Stormwater Management Facilities

Private Stormwater Report

Manzanita City Hall

HDG Job #: BAL001

Prepared For: City of Manzanita
167 S 5th St
Manzanita, OR 97130

Prepared By: Humber Design Group, Inc.

110 SE Main St. Suite 200
Portland, OR 97214
(P) 503 946 6690

'I hereby certify that this Stormwater Management Report for the Manzanita City Hall project has been prepared by me or under my supervision and meets minimum standards of City of Manzanita and normal standards of engineering practice.

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

REGISTERED PROFESSIONAL ENGINEER
87835PE
OREGON
SEPT 12, 2017
KRISTIAN ANN MCCOMBS
EXPIRES 12-31-2024

Date: November 10, 2023

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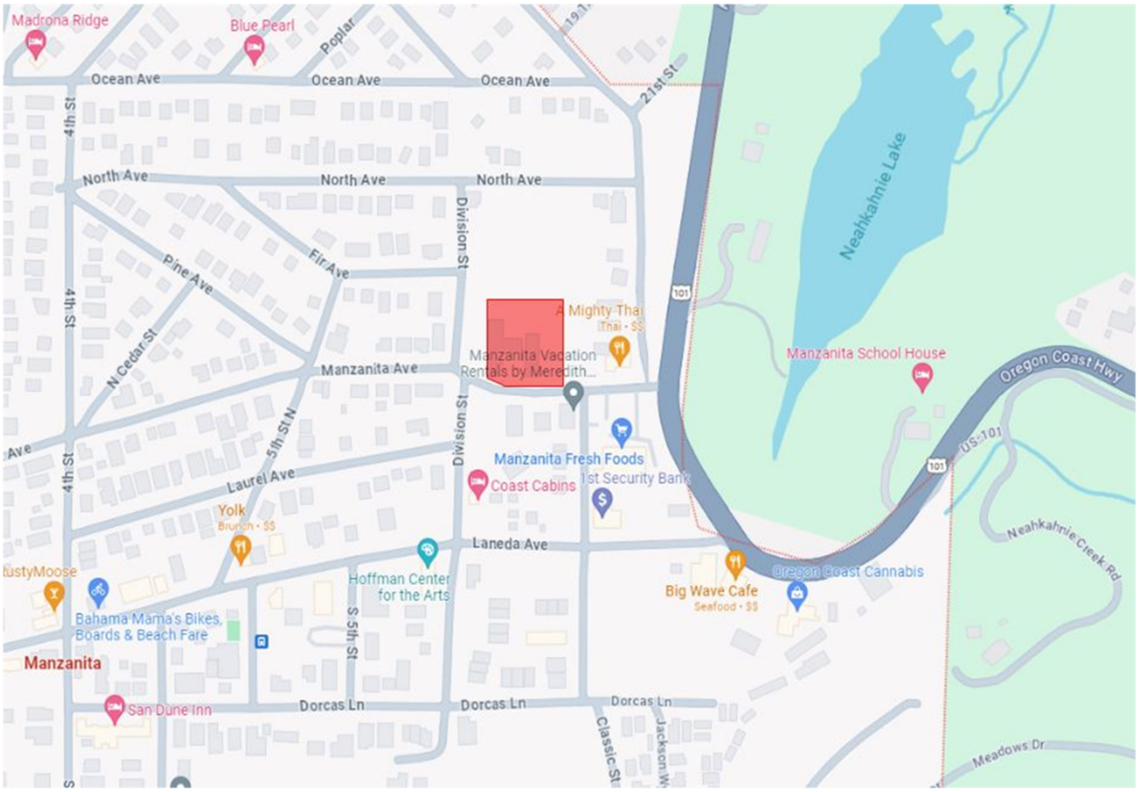
APPENDICES

Project Overview and Description

Location of Project	655 Manzanita Ave
Site Area/Acreage	2.66
Proposed Impervious Area	17,711 SF
Nearest Cross Street	Manzanita Ave
Property Zoning	MZ-C1
Existing Conditions	The existing site consists of (1) story building with ashalt and gravel parking lot.
Proposed Development	The proposed site will consist of 2 (1) story buildings with parking lot.
Tax Map	3N1029AD
Tax Lot	2500
Flood Zone	N/A
Permits Required	Building Permit

Vicinity Map



🟥 Site Location

Methodology

Existing Drainage	Stormwater on the site is currently conveyed to various catch basins and sent to the existing public storm pipe in Division Street.
Infiltration Results	
PRIVATE Proposed Stormwater Management Techniques	Stormwater runoff from site will be managed by underground rock infiltration systems.
Discharge Point	Drywell or Soakage Trench (UIC)

Analysis

Computational Method Used Using stormwater storage capacity requirement equation from Drywell & Infiltration System Standards manual of City of Manzanita.

Soil Types Beach and Dune Sands

Stormwater Management Narrative Stormwater runoff from the 12,497 SF of proposed impervious area will be treated by 3'X3' infiltration trench and 4,624 SF of proposed impervious area will be treated by (2) 1.5'X3' infiltration trench system. Overflow from infiltration trench will be delivered to the existing stormwater pipe located in Division Street.

Table 4 – Catchment Areas and Facility Table

Catchment/ Facility ID	Source (roof, road, etc.)	Treatment Area (sf)	Ownership (private/ public)	Facility Type/ Function	Facility Size
A	Roof, Pakring Lot, Sidewalk	12,497	Private	Infiltration Trench	3'X3'-115LF
B	Roof	719	Private	Infiltration Trench	1.5'X3'-10LF
C	Parking Lot	3,905	Private	Infiltration Trench	1.5'X3'-114LF

Engineering Conclusions

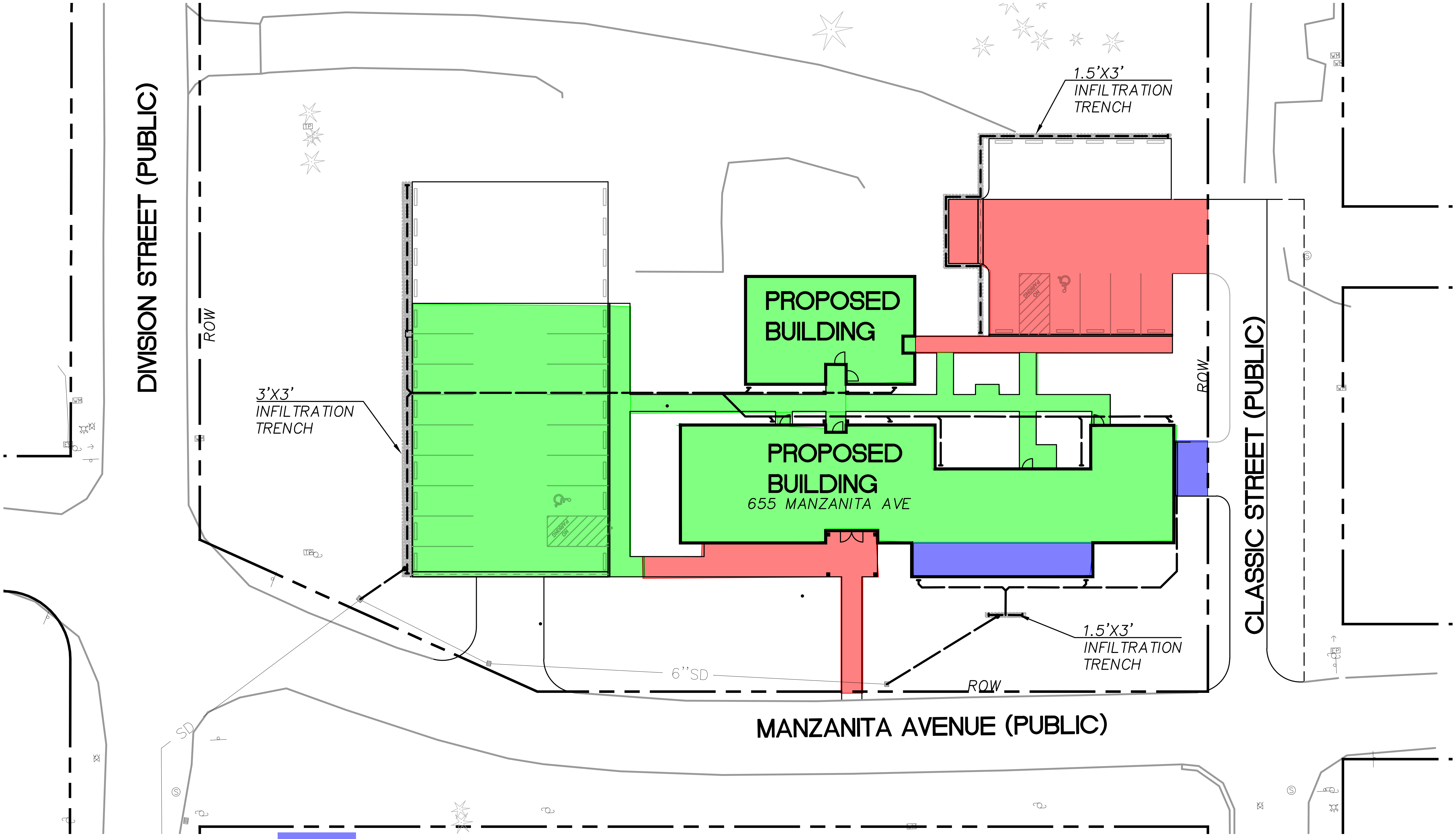
The preceding methodologies and calculations presented indicate compliance with the current jurisdictional stormwater management codes and requirements. A summarized breakdown is presented below:

Water Quality	The proposed development will meet the provisions for water quality per the Drywell & Infiltration System Standards Manual of City of Manzanita.
Water Quantity	The proposed development will meet the provisions for water quantity per the Drywell & Infiltration System Standards Manual of City of Manzanita.
Downstream / Upstream Impacts	There are no upstream or downstream impacts created by this proposed development.

Appendix A

Stormwater Facility Details / Exhibits

- Catchment Map
- Utility Map
- Rock Infiltraion Trench Details

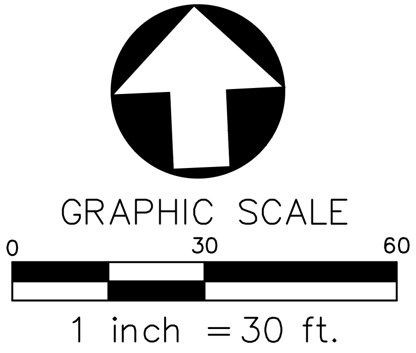


🟩 Catchment A = 12,497 SF

🟦 Catchment C = 3,905 SF

🟥 Catchment B = 719 SF

Total Impervious area = 17,121 SF



MANZANITA CITY HALL
CATCHMENT MAP

PROJECT NO.:	BAL001
DRAWN BY:	MCS
DESIGN BY:	MCS
REVIEWED BY:	KAM
DATE:	11/9/2023

Humber
Design
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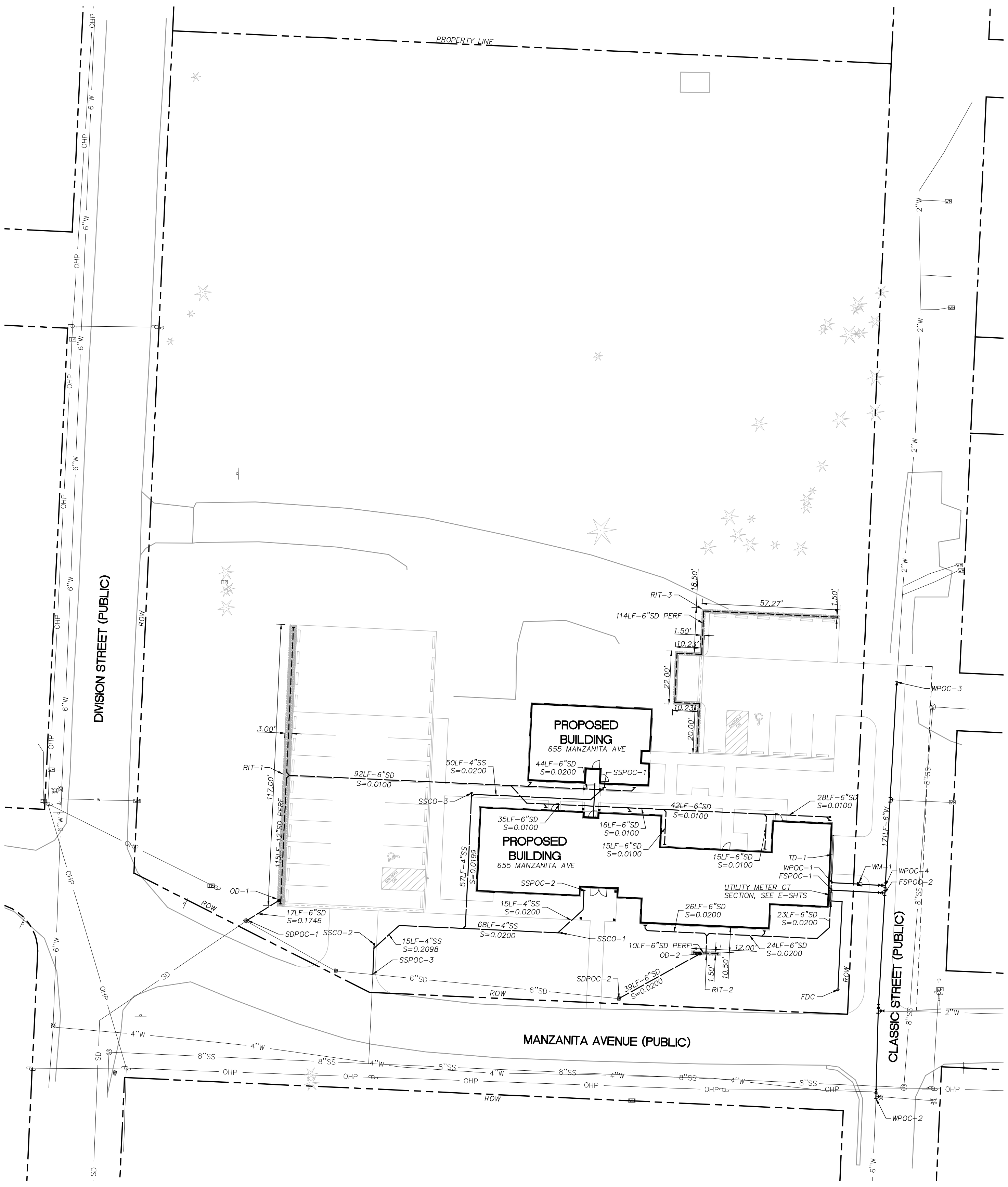
Civil Engineering ▪ 503.946.6690 ▪ hdgpdx.com

