DRAINAGE REPORT

To
City of Manzanita

For
Highlands, Phase 8

Submitted
November 26, 2024

Project Number
2160454.08

REGISTERED PROFESSIONAL ENGINEER
#56660
Ralph Henderson
OREGON
JULY 11, 2000
RALPH J.R. HENDERSON
EXPIRES: 6/30/25

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I. INTRODUCTION

The purpose of this report is to provide engineering documentation and storm drainage calculations to support the Manzanita Pines development in the City of Manzanita, Oregon. This report demonstrates the proposed stormwater management facility system's compliance with The City of Manzanita and its current Construction Standards (April 2015).

Manzanita Pines was outside the limits of the original storm report. The existing site at the Manzanita Pines location is undeveloped land area consisting of wooded areas and occasional sand dunes. The Phase 1 through Phase 5 development has already been constructed.

The previous phases of the development consist of residential lots, new roadways, and associated storm, sanitary, water, and electric utilities. The Phase 1 and Phase 2 roadway – Highlands Drive – connects to Classic Street approximately 400 feet north of Ridge Road. Phase 1 and Phase 2 also included Seaview Drive, which is approximately 1,000 feet long.

The site was outside City of Manzanita city limits but has recently been incorporated into the City. The lots are now zoned as Special Residential Recreational area (SRR).

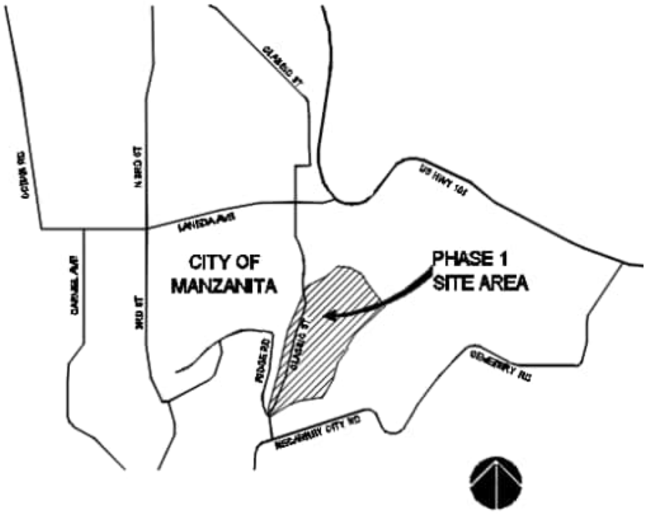


Figure 1: Vicinity Map

II. WATER QUALITY

The Natural Resources Conservation (NRCS) web service data exhibits the proposed Manzanita Pines development site soil consists of 100% Netarts fine sandy loam soil categorized under hydrologic soil group A. Based on the type of soil found at the site, two (2) infiltration ponds will be constructed with the project to meet the water quality requirements. These will be permanent ponds and will be located on the subject site.

The site has been broken up into two (2) drainage basins – Basin A and Basin B. Basin A is the majority of the site, and Basin B is the southern portion of the site. Pond A is sized to manage the stormwater collected from Basin A, and Pond B to manage storm drainage from Basin B.

The ponds will treat the collected stormwater by infiltrating the water through amended soil media and vegetation. See the basin map for basins and pond locations.

To size the infiltration ponds for Water Quality, one cubic foot of storage was provided for every 44 square feet (SF) of impervious surface developed per City of Manzanita storm requirements.

Basin A = 47,726 SF/44 = 1,084 cubic feet minimum.

Basin B = 28,590 SF/44 = 650 cubic feet minimum.

III. WATER QUANTITY

To meet City of Manzanita's water quantity requirement, two (2) separate infiltration ponds were designed to contain the 10-year storm event for the areas noted in Table 1 below.

All of Phases 1 through Phase 5 have already been built.

The total impervious area for Manzanita Pines North draining to the north pond = 47,726 SF.
The total impervious area for Manzanita Pine South draining to the south pond = 28,590 SF.

Table 1: Drainage Basins			
Drainage Basin	**Area - Pervious (ft^2)**	**Area - impervious (ft^2)**	**Total Area (ft^2)**
Basin A	30,682	47,726	78,408
Basin B	21,635	28,590	50,225

To size the infiltration ponds, one (1) cubic foot of storage was provided for every 44 square feet of impervious surface developed per City of Manzanita storm requirements. See calculations in WQ section above.

Table 2: Infiltration Ponds		
Drainage Basin	**Required Storage Volume (CF)**	**Provided Storage Volume (CF)**
Pond A	1084	2,438
Pond B	650	1,845

Based on the existing sandy soils, this water should all infiltrate from both ponds without any overflow. See the infiltration calculations in the appendices from Hydraflow. The infiltration calculations were based on a design infiltration rate of 20 inches per hour. The sandy soils infiltrate so fast that the measured infiltration rate was greater than 150 inches per hour.

Pond A has 12" of drain rock under the topsoil to provide additional storage and infiltration. Pond B does not have any rock underneath.

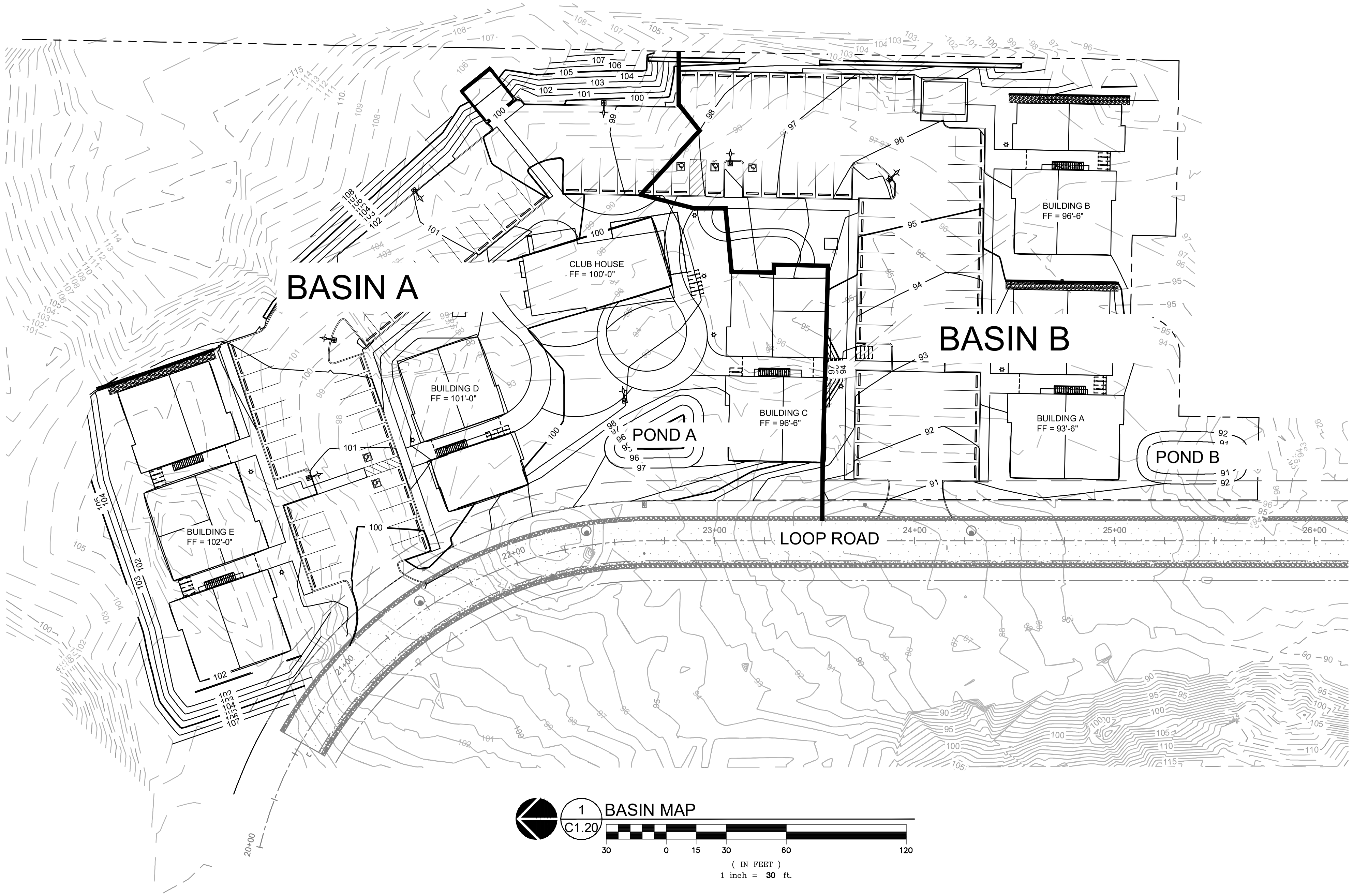
The pond sizes shown meet the infiltration requirements as listed the Hydraflow infiltration results.

IV. CONVEYANCE DESIGN

The rational method was used to size the storm pipes in the conveyance system. Sub-basins 1-6 are delineated and a 10-year storm runoff was used to analyze the proposed stormwater conveyance system. A time of concentration of 5.50 minutes and a runoff coefficient of 0.9 were assumed.

The conveyance system will be a maximum 12" pipe on site and a 12" pipe in Loop Road.

All the proposed roadways will have a uniform cross-slope of 2% towards a concrete gutter and rock overflow section. Catch basins are spaced out along the entire length of concrete gutter that will collect runoff from the various rights-of-way. There is not expected to be any drainage overflow, even up to the 50-year storm event; however, as with the previous phases, any overflow drainage beyond the maximum pond capacity will be dispersed via overland flow.



1 C1.20 BASIN MAP

THE SURVEY INFORMATION SHOWN AS A BACKGROUND SCREEN ON
THIS SHEET IS SHOWN FOR REFERENCE ONLY AND IS BASED ON
A SURVEY BY: HICKMAN & ASSOCIATES DATE:

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Planning ▪ Engineering

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Client
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Project
**HIGHLANDS
PHASE 8**

REVISION SCHEDULE

Delta	Issued As	Issue Date

SHEET TITLE:
BASIN MAP

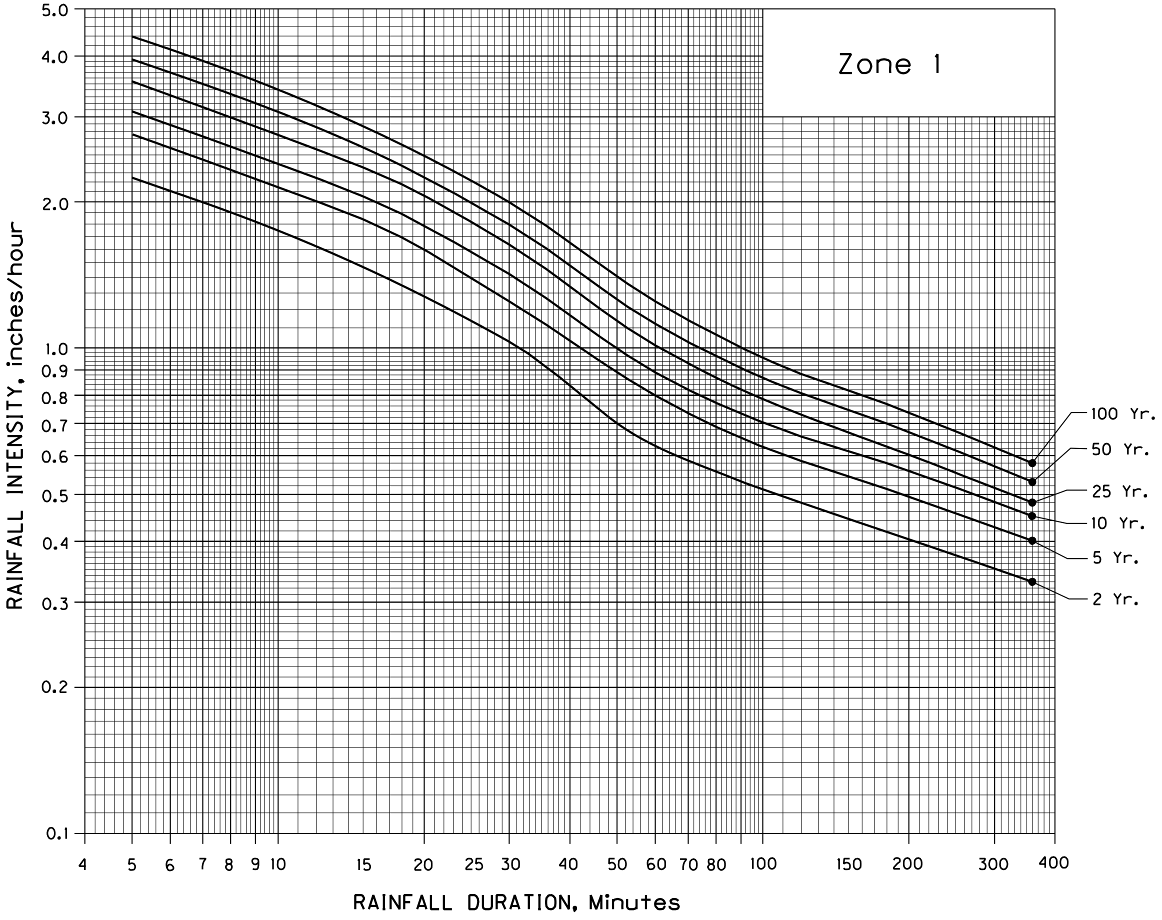
SHEET:
1

JOB NO. **2160454.08**

PRELIMINARY ONLY

216045400\4_DRAWINGS\CIVIL\PHASE 8\EXHIBITS\BASIN MAP.DWG:C1.20 RJH 11/22/24 10:34 1:30

RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES



Hydraflow Table of Contents

H:\Projects\216045400\Production\Calculations\Phase 8\storm-site.gpw

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021

Thursday, 11 / 21 / 2024

Hydrograph Return Period Recap

Hyd. No.	Hydrograph type (origin)	Inflow hyd(s)	Peak Outflow (cfs)								Hydrograph Description
			1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr	
1	SBUH Runoff	------	-------	-------	-------	-------	-------	-------	1.888	-------	Basin A - Developed
2	SBUH Runoff	------	-------	-------	-------	-------	-------	-------	1.133	-------	Basin B
4	Reservoir	1	-------	-------	-------	-------	-------	-------	1.142	-------	A
5	Reservoir	2	-------	-------	-------	-------	-------	-------	0.570	-------	B

Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SBUH Runoff	1.888	2	474	26,980	------	------	------	Basin A - Developed
2	SBUH Runoff	1.133	2	474	16,361	------	------	------	Basin B
4	Reservoir	1.142	2	490	26,980	1	96.56	1,899	A
5	Reservoir	0.570	2	496	16,360	2	92.34	1,660	B

H:\Projects\216045400\Production\Calculations\Phase 3\storm-site.gpw

Return Period: 50 Year

Thursday, 11 / 21 / 2024

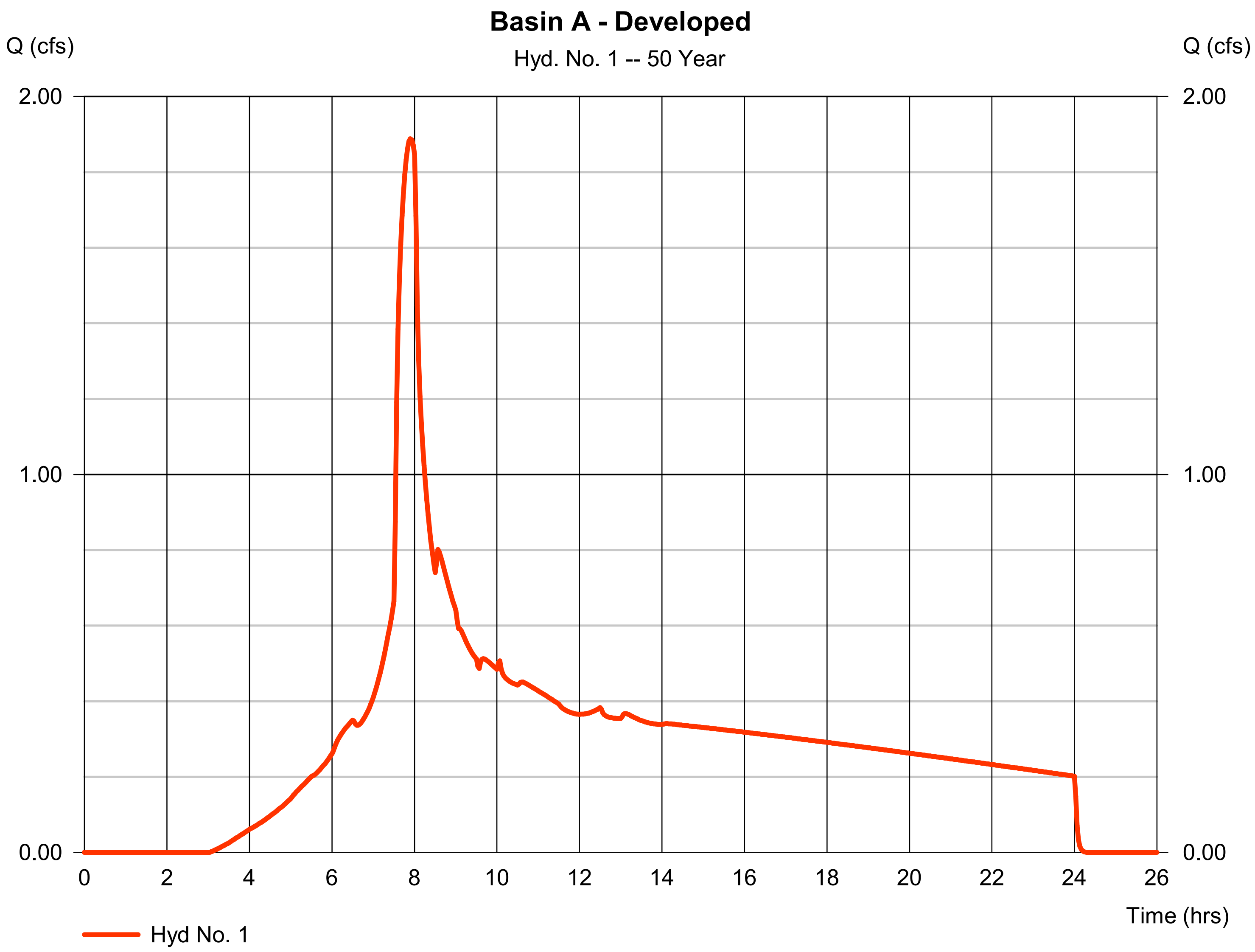
Hydrograph Report

Hyd. No. 1

Basin A - Developed

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.888 cfs
Storm frequency	= 50 yrs	Time to peak	= 7.90 hrs
Time interval	= 2 min	Hyd. volume	= 26,980 cuft
Drainage area	= 1.800 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 2.80 min
Total precip.	= 6.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(1.100 x 98) + (0.700 x 49)] / 1.800



Hydrograph Report

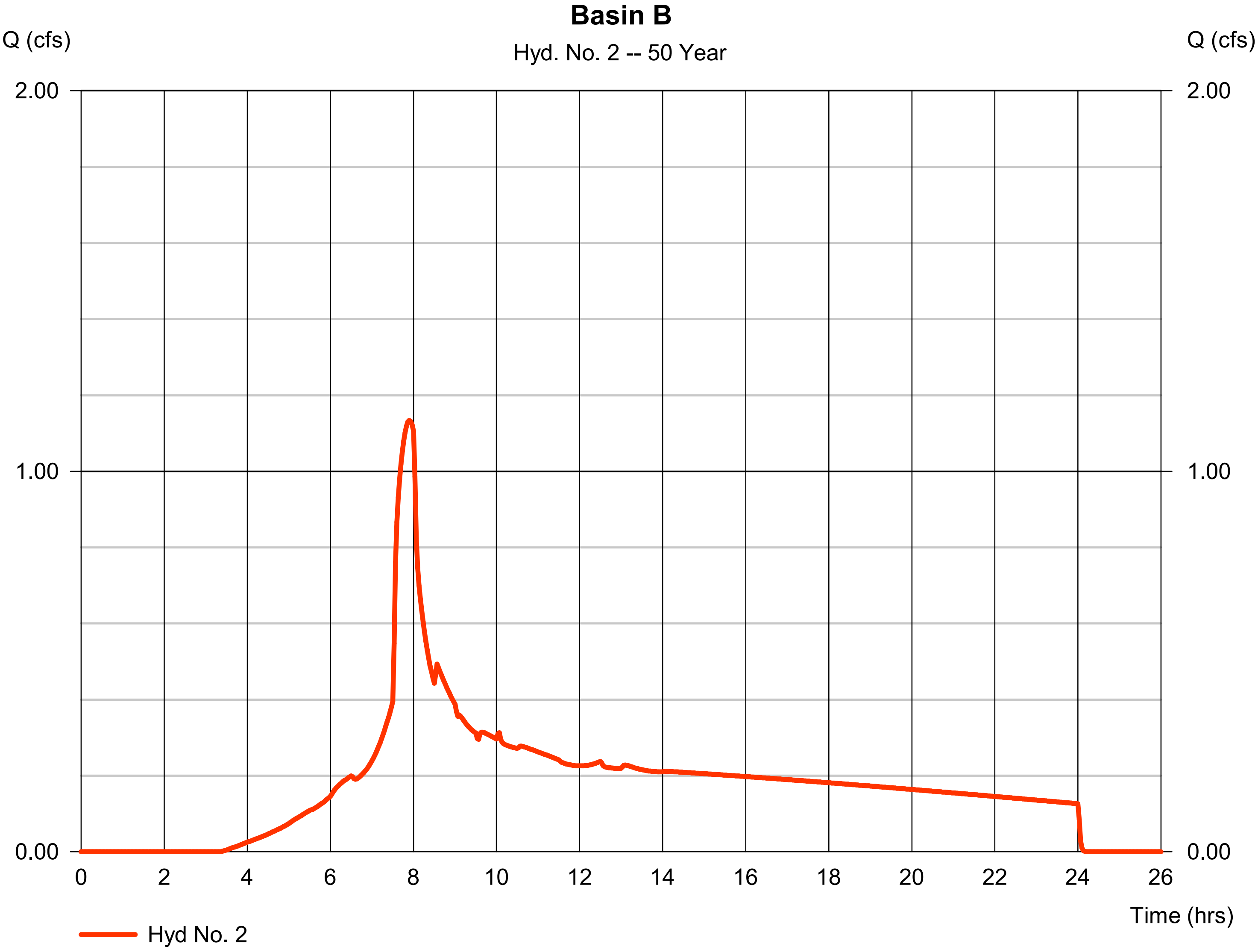
Hyd. No. 2

Basin B

Hydrograph type = SBUH Runoff
Storm frequency = 50 yrs
Time interval = 2 min
Drainage area = 1.150 ac
Basin Slope = 0.0 %
Tc method = TR55
Total precip. = 6.50 in
Storm duration = 24 hrs

Peak discharge = 1.133 cfs
Time to peak = 7.90 hrs
Hyd. volume = 16,361 cuft
Curve number = 77*
Hydraulic length = 0 ft
Time of conc. (Tc) = 2.00 min
Distribution = Type IA
Shape factor = n/a

* Composite (Area/CN) = [(0.660 x 98) + (0.490 x 49)] / 1.150



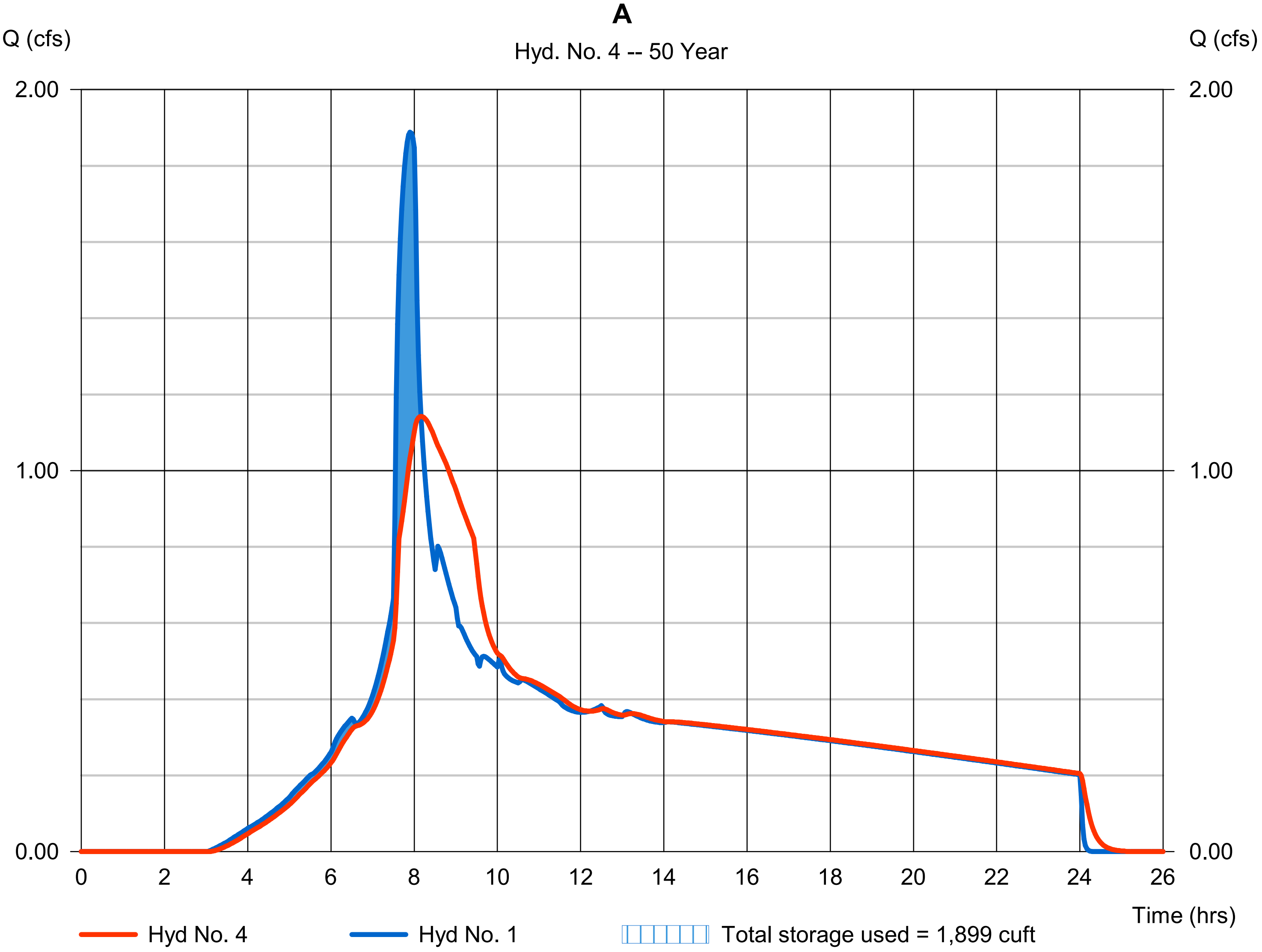
Hydrograph Report

Hyd. No. 4

A

Hydrograph type	= Reservoir	Peak discharge	= 1.142 cfs
Storm frequency	= 50 yrs	Time to peak	= 8.17 hrs
Time interval	= 2 min	Hyd. volume	= 26,980 cuft
Inflow hyd. No.	= 1 - Basin A - Developed	Max. Elevation	= 96.56 ft
Reservoir name	= Pond A	Max. Storage	= 1,899 cuft

Storage Indication method used.



