Preliminary Stormwater Report

Woodburn Community Center

LAND USE

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July 2025 | KPFF Project #1900192

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Joshua A Lighthipe, PE

08/01/2025

REGISTERED PROFESSIONAL ENGINEER

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Preliminary

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JUL. 9, 2002

JOSHUA A. LIGHTHIPE

EXPIRATION DATE: 12/31/ 26

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PROJECT OVERVIEW AND DESCRIPTION

Purpose of this Report

This report describes the stormwater management design strategies for the proposed development. The basis of this report is the City of Woodburn Storm Drainage Master Plan Chapters 7 and 11 and the Public Works Design and Construction Standards and the requirements outlined therein. The purpose of the proposed stormwater management facilities is to meet the city requirements for onsite conveyance and protect existing downstream stormwater infrastructure by providing flow control and detention.

Existing Conditions

The property is in Woodburn, Oregon. The site is currently used for an aquatic center, and portions of the existing building will remain with the renovation. It is located east of Interstate 5 between S Settlemier Ave and S Front Street. The site's existing storm infrastructure drains an existing storm culvert under the Front Street and the railroad tracks and outfalls to Mill Creek.

Proposed Onsite

The proposed site will construct a building addition and 4 parking lot areas. The onsite stormwater runoff will drain to shallow stormwater basins or swales that will treat the stormwater as it filters through the soil media to an underdrain layer. These facilities will contain enough storage volume above ground and in the subdrainage layer to meet the city detention requirements. A summary of the existing and post-development pervious and impervious areas is shown below in Table 1.

TABLE 1: EXISTING AND POST-DEVELOPED ONSITE AREAS

Basin	Impervious Hardscape Area	Impervious Roof Area	Total Impervious Area	Pervious Landscaped Area	Total Site Area	
	(sf)	(sf)	(sf)	(sf)	(sf)	(acres)
Existing	15,004	21,922	36,926	116,437	153,363	3.52
Post-Development	39,458	18,254	57,713	95,650	153,363	3.52

Proposed Offsite

The project involves upgrades to First Street, Oak Street, and 2nd Street, which will require modifications to current storm inlets and the addition of new storm inlets where necessary to accommodate the planned changes. According to guidance from Dago Garcia of the City Public Works Department, improvements to public right-of-way areas are not required to include storm treatment or detention. As a result, the public right-of-way are not part of the proposed stormwater facility design.

METHODOLOGY

Infiltration/Soils

A recent infiltration test performed 2025 by Geopacific showed little infiltration feasibility at 5' deep. The subgrade soils mostly consisted of Sandy SILT (ML). The testing results yielded an infiltration rate of 0.25 inches per hour. Additionally, there is known to be shallow groundwater and areas of undocumented fill. Based on these results and the geotech's recommendations, the storm design will not pursue infiltration as a stormwater management strategy. See **Appendix B** for Geotech Report and infiltration results.

Water Quality Treatment (On-site)

According to the City of Woodburn Storm Drainage Master Plan, there are no stormwater quality or treatment requirements for on-site runoff. As a result, although the proposed stormwater facilities will offer some level of stormwater treatment, calculations detailing the extent of treatment provided are not included in this report.

Detention & Flow Control (On-Site)

The proposed development increases the amount of impervious areas onsite by approximately 59%, therefore the post-development condition will increase the total runoff without mitigation.

According to the City Woodburn Storm Drainage Master Plan Chapter 7 Table 7-1 requires a detention volume of 18,883 CF per 10 acres (or 1888 CF per 1 acre), which is understood to be based on the difference in runoff volume from undeveloped to developed land for the 25-year storm. Additionally, each storm detention facility should discharge through a flow control orifice structure that limits the peak rate to the 5-year or less rate from undeveloped land, which is based on using the rational method with I=0.285 in/hr and C=0.25.