P:\BAL001 - Manzanita City Hall\7 - CAD Drawings\B - Exhibits\CHATCHMENT.dwg Nov 10, 2023 - 4:35pm



UTILITY PLAN
SCALE: 1"=20'

SHEET LEGEND

ITEM	DESCRIPTION	DETAIL
— SD —	STORM	
— SS —	SANITARY	
— W —	WATER	
— F —	FIRE SERVICE	
———	PERFORATED PIPE	
■	AREA DRAIN	15 C4.00
•	CLEANOUT	14 C4.00
⋈	WATER VALVE	
◪	WATER METER	

STORM SEWER SCHEDULE

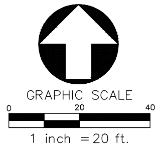
ITEM	DESCRIPTION	REFERENCE
SDPOC-1	STORMWATER POINT OF CONNECTION, CONNECT TO EXISTING CATCH BASIN IE=100.2	
SDPOC-2	STORMWATER POINT OF CONNECTION, CONNECT TO EXISTING CATCH BASIN IE=101.79	
OD-1	OVERFLOW DRAIN RIM=XXX IE=XXX	
OD-2	OVERFLOW DRAIN RIM=XXX IE=XXX	
RIT-1	ROCK INFILTRATION TRENCH, 3'X3', 115LF-12"SD.	17 C4.00
RIT-2	ROCK INFILTRATION TRENCH, 1.5'X3', 10LF-6"SD.	18 C4.00
RIT-3	ROCK INFILTRATION TRENCH, 1.5'X3', 114LF-6"SD.	19 C4.00
TD-1	TRENCH DRAIN, X"WIDE,S=X.XXXX	16 C4.00

WATER SCHEDULE

ITEM	DESCRIPTION	REFERENCE
WM-1	WATER METER, INSTALL 1" METER BY CITY OF MANZANITA. CONTRACTOR CONNECT TO THE SHORT STUB-OUT ON THE BACK SIDE OF THE METER BOX. 37.5 WSFU	
FSPOC-1	FIRE SERVICE POINT OF CONNECTION. SIZE= XXX, FOR BUILDING PLUMBING, SEE P-SHTS.	
FSPOC-2	FIRE SERVICE POINT OF CONNECTION. SIZE= XXX, CONNECTION TO PUBLIC WATER MAIN.	
WPOC-1	WATER POINT OF CONNECTION. SIZE= XXX, FOR BUILDING PLUMBING, SEE P-SHTS.	
WPOC-2	WATER POINT OF CONNECTION. REPLACE EXISTING 2" WATER MAIN AND CONNECT TO EXISTING 6" WATER MAIN.	
WPOC-3	WATER POINT OF CONNECTION. REPLACE EXISTING 2" WATER MAIN AND CONNECT TO EXISTING 2" WATER MAIN.	
WPOC-4	WATER POINT OF CONNECTION TO PUBLIC WATER MAIN.	
FDC	FIRE DEPARTMENT CONNECTION	

SANITARY SEWER SCHEDULE

ITEM	DESCRIPTION	REFERENCE
SSPOC-1	SANITARY SEWER CONNECTION, SIZE=XXX", IE=XXX', FOR BUILDING SEWER SEE PLUMBING, XXX DFU.	
SSPOC-2	SANITARY SEWER CONNECTION, SIZE=XXX", IE=XXX', FOR BUILDING SEWER SEE PLUMBING, XXX DFU.	
SSPOC-3	COMBINATION SEWER POINT OF CONNECTION, CONNECT TO EX. SEWER LATERAL, CONTRACTOR TO V.I.F. ASSUMED IE=96.17	
SSCO-1	SANITARY SEWER CLEANOUT TO GRADE, IE=XXX	14 C4.00
SSCO-2	SANITARY SEWER CLEANOUT TO GRADE, IE=XXX	14 C4.00
SSCO-3	SANITARY SEWER CLEANOUT TO GRADE, IE=XXX	14 C4.00



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SEPT 12, 2017
KRISTIAN ANN MCCOMBS
EXPIRES 12-31-2024

Printing	Date
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KAM

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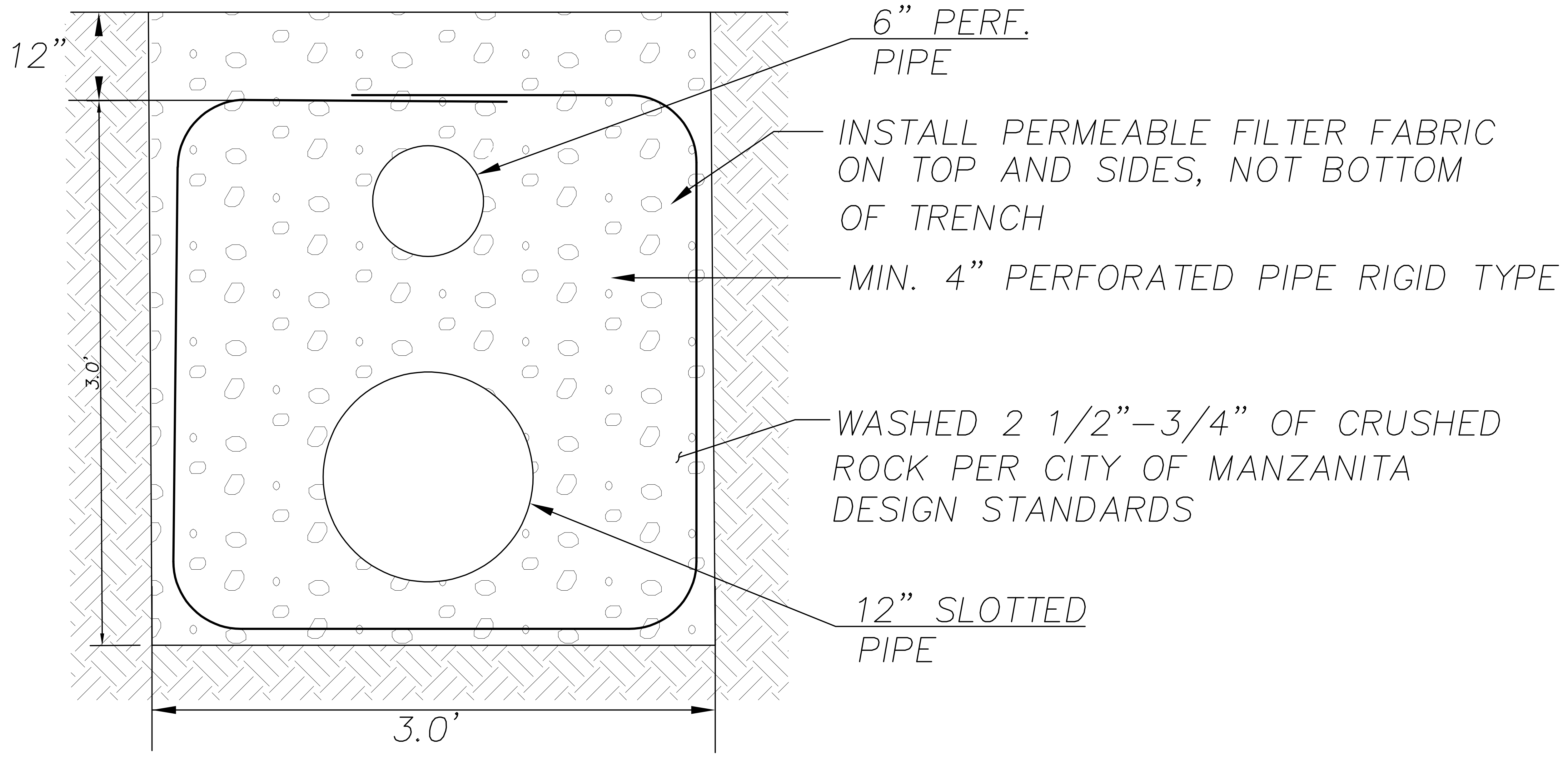
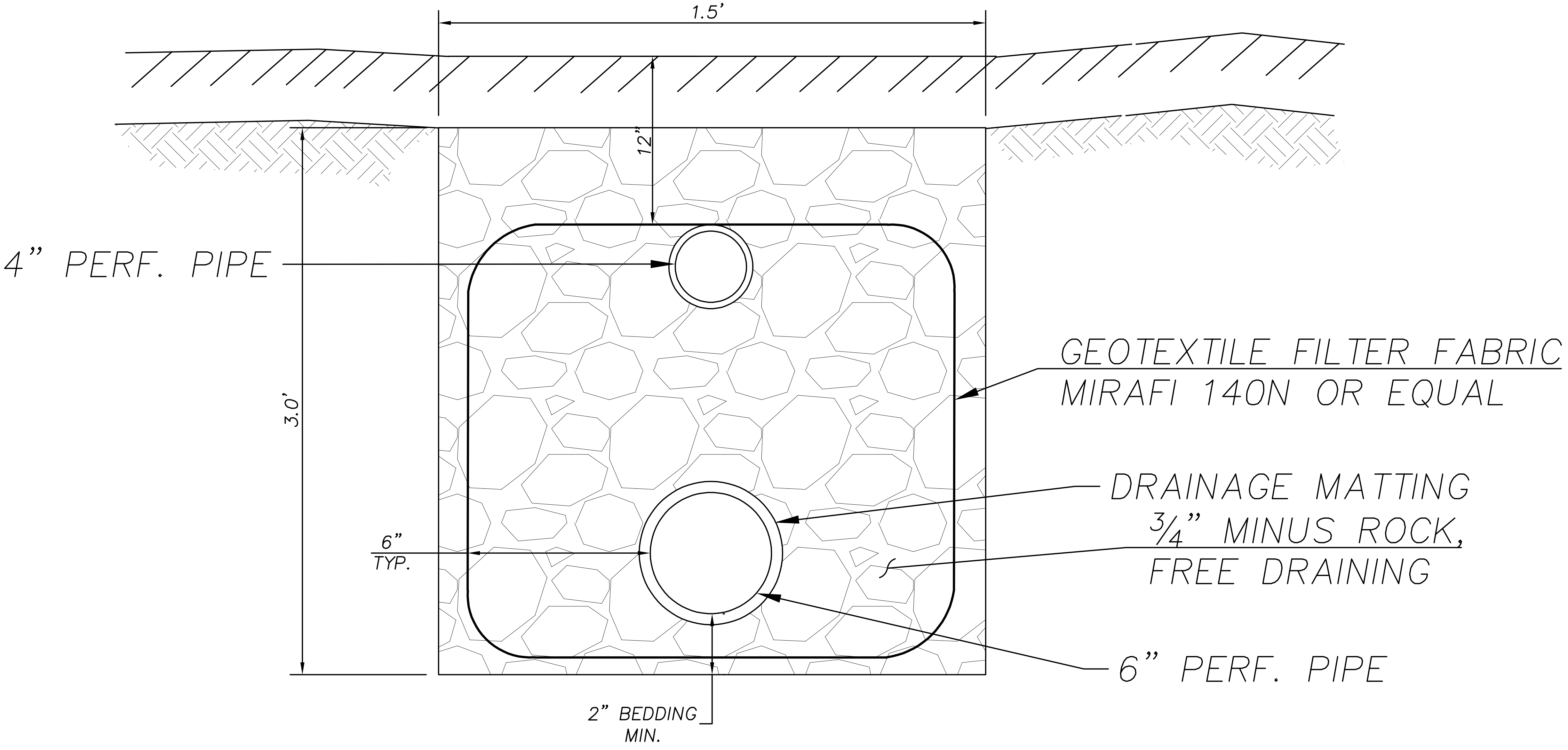
C3.00