Pond Report

Pond No. 1 - Pond A

Pond Data

Pond storage is based on user-defined values.

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	92.00	n/a	0	0
1.00	93.00	n/a	451	451
2.50	94.50	n/a	1	452
3.00	95.00	n/a	164	616
3.50	95.50	n/a	360	976
4.00	96.00	n/a	374	1,350
4.50	96.50	n/a	479	1,829
5.00	97.00	n/a	609	2,438

Culvert / Orifice Structures

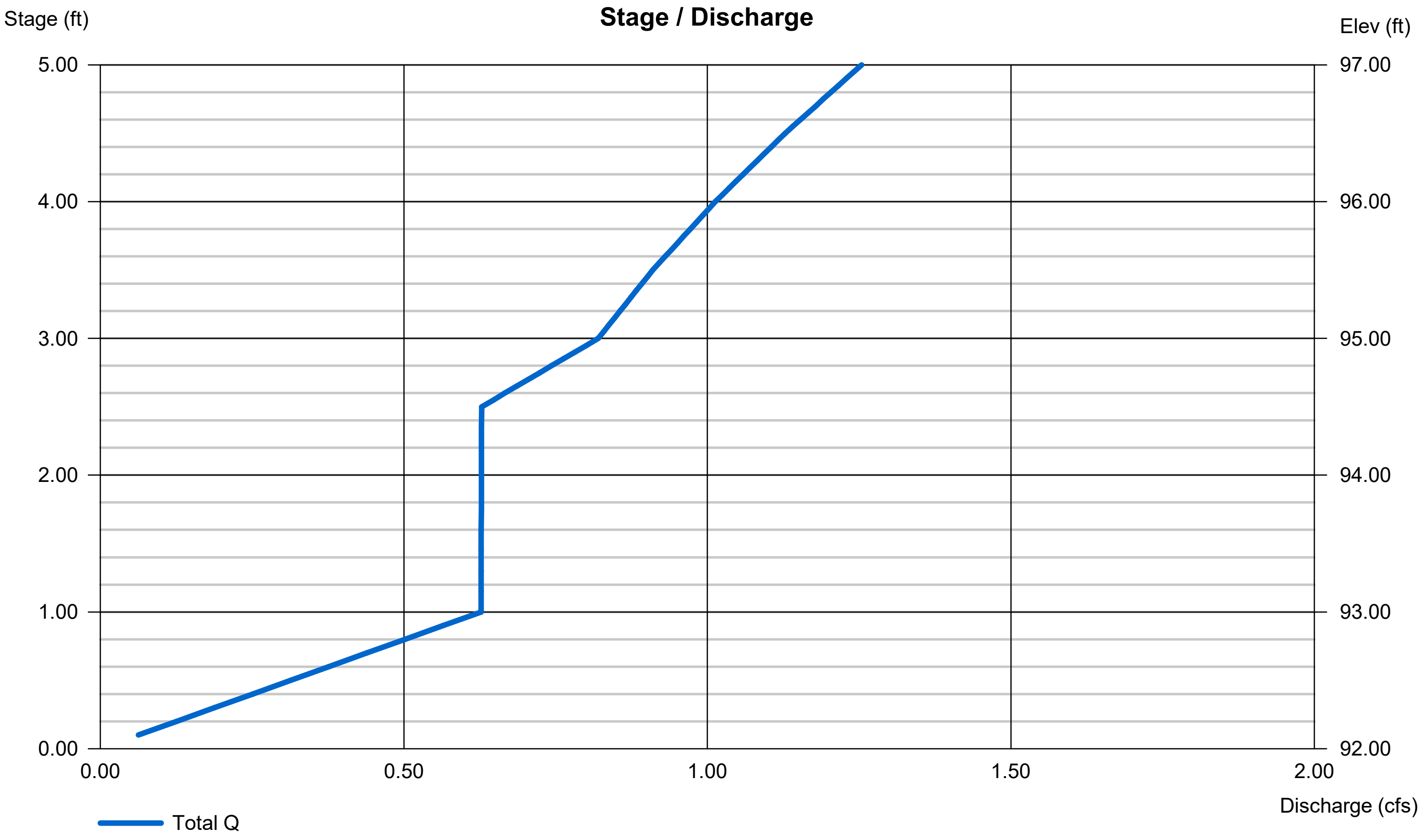
	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 0.00	0.00	0.00	0.00
Span (in)	= 0.00	0.00	0.00	0.00
No. Barrels	= 0	0	0	0
Invert El. (ft)	= 0.00	0.00	0.00	0.00
Length (ft)	= 0.00	0.00	0.00	0.00
Slope (%)	= 0.00	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 0.00	0.00	0.00	0.00
Crest El. (ft)	= 0.00	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= ---	---	---	---
Multi-Stage	= No	No	No	No

Exfil.(in/hr) = 0.000 (by Wet area)
TW Elev. (ft) = 0.00

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).



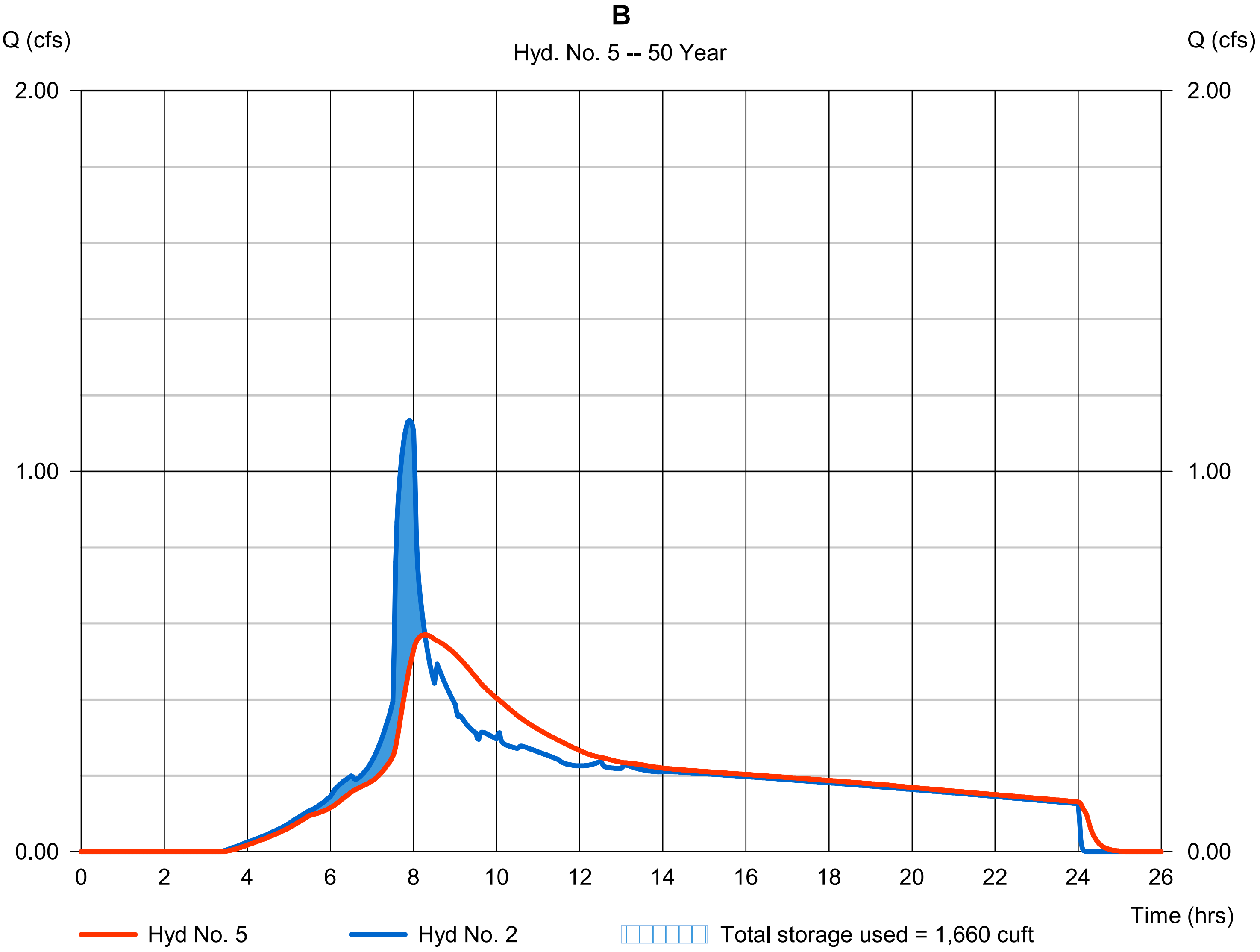
Hydrograph Report

Hyd. No. 5

B

Hydrograph type	= Reservoir	Peak discharge	= 0.570 cfs
Storm frequency	= 50 yrs	Time to peak	= 8.27 hrs
Time interval	= 2 min	Hyd. volume	= 16,360 cuft
Inflow hyd. No.	= 2 - Basin B	Max. Elevation	= 92.34 ft
Reservoir name	= Pond B	Max. Storage	= 1,660 cuft

Storage Indication method used.



Pond Report

Pond No. 2 - Pond B

Pond Data

Pond storage is based on user-defined values.

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	89.50	n/a	0	0
0.50	90.00	n/a	68	68
1.00	90.50	n/a	146	214
1.50	91.00	n/a	238	452
2.00	91.50	n/a	343	795
2.50	92.00	n/a	460	1,255
3.00	92.50	n/a	590	1,845

Culvert / Orifice Structures

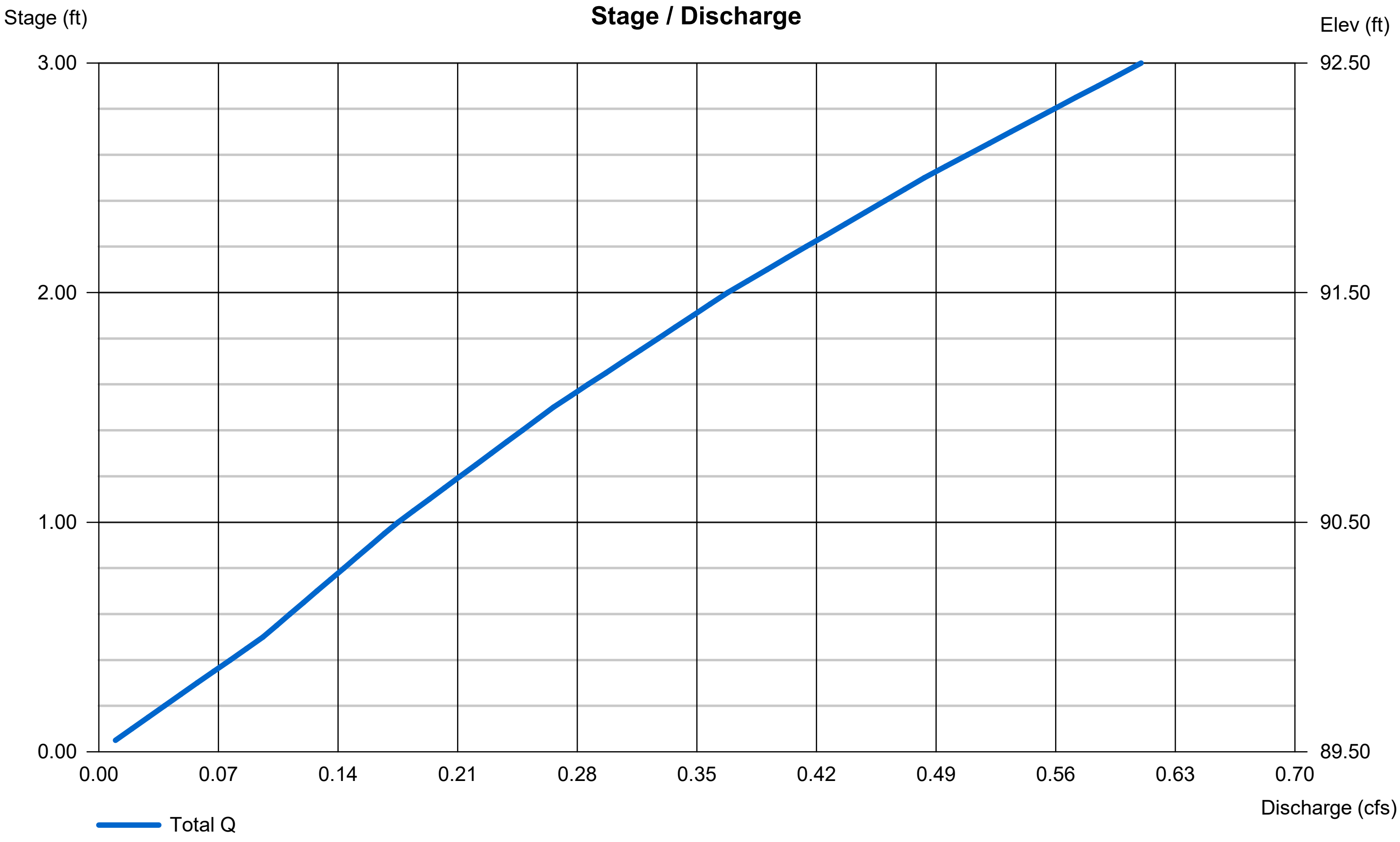
	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 0.00	0.00	0.00	0.00
Span (in)	= 0.00	0.00	0.00	0.00
No. Barrels	= 0	0	0	0
Invert El. (ft)	= 0.00	0.00	0.00	0.00
Length (ft)	= 0.00	0.00	0.00	0.00
Slope (%)	= 0.00	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	No	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 0.00	0.00	0.00	0.00
Crest El. (ft)	= 0.00	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= ---	---	---	---
Multi-Stage	= No	No	No	No

Exfil.(in/hr) = 0.000 (by Wet area)
TW Elev. (ft) = 0.00

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).