The proposed storm design strategy splits the development into 3 main catchment areas. Each drains to its own storm basin A, B or C that provides the required detention. Each basin is a vegetated rain garden type facility that is designed to pond up to 1.5' deep for basins A & B and 1' deep for basin C. The basins each contain an overflow inlet to ensure safe overtopping during extreme rain events and an underdrain layer to provide positive drainage and prevent standing water. The overflow inlet and the underdrain connect to a flow control structure within a vault located just outside of each basin. The flow control vault includes an orifice structure to limit stormwater peak rate discharge to the required amount. The discharge from the flow-control vault drains to public storm system.

Conveyance

Conveyance calculations will be provided in the final version of this report.

ANALYSIS

Stormwater Management

Water Quality (Onsite)

Not required. No sizing calculations provided in this report.

Water Quantity (Onsite)

Table 2 below shows a summary of the required storage volume and peak flow control rate for each onsite catchment area. The final columns show the actual storage volume provided and the peak flow rates through the orifice control structures.

For catchments B and C, the detention and flow control rates are fully meet with the proposed design.

For catchment A, which includes approximately 15,500 sf of existing roof (A1-EX) and 11,600 sf of existing tennis court (A2-EX), it is not required that these existing impervious areas contribute to the detention and flow control requirements. Therefore, the table shows the volume required by the proposed impervious areas. Basin A has been oversized to offset the unmanaged proposed impervious areas X1, X2 & X3 that due to site grading are impractical to collect and provide flow control runoff for separately.

Refer to Appendix A Exhibit B for information on these three catchments and their respective sub-basin areas.

TABLE 2: ONSITE CATCHMENT AND FACILITY TABLE

Catchment /Area	Sub-Area ID	Source (Roof, Pavement or other)	Area (SF)	C	Storm Facility: Storm Basin ID	Max. Discharge rate (cfs)	Vol Req (cu ft)	Actual vol (cu ft)	Y/N
						C * I *A = (.25) * (0.285 in/hr) * A	1888 CF/ 1 acre = 0.043 CF/SF		
A	A1-EX*	Ex. Roof	15,427	0.9	SP-A2		2,350.6	2,178.2	N
	A2-EX*	Ex. tennis Court	11,636	0.9					
	A3	New Roof	7,804	0.9					
	A4	New Hardscape	19,420	0.9					
		Total :	**54,287**	**SF**					
	Adjusted Total (A3 + A4 + X areas):		**32,900**			**0.05**	**1,425**	**2,178**	**Y**
B	B1	New Roof	10,383	0.9	SB-B1	**0.021**	**568.9**	**875.3**	**Y**
	B2	New Hardscape	2,755	0.9					
		Total :	**13,138**	**SF**					
C	C1	New Hardscape	11,010	0.9	SB-C1	**0.018**	**476.7**	**965.5**	**Y**
		Total :	**11,010**	**SF**					
UNMANAGED AREAS									
X**	X1	New Hardscape	144	0.9					
	X2	New Hardscape	689	0.9					
	X3	New Hardscape	4,843	0.9					
		Total :	**5,676**	**SF**					

*Area EX consists of existing impervious surfaces that are managed by stormwater facilities, but do not require stormwater mitigation because they are either undisturbed areas or are pavement replacement areas where subgrade is not disturbed. These "extra" mitigated areas are used to offset the un-mitigated runoff from Area X.

**Area X consists of redeveloped impervious surfaces that are impractical to capture and mitigate prior to discharge to storm system. Therefore these areas are not managed, but are offset by AREA EX mitigated areas.

CONCLUSION

Based on the requirements of the City of Woodburn and the engineering assumptions and calculations detailed in this report, all facility components have the capacity to manage flow control and treat to the necessary level of pollution reduction for the entire project site.

A copy of the onsite permit plans and details are included in **Appendix C**.

An Operations and Maintenance Manual will be is provided in the final version of the report.

