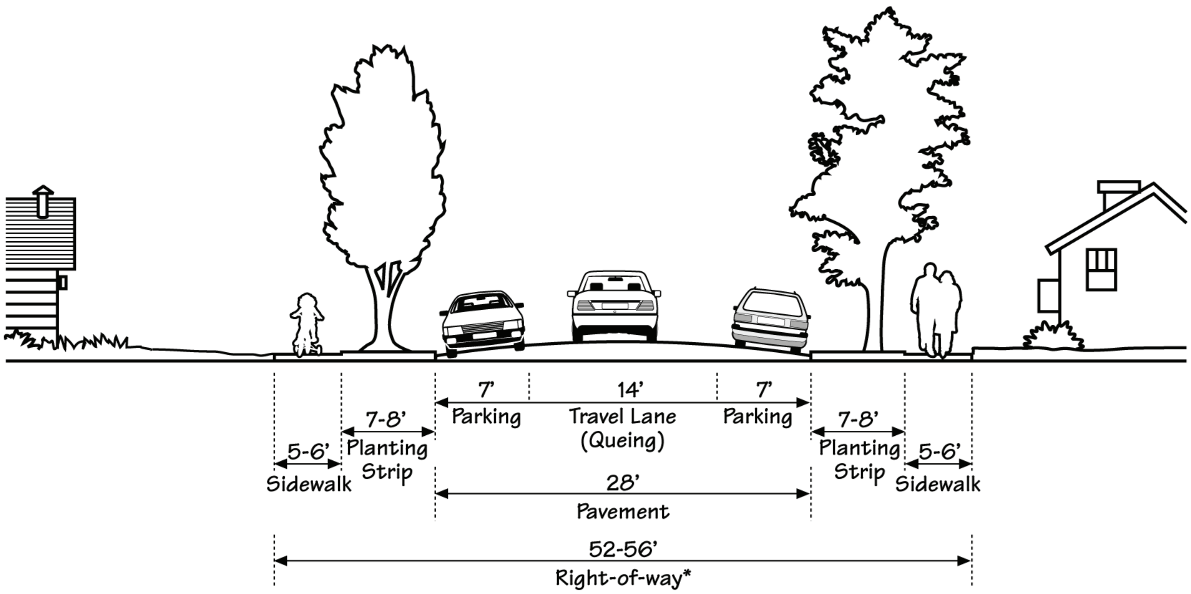
	Entering	Exiting	Total
AM Peak Hour	0	1	1
PM Peak Hour	1	0	1
Weekday	5	5	10

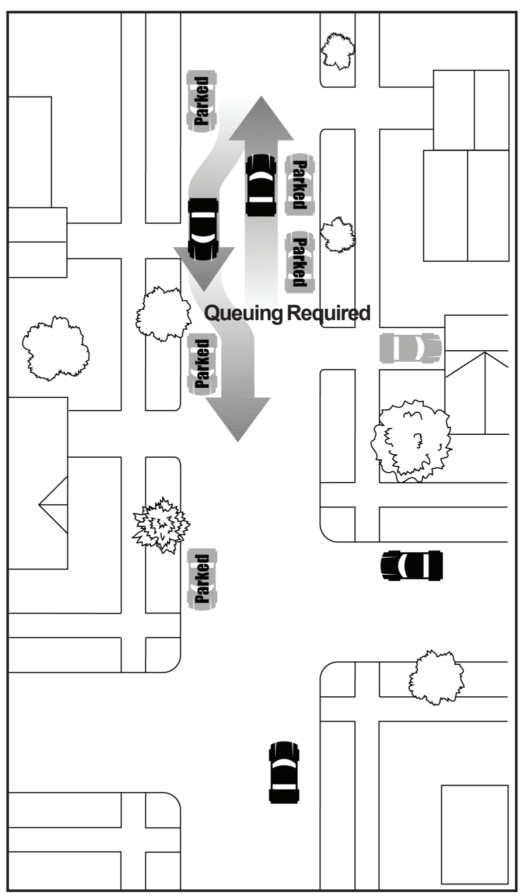
Data Source: *Trip Generation Manual, 10th Edition* , Institute of Transportation Engineers, 2017