1

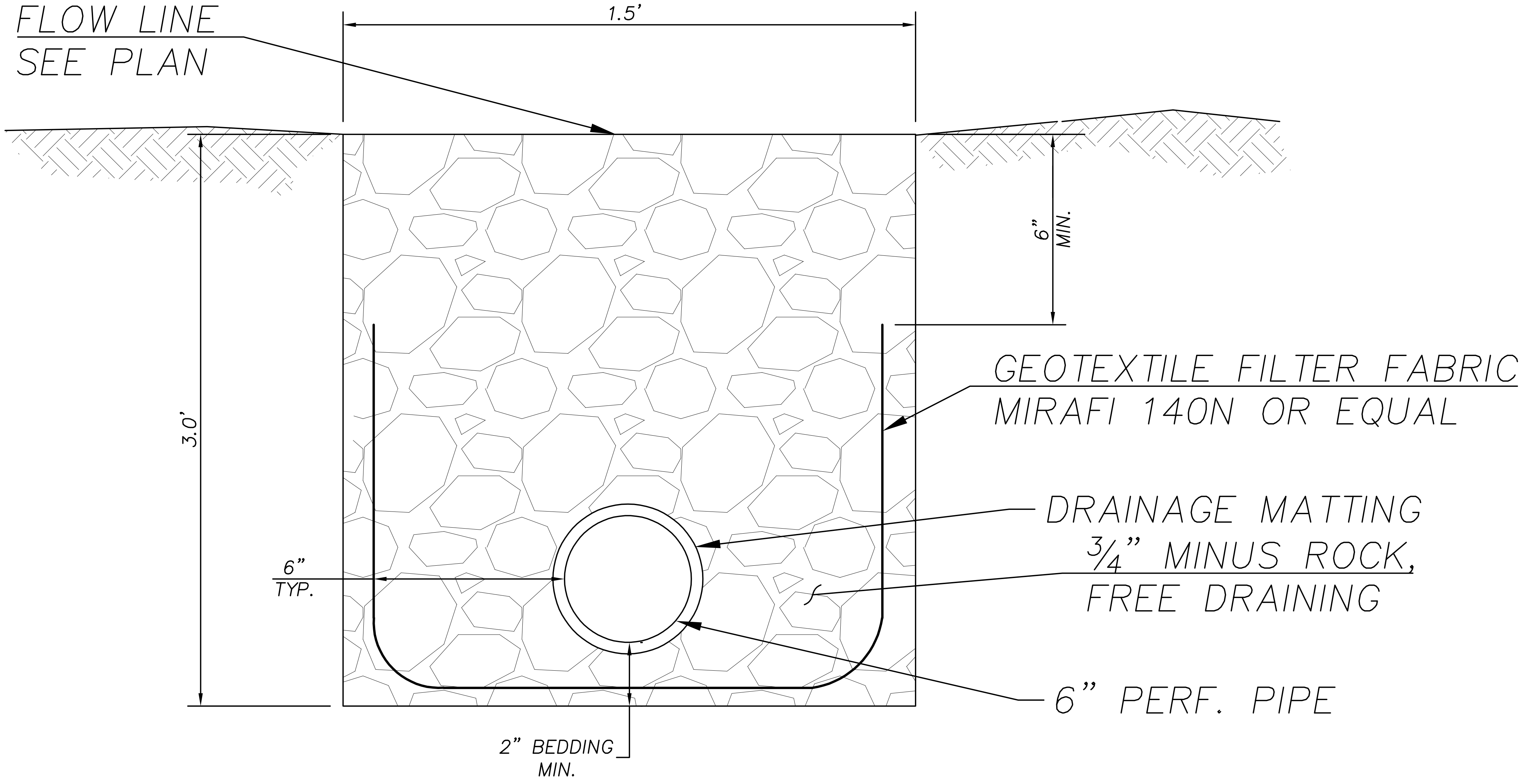
ROCK INFILTRATION TRENCH - 1

NTS



2 ROCK INFILTRATION TRENCH - 2

NTS



3 ROCK INFILTRATION TRENCH - 3

NTS

Appendix B

Support Calculations

Rock Infiltraion Trench Calculations

Humber
Design
Group, Inc.

Date: November 09, 2022

To: City of Manzanita
167 S 5th St
Manzanita, OR 97130

From: Min Chan Song, EIT
Humber Design Group, Inc. (HDG)

Subject: Stormwater Storage Capacity Calculations – Catchment A

Rational Formula: $1\ cubic\ foot\ for\ every\ 44\ square\ feet\ of\ impervious\ area.$

Assumptions:

Void of Rock $= 40\%$, Void of Perf. Pipe $= 100\%$

Rock Area Dimension,D $= 3' X\ 3'$

Radius of Perf. Pipe Size, r $= 6"\ and\ 3"$

Section Area with Void, $A = \left(D - (\pi r2) - (\pi r2)\right) * 40\% = 3.21\text{sqft}$

Calculations:

Impervious Area $= 12{,}497\ SF$

Storage required $= 12{,}497/\ 44\ = 284\ cuft$

Trench Length Required $= 284\ cuft/3.21\ sqft\ = 88.47\ ft$

Humber
Design
Group, Inc.

Date: November 09, 2022

To: City of Manzanita
167 S 5th St
Manzanita, OR 97130

From: Min Chan Song, EIT
Humber Design Group, Inc. (HDG)

Subject: Stormwater Storage Capacity Calculations – Catchment B

Rational Formula: $1\ cubic\ foot\ for\ every\ 44\ square\ feet\ of\ impervious\ area.$

Assumptions:

Void of Rock $= 40\%$, Void of Perf. Pipe $= 100\%$

Rock Area Dimension,D $= 1.5' X\ 3'$

Radius of Perf. Pipe Size, r $= 3"\ and\ 2"$

Section Area with Void, $A = \left(D - (\pi r2) - (\pi r2)\right) * 40\% = 1.69\text{sqft}$

Calculations:

Impervious Area $= 719\ SF$

Storage required $= 719/\ 44 = 16.3\ cuft$

Trench Length Required $= 16.3\ cuft/1.69 sqft = 9.65\ ft$

Humber
Design
Group, Inc.

Date: November 09, 2022

To: City of Manzanita
167 S 5th St
Manzanita, OR 97130

From: Min Chan Song, EIT
Humber Design Group, Inc. (HDG)

Subject: Stormwater Storage Capacity Calculations – Catchment C

Rational Formula: $1\ cubic\ foot\ for\ every\ 44\ square\ feet\ of\ impervious\ area.$

Assumptions:

Void of Rock $= 40\%$, Void of Perf. Pipe $= 100\%$

Rock Area Dimension,D $= 1.5' X\ 3'$

Radius of Perf. Pipe Size, r = 3"

Section Area with Void, $A = (D - \pi r2) * 40\% = 1.72\text{sqft}$

Calculations:

Impervious Area $= 3905\ SF$

Storage required $= 3905/\ 44\ = 88.75\ cuft$

Trench Length Required $= 88.75 cuft/1.72 sqft\ = 51.6\ ft$

Appendix C

Additional Forms & Associated Reports

Geotechnical Report

Draft Geotechnical Engineering Report

Manzanita City Hall
635-655 Manzanita Avenue
Manzanita, Oregon

Prepared for:
City of Manzanita
Leila Aman | Manzanita City Manager
PO Box 129
Manzanita, OR 97130

October 31, 2022
Project No. COM-2022-001

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SUPPORTING DATA

1.0 INTRODUCTION

This Geotechnical Engineering Report (GER) presents Rhino One Geotechnical (ROG) geotechnical engineering study for the proposed new City Hall for the City of Manzanita. The project location is shown on Figure 1, *Site Location Map* and attached in Appendix A.

The proposed project is located on an approximately 2.7-acre site at 635-655 Manzanita Avenue. There is an old existing school and quonset hut on the property that are both unoccupied. The new City Hall will be about 6,000 square feet along with on-site parking and other improvements. Parts of the existing structures may be renovated and incorporated as part of the new development.

The purpose of this study is to provide geotechnical data for the proposed buildings, pavements, and miscellaneous improvements. This report provides a summary of our field exploration, laboratory testing, geotechnical engineering analysis, seismic design criteria, geotechnical design criteria, and construction recommendations for the proposed project. This report may require modification as the project develops.

2.0 SCOPE OF SERVICES

The scope of services for ROG was completed in general accordance with our proposal dated July 25, 2022. Generally, the services consisted of the following major elements.

- Field Exploration and Laboratory Testing
- Geotechnical Engineering Analysis
- Preparation of this Geotechnical Engineering Report (GER)

3.0 EXPLORATION AND TESTING PROGRAM

3.1 Geotechnical Field Explorations

The subsurface exploration program for this project consisted of drilling five (5) borings with a full-size track-mounted drill rig operated by Crisman Pacific Strata Drilling L.L.C, of Donald, Oregon on October 10 and 11, 2022. The borings (B-01 to B-05) were drilled at the approximate locations shown on the *Site Exploration Plan* (Appendix A, Figure 2) and were drilled to depths between 11.5 and 81.5 feet below ground surface (BGS). The drilling was performed using mud-rotary drilling techniques. Disturbed Standard Penetration Test (SPT) soil samples were obtained at regular 2.5-foot or 5-foot intervals using a 140-pound automatic hammer with an average energy transfer ratio of 73.8% during the drilling in general accordance with the *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils* (ASTM D 1586).

The subsurface materials encountered were logged and field classified in general accordance with the ASTM *Manual-Visual Classification Method* (ASTM D 2488). The SPT samples were collected at desired depths and packaged in moisture-tight bags. Uncorrected blow counts from the SPT sampling are reported on the boring logs. Corrected blow counts [$(N1)_{60}$] were used for our analysis unless otherwise noted. Interpreted summary borings logs are presented in Appendix B. Table 1 describes the explorations completed for the project and as shown on the *Site Exploration Plan*.

Table 1 Geotechnical Exploration Program Summary

Exploration Number	Ground Elevation[1] (feet)	Termination Depth[2] (BGS, feet)	Exploration Method	Groundwater Depth[2] (BGS, feet)
B-01	104	81.5	MR	NE
B-02	106	51.5	MR	NE
B-03	105	16.5	MR	NE
B-04	104	11.5	MR	NE
B-05	105	21.5	MR	NE

Notes:
1: Elevation approximated from Google Earth
2: Below existing ground surface at time of drilling
MR = Mud rotary drilling technique using track-mounted rig
NE = not encountered due to use of mud rotary drilling techniques

3.2 Laboratory Testing

Laboratory tests were conducted to provide data on the physical characteristics and engineering properties of the soil essential for engineering studies and analyses. Laboratory tests were conducted on selected soil samples in accordance with standard ASTM methods. The tests conducted include:

- Natural moisture content of selected samples obtained from the borings in general accordance with guidelines presented in ASTM D 2216.
- Grain size analysis on selected samples obtained from the borings in general accordance with guidelines presented in ASTM D C136/117.

The results of these tests are attached in Appendix B, selected test results are presented on the boring logs, and a summary of test results is shown on Table 2.

Table 2 Laboratory Test Results Summary

Boring Designation, Sample Depth Interval (feet)	Percent Gravel	Percent Sand	Percent Silt and Clay
B-01, 5 – 6.5	0.0	99.5	0.5
B-01, 10– 11.5	0.0	99.4	0.6
B-01, 15 – 16.5	0.0	98.4	1.6
B-01, 20 – 21.5	0.0	96.9	3.1
B-01, 25 – 26.5	0.0	96.4	3.6
B-01, 30 – 31.5	0.0	97.3	2.7
B-01, 35 – 36.5	0.0	96.7	3.3
B-01, 40 – 41.5	0.0	96.5	3.5
B-01, 50 – 51.5	0.0	95.0	5.0
B-01, 60 – 61.5	0.0	95.3	4.7
B-01, 70 – 71.5	0.0	92.5	7.5
B-01, 80 – 81.5	0.0	90.7	9.3

4.0 SITE GEOLOGIC AND SUBSURFACE CONDITIONS

4.1 Geologic Information

Regional geology for the project vicinity was evaluated based on a review of existing geologic mapping, site reconnaissance, and subsurface explorations. Figure 2 (*Site Exploration Plan*) shows the approximate locations of exploration for this project.

4.1.1 Regional Published Geology Summary

The site is located along the northern Oregon coast of the Pacific Ocean. The region consists of accreted marine sedimentary rocks consisting of siltstone and sandstone. Exposures of invasive lava flows of the Columbia River Basalt Group lava is present north of the Neahkahnie Beach. Published mapping indicates bedrock underlying the project consists of Miocene to Oligocene aged marine sedimentary rocks (map units Tmo (Wells, Snavely, MacLeod, Kelly, & Parker, 1994). Surficial deposits consist of beach and dune sand deposits.

4.2 Interpreted Subsurface Conditions

As discussed above, to date we have completed five borings at the site. Subsurface data from the test borings were used to develop a general subsurface profile for the project location.

The materials encountered are described below. Additional observations regarding groundwater are included in Section 4.3.

4.2.1 Project Geotechnical Units

The materials encountered in our field explorations for the project are interpreted to represent beach and dune sand unit. Our interpretation of the subsurface conditions is based on our explorations and regional geologic information from published sources. The subsurface interpretation considered our borings from the current exploration program as well as geologic information from published sources. Subsurface conditions may vary between explorations differently from those discussed below or shown on the boring logs. The following sections are intended to provide the reader with a general overview of subsurface conditions. Individual borings logs should be reviewed to best understand the encountered subsurface conditions at specific locations. For detailed boring descriptions see attached boring logs.