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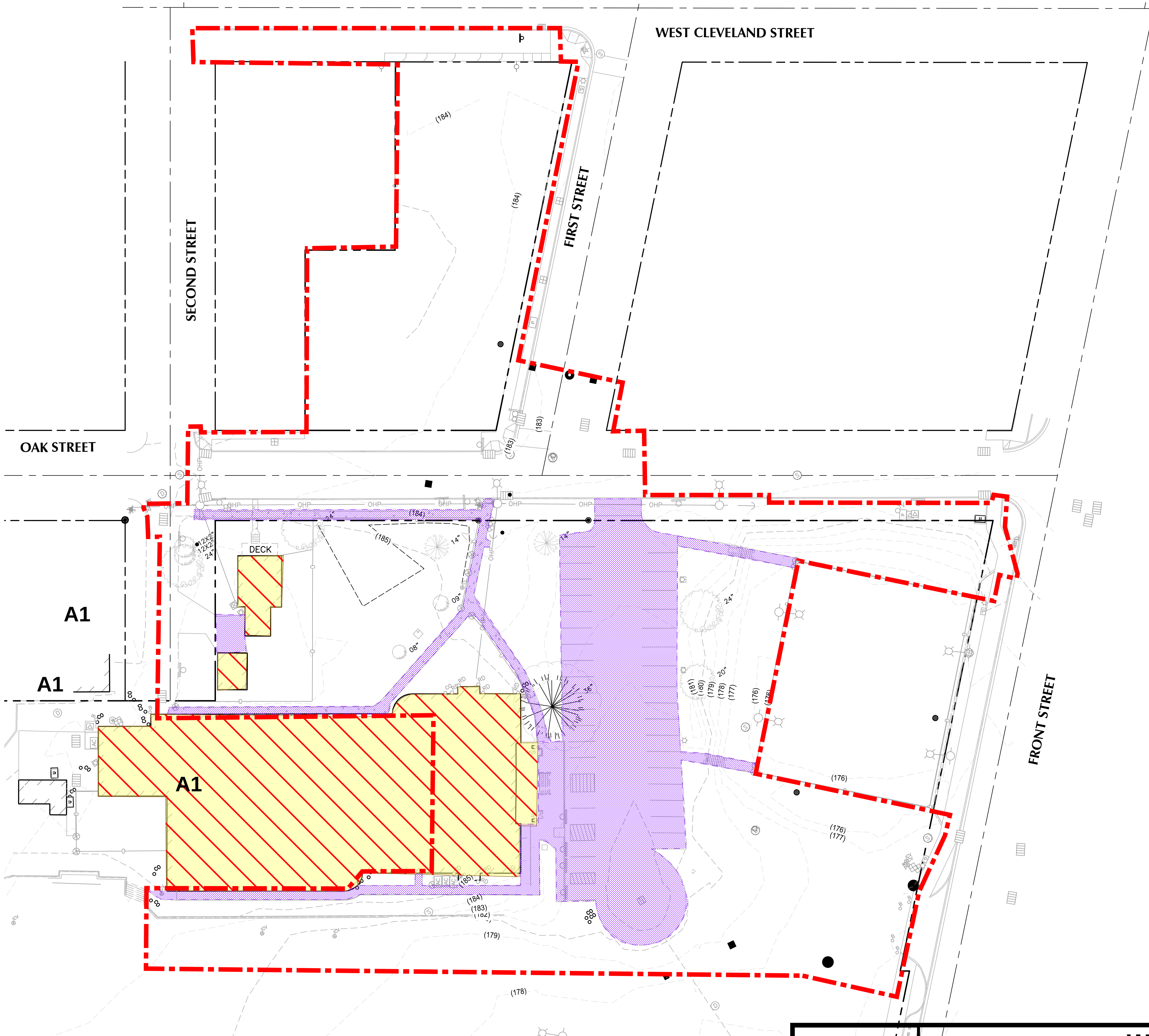
Appendix A

Exhibit A – Existing Conditions Area Map

Exhibit B – Proposed Conditions Basin Map

Exhibit C – Storm Basins B & C

Exhibit D – Storm Basins A



EXISTING CONDITIONS			
	Description	Quantity	Unit
	EXISTING IMPERVIOUS PAVEMENT	20,579	sf
	EXISTING. IMPERVIOUS ROOF	22,430	sf
	TOTAL PROJECT AREA	127,447	sf

SCALE: 1" = 60'
0 60'