Hydraflow Rainfall Report

Return Period (Yrs)	Intensity-Duration-Frequency Equation Coefficients (FHA)			
	B	D	E	(N/A)
1	0.0000	0.0000	0.0000	--------
2	6.9527	2.1000	0.6577	--------
3	0.0000	0.0000	0.0000	--------
5	9.9393	2.7000	0.6824	--------
10	10.2300	2.0000	0.6569	--------
25	11.8938	2.0000	0.6571	--------
50	13.7560	2.2000	0.6602	--------
100	15.0837	2.1000	0.6597	--------

File name: Portland IDF.IDF

Intensity = B / (Tc + D)^E

Return Period (Yrs)	Intensity Values (in/hr)											
	5 min	10	15	20	25	30	35	40	45	50	55	60
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	1.92	1.35	1.07	0.91	0.79	0.71	0.65	0.59	0.55	0.52	0.49	0.46
3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	2.47	1.75	1.40	1.18	1.03	0.92	0.83	0.77	0.71	0.66	0.62	0.59
10	2.85	2.00	1.59	1.34	1.17	1.05	0.95	0.88	0.82	0.76	0.72	0.68
25	3.31	2.32	1.85	1.56	1.36	1.22	1.11	1.02	0.95	0.89	0.83	0.79
50	3.74	2.64	2.10	1.78	1.55	1.39	1.26	1.16	1.08	1.01	0.95	0.90
100	4.14	2.91	2.32	1.96	1.71	1.53	1.39	1.28	1.19	1.11	1.05	0.99

Tc = time in minutes. Values may exceed 60.

Precip. file name: H:\Projects\220047300\Production\Calcs\Civil\Storm\CWS precipitation.pcp

Storm Distribution	Rainfall Precipitation Table (in)							
	1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr
SCS 24-hour	1.25	2.50	0.00	3.10	3.45	3.90	6.50	4.50
SCS 6-Hr	0.53	1.05	0.00	1.25	1.55	1.70	1.80	1.90
Huff-1st	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-2nd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-3rd	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-4th	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Huff-Indy	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Custom	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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Report of
Geotechnical Investigation
HFD-GLD Manzanita Housing
Tax Lot 1401
Tillamook County, Oregon

CGT Project Number G2305878

Prepared for

Rob Justus
Green Light - Home First, LLC
3050 SE Division Street, Suite 270
Portland, Oregon 97202

April 14, 2023

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April 14, 2023

Rob Justus
Green Light - Home First, LLC
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Portland, Oregon 97202

Report of
Geotechnical Investigation
HFD-GLD Manzanita Housing
Tax Lot 1401
Tillamook County, Oregon

CGT Project Number G2305878

Dear Rob Justus:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed HFD-GLD Manzanita Housing project. The site is located within the northeast portion of Tax Lot 1401 in Tillamook County, Oregon. We performed our work in general accordance with CGT Proposal GP23-017, dated February 16, 2023. Written authorization for our services was received on February 23, 2023.

We appreciate the opportunity to work with you on this project. Please contact us at (503) 601-8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
70773PE
OREGON
Jan. 10, 2007
BRAD MICHAEL WILCOX
EXPIRES: 6/30/2024

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Doc ID: G:\GEOTECH\PROJECTS\2023 Projects\G2305878 - HFD-GLD Manzanita Housing\G2305878 - GEO\008 - Deliverables\Report\G2305878.docx

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ATTACHMENTS

1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed HFD-GLD Manzanita Housing project. The site is located within the northeast portion of Tax Lot 1401 in Tillamook County, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Information

CGT developed an understanding of the proposed project based on our correspondence with HFD Partners (HFD) and project documents provided to us on February 6, 2023. The documents provided included a preliminary Site Plan, prepared by Polyphon Architecture & Design, LLC, and a marked up aerial image. Based on our review, we understand the project will include:

- Construction of a new common house and several new residential buildings at the site. Although no architectural plans have been provided, we anticipate the structures will be one to three stories, wood-framed, with slab on grade ground floors and/or post and beam ground floor construction (crawlspaces). The common house will incorporate a footprint of roughly 2,500 square feet, and the residential buildings will include a total of 60 units. No below-grade levels (basements) are anticipated for the proposed structures. For the purposes of this report, we have assumed maximum column, continuous wall, and uniform floor slab loads will be on the order of 50 kips, 4 kips per lineal foot (klf), and 150 pounds per square foot (psf), respectively.
- Construction of private driveways and parking areas to provide vehicular access to the new residential structures. We anticipate the new pavements will be surfaced with asphalt concrete (AC).
- Although no stormwater management plans have been provided, we understand stormwater collected from new impervious areas of the site will be disposed of, at least in part, via onsite infiltration. No details regarding the type or location of the proposed stormwater infiltration facility(ies) were available at the time of this assignment. Design of infiltration facility(s) will rest with others. Infiltration testing was requested at two locations at the site at a depth of 5 feet below ground surface (bgs).
- Although no grading plans have been provided, we anticipate permanent grade changes at the site will be relatively minimal, with maximum cuts and fills on the order of about 3 feet in depth.

1.2 Scope of Services

Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities within a 20-foot radius of our explorations at the site. CGT also subcontracted a private utility locator service to mark the locations of detectable private utilities within the same radius.
- Explore subsurface conditions at the site by advancing one hand auger boring to a depth of 10 feet bgs, and observing the excavation of nine test pits to depths of up to about 8½ feet bgs. Details of the subsurface investigation are presented in Appendix A.
- Conduct infiltration testing within two of the test pits. Results of the infiltration testing are presented in Appendix B.
- Classify the soils encountered in the explorations in general accordance with ASTM D2488 (Visual-Manual Procedure).
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.

- Provide recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a qualitative evaluation of seismic hazards at the site, including earthquake-induced liquefaction, landsliding, and surface rupture due to faulting or lateral spread.
- Provide geotechnical recommendations for site preparation and earthwork.
- Provide geotechnical engineering recommendations for use in design and construction of shallow foundations, floor slabs, site retaining walls, and pavements.
- Provide this written report summarizing the results of our geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Site Geology

Based on available geologic mapping[1,2] of the area, the site is underlain by Quaternary sediments consisting of unconsolidated, alluvial and estuarine clay, silt, sand, and gravel deposited along rivers and streams. Nearby cross sections and well logs suggest the Quaternary sediments are about 20 to 30 feet thick in the vicinity of the site and are underlain by Oligocene to Miocene aged sedimentary rocks (Unit Toms). The sedimentary rocks unit consists of thin- to mass-bedded, gray, tuffaceous siltstone and claystone with localized sandstone and shale. This sedimentary rock unit is very thick, extending to depths up to 5,000 feet below the site surface.

2.2 Site Surface Conditions

The site is bordered to the north, south, and east by undeveloped properties, and to the west by a newer residential development (under construction). At the time of our field investigation, the site gently descended to the south, and was generally vegetated with grasses, shrubs, and scattered coniferous and deciduous trees. The northeast portion of the site was densely vegetated with coniferous and deciduous trees. Site layout and surface conditions at the time of our field investigation are shown on the attached Site Plan (Figure 2) and Site Photographs (Figure 3).

2.3 Subsurface Conditions

2.3.1 Subsurface Investigation & Laboratory Testing

Our subsurface investigation consisted of one hand auger boring (HA-1) and nine test pits (TP-1 through TP-9) completed at the site on March 31, 2023. The approximate exploration locations are shown on the Site Plan, attached as Figure 2. In summary, the explorations extended to depths ranging from about 5 to 10 feet bgs. Details regarding the subsurface investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

2.3.2 Subsurface Materials

Logs of the explorations are presented in Appendix A. The following describes each of the subsurface materials encountered at the site.

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[1] Wells, R.E., Niem, A.R., MacLeod, N.S., Snavely, P.D., and Niem, W.A., 1983, Geologic Map of the West Half of the Vancouver 1ºx2º Quadrangle, Oregon: United States Geologic Survey, Open File Report, 83-59I, scale 1:250,000.

[2] Schlicker, H.G., Deacon, R.J., Beaulieu, J.D., and Olcott, G.W., 1972, Environmental geology of the coastal region of Tillamook and Clatsop Counties: Oregon Department of Geology and Mineral Industries, Bulletin 74, scale 1:62,500.

Sandy Organic Soil (OL)

Sandy organic soil was encountered at the surface of boring HA-1 and each test pit, and extended to a depth of about ½ foot bgs. This soil was generally brown to dark brown, moist, and contained abundant roots up to ½ inch in diameter, and fine- to medium-grained sand.

Poorly Graded Sand (SP)

Poorly graded sand was encountered below the organic soil in HA-1 and each test pit. This soil was generally loose to medium (based on digging effort), light brown to brown with orange and gray mottling, moist, fine- to medium-grained, and contained trace roots up to 1 inch in diameter. Minor to severe caving was observed below about 4 to 7 feet bgs within HA-1 and TP-1 through TP-9. The poorly graded sand extended the full depths explored at the site, about 5 to 10 feet bgs.

2.3.3 Groundwater

Groundwater was not encountered within the depths explored at the site on March 31, 2023. To determine approximate regional groundwater levels in the area, we researched well logs available on the Oregon Water Resources Department (OWRD)[3] website for wells located within Section 28, Township 3 North, Range 10 West, Willamette Meridian. Our review indicated that groundwater levels in the area generally ranged from about 30 to 50 feet bgs. More shallow water zones were reported at depths of about 17 feet bgs. It should be noted groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors.

3.0 SEISMIC CONSIDERATIONS

3.1 Seismic Design

Section 1613.2.2 of the 2022 Oregon Structural Specialty Code (2022 OSSC) requires that the determination of the seismic site class be in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). We have assigned the site as Site Class D (“Stiff Soil”) based on geologic mapping and subsurface conditions encountered during our investigation.

Earthquake ground motion parameters for the site were obtained in accordance with the 2022 OSSC using the Seismic Hazards by Location calculator on the ATC website. The site Latitude 45.716955° North and Longitude 123.922144° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

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[3] Oregon Water Resources Department, 2023. Well Log Records, *accessed April 2023*, from OWRD web site: http://apps.wrd.state.or.us/apps/gw/well_log/.

Table 1 Seismic Ground Motion Values

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	1.271g
	Spectral Acceleration, 1.0 second (S_1)	0.668g
Coefficients (Site Class D)	Site Coefficient, 0.2 second (F_A)	1.000
	Site Coefficient, 1.0 second (F_V)[1]	1.700
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 second (S_{MS})	1.271g
	MCE Spectral Acceleration, 1.0 second (S_{M1})	1.136g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 second (S_{DS})	0.847g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.757g
Seismic Design Category (Risk Category II)		D
[1] Value determined from 2022 OSSC Table 1613.2.3(2).		

3.2 Seismic Hazards

3.2.1 Liquefaction

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, pore water pressures can increase, approaching the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. When the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil approaches zero, and the soil can liquefy. The liquefied soils can undergo rapid consolidation or, if unconfined, can flow as a liquid. Structures supported by the liquefied soils can experience rapid, excessive settlement, shearing, or even catastrophic failure.

For fine-grained soils, susceptibility to liquefaction is evaluated based on penetration resistance and plasticity, among other characteristics. Criteria for identifying non-liquefiable, fine-grained soils are constantly evolving. Current practice to identify non-liquefiable, fine-grained soils is based on moisture content and plasticity characteristics of the soils[4,5,6]. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on penetration resistance, as measured using SPTs, CPTs, or Becker Hammer Penetration tests (BPTs).

As indicated in Section 2.3.3 above, groundwater was not encountered within the depths explored at the site on March 31, 2023. Additionally, review of well logs available on the OWRD website for wells located within the vicinity of the site indicated that groundwater levels in the area generally ranged from about 30 to 50 feet bgs. Based on the lack of saturated conditions, static groundwater, etc., the soils encountered within our explorations are considered non-liquefiable. Based on our previous experience in the area, we do not anticipate liquefiable conditions are present at depths below those explored as part of this assignment.

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[4] Seed, R.B. et al., 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. Earthquake Engineering Research Center Report No. EERC 2003-06.

[5] Bray, Jonathan D., Sancio, Rodolfo B., et al., 2006. Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, September 2006.

[6] Idriss, I.M., Boulanger, R.W., 2008. Soil Liquefaction During Earthquakes, Earthquakes Engineering Research Institute Monograph MNO-12.

3.2.2 Slope Instability

Review of the Statewide Landslide Information Database for Oregon (SLIDO), available at the DOGAMI website[7], shows no prehistoric or historic landslides on the project site. Pre-historic (over 150 years) landslides are mapped about 750 feet to the north of the site. No obvious signs of recent or on-going slope instability were observed at the site during our field investigation in March 2023. Recognizing the relatively gentle site grades, and provided the recommendations presented later in this report regarding grading are incorporated into design and development, the risk of seismically-induced landslides at the site is considered low.