4.2.1.1 Beach and Dune Sand

The encountered soils generally consist of loose to very dense, poorly graded sand (SP) and very dense, poorly graded sand with silt (SP-SM). The sand with silt typically have no plasticity. The soils were observed to be moist to wet. The soils correlate to beach and dune sand deposits. All borings were terminated in this unit. This unit was observed to for the full depths of exploration to 81.5 feet BGS in all borings.

The measured natural water content from the samples ranged from 10.0 to 24.5 percent with an average of 16.8 percent. The SPTs conducted in this layer ranged from 3 to 104 blows per foot (BPF) with an average of 24.1 BPF. The fines content of this material ranged from 0.5 to 9.3 percent with an average fines content of 3.8 percent.

4.3 Groundwater Observations

The borings were drilled using mud-rotary drilling techniques. Therefore, groundwater could not be directly measured at the time of drilling due to the introduction of artificial drilling slurry into the borehole. Groundwater was interpreted at depths between 30 and 35 feet BGS in the borings at the time of explorations based on moisture conditions. We also reviewed historical logs for Highway 101 for the tunnel crossing at Neahkahnie Creek. Groundwater was encountered at approximate elevation 55 to 62 feet (NGVD 29) in the piezometers installed for that study. Based on our observation and the data from historical logs, we recommend that groundwater be assumed at 30 feet BGS for design purposes.

5.0 SEISMIC HAZARDS EVALUATION

5.1 Seismic Design Considerations

This section presents the results of ROG's site-specific seismic hazard analysis for the proposed Project. The site location relative to surrounding physical features is shown on Figure 1 (Appendix A). This facility qualifies as an "Essential Structure" in accordance with Oregon Revised Statutes (ORS) Section 455.447(1).

This study has been completed as required in Section 1803.3.2 – "Seismic Site Hazard Study" with the reporting requirements of Section 1803.6.1 – "Seismic Site Hazard Report" of the Oregon Structural Specialty Code (OSSC, 2019)). The following sections of the report discuss these requirements along with the relevant section numbers from the code.

5.2 Geologic Profile (OSSC 1803.6.1, Item 1)

Figure 2, Appendix A, shows the location of test borings conducted for this study. Section 4 of the report describes materials encountered in detail to the depth explored. Borings logs and laboratory testing data are presented in Appendix B of the report. Section 4.3 of the report describes groundwater in detail. An estimated geologic profile at the site is presented in Table 3 below.

Table 3 Estimated Geologic Profile

Profile Depth (feet)	Geologic Unit	Shear Wave Velocity (feet per second)
0 to 20	Beach and Dune Sands	600 to 900
20 to 100	Beach and Dune Sands	900 to 1,200
> 100	Weathered Siltstone / Sandstone (Marine Sedimentary Rock)	1,500 to 2,500

5.3 Seismicity (OSSC 1803.6.1, Items 2 and 3)

Historic Seismicity

Information on the historical record of Oregon earthquakes dates back to approximately 1841. Prior to 1900, approximately 30 earthquakes had been recorded in the area. Several hundred earthquakes have been recorded in the State since 1900, especially since the 1980's when the University of Washington established a recording station as part of the Pacific Northwest Seismic Network (PNSN). Catalogues of earthquake events are available from Berg and Baker (1963) and Johnson et al. (1994). Additional summary of Oregon earthquakes resources includes Wong et al. (2000).

Oregon is a region of low to medium historical seismicity. Clusters of earthquakes are recorded in the Klamath Falls region (M = 6.0, approximately 260 miles from the Site), northeast Oregon (M= 5.0 Umatilla, M= 6.5 Milton Freewater, approximately 200 to 250 miles from the Site), and the Portland – Northern Willamette Valley (M= 5.6 Mt. Angel, approximately 100 miles from the Site).

Seismic Sources

Information provided by several references characterize the principal tectonic feature of the Pacific Northwest as the Cascadia Subduction Zone (CSZ). The subduction zone begins off the coast of Oregon and dips downward beneath Western Oregon. Two primary seismic source mechanisms are associated with the subduction zone: an interface source mechanism and an intraplate source mechanism. Additionally, several shallow crustal seismic faults of the North American Plate have also been mapped. The following subsections describe these three sources in detail. Volcanic sources beneath the Cascade Range are not considered in this study, as they rarely generate seismic events in excess of magnitude 5.0 and are not considered to pose significant ground shaking hazard at the Project Site.

Cascadia Subduction Zone - Interface Earthquake

CSZ represents the boundary between the subducting Juan de Fuca tectonic plate and the overriding North American tectonic plate. Interface earthquakes occur along the 1000 kilometer (km) thrust fault stretching from Northern Vancouver Island to Cape Mendocino California and are located at depths of less than approximately 30 km. Historically, earthquakes generated from subduction zone interface sources are the largest earthquakes observed worldwide. Geologic evidence from the coastal areas of Washington and Oregon indicates the CSZ has produced very large megathrust earthquakes of estimated moment magnitude (M_w) 8 to 9 originating at irregular intervals along the interface source in the past. The last such megathrust event occurred in January 1700 which likely ruptured much of the length of the CSZ. It is estimated the 1700 CSZ event had a moment magnitude (M_w) between 8.7 and 9.2 and the ground shaking may have continued for up to 3 to 4 minutes (Atwater, B. F., et al., 1995). Recurrence intervals for subduction zone megathrust interface earthquakes are based on studies of the geologic record. Studies indicate a recurrence interval ranging from 300 to 600 years.

The CSZ fault zone located offshore is considered to have the potential to generate a M9 event at a site distance of approximately 15 miles (Petersen, et al., 2014). The Oregon Structural Specialty Code (OSSC, 2019) requires the consideration of an earthquake on the seismogenic part of the CSZ interface with a minimum magnitude of 8.5 (1803.3.2.1(3), Design Earthquake), which likely corresponds to a 10 percent chance of being exceeded in 50 years. A moment magnitude event of 9.0 likely corresponds to a 2 percent chance of being exceeded in 50 years.

Cascadia Subduction Zone - Intraplate Earthquake

Intraplate earthquakes occur within the subducting Juan de Fuca plate where the plate bends below the North American plate. Intraplate earthquakes typically occur along normal faults and at greater depths than interface earthquakes. A number of researchers have noted the complete absence of intraplate seismicity in Western Oregon ((Ludwin, R.S., Weaver, C.S., and Crosson, R.S, 1991) and (Rogers, A. M., Walsh, T. J., Kockelman, W. J., and Priest, G. R., 1996)). With the possible exception of the 1873 Crescent City Earthquake Richter magnitude 6.75, no moderate to large intraplate earthquakes (M_w greater than 5.0) have occurred within the CSZ from south of Puget Sound to Cape Mendocino. These earthquakes are postulated to have a deep focus of 40 to 80 kilometers in the subducted Juan de Fuca Plate, and theoretical magnitudes of up to 7.8.

This fault is considered to be capable of generating a M6.9 at a distance of about 50 miles from the site (Petersen, et al., 2014). The Oregon Structural Specialty Code (OSSC, 2019) requires the consideration of a deep earthquake on the seismogenic part of the subducting CSZ with a moment magnitude greater than 7 (1803.3.2.1(2), Design Earthquake).

Crustal Sources

Crustal source seismic events are shallow earthquakes occurring within the North American plate. Due to their proximity, crustal faults are significant seismic sources for ground motion. A review of the (US Geological Survey (USGS), May 26 2015) and a study by (Geomatrix Project Number 2442, 1995) indicates there are 4 crustal faults within a 20-mile radius of Site. Table 4 and Figure 3 present the mapped crustal faults within 20-mile radius of the Project Site.

Table 4 Potentially Active Crustal Fault Summary

Fault Name	Fault Number	Closest Distance to Site (mile)	Fault Type	Most Recent Deformation (year)	Mapped Length (km)
Tillamook Bay Fault Zone	881	9.9	Reverse	< 1.6 M	32
Fault H	790	12.9	Normal	< 15 ka	49
Nehalem Bank Fault	789	17.1	Right Lateral	< 15 ka	101
Gales Creek Fault Zone	718	17.3	Right Lateral	< 1.6 M	73

Recorded seismicity due to these sources in the Site vicinity is relatively limited, with only a few recorded earthquakes in the region exceeding moment magnitude (M_w) 3.0 and none exceeding 5.0. Studies (Yelin, T. S. and Patton, H. J. , 1991) of small earthquakes in the region indicate most crustal earthquake activity is occurring at depths of 10 to 20 kilometers.

Seismic Sources

The contribution of earthquake hazards for the PGA from various seismogenic sources was evaluated using the interactive deaggregation tool provided by the USGS (U.S. Department of the Interior, U.S. Geological Survey, 2020). The interactive deaggregation tool incorporates the results of the 2014 National Seismic Hazard Mapping Program (NSHMP) and separates the earthquake hazards into four sources: interface, slab, fault and grid. The interface and slab categories are from the CSZ, and the fault and grid categories represent the shallow crustal sources (the “fault” category is the hazard from discrete crustal faults in the USGS 2014 NSHMP seismic source model; the “grid” category is the hazard also from crustal seismicity but from as-of-yet unknown or discretely modeled faults). The seismic hazard at the site is dominated by Interface Earthquake with a Moment Magnitude of 9.08 a distance of 31.5 KM from the site with a PGA of 0.77g. This earthquake scenario was analyzed further for liquefaction and lateral spreading analysis.

5.4 Recommended Design Spectra

The design of the new structure will be governed by the 2019 Oregon Structural Specialty Code (International Code Council, Inc., 2019). Therefore, the seismic design will be completed in accordance with ASCE 7-16. The following sections provide seismic design criteria in accordance with ASCE 7-16.

The code-based design earthquake spectral response acceleration parameters were determined using ASCE’s online Hazard Tool (American Society of Civil Engineers, 2020). Section 11.4.8 of ASCE/SEI 7-16 recommends a ground motion hazard analysis be completed for structures on Site Class D with S_1 greater than or equal to 0.2g. This site has S_1 of 0.671g which is greater than 0.2g. Therefore, the values for long-period site coefficient (F_v), spectral response acceleration parameter adjusted for site class effects (S_{M1}) and design spectral response acceleration parameter (S_{D1}) at a period of 1 second are not listed in the ASCE's online Hazard Tool. Based on our discussions with the structural engineer, the proposed structure is a two-story building with a fundamental period of vibration less than 0.5 seconds. Therefore, a site response analysis is not required, rather a site class is permitted to be determined in accordance with Section 20.3 of ASCE 7-16 and corresponding values of F_a and F_v determined from tables 11.4-1 and 11.4-2.

Following the requirements of Section 21.2 of ASCE/SEI 7-16, a probabilistic ground motion analysis was completed using USGS's *Unified Hazard Tool*, version 4.2.0, Dynamic: Conterminous U.S. 2014 (update) for Site Class D (U.S. Department of the Interior, U.S. Geological Survey, 2020). The Risk Targeted Ground Motions (RTGM) was calculated using the USGS RTGM Calculator (U.S. Department of the Interior, U.S. Geological Survey, 2020) in accordance with "Method 2" of Section 21.2.1.2 of ASCE/SEI 7-16. The maximum RTGM spectral acceleration was calculated as 1.856g at a period of 0.3 second. Since this value is less than 1.2 F_a, deterministic ground motion analyses are not required. The design spectral accelerations can therefore be taken as 2/3 of the RTGM spectral accelerations unless it is less than 80% of the spectral accelerations evaluated using the procedure in Section 11.4.6 with F_v = 2.5. Table 5 summarizes the spectral accelerations based on the method discussed above and the site-specific seismic analysis.

Table 5. ASCE 7-16 Seismic Design Parameters (OSSC 2019)

	Short Period (0.2 Sec)	**Long Period (1 Sec)**
Maximum Credible Earthquake Spectral Acceleration (g)	S_s = 1.278 g	S_1 = 0.671 g
Site Class	D	
Results from ASCE 7-16 Online Calculator		
Site Coefficient	F_a = 1.2	F_v = N/A
Adjusted Spectral Acceleration (g)	S_{MS} = 1.533	S_{M1} = N/A
Design Spectral Response Acceleration Parameters (g)	S_{DS} = 1.022	S_{D1} = N/A
Mapped PGA (g)	0.64 g	
F_{PGA}	1.20	
PGA_M (g)	0.768 g	
"Method 2" of Section 21.2.1.2 of ASCE/SEI 7-16		
Risk Targeted Ground Motions Spectral Acceleration (Section 21.2)[1]	1.828 g	1.482 g
Design Spectral Response Acceleration Parameters (Method 2)	S_{DS} = 1.219 g	S_{D1} = 0.988 g
Design Spectral Acceleration in accordance with Section 11.4.6 with F_v = 2.5	1.022 g	1.118 g
80% Design Spectral Acceleration in accordance with Section 11.4.6 with F_v = 2.5	0.818 g	0.894 g
Recommended Design Spectral Response Acceleration Parameters	S_{DS} = 1.219 g	S_{D1} = 0.988 g

Notes:
1: Uses scale factors of 1.1 at 0.2s and 1.3 at 1s

5.5 Seismic Hazards (OSSC 1803.6.1, Item 6):

In addition to ground shaking, site-specific geologic conditions can influence soil strength behavior and permanent ground deformations. Based on our subsurface exploration, analysis, literature review and experience, a summary of the potential geologic and seismic hazards at the site are discussed in the following sections.