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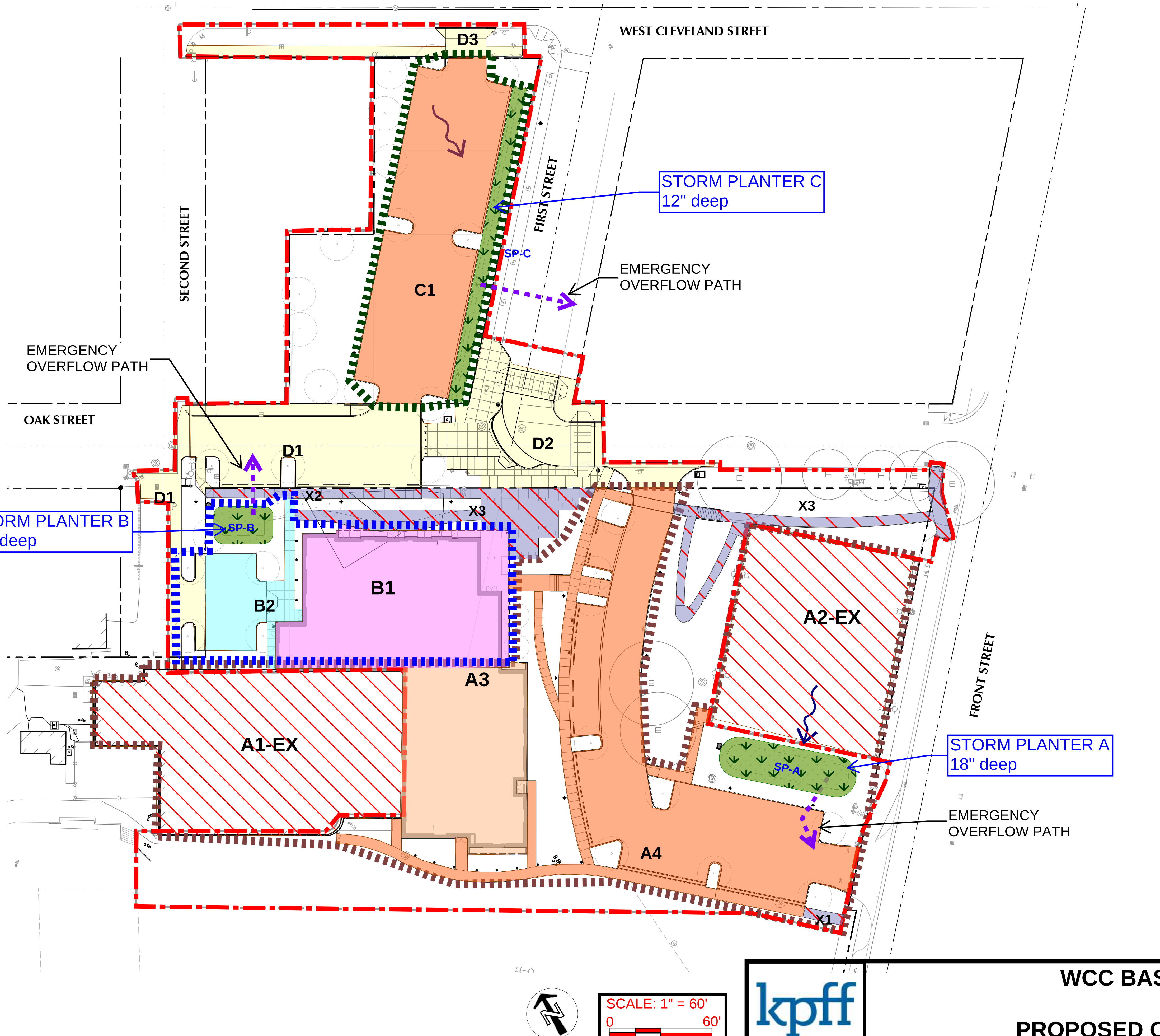
WCC BASIN MAP