Scenario 1

28 Ft. Streets

Parking on both sides

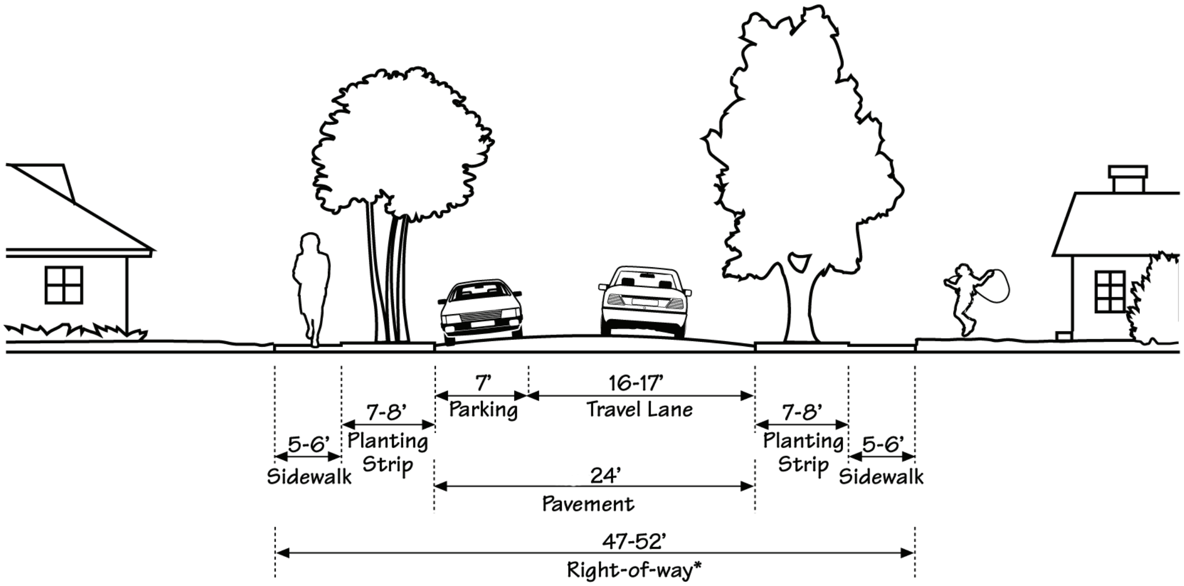


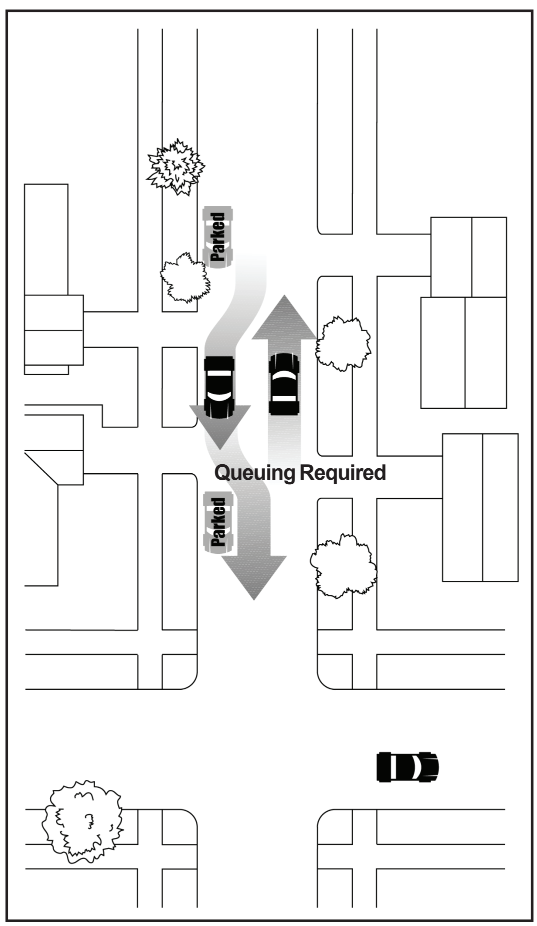


Scenario 2

24 Ft. Streets

Parking on one side only

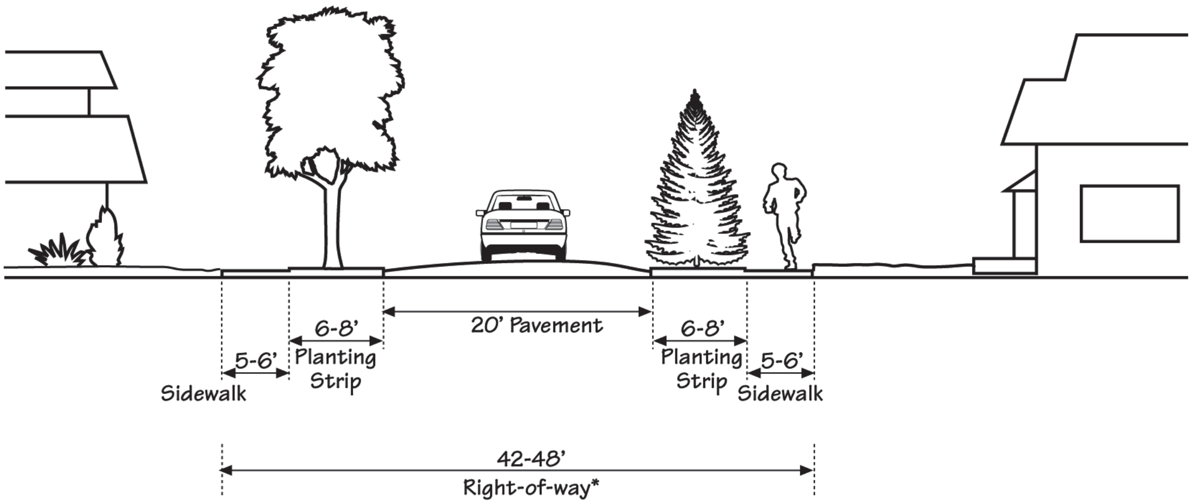


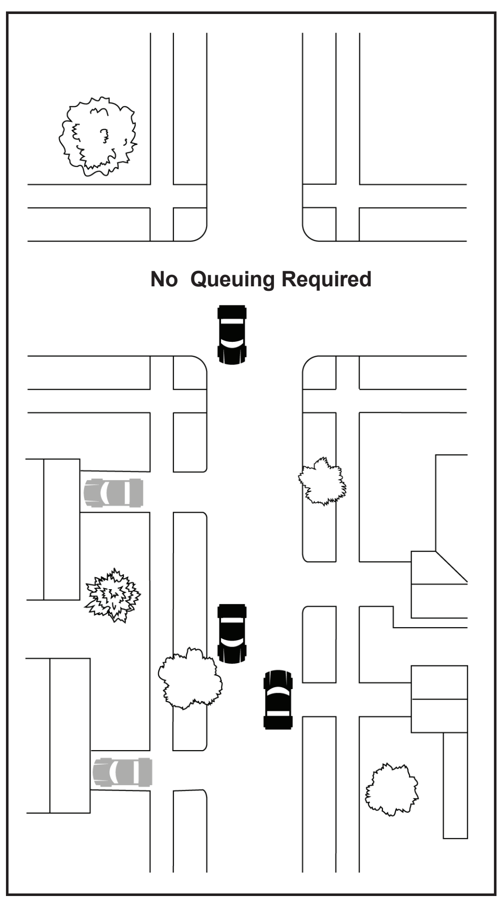


Scenario 3

20 Ft. Streets

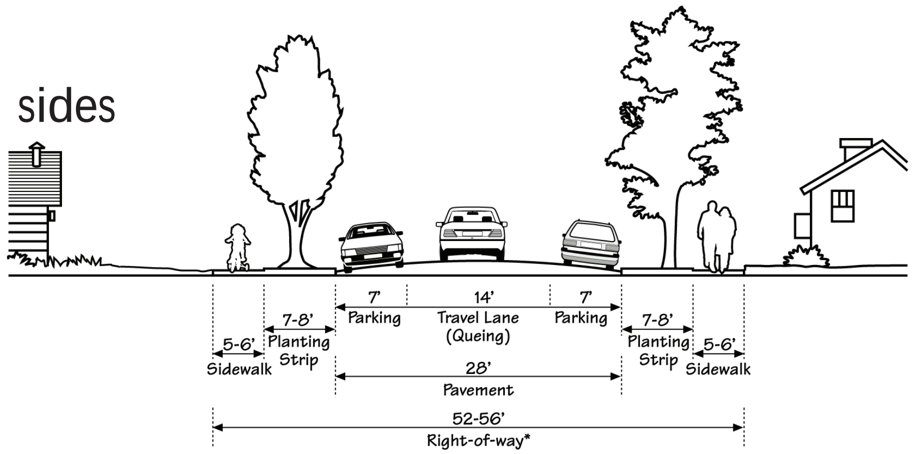
No parking allowed



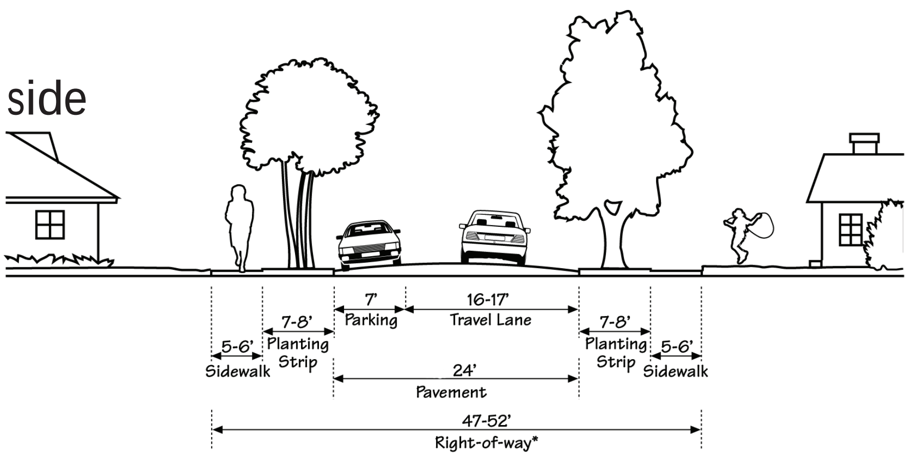


Summary of Three Potential Scenarios

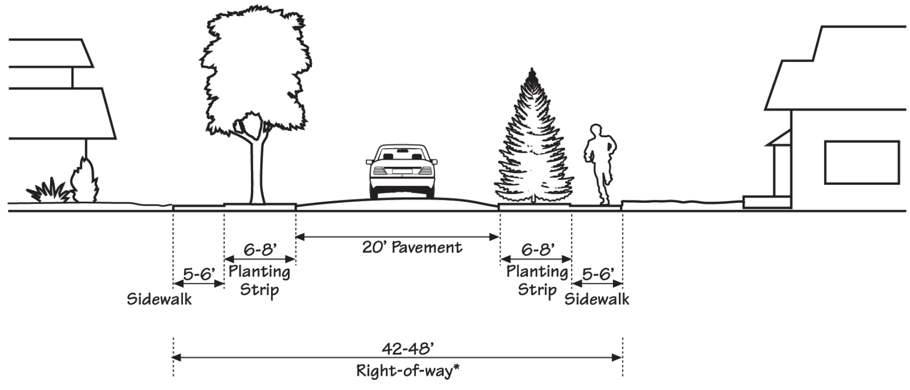
28 Ft Street
Parking on both sides



24 Ft Street
Parking on one side



20 Ft Street
No on-street parking allowed



21370 SW Langer Farms Pkwy
Suite 142, Sherwood, OR 97140

REGISTERED PROFESSIONAL ENGINEER
54983PE
OREGON
JULY 10, 2002
MICHAEL T. ARD
EXPIRES: 12/31/2021

Technical Memorandum

To: Mark Handris, Icon Construction

From: Michael Ard, PE

Date: October 28, 2020

Re: Willow Ridge – Response to Neighborhood Resident Concerns