Earthquake Induced Slope Instability

The site slopes gently towards the east and west at an average slope of flatter than 5H: 1V. Earthquake induced Slope stability is not a concern at these flat slopes.

Liquefaction and Differential Settlements

We conducted a preliminary screening for liquefiable soils based on the Bray and Sancio (Bray & Sancio, 2006) criteria, which suggest soils with plasticity indices below 12 with a natural moisture content greater than 0.85 times the liquid limit are susceptible to liquefaction, and using the Boulanger and Idriss (2006) method which provides recommendations the fine-grained soils with a plasticity index less than 7 are susceptible to liquefaction. Soils which met these criteria were considered potentially liquefiable.

Liquefaction triggering analyses were completed for a magnitude 9.1 earthquake with a peak ground acceleration of 0.77 g. For this analysis, groundwater was assumed at a depth greater than 30 feet BGS. Liquefaction triggering analysis was completed using the method outlined by Boulanger and Idriss (2014). The computer programs LiqSVs version 2.0.1.6 by GeoLogismiki were used to analyze the SPT data. A liquefaction hazard was assumed to exist if the calculated factor of safety against liquefaction was less than 1.2. The loose to medium dense material consisting of poorly graded sand from a depth of 30 feet (assumed groundwater level) to 40 feet are potentially liquefiable. Our analysis indicates post liquefaction settlement on the order of 2- to 5- inches could result during a design seismic event.

Surface Displacement Due to Faulting or Lateral Spreading

The nearest mapped crustal fault is located approximately ten miles from the project site. This fault is not considered to be active. Therefore, fault rupture at the project site is not a seismic hazard. Lateral spreading on the site could occur towards the ocean or Neahkahnie Lake. Based on our preliminary analysis, lateral spreading could range from 12 to 18 inches during design seismic event due to the liquefaction of the later between 30 to 40 feet BGS.

Tsunami or Seiche Inundation

The site is inland and elevated at an elevation of 100 feet above MSL. Accordingly, tsunami or seiche events do not represent a seismic hazard to the site.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

6.1 General

The new City Hall is proposed to be approximately 6,000 square feet two-story building with on-site parking and other miscellaneous improvements. Parts of the existing structures may be renovated and incorporated as part of the new development. The location of the new City Hall was chosen, in part, due to it being outside of the tsunami inundation zone. As such, it may be used as a shelter and muster point in the event of an earthquake and accompanying tsunami. Therefore, the new City Hall may be designed as a risk category IV. Building loads are not known at this time. For the purposes of this report, we have assumed loads on the order of 250 kips and 6 kips per feet for isolated columns and perimeter foundations.

6.2 Design Profile Recommendations

All borings were advanced in the vicinity of the proposed building during our exploration program. These borings were drilled to depths of 11.5 to 81.5 feet BGS. Based on the subsurface conditions encountered in these borings, we have developed the following profile for geotechnical analysis of the bridge and retaining walls.

Table 6 Recommended Soil Strength Properties

Soil Type	Effective Unit Weight, γ (pcf)	Angle of Internal Friction, Φ (degree)	Cohesion, c (psf)	L-Pile Soil Model Type	Non-Default, k (pci)	Strain Factor, ε_{50}
Loose to Medium Dense Poorly-graded Sand (SP) (0 – 10 feet)	110	30	0	Sand	25	NA
Medium Dense Poorly-graded Sand (SP) (10 – 25 feet)	52.6	30	0	Sand	60	NA
Dense Poorly-graded Sand (SP) (25 – 45 feet)	52.6	34	0	Sand	90	NA
Very Dense Poorly-graded Sand (SP) (45 – 80 feet)	57.6	39	0	Sand	125	NA

Notes:
pci: pounds per cubic inch
psf: pounds per square foot
Groundwater was encountered at depths of 10 to 15 feet at time of drilling. For design assume groundwater at 25 feet below ground surface

6.3 Building Foundations Design Approach

We understand that part of this building will be used as an emergency response center and will therefore be designed as a risk category IV structure. Table 12.13-2 within ASCE 7-16 defines the upper limit on lateral ground displacement for use of shallow foundations. Based on our calculations, the lateral spreading is on the order of 12- to 18- inches towards the ocean and the Lake Neahkahnie during a design seismic event. The vertical differential settlements are specified in ASCE 7-16 Table 12.13-3 to be less than 0.002L for risk category IV structures. For a 50 feet span, these differential settlements are calculated as 1.2 inches. As noted, before, the post-liquefaction settlements are calculated as 2 to 5 inches with an estimated differential settlement of 1 to 2.5 inches which is larger than the 1.2 inches allowed for a 50 feet span. We recommend that the owner/design team evaluate these lateral spreading and settlements estimates limits for the type of building and decide if shallow spread footing or deep foundations systems are required.

Based on this analysis, the building foundations should be supported on deep foundations for a risk category IV structure. This deep foundation foundations system should also be designed to resist additional lateral loads due to lateral spreading. We have however provided preliminary recommendations for both shallow spread footings and deep foundations (Continuous Flight Auger (CFA)). Once the building types are decided, these recommendations will be updated as needed.

6.4 Spread Foundations Design Recommendations

Shallow spread footings can be placed on firm native subgrade or on top of engineered fill. Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 24 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 18 inches below the base of the floor slab. Due to the settlement risks associated with post-liquefaction settlements, we recommend that the following two measures be used for the design of foundations:

- Use a minimum of 12 inches of compacted gravels below the spread footings, wall footings and grade beams.
- The structural engineer should evaluate if the calculated post-liquefaction differential settlement of 1 to 2.5 inches is detrimental to the building performance for code required life-safety requirements. Remedial measures like tying the building together using grade beams or other structural measures should be instituted if needed.

The nominal bearing capacity (un-factored) for footings meeting the above minimum dimensions is 6,000 pounds per square feet (psf). Footings bearing on compact native soils should be sized for an allowable bearing capacity of 2,000 psf (Factor of Safety = 3). This is a net bearing pressure. The weight of the footing and overlying backfill can be disregarded in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term-live loads, and this bearing pressure may be doubled for short-term loads such as those resulting from wind or seismic forces.

Based on our analysis, total post-construction settlements were calculated to be less than 1 inch, with post-construction differential settlement of less than 0.5 inch over a 50-foot span for maximum column and perimeter footing loads of less than 250 kips and 6 kips per linear foot.

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction at the base of the footings. A passive earth pressure of 300 pounds per cubic foot (pcf) may be used for footings confined by native soils. Adjacent floor slabs, pavements, or the upper 24-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. For footings in contact with native material, use a coefficient of friction equal to 0.53 when calculating resistance to sliding. Both of these numbers do not include a factor of safety. We recommend a minimum factor of safety of 1.5 be used with these numbers.

The footings should be founded below an imaginary line projecting at a 1-horizontal to 1-vertical (1H:1V) slope from the base of any adjacent parallel utility trenches. The footings must be embedded so there is a minimum of 10 feet of horizontal distance between the base of the footings and any adjacent slope.

A geotechnical engineer or their representative from ROG should confirm suitable bearing conditions and evaluate footing subgrades. Observations should also confirm loose or soft material, organics, unsuitable fill, and topsoil zones were removed. Localized deepening of excavations may be required to penetrate deleterious materials. Any resulting excavations should be backfilled with compacted granular material.

6.5 Deep Foundation Design Recommendations

Our analysis indicates that soils from 30 to 40 feet are potentially liquefiable and therefore the deep foundations should be supported below 40 feet BGS. Continuous flight auger (CFA) piles are an economical method of supporting the proposed structures. We recommend the CFA piles be installed at least 10 feet into the dense to very dense sands which were encountered below a depth of 40 feet in the borings. The minimum depth of the CFA piles is recommended to be 50 feet BGS.

We analyzed the vertical compressive and tension (uplift) load capacities of 18-inch and 24-inch diameter CFA piles using the computer program Shaft® version 2017.8.10 by Ensoft, Inc. We also completed lateral load analysis using computer program LPile® version 2019.11.07 by Ensoft, Inc. The following table summarizes the results of our analysis.

Table 7. Preliminary CFA Pile Capacities

CFA Pile Diameter (inch)	Pile Depth[1] (feet)	Allowable Vertical Capacity (kip)[2] (FS = 3)	Allowable Uplift Capacity (kip)[2] (FS = 3)	Lateral Load at ½ -inch Deflection (Free Head) (kip)	Lateral Load at ½ -inch Deflection (Fixed Head) (kip)
18	50	62	40	13	20
18	55	88	56		
18	60	104	74		
24	50	92	56	20	35
24	55	128	80		
24	60	162	106		

Notes:
1: The depth of embedment is measured from existing ground surface
2: The capacities can be increased by 1/3 during seismic events
3: Additional downdrag loads of 75 kips should be added to piles during design seismic event

The individual nominal vertical resistance of each CFA should be reduced by a factor η for an isolated CFA piles taken as:

- η = 0.65 for a center-to-center spacing of 2.5 diameters
- η = 1.0 for a center-to-center spacing of 4.0 diameters or more

For intermediate spacings, the value of η may be, determined by linear interpolation.

Similarly, p-y multipliers should be used for lateral load capacities depending on the pile spacing. The lateral load analysis should be updated with these multipliers once the pile spacing is decided. The p- multiplier for CFA piles shall be taken as:

- η = 0.70, 0.5, and 0.35 for rows 1, 2, and 3 (and higher) respectively for a center-to-center spacing of 3 diameters
- η = 1, 0.85, and 0.70 for rows 1, 2, and 3 (and higher) respectively for a center-to-center spacing of 5 diameters

For intermediate spacings, the value of η may be determined by linear interpolation.

Additional bending moment and shear forces will be applied to the piles during a lateral spreading event. ROG should evaluate these forces as the design of the building proceeds in conjunction with the structural engineer.

6.6 Foundation Drain Recommendations

We recommend foundation drains around the perimeter foundations of all structures. The foundation drains should be at least 12 inches below the base of the slab. The foundation drain should consist of perforated collector pipes embedded in a minimum 2-foot-wide zone of angular drain rock. The drain rock should consist of drain rock meeting the specifications provided in 2021 version of Oregon State Standard Specifications for Construction (ODOT-SS, 2021) 00430.11 – Granular Drain Backfill Material. The drain rock should be wrapped in a geotextile fabric geotextile fabric meeting the specifications provided in ODOT-SS 2320.20 for soil separation and/or stabilization. The collector pipes should discharge at an appropriate location away from the base of the footings.

6.7 Floor Slab Design Recommendations

The floor slabs should be placed on top of imported granular materials. For on-grade slabs, we recommend an 8-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade. Imported granular material should be crushed rock or crushed gravel and fairly well-graded between coarse and fine, contain no deleterious materials, have a maximum particle size of 1-inch, and have less than 5-percent by weight passing the U.S. Standard No. 200 Sieve. A subgrade modulus of 150 pounds per cubic inch (pci) may be used to design the floor slab for static conditions. Please note that during the design seismic event, distress to the building slabs may occur due to differential settlements related to liquefaction.

The owner and design team should evaluate whether a vapor barrier is needed under the new slab areas. A vapor barrier will reduce the potential for moisture transmission through and efflorescence growth on the floor slabs. Additionally, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and will warrant their product only if a vapor barrier is installed according to their recommendations. Actual selection and design of an appropriate vapor barrier, if needed, should be based on discussions between the owner and members of the design team.

6.8 Retaining Wall Design Recommendations

The retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls, (2) the walls are less than 10 feet in height, and (3) the backfill is drained. Review of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

Unrestrained site walls which retain native soils should be designed to resist active fluid unit weight of 40 pounds per cubic foot (pcf) where supporting slopes are flatter than 4H:1V. The active fluid unit weight shall be increased to 67 pcf where supporting slopes are 2H:1V. A superimposed seismic lateral force based on a dynamic force of $18H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet, and applied at 0.6H from the base of the wall should also be applied to walls supporting slopes flatter that 4H:1V.

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction at the base of the footings. Nominal (Un-factored) passive earth pressure of 300 pounds per cubic foot (pcf) may be used for footings confined by native soils. Adjacent floor slabs, pavements, or the upper 24-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. For footings in contact with native material, use a coefficient of friction equal to 0.53 when calculating resistance to sliding. These numbers do not contain a factor of safety. We recommend a minimum factor of safety of 1.5.

If other surcharges (Foundations, vehicles, etc.) are located within a horizontal distance from the back of a wall equal to twice the height of the wall, then additional pressures will need to be accounted for in the wall design. Contact our office for the appropriate wall surcharges based upon the actual magnitude and configuration of the applied loads. The wall footings should be designed in accordance with the guidelines provided in the “Spread Footing Design Recommendation” section of this report.

The design parameters provided assume back-of-wall drains will be installed in order to prevent buildup of hydrostatic pressures behind all walls. A minimum 12-inch-wide zone of drain rock, extending from the base of the wall to within 6 inches of finished grade, should be placed against the back of all retaining walls. Perforated collector pipes should be embedded at the base of the drain rock. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The backfill material placed behind the walls and extending a horizontal distance equal to at least the height of the retaining wall should consist of granular retaining wall backfill material meeting specifications provided in Oregon's Department of Transportation/City of Portland Standard Specifications for Construction 2021 (ODOT-SS) Section 510.12. We recommend the select granular wall backfill be separated from general fill, native soil and/or topsoil using a geotextile fabric which meets the requirements provided in ODOT-SS 2320.20 for drainage geotextiles. The wall backfill should be compacted to a minimum of 92 percent of the maximum dry density, as determined by ASTM D 1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (e.g., jumping jack or vibratory plate compactors).