EXISTING CONDITIONS

#2500050

EXHIBIT A

05/30/2025



CATCHMENT TOTAL AREAS

Catchment A	68,505 sf
Catchment B	16,430 sf
Catchment C	14,161 sf

TOTAL PROJECT AREA 126,300 sf

STORMWATER FACILITY AREA A

STORM PLANTER A	1,854 sf
STORM PLANTER B	748 sf
STORM PLANTER C	1,520 sf

IMPERVIOUS AREAS

Label	Description	Quantity	Unit
A1-EX	EX. IMPERVIOUS ROOF	15,427	sf
A2-EX	EX. Tennis Court	11,636	sf
A3	PROP. IMPERVIOUS ROOF	7,804	sf
A4	PROP. IMPERVIOUS HARDSCAPE	19,420	sf
B1	PROP. IMPERVIOUS ROOF	10,383	sf
B2	PROP. IMPERVIOUS HARDSCAPE	2,755	sf
C1	PROP. IMPERVIOUS HARDSCAPE	11,010	sf
D1	PROP. Street Improvements	8,344	sf
D2	PROP. Street Improvements	6,748	sf
D3	PROP. Street Improvements	1,475	sf
X1	PROP. IMPERVIOUS HARDSCAPE	144	sf
X2	PROP. IMPERVIOUS HARDSCAPE	689	sf
X3	PROP. IMPERVIOUS HARDSCAPE	4,843	sf

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WCC BASIN MAP

PROPOSED CONDITIONS

#2500050

EXHIBIT B

05/30/2025

Appendix B

Geotech Report

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Report

Project Information:	Woodburn Community Center GeoPacific Project No. 25-6755 March 21, 2025
Site Location:	190 Oak Street Woodburn, OR 97071 Marion County Tax Lots 051W18BA 10200, 10300, 12000, 12100, 12400, 12500, & 12700
Client:	Dago Garcia, City Engineer Public Works Engineering Department 190 Garfield Street Woodburn, OR 97071 Phone (503) 982-5248 Email: dago.garcia@ci.woodburn.or.us

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Portland, Oregon 97224 **Fax (503) 941-9281**

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1.0 PROJECT INFORMATION

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site, assess potential hazards at the property, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-9059, dated February 21, 2025, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

2.0 SITE AND PROJECT DESCRIPTION

As indicated on Figures 1 through 3, the subject site is located at 190 Oak Street and consists of Marion County Tax Lots 10200, 10300, 12000, 12100, 12400, 12500, & 12700, of tax map 051W18BA. While the overall property is much larger, the area where site work is currently planned is approximately 2 to 3 acres in size. The area where site work is currently planned is in the immediate vicinity of the existing Woodburn Aquatic Center building. There is an existing parking lot to the east of the Aquatic Center and to the east of the parking lot there is a tennis court. There is an existing single-family residential structure to the northwest of the Aquatic Center and a sports field to the south of the Aquatic Center.

Topography onsite generally slopes down very gently to the south. Between the parking lot and the tennis court, grades slope down to the east at an inclination of about 20 percent with a total vertical relief of about 8 feet. Vegetation on the site generally consists of short grasses, but there are some trees in the northern portion of the site.

Conceptual site plans indicate that development will consist of the demolition of the eastern portion of the existing Woodburn Aquatic Center building, the construction of a new 16,000 to 18,000 square foot addition, and site work. Site work may include the construction of new parking areas and driving lanes. It is our understanding that subsurface infiltration of stormwater is desired for the site. We anticipate that cuts and fills will be on the order of 6 feet or less. For the proposed structure, the minimum and maximum column loads are expected to be 75 and 225 kips, respectively. The minimum and maximum wall loads are expected to be 4 and 8 kips, respectively. We understand that the building will have a maximum occupancy of 300 or greater, meaning that the building will classify as “special occupancy”, per ORS 455.447. Due to the proposed occupancy, and risk category of the proposed building, a Seismic Site Hazard Investigation per OSCC 1803.3.2 was performed to obtain seismic design parameters as per ASCE 7-16 and OSSC 2022.