3.2.3 Surface Rupture

3.2.3.1 Faulting

Although the site is situated in a region of the country with known active faults and historic seismic activity, no known faults exist on or immediately adjacent to the site. Therefore, the risk of surface rupture at the site due to faulting is considered negligible.

3.2.3.2 Lateral Spread

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Given the lack of liquefiable soils, the risk of surface rupture due to lateral spread is considered very low.

4.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1 of this report, provided the recommendations presented in this report are incorporated into the design and development. Satisfactory subgrade support for planned shallow foundations, floor slabs, and pavements can be achieved by the native, near-surface, poorly graded sand (SP) or structural fill that is properly placed and compacted on that material during construction. The native poorly graded sand was encountered at depths of about ½-foot bgs in our explorations. Geotechnical recommendations for use in design and construction of the proposed project are presented in the following section of this report.

5.0 RECOMMENDATIONS

The recommendations presented in this report are based on the information provided to us, results of our field investigation and analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

5.1 Site Preparation

5.1.1 Stripping & Grubbing

Existing vegetation, topsoil, and rooted soils (OL) should be removed from within, and for a minimum 5-foot margin around, proposed building pad, structural fill, and pavement areas. Based on the results of our field

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[7] Oregon Department of Geology and Mineral Industries, 2023. Statewide Landslide Information Database for Oregon (SLIDO), *accessed April 2023*, from DOGAMI web site: https://gis.dogami.oregon.gov/maps/slido/.

explorations, topsoil stripping depths are anticipated to be on the order of about ½ foot bgs. These materials may be deeper or shallower at locations away from the completed explorations. The geotechnical engineer's representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and rooted soils should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

Grubbing of trees should include the removal of the root mass and roots greater than ½ inch in diameter. Grubbed materials should be transported off-site for disposal. Root masses from larger trees may extend greater than 3 feet bgs. Where root masses are removed, the resulting excavation should be properly backfilled with structural fill in conformance with Section 5.4 of this report.

Any areas in which densely-rooted soils are encountered should be scarified to a minimum depth of 12 inches below the current (prepared) site grades using suitable earthwork equipment (such as "ripping" blades on a bulldozer). This should be performed within, and for a 5-foot margin around (where feasible), the proposed structural fill areas, building pads, and pavement areas. The purpose of this earthwork is to help remove any remaining large and/or heavy concentrations of tree roots. Where encountered, heavy concentrations of organics and/or roots in excess of 1 inch in diameter should be removed (processed) from the scarified subgrade. Following the root processing, the scarified subgrade should be moisture conditioned and compacted to at least 90 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor).

5.1.2 Test Pit Backfills

The test pits conducted at the site were loosely backfilled during our field investigation. Where test pits are located within finalized building, structural fill, or pavement areas, the loose backfill materials should be re-excavated. The resulting excavations should be backfilled with structural fill in conformance with Section 5.4 of this report.

5.1.3 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new buildings, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 5.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 5.4 of this report.

5.1.4 Subgrade Preparation - Building Pads & Pavement Areas

After site stripping as recommended above, but prior to placement of structural fill or base rock, the prepared sandy subgrade soils should be surface compacted with suitable equipment (e.g. smooth drum roller). The subgrade soils should be compacted to not less than 90 percent of the material's maximum dry density as determined by ASTM D1557 (Modified Proctor). The geotechnical engineer or his representative should perform in-place density testing of the compacted subgrade to confirm proper compaction. If areas of soft soil or excessive yielding are identified, the affected material should be repaired as recommended by the geotechnical engineer or his representative.

5.1.5 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

5.2 Temporary Excavations

5.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts as described earlier in this report. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person,” as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT’s current role on the project does not include review or oversight of excavation safety.

5.2.2 OSHA Soil Type

For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type “C” should be used for the poorly graded sand (SP) encountered at the site. As evidenced in several of the test pits, caving of excavations extending beyond depths of about 5 feet bgs should be expected.

5.2.3 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native, poorly graded sand encountered near the surface of the site. As evidenced in several of the test pits, caving of trench cuts extending beyond depths of about 5 feet bgs should be expected. If groundwater seepage undermines the stability of the trench, or if sidewall caving is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. If groundwater is encountered, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 5.4.3.

5.2.4 Excavations Near Foundations

Excavations near footings should not extend within a 1½ horizontal to 1 vertical (1½H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

5.3 Wet Weather Considerations

Due to its very low concentration of fine-grained particles (i.e. silt or clay), the native poorly graded sand (SP) is not considered susceptible to disturbance from wet weather. However, sandy soils are susceptible to raveling under construction traffic and may result in loosening of the surface sands. If the soils become loose due to construction traffic, they should be moisture-conditioned (as necessary) and compacted to a well-keyed condition in accordance with Section 5.1.4 of this report.

5.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[8]. The geotechnical engineer's representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed.

5.4.1 On-Site Soils – General Use

5.4.1.1 Poorly Graded Sand (SP)

Re-use of the on-site, relatively clean, poorly graded sand as structural fill is feasible, provided the material is kept clean of organics, debris, and particles larger than 1½ inches in diameter. If reused as structural fill, the material should be prepared in general accordance with Section 5.4.2 below.

If the on-site materials cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

5.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 90 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by proof roll test observation (deflection tests), where accepted by the geotechnical engineer.

5.4.3 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

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[8] Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

5.4.4 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 2 Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas^{1,2}	Landscaping Areas
Pipe Base and Within Pipe Zone	88% ASTM D1557 or pipe manufacturer's recommendation	85% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	90% ASTM D1557	88% ASTM D1557
Within 3 Feet of Design Subgrade	90% ASTM D1557	88% ASTM D1557

[1] Includes proposed buildings, pavement areas, structural fill areas, exterior hardscaping, etc.

[2] Or as specified by the local jurisdiction where located in the public right of way.

5.4.5 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

5.5 Permanent Slopes

5.5.1 Overview

Permanent cut or fill slopes constructed at the site, if any, should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

5.5.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where existing (native) slopes exceed 5H:1V, the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 4. If subdrains are needed on benches, subject to the review of the

CGT geotechnical representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well-compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. A representative from CGT should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

5.6 Shallow Foundations

5.6.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations can be obtained from the native, near-surface, poorly graded sand (SP), or new structural fill that is properly placed and compacted on that material during construction. Due to its generally loose near-surface relative density, the native sandy soils should be moisture-conditioned (as necessary) and surface compacted using suitable equipment (e.g. jumping jack compactor, vibrating plate compactor, etc.) until achieving a well-keyed condition.

The geotechnical engineer's representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, excessively loose, organic-laden, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 5.4.2. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

5.6.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the most recent Oregon Structural Specialty Code (OSSC). As a guideline, CGT recommends individual spread footings have a minimum width of 24 inches. For one-story, light-framed structures, we recommend continuous wall footings have a minimum width of 12 inches. Similarly, for two-story, light-framed structures, we recommend continuous wall footings have a minimum width of 15 inches. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade for frost protection.

5.6.3 Horizontal Setback from Descending Slopes

Foundations constructed within or near descending slopes should be setback a minimum of 5 feet from the slope surface. This distance should be measured between the face of the slope and the bottom, outside edge of the respective foundation. Organic topsoil and loose surface soils (if present) should not be included when determining this distance. The geotechnical engineer or his representative should be contacted to observe foundation subgrade conditions and confirm this recommended minimum setback is achieved.

5.6.4 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 1,500 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. Differential settlements between adjacent columns and/or bearing walls should not exceed ½ inch. If an increased allowable soil bearing pressure is desired, the geotechnical engineer should be consulted.

5.6.5 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.40 may be used when calculating resistance to sliding for footings founded on the native sandy soils described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

5.7 Rigid Retaining Walls

5.7.1 Footings

Retaining wall footings should be designed and constructed in conformance with the recommendations presented in Section 5.6, as applicable.

5.7.2 Wall Drains

We recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

5.7.3 Wall Backfill

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 5.4.2 and contain less than 5 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

5.7.4 Design Parameters & Limitations

For rigid retaining walls founded, backfilled, and drained as recommended above, the following table presents parameters recommended for design.

Table 3 Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Seismic Equivalent Fluid Pressure (S_{AE}) [1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i=0)	28 pcf	42 pcf	0.22*q
Restrained from Rotation	Level (i=0)	50 pcf	63 pcf	0.38*q

[1] Refer to the attached Figure 5 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual. Static and seismic equivalent fluid pressures are <u>not</u> additive.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 5).
- The walls are 10 feet or less in height.
- The backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
- No point, line, or strip load surcharges are imposed behind the walls.
- The grade behind the wall is level, or sloping down and away from the wall, for a distance of 10 feet or more from the wall.
- The grade in front of the walls is level or ascending for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

5.8 Floor Slabs

5.8.1 Subgrade Preparation

Satisfactory subgrade support for slabs constructed on grade, supporting up to 150 psf area loading, can be obtained from the native, near-surface, poorly graded sand (SP), or new structural fill that is properly placed and compacted on that material during construction. Due to its generally loose near-surface relative density, the native sandy soils should be moisture-conditioned (as necessary) and surface compacted using suitable equipment (e.g. vibrating plate compactor, smooth drum roller, etc.) until achieving a well-keyed condition.

The geotechnical engineer's representative should observe floor slab subgrade soils to evaluate surface relative densities. If soft, excessively loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by CGT geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill as described in Section 5.4.2 of this report.

5.8.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 4-inch-thick layer of crushed rock (base rock).

5.8.2.1 Conventional Base Rock

Floor slab base rock should consist of well-graded granular material (crushed rock) containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Floor slab base rock should be placed in one lift and compacted to not less than 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). We recommend "choking" the surface of the base rock with sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing. Choking the base rock also reduces punctures in vapor retarding membranes due to foot traffic where such membranes are used.

5.8.2.2 Gas Permeable Base Rock

Floor slab base rock in areas where radon gas mitigation is desired should consist of open-graded crushed rock containing no organic matter or debris, with all material passing through a 1-inch sieve, less than 10 percent passing the ½-inch sieve, no fines (0 percent passing the U.S. Standard No. 200 sieve), and a free void space of approximately 50 percent in accordance with Section 1811.2.1.1 of the 2022 OSSC.

CGT recommends that a minimum 10-mil polyethylene sheeting or equivalent material with equal or greater tensile strength, resistance to puncture, resistance to deterioration, and resistance to water-vapor transmission be placed on top of the gas-permeable base rock to act as a soil-gas-retarder. Placement and installation of this sheeting should be in conformance with that indicated in Section 1811.2.2 of the 2022 OSSC.

5.8.3 Design Considerations

For floor slabs constructed with a 4-inch thick base rock layer as recommended, an effective modulus of subgrade reaction of 200 pounds per cubic inch (pci) is recommended for the design of the floor slab. A higher effective modulus of subgrade reaction can be obtained by increasing the base rock thickness. Please contact the geotechnical engineer for additional recommendations if a higher modulus is desired. Floor slabs constructed as recommended will likely settle less than ½ inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

5.8.4 Subgrade Moisture Considerations

Liquid moisture and moisture vapor should be expected at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized, e.g. impervious floor coverings, storage of moisture sensitive materials directly on the slab surface, etc., a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the architect and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. Please note that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

5.9 Pavements

5.9.1 Subgrade Preparation

Pavement subgrade preparation should be performed in general accordance with the recommendations presented in Section 5.1.4 above. Subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

5.9.2 Traffic Levels

Recognizing that traffic data has not been provided, CGT has considered three levels of traffic demand for review and design of pavement sections. We modeled the following three design cases (traffic levels) developed from the Asphalt Pavement Association of Oregon (APAO):

- *APAO Level I (Very Light):* This design case considers typical average daily truck traffic (ADTT) of 1 per day over 20 years. Among others, examples under this loading consist of passenger car parking stalls, residential driveways, and seasonal recreational roads.
- *APAO Level II (Light):* This design case considers typical ADTT of 2 to 7 per day over 20 years. Examples under this loading consist of residential streets and parking lots of less than 500 stalls.
- *APAO Level III (Low Moderate):* This design case considers typical ADTT of 7 to 14 per day over 20 years. Among others, examples under this loading consist of urban minor collector streets and parking lots with more than 500 stalls.

5.9.3 Input Parameters

Our asphalt concrete (AC) pavement section designs were based on the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual. A number of design assumptions and variables were required in order to develop design sections for pavements proposed at the site. The following table presents the input parameters assumed for the design:

Table 4 Input Parameters Used in AC Pavement Design

Input Parameter	Design Value[1]	Input Parameter		Design Value[1]
Pavement Design Life	20 years	Resilient Modulus	Subgrade (Compacted Sand)[3]	10,000 psi
Annual Percent Growth	0 percent		Crushed Aggregate Base	20,000 psi
Initial Serviceability	4.2 initial	Structural Coefficient	Crushed Aggregate Base	0.10
Terminal Serviceability	2.5 terminal		Asphalt	0.42
Reliability	75 percent	Vehicle Traffic[4] (range in ESAL)	Level I (Very Light)	Less than 10,000
Standard Deviation	0.49		Level II (Light)	Less than 50,000
Drainage Factor[2]	1.0		Level III (Low Moderate)	Less than 100,000

[1] If any of the above parameters are incorrect, please contact us so that we may revise our recommendations, if warranted.