Settlements of up to 1% of the wall height commonly occur immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend construction of flat work adjacent to retaining walls be postponed at least four (4) weeks after backfilling of the wall, unless survey data indicates settlement is complete prior to that time.

6.9 Excavation and Temporary Shoring Design Recommendations

The proposed cuts are minimal on the order of less than 4 feet bgs for construction of foundations and utilities for the project. Trench cuts should stand vertical to a depth of approximately 4 feet, provided no groundwater seepage is present in the trench walls. Open excavation may be used to excavate with the walls of the excavation cut at a slope of 1½H:1V, provided groundwater seepage is not present and with the understanding some sloughing may occur. We did not encounter groundwater during our exploration. However, if perched groundwater is encountered provisions should be made to keep groundwater at least 2 feet below the bottom of the excavation.

If shoring or dewatering is used, we recommend the type and design of the shoring and dewatering systems be the responsibility of the contractor who is in the best position to choose systems which fit the overall plan of operation. These excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations.

6.10 Pavement Design Recommendations

Our pavement recommendations are based on the following assumptions:

- A resilient modulus of 4,500 psi for the native site soils.
- A resilient modulus of 20,000 psi estimated for the base rock.
- Initial and terminal serviceability index of 4.2 and 2.5, respectively.
- Reliability and standard deviation of 85% and 0.45, respectively.
- Structural coefficient of 0.42 and 0.10 for the asphalt and base rock, respectively.
- We assumed several Equivalent Single Axle Loads (ESALs) for pavement design. The actual ESALs should be selected based on traffic levels anticipated as the project moves forward.

If any of these assumptions are incorrect, contact our office with the appropriate information so we may revise the pavement design recommendations. Pavement design recommendations were based on the 1993 AASHTO pavement design equations. The development of pavement designs for the project pavements are in general accordance with the design guidelines and procedures of the American Association of State Highway and Transportation Officials (AASHTO) and the Oregon Department of Transportation (ODOT) Pavement Design Manual. Summary of our pavement design recommendations are in the table below.

Table 8 Minimum Pavement Sections

Traffic Loading (ESALs)	Asphalt Cement Concrete (inch)	Aggregate Base Rock (inch)
10,000	3	8
50,000	4	10
100,000	4.5	12
250,000	5.5	12
500,000	6	15
1,000,000	7	15

The thicknesses shown in the table above are intended to be minimum acceptable values.

The asphalt cement (AC) binder should be PG 64-22 Performance Grade Asphalt Cement according to ODOT SS 00744.11 – Asphalt Cement and Additives. The AC should consist of dense graded Level 3, ½-inch hot mix asphalt. The minimum lift thicknesses should be 2-inches. The AC should conform to ODOT SS 00744.13 and be compacted to 91% of Rice Density of the mix, as determined in accordance with ASTM D 2041.

The pavement subgrade should be prepared in accordance with the “Site Preparation” and “Structural Fill” sections of this report.

Construction traffic should be limited to non-building, unpaved portions of the project site or haul roads. Construction traffic should be prohibited on new pavements. If construction traffic is allowable on newly constructed road sections, an allowance for this additional traffic is necessary in the design pavement section.

If moist soil conditions make it difficult to properly moisture condition and compact the roadway subgrade, the use of cement amendment should be considered as alternative to moisture conditioning and compaction. The use of cement amendment will allow for construction of the pavement sections without disturbing the sensitive soil subgrade. If this method is chosen, contact ROG for additional recommendations and alternative pavement sections.

7.0 CONSTRUCTION RECOMMENDATIONS

The construction should be carried out as indicated in accordance with 2021 Oregon Standard Specifications for Construction (APWA Oregon Chapter) (ODOT–SS). We assume these specifications will serve, in part, as the project specifications for items contained within and for those not included in this report.

7.1 Site Preparation

The existing subgrade consist generally of sand. We understand that the existing building and pavements will be demolished and hauled off from the site. The existing near-surface root zone should be stripped and removed from the project site in all proposed new structure or pavement areas. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or stockpiled for use in landscaped areas or as directed by the Owner.

Trees and shrubs should be removed from all new improvement areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet bgs. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend soil disturbed during grubbing operations be removed to expose firm undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

Demolition should include removal of existing improvements throughout the project site. Underground utility lines, vaults, basement walls, or tanks should also be removed or grouted full if left in place. The voids resulting from removal of footings, buried tanks, etcetera, or loose soil in utility lines should be backfilled with compacted structural fill. The base of these excavations should be excavated to firm subgrade before filling with sides sloped at a minimum of 1H: 1V to allow for uniform compaction.

Materials generated during demolition of existing improvements should be transported off site or stockpiled in areas designated by the owner. Asphalt, concrete, gravel fill, and base rock materials may be crushed and recycled for use as general fill.

Following stripping and prior to placing foundations, the exposed subgrade should be evaluated by proof-rolling. The subgrade should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tire construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe the proof-rolling. Soft or loose zones identified during the field evaluation should be compacted to an unyielding condition or be excavated and replaced with structural fill.

7.2 Wet-Weather/Wet-Soil Conditions

Trafficability on the near-surface soils may be difficult during or after extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. Soils which have been disturbed during site-preparation activities, or soft or loose zones identified during probing or proof-rolling, should be removed and replaced with compacted structural fill.

Track-mounted excavating equipment may be required during wet weather. The thickness of the granular material for haul roads and staging areas will depend on the amount and type of construction traffic. A 12- to 18-inch-thick mat of imported granular material is sufficient for light staging areas. The granular mat for haul roads and areas with repeated heavy-construction traffic typically needs to be increased to between 18- to 24-inches. The actual thickness of haul roads and staging areas should be based on the contractor’s approach to site development and the amount and type of construction traffic. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller. Additionally, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material should be 4- to 6-inch minus pit run rock with less than 10% passing a Standard #200 sieve. Note that it is the contractor’s responsibility to protect the subgrade during construction.

7.3 Structural Fills

Fills should be placed over subgrade prepared in conformance with the previous section of this report. Material used as structural fill should be free of organic matter or other unsuitable materials and should meet specifications provided in Oregon Department of Transportation Standard Construction Specifications, 2021 version (ODOT SS), depending upon the application. Discussion of these materials is in the following sections.

7.3.1 Native Soils

The moisture content of the native soils is on the order of 10 to 25 percent. Proper moisture conditioning for structural fill will require large areas and dry summer weather. These soils if properly processed can be used as structural fills. For structural fills these native soils should be placed in lifts with a maximum un-compacted thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density as determined by ASTM D 1557.

7.3.2 Imported Granular Fill

Imported granular material should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in ODOT SS 00330.14 – Selected Granular Backfill, and ODOT SS 00330.15 – Selected Stone Backfill. The imported granular material should be fairly well graded between coarse and fine material and have less than 5% by weight passing the U.S. Standard No. 200 Sieve.

Imported granular material should be placed in lifts with a maximum non-compacted thickness of 8 to 12 inches and be compacted to at least 95% of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 18 inches in non-compacted thickness and should be compacted with a smooth-drum roller without using vibratory action.

Where imported granular material is placed over wet or soft soil subgrades, we recommend a geotextile be placed as a barrier between the subgrade and imported granular material. The geotextile should meet ODOT SS 2320.20 for soil separation and/or stabilization. The geotextile should be installed in conformance with ODOT SS 00350.40 – Geosynthetic Construction.

7.3.3 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (e.g., the pipe zone) should consist of well-graded, granular material with a maximum particle size of 1.5 inches, have less than 10% by weight passing the U.S. Standard No. 200 Sieve, and meet ODOT SS 405.12 - Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90% of the maximum dry density, as determined by ASTM D 1557 or as required by the pipe manufacturer or local building department.

Within roadway alignments or beneath building pads, the remainder of the trench backfill should consist of well-graded, granular material with a maximum particle size of 2.5 inches, have less than 10% by weight passing the U.S. Standard No. 200 Sieve, and meet ODOT SS 405.14 - Trench Backfill, Class B. This material should be compacted to at least 92% of the maximum dry density as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 2-feet of the trench backfill should be compacted to at least 95% of the maximum dry density as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill materials free of organics and materials over 6 inches in size, and meet ODOT SS 405.14 - Trench Backfill, Class A, C, or D. This general trench backfill should be compacted to at least 90% of the maximum dry density, as determined by ASTM D 1557 or as required by the pipe manufacturer or local building department.

7.3.4 Retaining Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of 0.5H, where H is the height of the retaining wall, should consist of select granular material meeting ODOT SS 510.12 – Granular Wall Backfill. We recommend the select granular wall backfill be separated from general fill, native soil and/or topsoil using a geotextile fabric that meets the requirements provided in ODOT SS 2320.20 for drainage geotextiles. The geotextile should be installed in conformance with ODOT SS 00350.40 – Geosynthetic Construction.

7.3.5 Drain Material

Backfill for subsurface trench drains and for a minimum 1-foot-wide zone against the back of retaining walls should consist of drain rock meeting the specifications provided in ODOT SS 00430.11 – Granular Drain Backfill Material. A pre-fabricated drain board can be substituted for the drain rock. The drain rock should be wrapped in a geotextile fabric meeting the specifications provided in ODOT SS 2320.20 for soil separation and/or stabilization. The geotextile should be installed in conformance with ODOT SS 00350.40 – Geosynthetic Construction.

7.3.6 Floor Slab Base Rock

Base aggregate for floor slabs should be clean, crushed rock or crushed gravel. The base aggregate should contain no deleterious materials, meet specifications provided in ODOT SS 02630.10 – Dense Graded Aggregate 1”-0”, and have less than 5% by weight passing the U.S. Standard No. 200 Sieve. The imported granular material should be placed in one lift and compacted to at least 95% of the maximum dry density, as determined by ASTM D 1557.

7.3.7 Pavement Base Aggregate

Imported base aggregate for roads and parking lots should be clean, crushed rock or crushed gravel. The base aggregate should meet the gradation defined in ODOT SS 02630.10 – Dense Graded Aggregate 1”-0,” with the exception that the aggregate should have less than 5% passing a U.S. Standard No. 200 Sieve. The base aggregate should be compacted to at least 95% of the maximum dry density, as determined by ASTM D 1557.

7.4 Drainage Considerations

The Contractor shall be made responsible for temporary drainage of surface water and groundwater as necessary to prevent standing water and/or erosion at the working surface. We recommend removing only the foliage necessary for construction to help minimize erosion.

The ground surface around the structures should be sloped to create a minimum gradient of 2% away from the building foundations for a distance of at least 5 feet. Surface water should be directed away from all buildings into drainage swales or into a storm drainage system. “Trapped” planting areas should not be created next to any building without providing means for drainage. The roof downspouts should discharge onto splash blocks or pavement surfaces which direct water away from the buildings, or into smooth-walled underground drain lines that carry the water to appropriate discharge locations at least 10 feet away from any buildings.

8.0 CONSTRUCTION OBSERVATION RECOMMENDATIONS

Satisfactory earthwork performance depends on the quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining the work is completed in accordance with the construction drawings and specifications. We recommend a geotechnical engineer be retained to observe general excavation, stripping, fill placement, deep foundation installation, footing subgrades, temporary shoring, and subgrades and base rock for floor slabs and pavements. The geotechnical engineer should confirm suitable bearing conditions and evaluate footing subgrades prior to placement of any structural fill for the new structures.

9.0 LIMITATIONS

This report has been prepared for the exclusive use of the addressee and engineers, and for aiding in the design of the proposed project as discussed above. The opinions, comments, and conclusions presented in this report were based upon information derived from our literature review, field investigation, and laboratory testing. Conditions between or beyond our explorations may vary from those encountered. Unanticipated soil conditions and seasonal soil moisture variations are commonly encountered and cannot be fully determined by merely taking soil samples or soil explorations. Such variations may result in changes to our recommendations and may require additional expenditures be made to attain a properly constructed project.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, it is recommended this report be reviewed to determine the applicability of the conclusions and recommendations.

10.0 RESTRICTIONS

This report is for the exclusive use of the client for design of the development, as described in our proposal for this particular project, and this report is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without the expressed written consent of the client and ROG.