3.0 REGIONAL GEOLOGIC SETTING

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

The subject site is underlain by the Quaternary age (last 1.6 million years) Catastrophic Flood Deposits associated with repeated glacial outburst flooding of the Willamette Valley (Madin, 1990). The last of these outburst floods occurred about 10,000 years ago. These deposits typically consist of horizontally layered, micaceous, silt to coarse sand and gravel forming poorly-defined to distinct beds less than 3 feet thick. Locally, the flood deposits are mantled by a thin layer of loess (windblown silt) that is difficult to distinguish from the water deposited silt. Regional studies indicate that the thickness of the Catastrophic Flood Deposits in the vicinity of the subject site is approximately 50 feet (Madin, 1990).

Underlying the Catastrophic Flood Deposits is the Tertiary aged Troutdale Formation that consists of weak to moderately strong conglomerate with interbeds of claystone, siltstone, and sandstone (Beeson, et al., 1991). The Troutdale Formation is underlain by an unnamed sequence of non-marine, fine-grained strata that consists of moderately to poorly lithified siltstone, sandstone, mudstone, and claystone with common wood fragments and minor volcanic ash and pumice (Yeats et al., 1996). These rocks are tentatively correlated with the Sandy River Mudstone, and the Troutdale and Helvetia Formations. The estimated thickness of unnamed sedimentary rock beneath the subject site is about 500 feet. The unnamed strata rest on Miocene (about 14.5 to 16.5 million years ago) Columbia River Basalt, a thick sequence of lava flows which forms the crystalline basement of the basin.

4.0 REGIONAL SEISMIC SETTING

At least two major fault zones capable of generating damaging earthquakes are thought to exist in the vicinity of the subject site. These include the Gales Creek-Newberg-Mt. Angel Structural Zone and the Cascadia Subduction Zone. The location of the site relative to the major fault zones is shown on Figure 3.

4.1 Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW trending faults that lies about 0.3 miles northwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault or Newberg Fault (the fault closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

According to the USGS Earthquake Hazards Program, the Mount Angel fault is mapped as a high-angle, reverse-oblique fault, which offsets Miocene rocks of the Columbia River Basalts, and Miocene and Pliocene sedimentary rocks. The fault appears to have controlled emplacement of the Frenchman Spring Member of the Wanapum Basalts, and thus must have a history that predates the Miocene age of these rocks. No unequivocal evidence of deformation of Quaternary deposits has been described, but a thick sequence of sediments deposited by the Missoula floods covers much of the southern part of the fault trace.

4.2 Cascadia Subduction Zone

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

5.0 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our subsurface explorations for this report were conducted on February 28, 2025 . Three exploratory borings (B-1 through B-3) were advanced to a maximum depth of 101.5 feet below the ground surface (bgs). Additionally, ten exploratory hand auger borings (HA-1 through HA-10) were advanced at the site to maximum depths of 10 feet bgs. A ReMi Array was placed on the ground surface of the proposed development to measure shear wave velocities of soil within the upper 145 feet below the ground surface to obtain data for site-response. The results of the ReMi analysis are presented in the appendix of this report.

Explorations were conducted under the full-time observation of a GeoPacific engineering staff member. During the explorations pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in accordance with the Unified Soil Classification System (USCS). Soil samples obtained from the explorations were placed in relatively air-tight plastic bags. At the completion of the investigation, the borings were backfilled with bentonite chips and, where necessary, patched with asphalt cold patch. Hand auger borings were loosely backfilled with onsite soil. The approximate locations of the explorations are indicated on Figures 2 and 3.

It should be noted that exploration locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate. Summary exploration logs are attached. The stratigraphic contacts shown on the individual exploration logs represent the approximate boundaries between soil types. The actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times. Soil and groundwater conditions encountered in the explorations are summarized below.

5.1 Soil Descriptions

Topsoil Horizon: At the locations of all of our explorations aside from soil boring B-1 and hand auger borings HA-3 and HA-6, the ground was surfaced with a layer of topsoil. The topsoil layer generally consisted of moderately to highly Organic SILT (OL-ML) with fine roots. The topsoil layer extended to approximately 6 to 12 inches below the ground surface (bgs) in our explorations.