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This memorandum is written in response to concerns raised by local residents regarding the potential transportation impacts of the proposed Willow Ridge Subdivision. To make the responses clear, the public comments are summarized and numbered below, with a response immediately following.

1) There is no traffic impact analysis for the Tentative Plan, which is the hammer termination of Landis for the Willow Ridge Development.

The tentative site plan generates traffic volumes far below the threshold at which a detailed traffic impact study is required. Due to the lack of a connection to Cornwall Street, this scenario would not include any other changes to traffic patterns that would justify a more detailed analysis. Regardless, a brief analysis of the tentative site plan is included in the updated traffic analysis dated October 28, 2020 which has been provided for the project.

2) A total of 242 daily trips would be expected on Landis Street under the tentative plan, which is significant.

The 26 homes accessed via Landis Street under the tentative plan would result in 246 daily trips. The actual trip rate for residential homes averages 9.44 trips per dwelling. This volume equates to 26 trips during the highest volume hour of the day, or approximately one trip every 2.3 minutes. This traffic volume is not considered "significant" and represents only one quarter of the traffic volume typically permitted on constrained local residential streets.

3) There is no definition of what a "trip" is, and this must be stated in the analysis.

The definition of a trip is standardized within the transportation engineering community. A trip consists of a single vehicle either entering or exiting a site. Accordingly, a home which has 5 vehicles depart from the site and return during the course of a day would generate 10 total daily trips.

4) The Tanner Stonegate Board of Directors believe the trip numbers may not necessarily correct and be understated. There is considerably more online shopping resulting in more deliveries from USPS, Amazon, UPS, FedEx, grocery stores, restaurants plus city and homeowner service vehicles, friend and family visits, etc. which we feel have not been properly identified. The trip definition and number of daily trips per household must be revised.