[2] Assumes good drainage away from pavement, base, and subgrade is achieved by proper crowning of subgrades.

[3] Values based on experience with similar soils.

[4] ESAL = Total 18-Kip equivalent single axle load. Refer to Section 5.9.2 for additional discussion. If actual traffic levels will be above those identified above, the geotechnical engineer should be consulted.

5.9.4 Recommended Minimum Sections

The following table presents the minimum AC pavement sections for the traffic loads indicated in the preceding table, based on the referenced AASHTO procedures.

Table 5 Recommended Minimum AC Pavement Sections

Material	**Level I (Very Light Traffic)**	**Level II (Light Traffic)**	**Level III (Low Moderate Traffic)**
Asphalt Pavement (inches)	3	3½	4
Crushed Aggregate Base (inches)	4	6	6
Subgrade Soils	Prepared in conformance with Section 5.6.1 of this report.		

5.9.5 Pavement Materials

We recommend pavement aggregate base consist of dense-graded aggregate in conformance with Section 02630.10 of the most recent ODOT SSC, with the following additional considerations. We recommend the material consist of crushed rock or gravel, have a maximum particle size of 1½ inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Aggregate base should be compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor).

We recommend asphalt pavement consist of Level 2, ½-inch, dense-graded AC in conformance with the most recent ODOT SSC. Asphalt pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity).

5.10 Additional Considerations

5.10.1 Drainage

Subsurface drains, if incorporated, should be connected to the nearest storm drain, on-site infiltration system (to be designed by others) or other suitable discharge point. Paved surfaces and grading near or adjacent to the buildings should be sloped to drain away from the buildings. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains (if incorporated), retaining wall drains, or onto site slopes.

5.10.2 Expansive Potential

The near surface native soils consist of non-plastic sandy soils. These soils are not considered to be susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at the site.

6.0 RECOMMENDED ADDITIONAL SERVICES

6.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

6.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend geotechnical engineer's representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer's representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping and Grubbing
- Subgrade Preparation for Shallow Foundations, Retaining Walls, Structural Fills, Floor Slabs, and Pavements
- Compaction of Structural Fill, Retaining Wall Backfill, and Utility Trench Backfill
- Compaction of Base Rock for Floor Slabs and Pavements
- Compaction of Asphalt Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

7.0 LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

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

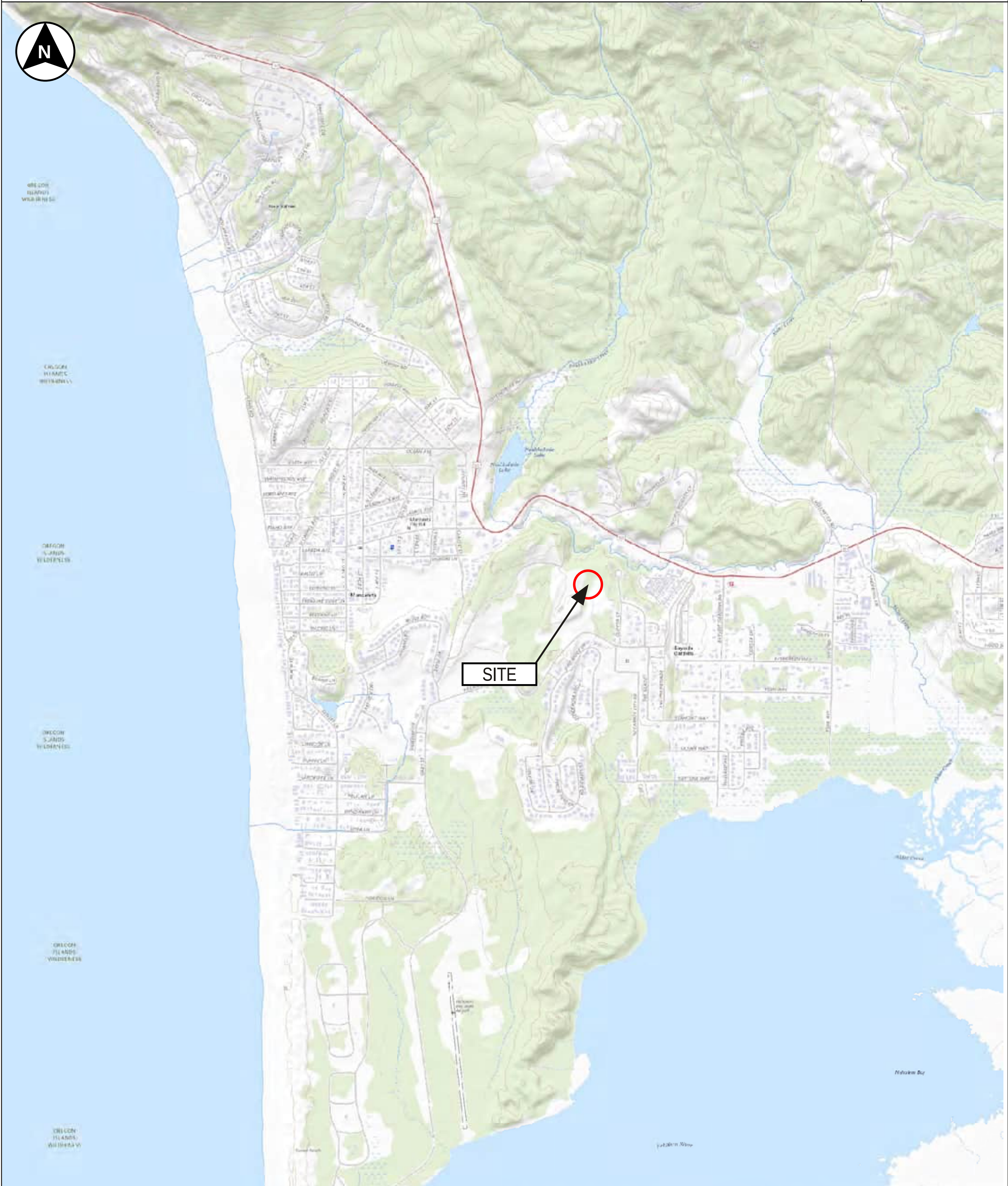
Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

HFD-GLD MANZANITA HOUSING - TILLAMOOK COUNTY, OREGON

Project Number G2305878

FIGURE 1

Site Location



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Drafted by: BLN

USGS Topographic base map created with The National Map, 2022, at https://viewer.nationalmap.gov/advanced-viewer/

Township 3 North, Range 10 West, Section 28, Willamette Meridian

Latitude: 45.716955° North
Longitude: 123.922144° West

1 Inch = 2,000 feet

0 2000 4000

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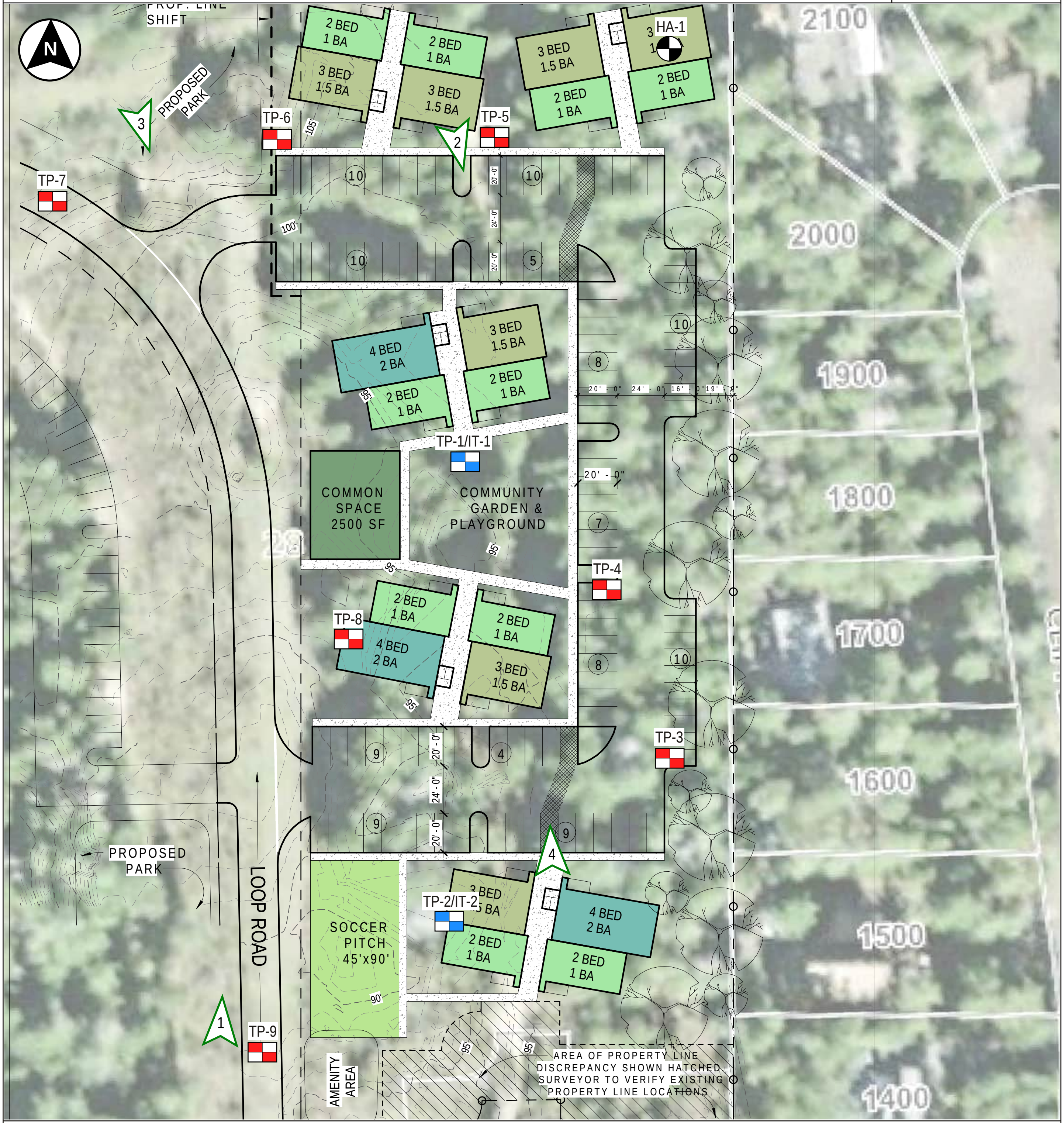
Project Number G2305878

FIGURE 2

Site Plan



LEGEND

HA-1 ◐ Hand auger boring

TP-3 🟥 Test pit exploration

TP-1/IT-1 🟦 Test pit exploration and infiltration test

△1 Orientation of site photographs shown on Figure 3

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Drafted by:BN/BG

NOTES: Drawing based on observations made while on site and site plans provided by client. All locations are approximate.