Sincerely,
RhinoOne Geotechnical

Levi Good, EIT
Staff Geotechnical Engineer

Rajiv Ali, PE, GE
Principal Geotechnical Engineer

11.0 REFERENCES

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APPENDIX A

Site Location Map
Site Exploration Plan
Quaternary Fault Map

RhinoOne
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12308 NE 56th Street, #1107
Vancouver, WA 98682
360-258-1738

MANZANITA TOWN HALL

MANZANITA AVENUE AND DIVISION STREET
MANZANITA, TILLAMOOK COUNTY, OREGON 97130

FIGURE 1 - SITE LOCATION MAP

PROJECT
COM-2022-001

DATE
NOV 2022

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LEGEND

⌖ *B-0#*

TEST BORING NUMBER AND
APPROXIMATE LOCATION

RhinoOne
GEOTECHNICAL

12308 NE 56th Street, #1107
Vancouver, WA 98682
360-258-1738

MANZANITA TOWN HALL

MANZANITA AVENUE AND DIVISION STREET
MANZANITA, TILLAMOOK COUNTY, OREGON 97130

FIGURE 2 - SITE EXPLORATION PLAN

PROJECT
COM-2022-001

DATE
NOV 2022

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U.S. Geological Survey Quaternary Faults

10/27/2022, 10:38:23 AM

Fault Areas

- ▨ Class B
- ▨ historic
- ▨ late Quaternary
- ▨ latest Quaternary
- ▨ middle and late Quaternary

National Database

- — Historic (< 150 years), well constrained location
- - - - Historic (< 150 years), moderately constrained location
- • • • • Historic (< 150 years), inferred location
- — Latest Quaternary (<15,000 years), well constrained location
- - - - Latest Quaternary (<15,000 years), moderately constrained location
- • • • • Latest Quaternary (<15,000 years), inferred location
- — Late Quaternary (< 130,000 years), well constrained location
- - - - Late Quaternary (< 130,000 years), moderately contrained location
- • • • • Late Quaternary (< 130,000 years), inferred location
- — Middle and late Quaternary (< 750,000 years), well constrained location
- - - - Middle and late Quaternary (< 750,000 years), moderately constrained location
- • • • • Middle and late Quaternary (< 750,000 years), inferred location
- — Undifferentiated Quaternary (< 1.6 million years), well constrained location
- - - - Undifferentiated Quaternary (< 1.6 million years), moderately constrained location
- • • • • Undifferentiated Quaternary (< 1.6 million years), inferred location

1:577,791

0 4.75 9.5 19 mi

0 5 10 20 km

National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp.

USGS

National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp. | USGS |

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Vancouver, WA 98682
360-258-1738

MANZANITA TOWN HALL
MANZANITA AVENUE AND DIVISION STREET
MANZANITA, TILLAMOOK COUNTY, OREGON 97130

FIGURE 3 - QUATERNARY FAULTS MAP

PROJECT
COM-2022-001

DATE
NOV 2022

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APPENDIX B

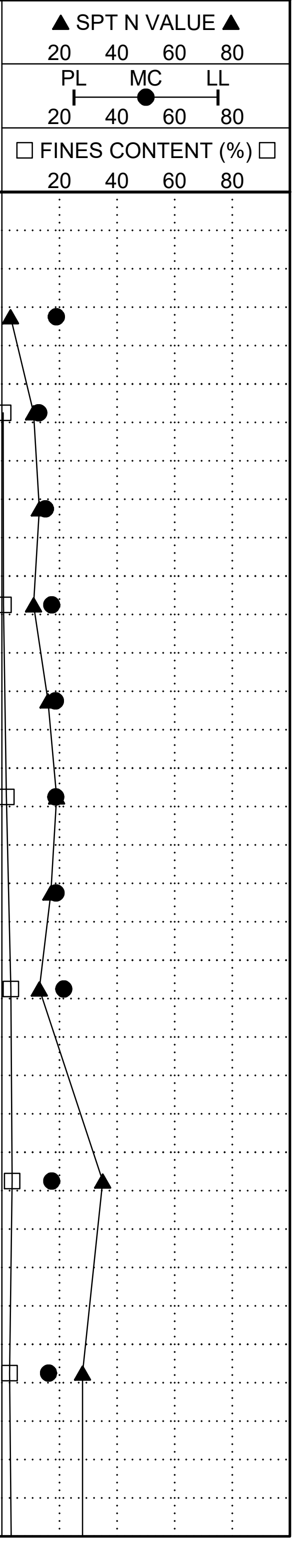
Interpreted Summary Boring Logs
Results of Laboratory Testing

RhinoOne
GEOTECHNICAL

BORING NUMBER B-01

CLIENT City of Manzanita
PROJECT NAME Manzanita City Hall
PROJECT NUMBER COM-2022-001
PROJECT LOCATION Manzanita, OR
DATE STARTED 10/10/22 **COMPLETED** 10/10/22
NORTH **EAST**
DRILLING CONTRACTOR Crisman Pacific Strata Drilling LLC
GROUND ELEVATION 104 ft **HOLE SIZE** 4 inches
DRILLING METHOD Mud Rotary with Auto Hammer
GROUND WATER LEVELS:
LOGGED BY LG **CHECKED BY** PH
AT TIME OF DRILLING --- Not Observed
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0		Poorly-graded SAND (SP); Grey to orange, Moist, Very loose, Fine sand (Beach and Dune Sands)					
			SPT 1	44	1-1-2 (3)		
5		Becomes Grey-brown and Medium dense	SPT 2	61	3-5-6 (11)		
			SPT 3	56	3-5-8 (13)		
10			SPT 4	61	4-5-6 (11)		
			SPT 5	61	5-7-9 (16)		
15			SPT 6	61	6-8-11 (19)		
			SPT 7	56	6-7-10 (17)		
20			SPT 8	61	4-6-7 (13)		
25		Becomes Light brown and Dense	SPT 9	72	9-14-21 (35)		
30		Becomes Medium dense	SPT 10	67	8-12-16 (28)		
35							



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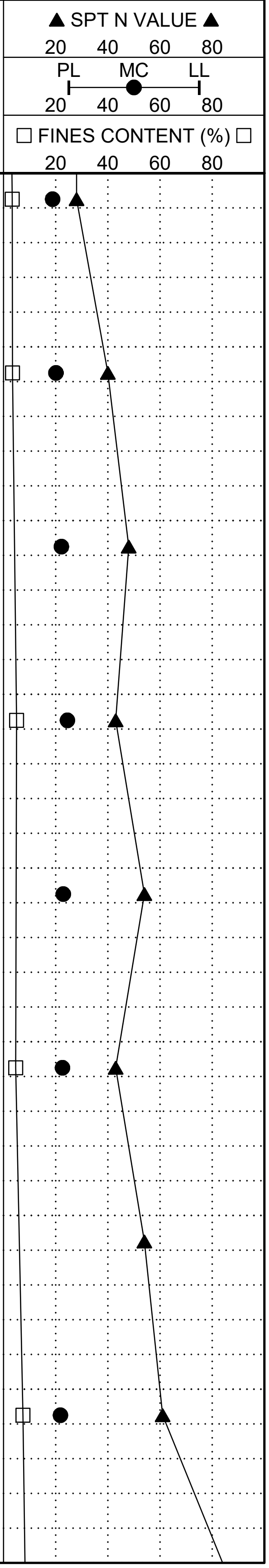
BORING NUMBER B-01

CLIENT City of Manzanita
PROJECT NAME Manzanita City Hall
PROJECT NUMBER COM-2022-001
PROJECT LOCATION Manzanita, OR

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
35	Poorly-graded SAND (SP); Grey to orange, Moist, Medium dense, Fine sand	SPT 11	61	9-12-16 (28)		
40	Becomes Brown and Dense to very dense	SPT 12	78	13-19-21 (40)		
45		SPT 13	100	11-21-27 (48)		
50		SPT 14	61	14-19-24 (43)		
55		SPT 15	89	21-26-28 (54)		
60		SPT 16	61	15-19-24 (43)		
65		SPT 17	0	18-23-31 (54)		
70	Poorly-graded SAND with silt (SP); Light brown, Moist, Very dense, Fine sand	SPT 18	50	18-28-33 (61)		
75						



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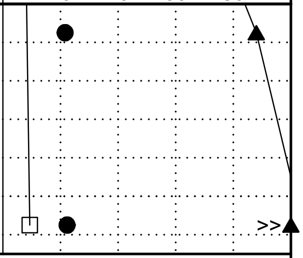
CLIENT City of Manzanita
PROJECT NAME Manzanita City Hall
PROJECT NUMBER COM-2022-001
PROJECT LOCATION Manzanita, OR

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 20 40 60 80 PL MC LL 20 40 60 80 ☐ FINES CONTENT (%) ☐ 20 40 60 80
75		Poorly-graded SAND with silt (SP); Light brown, Moist, Very dense, Fine sand *(continued)*	SPT 19	67	22-39-49 (88)			
80			SPT 20	89	36-46-58 (104)			>>

Bottom of borehole at 81.5 feet.

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BORING NUMBER B-02

CLIENT City of Manzanita

PROJECT NAME Manzanita City Hall

PROJECT NUMBER COM-2022-001

PROJECT LOCATION Manzanita, OR

DATE STARTED 10/11/22 **COMPLETED** 10/11/22

NORTH **EAST**

DRILLING CONTRACTOR Crisman Pacific Strata Drilling LLC

GROUND ELEVATION 106 ft **HOLE SIZE** 4 inches

DRILLING METHOD Mud Rotary with Auto Hammer

GROUND WATER LEVELS:

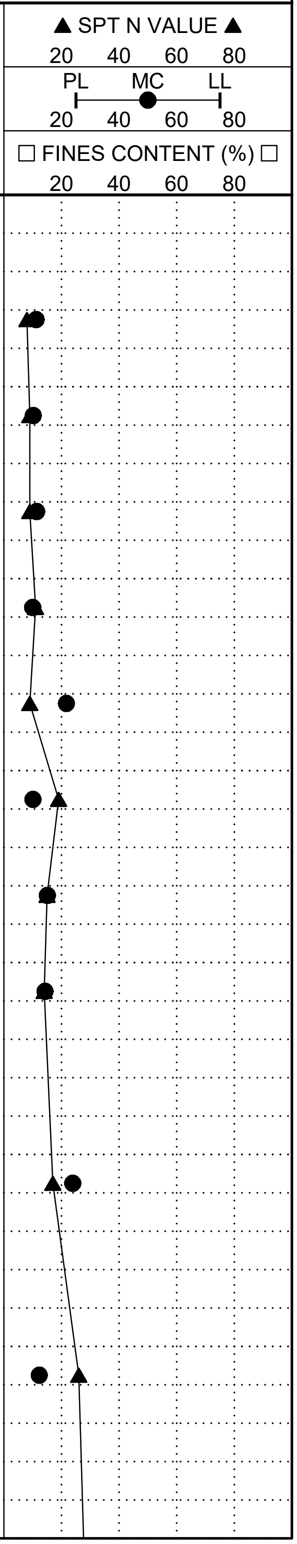
LOGGED BY LG **CHECKED BY** PH

AT TIME OF DRILLING --- Not Observed

NOTES

AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0	Poorly-graded SAND (SP); Light brown, Moist, Loose, Fine sand (Beach and Dune Sands)					
		SPT 1	44	3-3-5 (8)		
5						
		SPT 2	61	2-4-5 (9)		
		SPT 3	44	3-5-4 (9)		
10						
	Becomes Medium dense to Loose	SPT 4	50	3-5-6 (11)		
		SPT 5	50	3-4-5 (9)		
15						
	Becomes Orange-brown and Medium dense	SPT 6	67	5-9-10 (19)		
	Becomes Light brown	SPT 7	56	6-6-9 (15)		
20						
		SPT 8	56	5-6-8 (14)		
25						
		SPT 9	50	6-8-9 (17)		
30						
		SPT 10	61	9-12-14 (26)		
35						



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BORING NUMBER B-02

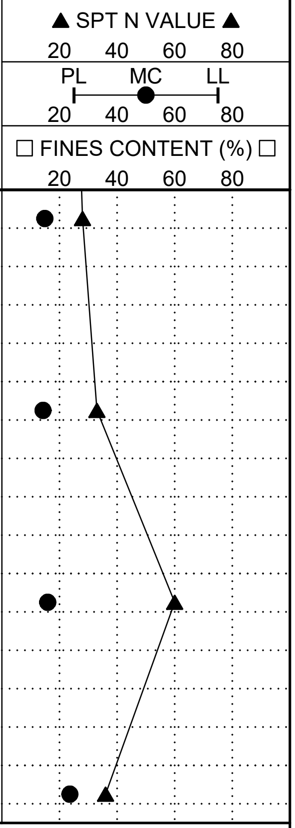
CLIENT City of Manzanita

PROJECT NAME Manzanita City Hall

PROJECT NUMBER COM-2022-001

PROJECT LOCATION Manzanita, OR

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
35		Poorly-graded SAND (SP); Light brown, Moist, Medium dense, Fine sand	SPT 11	56	9-13-15 (28)		
40		Becomes Dense to very dense	SPT 12	61	10-14-19 (33)		
45			SPT 13	72	16-24-36 (60)		
50			SPT 14	67	14-16-20 (36)		

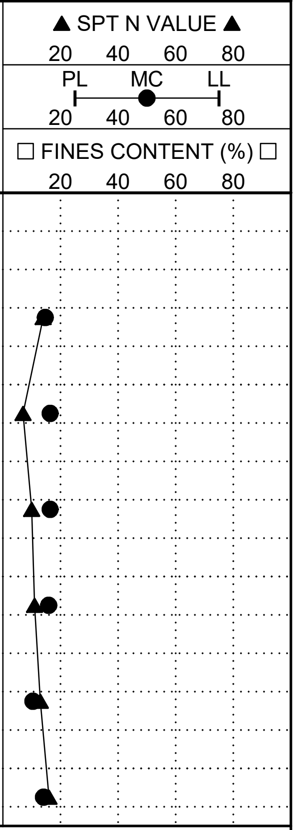


Bottom of borehole at 51.5 feet.