The trip generation data from the Trip Generation Manual published by the Institute of Transportation Engineers was developed using data from 159 residential developments, with an average of 264 dwelling units per study location. Accordingly, the data represents an average of approximately 42,000 homes. The data for single-family homes is more robust than any other land use category. Although there can be variations in the trip rate is small communities, requirements for mitigation must be proportional to the impacts of the development, and since the actual trips generated by a development vary over time and cannot be measured in advance of project completion, the only way to ensure compliance with this requirement of federal law is to base the analysis on average trip rates. Regardless, the error in projection would need to be off by a factor of four in order to result in traffic volumes that would be problematic under the tentative plan. This is well beyond the range of variation observed at any of the 159 study sites. The projections were therefore appropriate and any local variation would not rise to a level that would result in any change in the conclusions of the traffic study.

Further, CDC Section 85.170(B)(2)(b) states:

> *b. Typical average daily trips. The latest edition of the Trip Generation manual, published by the Institute of Transportation Engineers (ITE) shall be used as the standards by which to gauge average daily vehicle trips.*

Accordingly, the projection method used for estimating site trips is not only best practice from the perspective of a transportation engineer, but is explicitly required by West Linn's Community Development Code.

5) There is a school bus stop at Stonegate Lane and Beacon Hill Drive and parents wait in the cars during drop off and pick up creating congestion. Increased Willow Ridge traffic will elevate risk for students and add more congestion in that area during the school year.

School bus stops are typical on local residential streets, and parking is also permitted. Neither the presence of school buses nor parent vehicles create an unusual or unsafe condition. Although there may be some brief congestion which occurs at the times of school bus activities, congestion surrounding school activities is generally considered to be acceptable or even desirable, since it results in decreased travel speeds and increased caution in the vicinity.

6) The Traffic Impact Analysis must define total impact on Landis Street, Stonegate Lane and Beacon Hill Drive, address increased traffic and congestion related issues and plan for student safety for the Alternative Plan and Tentative Plan.

Impacts on Landis Street and Stonegate Lane were identified in the traffic study. No significant change would be projected on Beacon Hill Drive under either the tentative plan or the alternative plan.

7) Heading east on Stonegate Lane there is a slight hill where it intersects with Landis Street and the corner is blind. Also, heading north on Landis Street, starting at the north end at lot 37,

there is a large stone retaining wall and a right curve in the road that presents a blind turn. The road also narrows at the large retaining wall and cars parking on the west side of Landis Street across from the retaining wall further decreases street width at the blind curve.

Stonegate Lane and Landis Street were constructed in 2002 to modern design standards. The presence of hilly terrain, curves and walls are not unusual conditions, particularly in areas where natural terrain makes these features necessary. The complete lack of crash history on either Stonegate Lane or Landis Street also demonstrates that the road design has not been problematic.

8) Children safety on Landis Street is an issue. Due to relatively small yards, children are riding bikes, scooters, and generally playing, etc. on their driveway, sidewalk and sometimes the street. The increased traffic is a neighborhood concern for the safety of children.

Again, the presence of children and potential conflicts with traffic are typical conditions on local residential streets.

9) The Tanner Stonegate BOD would respectively ask the city to propose how traffic safety issues will be mitigated for blind spots and children safety before approving the Willow Ridge development. Future development of the farm property north of Stonegate Lane should be considered when developing the mitigation plan.

Based on the analysis, no operational or safety mitigations are necessary in conjunction the proposed development.

10) Tanner Stonegate BOD is asking if the city would review the Master Plan and not have Landis Street connect to Cornwall Street. This would create a short cut to Sunset Avenue and put an unnecessary traffic burden on Landis Street and Cornwall Street.

The updated analysis provided for the proposed subdivision provides information regarding the volume of traffic that would use this potential street connection. It is acknowledged that providing this connection would result in increased traffic volumes on Stonegate Lane, Landis Street and Cornwell Street. Ultimately, the decision as to whether this street connection will be required must be decided through the land use approval process.

Conclusions

The scope of the analysis memorandum dated October 28, 2020 far exceeded the requirements of Public Works Design Standard Section 5.0014 and CDC 85.170(B)(2) by providing a detailed analysis for a project which generates only 5 percent and 20 percent respectively of the traffic which would normally require a traffic impact study. In addition to the analysis memorandum, we have herein addressed the questions and concerns raised by local residents in the project vicinity. Based on the results of the analysis, the traffic-related code requirements of the City of West Linn described in CDC Chapters 48, 55 and 85 are satisfied.