1 Inch = 60 Feet

0 60 120

Photograph 1

Photograph 2

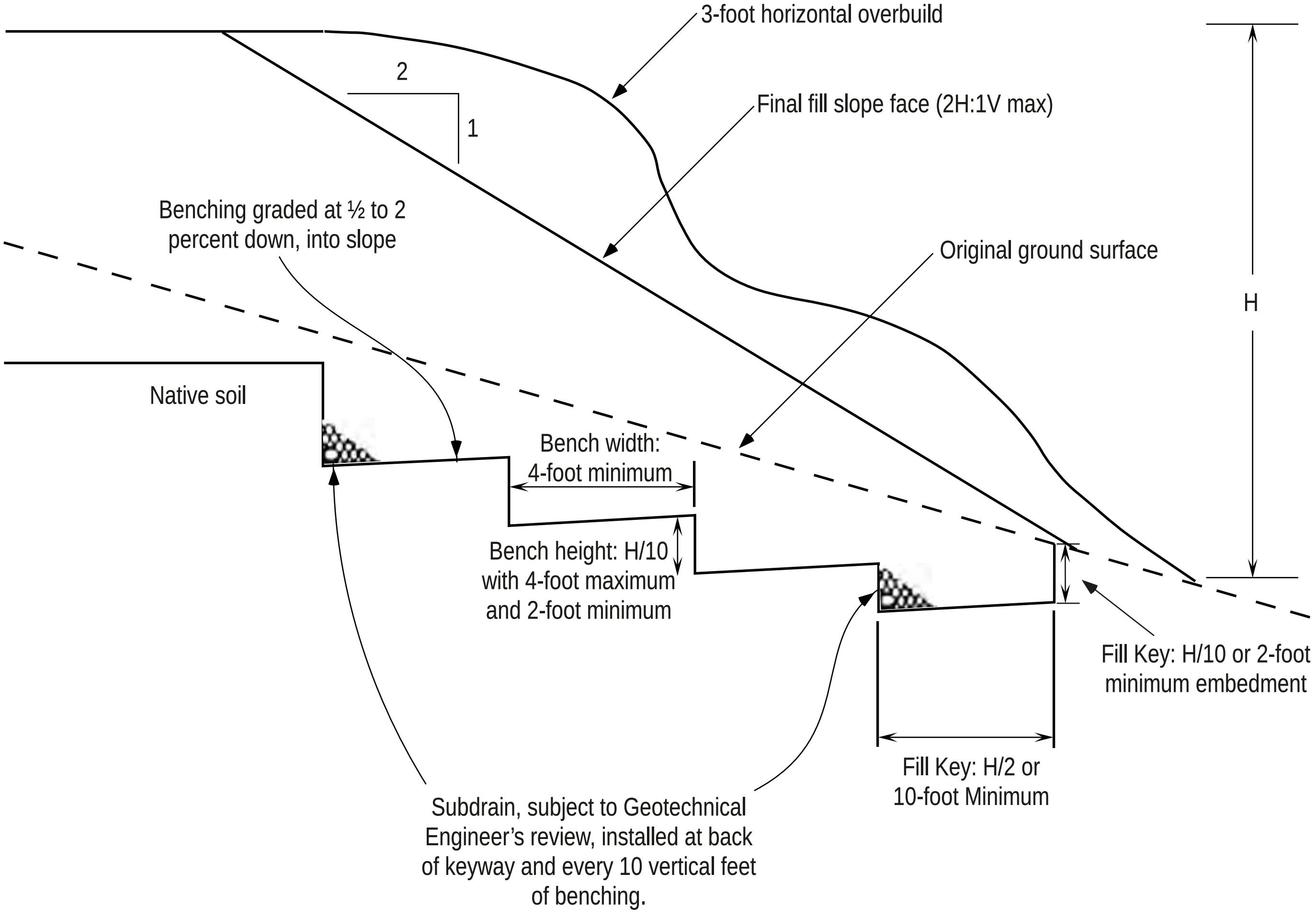
Photograph 3

Photograph 4

See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

FIGURE 4

Fill Slope Detail



NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

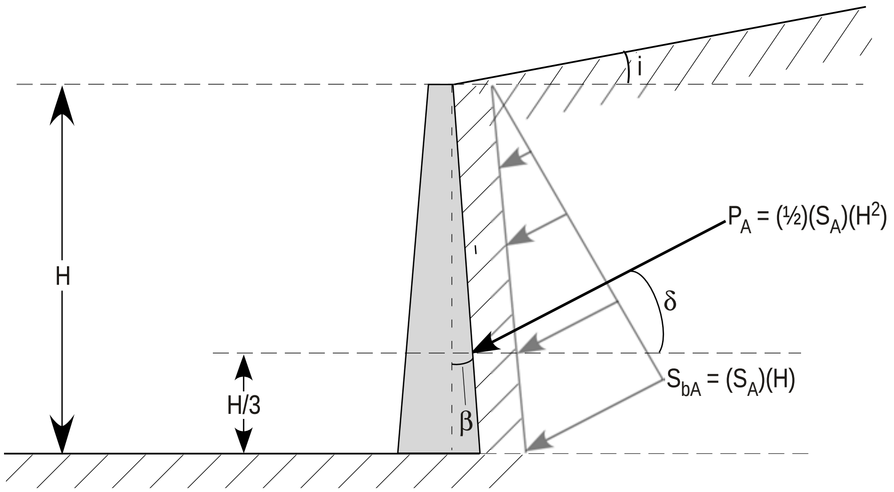
HFD-GLD MANZANITA HOUSING - TILLAMOOK COUNTY, OREGON
Project Number G2305878

FIGURE 5

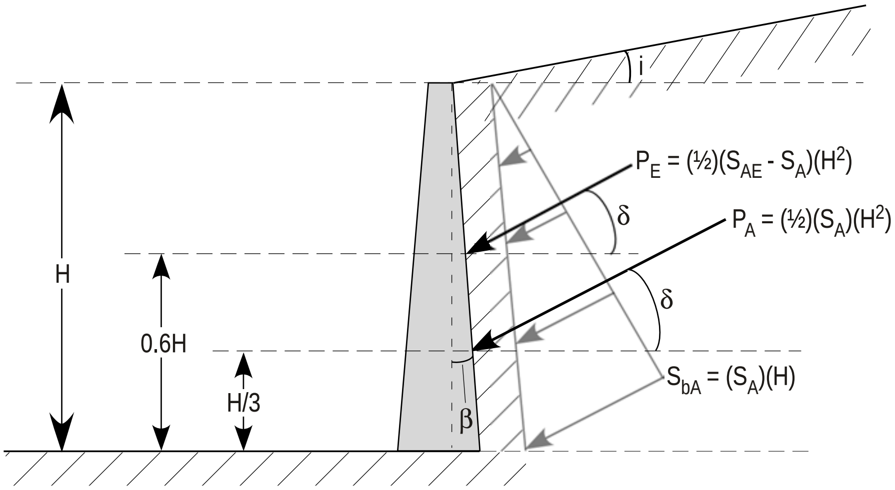
Retaining Walls

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

S_A = Active lateral equivalent fluid pressure (lb/ft^3)*

S_{bA} = Active lateral earth pressure (static) at the bottom of wall (lb/ft^3)

S_{AE} = Active total (static + seismic) equivalent fluid pressure (lb/ft^3)*

i = Slope of backfill, relative to horizontal (degrees)**

β = Slope of back of wall, relative to vertical (degrees)**

P_A = Static active thrust force acting at H/3 from bottom of retaining wall (lb/ft)

P_E = Dynamic active thrust force acting at 0.6H from bottom of retaining wall (lb/ft)

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al, 1982 and Whitman, 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman, 1990].

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Appendix A: Subsurface Investigation and Laboratory Testing

HFD-GLD Manzanita Housing
Tax Lot 1401
Tillamook County, Oregon

CGT Project Number G2305878

April 14, 2023

Prepared For:

Green Light - Home First, LLC
Attn: Rob Justus
3050 SE Division Street, Suite 270
Portland, Oregon 97202

Prepared by
Carlson Geotechnical

A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of one hand auger boring and nine test pits completed at the site on March 31, 2023. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. The exploration locations were recorded in the office using desktop GIS software and located in the field using a cellular telephone, and are approximate (+/- 30 feet horizontally). Surface elevations indicated on the logs were estimated based on the topographic contours (by others) shown on the referenced Site Plan and are approximate. The attached figures detail the exploration methods (Figure A1), soil classification criteria (Figure A2), and present detailed logs of the explorations (Figure A3 through A12), as discussed below.

A.1.1 Hand Auger Borings

CGT advanced one hand auger boring (HA-1) to a depth of about 10 feet bgs. The boring was advanced using a manual, 3-inch-diameter hand auger. The hand auger boring was loosely backfilled with the excavated materials upon completion.

A.1.2 Test Pits

CGT observed the excavation of nine test pits (TP-1 through TP-9) at the site to depths of about 5 to 8½ feet bgs. The test pits were excavated using a John Deere 35G mini-excavator provided and operated by our excavation subcontractor, Doug Shepherd's Dirtworks of Keizer, Oregon. The test pits were loosely backfilled with the excavated materials upon completion.

A.1.3 In-Situ Testing

A.1.3.1 Dynamic Cone Penetrometer Test

In conjunction with the hand auger boring, we advanced one dynamic cone penetrometer test to a depth of 11 feet bgs. The test was performed using a Wildcat Dynamic Cone Penetrometer (WDCP) provided and operated by CGT. The WDCP test is described on the attached Exploration Key, Figure A1. Results of the WDCP test are provided on the log for boring HA-1.

A.1.3.2 Infiltration Tests

CGT performed two infiltration tests (IT-1 and IT-2) at the site within test pits TP-1 and TP-2, respectively, at a depth of about 5 feet bgs. Details regarding the test procedure and results of the tests are presented in Appendix B.

A.1.4 Material Classification & Sampling

Representative disturbed (grab) samples of the soils encountered were obtained at selected intervals within the test pits and hand auger boring. Qualified members of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). An explanation of this classification system is attached as Figure A2. The samples were stored in sealable plastic bags and transported to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in order to refine the initial field classifications.

A.1.5 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figure A3 through A12.

A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing included the following:

- Ten moisture content determinations (ASTM D2216).
- Two percentage passing the U.S. Standard No. 200 Sieve tests (ASTM D1140).

Results of the laboratory tests are shown on the exploration logs.

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Project Number G2305878

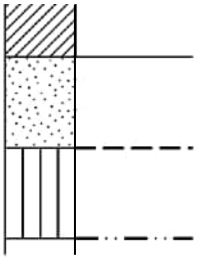
FIGURE A1
Exploration Key

PL ● LL MC	Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216)
☐ FINES CONTENT (%)	Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140)

SAMPLING

GRAB	Grab sample
BULK	Bulk sample
SPT	**Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and N_{60} are noted on the boring logs.
MC	**Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM G3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT N_{60} value per Lacroix and Horn, 1973.
CORE	**Rock Coring** interval
SH	**Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils.
WDCP	**Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT N_{60} values.
DCP	**Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer.
POCKET PEN. (tsf)	**Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils.

CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

ADDITIONAL NOTATIONS

Italics	Notes drilling action or digging effort
{ Braces }	Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt })

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All measurements are approximate.

HFD-GLD MANZANITA HOUSING - TILLAMOOK COUNTY, OREGON
Project Number G2305878

FIGURE A2
Soil Classification

Classification of Terms and Content

NAME: Group Name and Symbol
Relative Density or Consistency
Color
Moisture Content
Plasticity
Other Constituents
Other: Grain Shape, Approximate Gradation
Organics, Cement, Structure, Odor, etc.
Geologic Name or Formation

Grain Size

		U.S. Standard Sieve
Fines		<#200 (0.075 mm)
Sand	Fine	#200 - #40 (0.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75 mm)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches
Boulders		> 12 inches

Coarse-Grained (Granular) Soils

Relative Density

SPT N_{60}-Value	Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
>50	Very Dense

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace silt"
5 - 15%	"With" as part of group name	"**POORLY GRADED SAND WITH SILT**"
15 - 49%	Modifier to group name	"**SILTY SAND**"

Fine-Grained (Cohesive) Soils

SPT N_{60}-Value	Torvane tsf Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
<2	<0.13	<0.25	Very Soft	Thumb penetrates more than 1 inch
2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace fine-grained sand"
5 - 15%	"Some" as part of soil description	"some fine-grained sand"
15 - 30%	"With" as part of group name	"**SILT WITH SAND**"
30 - 49%	Modifier to group name	"**SANDY SILT**"

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

Visual-Manual Classification

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel/sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel/sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel/sand/silt mixtures
			GC	Clayey gravels, gravel/sand/clay mixtures
	Sands: More than 50% *passing* the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand/silt mixtures
			SC	Clayey sands, sand/clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic soil of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic soil of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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References:
ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.

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FIGURE A3

Boring HA-1

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CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 110 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 50ºF
SURFACE Sand
LOGGED BY BJG
REVIEWED BY BMW
DRILLING CONTRACTOR CGT
SEEPAGE ---
EQUIPMENT Manual Hand Auger & WDCP
GROUNDWATER DURING DRILLING ---
DRILLING METHOD Manual Hand Auger
GROUNDWATER AFTER DRILLING ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft) 0	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
		OL	**SANDY ORGANIC SOIL**: Loose, dark brown, moist, and contained abundant rootlets/roots up to ¼-inch in diameter, and fine- to medium-grained sand.					2		
								5		
			POORLY GRADED SAND: Loose, tan with orange mottling, moist, and contained some rootlets within the upper 6 inches.					5		
								6		
								10		
108					2			10		
			Medium dense below about 2 feet bgs.					11		
								11		
								11		
								12		
								11		
106					4			11		
			Loose below about 4 feet bgs					8		
								9		
								8		
		SP				GRAB 1	100	8		
								5		
104					6			6		
								4		
								4		
								3		
			Minor caving below about 7 feet bgs.					2		
								3		
102					8			3		
								3		
								4		
								4		
								4		
								4		
100					10	GRAB 2	100	4		
								5		
			• Boring terminated at 10 feet bgs. • Minor caving encountered below about 7 feet bgs. • No groundwater encountered. • Boring loosely backfilled with excavated materials upon completion.					6		
								6		
								8		
98										

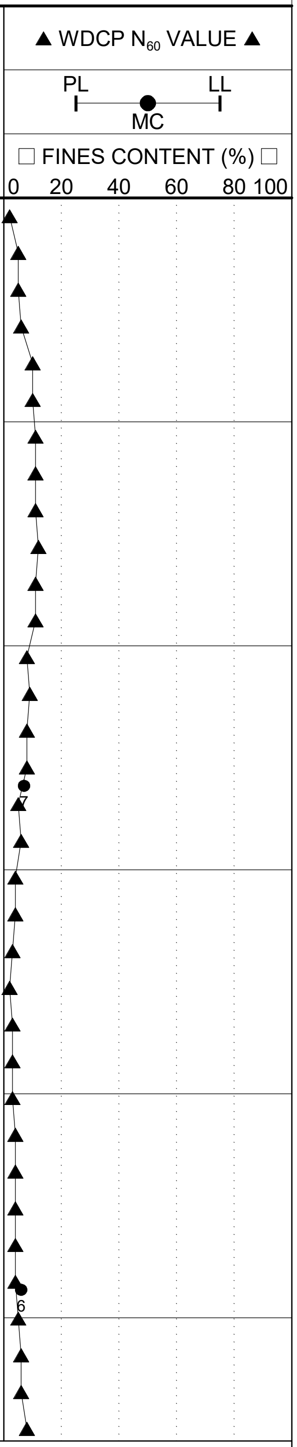

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FIGURE A4

Test Pit TP-1

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CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 94 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 50ºF
SURFACE Sand
LOGGED BY AET
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
					0						
		OL	**SANDY ORGANIC SOIL**: Dark gray, moist, and contained abundant rootlets/roots up to ¼-inch in diameter and fine- to medium-grained sand.								
		SP	**POORLY GRADED SAND**: *Loose*, brown with gray mottling, moist, fine- to medium-grained, and contained trace roots up to 1 inch in diameter.								
92					2						
						GRAB 1	100				
			Light gray below about 3 feet bgs.								
90					4						
			No roots below 4 feet bgs.								
						GRAB 2	100				1 5
88											
86											
84											

- Test pit terminated a 5 feet bgs.
- Infiltration test conducted at 5 feet bgs. Refer to Appendix B for test results.
- No caving or groundwater encountered.
- Test pit loosely backfilled with excavated materials upon completion.