BORING NUMBER B-03

CLIENT City of Manzanita
PROJECT NAME Manzanita City Hall
PROJECT NUMBER COM-2022-001
PROJECT LOCATION Manzanita, OR
DATE STARTED 10/11/22 **COMPLETED** 10/11/22
NORTH **EAST**
DRILLING CONTRACTOR Crisman Pacific Strata Drilling LLC
GROUND ELEVATION 105 ft **HOLE SIZE** 4 inches
DRILLING METHOD Mud Rotary with Auto Hammer
GROUND WATER LEVELS:
LOGGED BY LG **CHECKED BY** PH
AT TIME OF DRILLING --- Not Observed
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0		Poorly-graded SAND (SP); Brown, Moist, Medium dense, Fine sand (Beach and Dune Sands)					
			SPT 1	78	6-7-7 (14)		
5		Becomes Loose	SPT 2	61	3-3-4 (7)		
		Becomes Medium dense	SPT 3	67	3-4-6 (10)		
10			SPT 4	56	3-5-6 (11)		
			SPT 5	56	4-5-8 (13)		
15			SPT 6	50	4-7-9 (16)		



Bottom of borehole at 16.5 feet.

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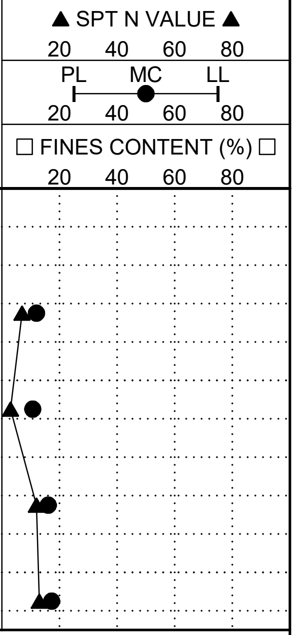
BORING NUMBER B-04

PAGE 1 OF 1

CLIENT City of Manzanita
PROJECT NAME Manzanita City Hall
PROJECT NUMBER COM-2022-001
PROJECT LOCATION Manzanita, OR
DATE STARTED 10/11/22 **COMPLETED** 10/11/22
NORTH **EAST**
DRILLING CONTRACTOR Crisman Pacific Strata Drilling LLC
GROUND ELEVATION 104 ft **HOLE SIZE** 4 inches
DRILLING METHOD Mud Rotary with Auto Hammer
GROUND WATER LEVELS:
LOGGED BY LG **CHECKED BY** PH
AT TIME OF DRILLING --- Not Observed
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
0		Poorly-graded SAND (SP); Brown, Moist, Loose, Fine sand (Beach and Dune Sands)					
			SPT 1	56	2-3-4 (7)		
5		Becomes Very loose	SPT 2	56	2-1-2 (3)		
		Becomes Grey and Medium dense	SPT 3	56	2-4-8 (12)		
10		Becomes Light brown	SPT 4	67	4-6-7 (13)		

Bottom of borehole at 11.5 feet.



BORING NUMBER B-05

PAGE 1 OF 1

CLIENT City of Manzanita

PROJECT NAME Manzanita City Hall

PROJECT NUMBER COM-2022-001

PROJECT LOCATION Manzanita, OR

DATE STARTED 10/11/22 **COMPLETED** 10/11/22

NORTH **EAST**

DRILLING CONTRACTOR Crisman Pacific Strata Drilling LLC

GROUND ELEVATION 105 ft **HOLE SIZE** 4 inches

DRILLING METHOD Mud Rotary with Auto Hammer

GROUND WATER LEVELS:

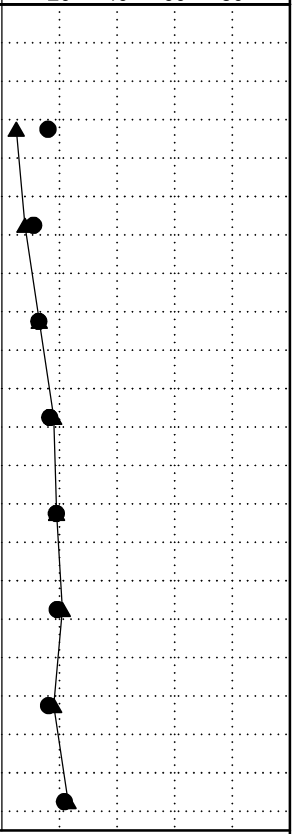
LOGGED BY LG **CHECKED BY** PH

AT TIME OF DRILLING --- Not Observed

NOTES

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 20 40 60 80 PL MC LL 20 40 60 80 ☐ FINES CONTENT (%) ☐ 20 40 60 80
0		Poorly-graded SAND (SP); Grey to orange-brown, Moist, Loose, Fine sand (Beach and Dune Sands)						
			SPT 1	50	0-1-4 (5)			
5		Becomes Brown to light brown	SPT 2	50	3-4-4 (8)			
		Becomes Medium dense	SPT 3	61	5-6-7 (13)			
10			SPT 4	61	5-8-10 (18)			
			SPT 5	72	5-8-11 (19)			
15			SPT 6	78	7-10-11 (21)			
			SPT 7	67	7-8-10 (18)			
20			SPT 8	56	7-12-11 (23)			



Bottom of borehole at 21.5 feet.

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Laboratory Results
Oven Dry Moisture Content (ASTM D 2216)

Project Name:	City of Manzanita	Date:	12-Oct-22
Project Number:	COM-2022-001	Tested By:	RA
Location:	Manzanita, Oregon	Laboratory Number:	2022M 0176

Boring Number	Depth			Tare Number	Weight of Tare	Weight of Tare + Wet Soil	Weight of Tare + Dry Soil	Weight of Dry Soil	Weight of Water	Water Content by Weight
B-01	2.5	-	4	001	51.25	163.04	145.29	94.04	17.75	18.9%
	5	-	6.5	002	51.58	154.31	142.66	91.08	11.65	12.8%
	7.5	-	9	003	51.25	182.42	165.2	113.95	17.22	15.1%
	10		11.5	004	51.36	174.69	156.49	105.13	18.2	17.3%
	12.5	-	14	005	51.73	179.69	159.6	107.87	20.09	18.6%
	15		16.5	006	51.12	176.14	156.42	105.3	19.72	18.7%
	17.5		19	007	51.72	189.98	168.13	116.41	21.85	18.8%
	20		21.5	008	51.82	154.41	136.28	84.46	18.13	21.5%
	25		26.5	009	51.49	155.01	139.77	88.28	15.24	17.3%
	30		31.5	010	51.56	161.06	145.82	94.26	15.24	16.2%
	35		36.5	011	51.61	161.43	144.07	92.46	17.36	18.8%
	40		41.5	012	51.54	150.23	133.7	82.16	16.53	20.1%
	45		46.5	013	50.29	152.29	133.73	83.44	18.56	22.2%
	50		51.5	014	51.22	155.08	134.65	83.43	20.43	24.5%
	55		56.5	015	51.55	181.77	157.51	105.96	24.26	22.9%
	60		61.5	016	51.66	167.27	145.99	94.33	21.28	22.6%
	70		71.5	018	51.58	185.86	161.8	110.22	24.06	21.8%
	75		76.5	019	51.37	195.95	170.3	118.93	25.65	21.6%
	80		81.5	020	51.76	170.67	148.99	97.23	21.68	22.3%
B-02	2.5		4	021	51.01	189.55	175.64	124.63	13.91	11.2%
	5		6.5	022	51.79	167.7	156.94	105.15	10.76	10.2%
	7.5		9	023	51.75	154.35	143.85	92.1	10.5	11.4%
	10		11.5	024	51.7	162.19	152.13	100.43	10.06	10.0%
	12.5		14	025	51.66	183.92	160.38	108.72	23.54	21.7%
	15		16.5	026	51.94	171.09	160.14	108.2	10.95	10.1%

Project Name:	City of Manzanita	Date:	12-Oct-22
Project Number:	COM-2022-001	Tested By:	RA
Location:	Manzanita, Oregon	Laboratory Number:	2022M 0176

Boring Number	Depth			Tare Number	Weight of Tare	Weight of Tare + Wet Soil	Weight of Tare + Dry Soil	Weight of Dry Soil	Weight of Water	Water Content by Weight
	17.5		19	027	51.58	158.67	144.64	93.06	14.03	15.1%
	20		21.5	028	51.48	176.64	160.94	109.46	15.7	14.3%
	25		26.5	029	51.55	163.89	142.23	90.68	21.66	23.9%
	30		31.5	030	51.67	173.68	160.27	108.6	13.41	12.3%
	35		36.5	031	51.46	150.15	137.35	85.89	12.8	14.9%
	40		41.5	032	50.88	156.96	143.72	92.84	13.24	14.3%
	45		46.5	033	50.88	176.66	159.43	108.55	17.23	15.9%
	50		51.5	034	52	164.64	143.1	91.1	21.54	23.6%
B-03	2.5		4	036	51.17	176.92	160.83	109.66	16.09	14.7%
	5		6.5	038	51.82	183.22	164.73	112.91	18.49	16.4%
	7.5		9	001	51.25	154.01	139.51	88.26	14.5	16.4%
	10		11.5	002	51.58	196.73	176.85	125.27	19.88	15.9%
	12.5		14	003	51.25	167.4	156.49	105.24	10.91	10.4%
	15		16.5	004	51.36	152.29	139.82	88.46	12.47	14.1%
B-04	2.5		4	005	51.73	150.93	140.28	88.55	10.65	12.0%
	5		6.5	006	51.12	157.61	147.32	96.2	10.29	10.7%
	7.5		9	007	51.72	151.62	137.8	86.08	13.82	16.1%
	10		11.5	008	51.82	185.43	165.68	113.86	19.75	17.3%
B-05	2.5		4	009	51.49	167.83	151.81	100.32	16.02	16.0%
	5		6.5	010	51.56	155.12	144.82	93.26	10.3	11.0%
	7.5		9	011	51.61	162.51	149.9	98.29	12.61	12.8%
	10		11.5	012	51.54	175.36	157.69	106.15	17.67	16.6%
	12.5		14	013	50.29	183.18	162.04	111.75	21.14	18.9%
	15		16.5	014	51.22	178.86	158.29	107.07	20.57	19.2%

Laboratory Results

Oven Dry Moisture Content (ASTM D 2216)

Project Name:	City of Manzanita	Date:	12-Oct-22
Project Number:	COM-2022-001	Tested By:	RA
Location:	Manzanita, Oregon	Laboratory Number:	2022M 0176

Boring Number	Depth			Tare Number	Weight of Tare	Weight of Tare + Wet Soil	Weight of Tare + Dry Soil	Weight of Dry Soil	Weight of Water	Water Content by Weight
	17.5		19	015	51.55	179.05	161.31	109.76	17.74	16.2%
	20		21.5	016	51.66	189.97	165.27	113.61	24.7	21.7%

RhinoOne GEOTECHNICAL

Laboratory Results
Material Finer Than The #200 Sieve (ASTM D 1140)

Project Name:	Manzanita City Hall	Date:	
Project Number:	COM-2022-001	Tested By:	
Location:	Manzanita Ave & Classic St	Laboratory Number:	

									Post Wash					
Boring Number	**Depth**	**Tare**	**Weight of Tare**	**Weight of Tare + Wet Soil**	**Weight of Tare + Dry Soil**	**Weight of Dry Soil**	**Weight of Water**	**Water Content by Weight**	**Weight of Dry Soil + Tare > #200**	**Weight of Dry Soil > #200**	**Weight of Dry Soil > #4**	**Percent Gravels**	**Percent Sands**	**Percent Fines**
B-1	5.0 - 6.5	B	196.06	329.85	313.84	117.78	16.01	13.6%	313.26	117.20	0.00	0.0%	99.5%	0.5%
B-1	10.0 - 11.5	M	193.65	312.08	297.31	103.66	14.77	14.2%	296.66	103.01	0.00	0.0%	99.4%	0.6%
B-1	15.0 - 16.5	J	193.37	314.02	294.95	101.58	19.07	18.8%	293.29	99.92	0.00	0.0%	98.4%	1.6%
B-1	20.0 - 21.5	N	193.30	369.27	338.26	144.96	31.01	21.4%	333.79	140.49	0.00	0.0%	96.9%	3.1%
B-1	25.0 - 26.5	Q	193.79	353.85	331.07	137.28	22.78	16.6%	326.13	132.34	0.00	0.0%	96.4%	3.6%
B-1	30.0 - 31.5	R	192.91	321.80	304.06	111.15	17.74	16.0%	301.01	108.10	0.00	0.0%	97.3%	2.7%
B-1	35.0 - 36.5	F	194.98	346.41	322.58	127.60	23.83	18.7%	318.32	123.34	0.00	0.0%	96.7%	3.3%
B-1	40.0 - 41.5	D	194.80	351.63	324.15	129.35	27.48	21.2%	319.59	124.79	0.00	0.0%	96.5%	3.5%
B-1	50.0 - 51.5	K	186.10	370.88	335.87	149.77	35.01	23.4%	328.45	142.35	0.00	0.0%	95.0%	5.0%
B-1	60.0 - 61.5	L	193.24	386.19	348.17	154.93	38.02	24.5%	340.91	147.67	0.00	0.0%	95.3%	4.7%
B-1	70.0 - 71.5	P	193.15	372.90	340.21	147.06	32.69	22.2%	329.19	136.04	0.00	0.0%	92.5%	7.5%
B-1	80.0 - 81.5	H	193.93	439.29	395.62	201.69	43.67	21.7%	376.92	182.99	0.00	0.0%	90.7%	9.3%