Undocumented Fill: At the ground surface in the locations of soil boring B-1 and hand auger borings HA-3 and HA-6, we observed undocumented fill at the ground surface. The undocumented fill material observed in B-1 consisted of SILT with Gravel (ML) and extended to a depth of approximately 1 foot below the ground surface. The undocumented fill observed in hand auger boring HA-3 consisted of a layer of crushed aggregate underlain by stiff SILT with Gravel (ML), which extended to a depth of approximately 5 feet bgs. In hand auger boring HA-6, we observed undocumented fill which consisted of SILT with Gravel (ML) and GRAVEL with Sand (GP). We encountered practical refusal on gravel at a depth of 3 feet in hand auger boding HA-2 and at a depth of 1.75 feet in hand auger boring HA-6. The undocumented fill in hand auger borings HA-2 and HA-6 extended beyond the maximum depth of the exploration.

Buried Topsoil Horizon: Underlying the undocumented fill in hand auger boring HA-3, we observed a buried topsoil horizon layer consisting of soft, gray Organic SILT (OL-ML). This layer extended to a depth of approximately 5.5 feet bgs.

Catastrophic Flood Deposits: Underlying the undocumented fill in B-1, the buried topsoil horizon in HA-3, and the topsoil horizon in all other exploration locations, we observed Catastrophic Flood Deposits. The upper portion of the Catastrophic Flood Deposit soils generally consisted of Sandy SILT (ML) which was brown, gray, blue, or black and varied in sand content. Although some layers within the upper portion of the Catastrophic Flood Deposit soils were medium stiff to stiff, most of these soils were very soft to soft, with SPT N-values as low as 1 to 4. Soils laboratory testing conducted on representative samples collected from our explorations indicated that the SILT (ML) encountered in B-1 contained 65.7 to 69.0 percent by weight passing the U.S. No. 200 sieve, and moisture content of 30.7 to 43.4 percent.

At a depth of approximately 25 feet bgs in B-1, the Catastrophic Flood Deposit soils graded to brown, medium dense Silty SAND (SM). SPT tests within the Silty SAND (SM) indicated that the N-values generally ranged from 15 to 24.

At a depth of approximately 70 feet bgs, the Catastrophic Flood Deposit soils graded to gray and black, dense to very dense, Poorly Graded SAND (SP). The Poorly Graded SAND (SP) ranged from dense to very dense, with SPT N-values ranging from 31 to 69. The Poorly Graded SAND (SP) extended beyond the maximum extent of our explorations (101.5 feet).

5.2 ReMi Array

A seismic survey was performed in one array location to determine the shear wave velocity of the soil profile for liquefaction analysis. The surveys were performed by recording active and/or ambient (passive) seismic sources. The seismic recording array for these surveys consisted of 12, 4.5 Hz geophones at 26.3 ft spacing, for a total survey length of 290 feet. Noise generated by off-end Hammer Blows and walking along the array line during data acquisition while ambient noise was generated from traffic along the nearby roads. The seismic data were acquired using a ReMiDAQ™ 5-12 channel seismograph, while data was processed using Terēan's ReMi™ software (terean.com/products). Survey results indicate weighted-average soil shear wave velocities of 855 ft/s at the location of Array 1, which extended to an inferred depth of approximately 145 feet below the ground surface. These results indicate Seismic Site Class D according to Table ASCE 7-16. Results of the ReMi analysis data are attached to this report.

5.3 Shrink-Swell Potential

Low plasticity, fine-grained and coarse-grained soils were encountered near the ground surface within subsurface explorations conducted at the site. Based upon the results of our observations, laboratory testing, and our local experience with the soil layers in the vicinity of the subject site, the shrink-swell potential of the soil types is considered to be low. Special design measures are not considered necessary to minimize the risk of uncontrolled damage of foundations as a result of potential soil expansion at this site.