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FIGURE A5

Test Pit TP-2

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CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 94 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 50ºF
SURFACE Sand
LOGGED BY AET
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ●MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**SANDY ORGANIC SOIL**: Light gray, moist, and contained abundant rootlets/roots up to ¼-inch in diameter, and fine- to medium-grained sand.		0						
		SP	**POORLY GRADED SAND**:*Loose*, brown with gray mottling, moist, fine- to medium-grained, and contained trace roots up to 1 inch in diameter.								
92					2						
			Light gray with brown mottling below about 3 feet bgs.			GRAB 1	100				
90					4						
						GRAB 2	100				☐ 1 ● 7
88			• Test pit terminated a 5 feet bgs. • Infiltration test conducted at 5 feet bgs. Refer to Appendix B for test results. • No caving or groundwater encountered. • Test pit loosely backfilled with excavated materials upon completion.								
86											
84											

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FIGURE A6

Test Pit TP-3

CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon

DATE STARTED 3/31/23
GROUND ELEVATION 98 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 50ºF
SURFACE Sand
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
					0						
		OL	**SANDY ORGANIC SOIL**: Brown, moist, and contained some rootlets, and fine- to medium-grained sand.								
		SP	**POORLY GRADED SAND**: *Loose*, tan with orange mottling, moist, and fine- to medium-grained.								
96					2						
94					4						
			Minor caving below about 5 feet bgs.								
92					6						
			Severe caving below about 7 feet bgs.								
90					8	GRAB 1	100				● 6
88											

- Test pit terminated at 8 feet bgs due to caving.
- Minor to severe caving encountered below about 5 to 7 feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

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FIGURE A7

Test Pit TP-4

PAGE 1 OF 1

CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 96 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 50ºF
SURFACE Sand
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**SANDY ORGANIC SOIL**: Brown, moist, and contained some rootlets, and fine- to medium-grained sand.		0						
		SP	**POORLY GRADED SAND**: *Loose*, tan with orange mottling, moist, and fine- to medium-grained.								
94					2						
92					4						
						GRAB 1	100				● 5
90			Minor caving below about 6 feet bgs.		6						
			Severe caving below about 7 feet bgs.			GRAB 2	100				
88											
86											

- Test pit terminated at 7½ feet bgs due to caving.
- Minor to severe caving encountered below about 6 to 7 feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

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FIGURE A8

Test Pit TP-5

CLIENT Green Light - Home First, LLC

PROJECT NAME HFD-GLD Manzanita Housing

PROJECT NUMBER G2305878

PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon

DATE STARTED 3/31/23

GROUND ELEVATION 102 ft

ELEVATION DATUM Topographic contours shown on Figure 2

WEATHER Rain, 50ºF

SURFACE Sand

LOGGED BY BJG

REVIEWED BY BMW

EXCAVATION CONTRACTOR Doug Shepherd Dirtworks

SEEPAGE ---

EQUIPMENT John Deer 35G with 18-inch wide smooth bucket

GROUNDWATER DURING DRILLING ---

EXCAVATION METHOD Test Pit

GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**SANDY ORGANIC SOIL**: Brown, moist, and contained some rootlets, and fine- to medium-grained sand.		0						
		SP	**POORLY GRADED SAND**: *Loose*, tan with orange mottling, moist, and fine- to medium-grained.								
100					2						
98					4						
						GRAB 1	100				● 5
			Moderate caving below about 5½ feet bgs.								
96					6						
			Severe caving below about 7 feet bgs.			GRAB 2	100				● 5

- Test pit terminated at 7 feet bgs due to caving.
- Moderate to severe caving encountered below about 5½ to 7 feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

94

92

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FIGURE A9

Test Pit TP-6

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CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 94 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 49ºF
SURFACE Sand
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**SANDY ORGANIC SOIL**: Brown, moist, and contained some rootlets/roots up to ½-inch in diameter, and fine- to medium-grained sand.		0						
		SP	**POORLY GRADED SAND**: *Loose*, tan with orange mottling, moist, and fine- to medium-grained.								
92					2						
90					4						
			Moderate caving below about 5 feet bgs.								
88					6						
						GRAB 1	100				● 5
			Severe caving encountered below about 7 feet bgs.								
86											
84											

- Test pit terminated at 7½ feet bgs due to severe caving.
- Moderate to severe caving encountered below 5 to 7 feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

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FIGURE A10

Test Pit TP-7

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CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 105 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 49ºF
SURFACE Sand
LOGGED BY AET
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**SANDY ORGANIC SOIL**: Dark gray, moist, and contained abundant rootlets/roots up to ¼-inch in diameter, and fine- to medium-grained sand.		0						
104		SP	**POORLY GRADED SAND**: *Loose*, brown with gray mottling, moist, fine- to medium-grained, and contained trace roots up to 1 inch in diameter.								
					2						
102											
					4						
100											
					6						
			Severe caving below about 6½ feet bgs.								
98											
					8	✋ GRAB 1	100				● 6
96											

- Test pit terminated at 8½ feet bgs due to caving.
- Severe caving encountered below about 6½ feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

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FIGURE A11

Test Pit TP-8

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CLIENT Green Light - Home First, LLC

PROJECT NAME HFD-GLD Manzanita Housing

PROJECT NUMBER G2305878

PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon

DATE STARTED 3/31/23

GROUND ELEVATION 100 ft

ELEVATION DATUM Topographic contours shown on Figure 2

WEATHER Rain, 49ºF

SURFACE Sand

LOGGED BY AET

REVIEWED BY BMW

EXCAVATION CONTRACTOR Doug Shepherd Dirtworks

SEEPAGE ---

EQUIPMENT John Deer 35G with 18-inch wide smooth bucket

GROUNDWATER DURING DRILLING ---

EXCAVATION METHOD Test Pit

GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ / PL ● MC LL / ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**SANDY ORGANIC SOIL**: Dark gray, moist, and contained abundant rootlets/roots up to ½-inch in diameter, and fine- to medium-grained sand.		0						
		SP	**POORLY GRADED SAND**: *Loose*, brown with gray mottling, moist, fine- to medium-grained, and contained trace roots up to 1 inch in diameter.								
98					2						
96			Gray with brown mottling, and moderate caving below about 4 feet bgs.		4						
94					6	GRAB 1	100				● 5
92					8						

- Test pit terminated at 8 feet bgs due to caving.
- Moderate caving encountered below about 4 feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

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FIGURE A12

Test Pit TP-9

PAGE 1 OF 1

CLIENT Green Light - Home First, LLC
PROJECT NAME HFD-GLD Manzanita Housing
PROJECT NUMBER G2305878
PROJECT LOCATION Tax Lot 1401, Manzanita, Oregon
DATE STARTED 3/31/23
GROUND ELEVATION 90 ft
ELEVATION DATUM Topographic contours shown on Figure 2
WEATHER Rain, 49ºF
SURFACE Sand
LOGGED BY AET
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT John Deer 35G with 18-inch wide smooth bucket
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD Test Pit
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
					0						
		OL	**SANDY ORGANIC SOIL**: Dark gray, moist, and contained abundant rootlets/roots up to ¼-inch in diameter, and fine- to medium-grained sand.								
		SP	**POORLY GRADED SAND**: *Loose*, brown with gray mottling, moist, fine- to medium-grained, and contained trace roots up to 1 inch in diameter.								
88					2						
			Gray with brown mottling below about 3 feet bgs.								
86					4						
			Moderate caving below about 5 feet bgs.								
84					6						
						GRAB 1	100				● 6

- Test pit terminated at 6½ feet bgs due to caving.
- Moderate caving encountered below about 5 feet bgs.
- No groundwater encountered.
- Test pit loosely backfilled with excavated material upon completion.

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Tigard Office	**(503) 684-3460**

Appendix B: Results of Infiltration Testing

HFD-GLD Manzanita Housing
Tax Lot 1401
Tillamook County, Oregon

CGT Project Number G2305878

April 14, 2023

Prepared For:

Green Light- Home First, LLC
Attn: Rob Justus
3050 SE Division Street, Suite 270
Portland, Oregon 97202

Prepared by
Carlson Geotechnical

B.1.0 INTRODUCTION

Our client requested two infiltration tests at the project site. The tests were performed in test pits TP-1 and TP-2 on the Site Plan, which is attached to the main report as Figure 2.

B.2.0 TEST PROCEDURE

Two infiltration tests (IT-1 and IT-2) were performed in general accordance with the Falling Head Infiltration Test method as described in Chapter 3 of the 1980 EPA Onsite Wastewater Treatment and Disposal Systems Design Manual (1980 EPA).

The tests were performed within prepared test pits TP-1 and TP-2, which were advanced to the infiltration test depth (5 feet bgs) with a John Deere 35G mini-excavator with a 2-foot-wide toothed bucket. Once the test pits were advanced to the infiltration test depth, a 6-inch diameter PVC pipe was pushed about 6 inches into the soil at the test depth to obtain a proper seal between the PVC pipe and surrounding soils. A thin layer of clean gravel was placed within each pipe to prevent scouring the soil with water during testing.

We attempted to soak the subsurface soils within TP-1 and TP-2 by pouring an approximate 12-inch column of water into the test pipes. The water infiltrated into the subsurface soils in less than 10 minutes. This was repeated a second time with similar results; therefore, we immediately proceeded with the infiltration test in general accordance with the referenced test method. We poured about 6 inches of water into each test pipe and recorded the time required for the water to completely infiltrate into the subsurface materials during each trial. We administered several trials in TP-1 and TP-2.

B.3.0 INFILTRATION TEST RESULTS

The following table presents the details, raw data, and calculated infiltration rates observed during testing. Please note that the calculated infiltration rates do <u>not</u> include any safety or correction factors.

Table B1 Results of Infiltration Test IT-1

Location:	See Figure 2	Date:	3-31-23	Exploration Number:	TP-1
Test Method:	1980 EPA Falling Head Test Method.	Inner Diameter of Pipe:	6 inches	Infiltration Test Depth:	5 feet
Soil at infiltration test depth:	Poorly Graded Sand (SP)				
Saturation Start Time:	11:28:00 a.m.	Excavation could not maintain head. Test pipe filled twice with 12 inches of water, and water completely drained out of test pipe within less than 10 minutes.			
Saturation End Time:	11:34:00 a.m.				

	Time	Time Interval (Minutes)	Measurement* (inches)	Drop in Water level* (inches)	Infiltration Rate** (inches per hour)	Remarks
Trial 1	11:36:00 a.m.	---	41½	---	---	Water level adjusted
	11:41:10 a.m.	5.2	47½	6	69.23	Trial 1 concluded
Trial 2	11:42:00 a.m.	---	41½	---	---	Water level adjusted
	11:45:58 a.m.	4.0	47½	6	90.00	Trial 2 concluded
Trial 3	11:47:00 a.m.	---	41½	---	---	Water level adjusted
	11:51:30 a.m.	4.5	47½	6	80.00	Trial 3 concluded
Trial 4	11:52:00 a.m.	---	41½	---	---	Water level adjusted
	11:56:48 a.m.	4.8	47½	6	75.00	Trial 4 concluded
Measured Infiltration Rate					**75 Inches per hour**	

* Measured to the nearest one-sixteenth of an inch using a measuring tape.

** Values calculated are raw (unfactored) rates.

Table B2 Results of Infiltration Test IT-2

Location:	See Figure 2	Date:	3-31-23	Exploration Number:	TP-2
Test Method:	1980 EPA Falling Head Test Method.	Inner Diameter of Pipe:	6 inches	Infiltration Test Depth:	5 feet
Soil at infiltration test depth:	Poorly Graded Sand (SP)				
Saturation Start Time:	10:23:00 a.m.	Excavation could not maintained head. Test pipe filled twice with 12 inches of water, and water completely drained out of test pipe within less than 10 minutes.			
Saturation End Time:	10:46:00 a.m.				

	Time	Time Interval	Measurement*	Drop in Water level*	Infiltration Rate**	Remarks
		(Minutes)	(inches)	(inches)	(inches per hour)	
Trial 1	10:46:00 a.m.	---	56¼	---	---	Water level adjusted
	10:51:10 a.m.	5.2	62¼	6	69.23	Trial 1 concluded
Trial 2	10:52:00 a.m.	---	56¼	---	---	Water level adjusted
	10:56:43 a.m.	4.7	62¼	6	76.60	Trial 2 concluded
Trial 3	10:58:00 a.m.	---	56¼	---	---	Water level adjusted
	11:02:53 a.m.	4.9	62¼	6	73.47	Trial 3 concluded
Trial 4	11:10:00 a.m.	---	56¼	---	---	Water level adjusted
	11:14:47 a.m.	4.8	62¼	6	75.00	Trial 4 concluded
Measured Infiltration Rate					**75 Inches per hour**	

* Measured to the nearest one-sixteenth of an inch using a measuring tape.

** Values calculated are raw (unfactored) rates.

B.4.0 DISCUSSION

As detailed above, the measured raw (unfactored) infiltration rate was 75 inches per hour at the tested locations and depth. Please note this infiltration rate does not include any safety or correction factors. We recommend the stormwater infiltration system designer consult the appropriate design manual in order to assign appropriate safety/correction factors to calculate the design infiltration rate for the proposed infiltration system.

Once the design is completed, we recommend the infiltration system design (provided by others) and location be reviewed by the geotechnical engineer. If the location and/or depth of the system change from what was indicated at the time of our fieldwork, additional testing may be recommended.