5.4 Groundwater and Soil Moisture

On February 28, 2025, observed soil moisture conditions above the groundwater table were generally .generally moist to very moist. Groundwater was measured at a depth of approximately 7 feet in soil boring B-1 and at a depth of approximately 6 feet in hand auger boring HA-5. Soil borings B-2 and B-3 were drilled with mud rotary drilling methods, and therefore it was not possible to obtain groundwater measurements in those borings. According to local well logs, groundwater has been recorded at depths ranging from10 to 20 feet bgs in the vicinity of the subject site.

It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored and may become evident during site grading.

5.5 Infiltration Testing

We performed soil infiltration testing at the site using the open-hole falling head method within hand auger boring HA-4. The test was conducted in native soils at an approximate depth of 5.0 feet below the ground surface. The test hole was pre-saturated prior to recording measurements. Tested native soils classified as Catastrophic Flood Deposits consisting of Sandy SILT (ML). During testing, we measured the water level to the nearest 0.01 foot (1/8 inch) from a fixed point and the change in water level was recorded at regular intervals until three successive measurements showing a consistent infiltration rate were achieved. The native Sandy SILT (ML) exhibited an infiltration rate of 0.25 inches per hour.

The measured infiltration rate does not incorporate factors of safety and reflects vertical flow pathways only. The presence of shallow groundwater, undocumented fill material, and the contact between the fill material and native soil should be considered in the design of stormwater management systems, as discussed in the section of this report titled Stormwater Management.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicates that the proposed construction appears to be geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The primary geotechnical concerns for the proposed development are:

1. The risk of damage to the proposed structure due to liquefaction and/or cyclic softening. In the event of a large earthquake, our analyses show that some soil layers in the subsurface profile could experience liquefaction and cyclic softening. The majority of the liquefaction and cyclic softening is expected to occur within the upper 20 feet of the soil profile. Without remedial measures, there would be a risk of damage to the proposed structure due to differential settlement and reduction of bearing capacity support.

   Without ground improvements, we estimate that the total dynamic settlement expected at the subject site due to soil liquefaction and cyclic softening would be up to 3.9 inches, and that the differential settlement expected due to soil liquefaction and cyclic softening would be up to 2.0 inches over a horizontal distance of 20 feet.

2. The presence of soft soils on the site. The near-surface soils in the vicinity of the proposed structure varied in stiffness from very soft to medium stiff. Since some of the soils were very soft to soft and because soil stiffness varies across the footprint of the proposed structure, unless remedial measures are implemented, the proposed structure could experience damage due to differential settlement.

3. The presence of undocumented fill and buried topsoil on the site. During our site investigation, we encountered undocumented fill to a depth of up to 5 feet below the ground surface in some areas of the site. In hand auger boring HA-3, we encountered buried topsoil to a depth of approximately 5.5 feet. We understand that the properties in the northern portion of the proposed development area, in the vicinity of hand auger borings HA-6 through HA-8, were previously occupied by residential structures. We anticipate that some debris associated with these structures, such as concrete slabs, septic tanks, etc., may be encountered in this area.

   Where encountered, undocumented fill material should be completely removed from the influence zone of the proposed structures. In areas where parking and drive areas are proposed and undocumented fill is present, it may be feasible to allow some of the undocumented fill soils to remain in place, pending evaluation by GeoPacific and provided that some vertical settlement and maintenance is acceptable to the owner. The existing undocumented fill soils should be evaluated in areas of proposed parking and drive areas by proofrolling with fully loaded haul trucks and by potholing. The existing fill material should be free of buried organic debris, voids, etc. If existing soils are to remain in place in parking and drive areas, the upper portion of existing undocumented fill soils in parking and drive areas may need to be ripped and recompacted.

In order to mitigate the main geotechnical issues for the proposed building, we recommend that ground improvement be installed to stiff or medium dense soils, which were encountered at a depth of 25 feet below the existing ground surface in soil boring B-1. We recommend that the ground improvement generally consist of engineered aggregate piers installed with displacement methods, so that no casing will be required. We anticipate that the use of ground improvements to an anticipated depth of 25 feet can lower the estimated magnitude of differential settlement to levels of less than 0.5 inches over a horizontal distance of 40 feet and mitigate the risk of damage to the structure due to loss of bearing capacity support, thereby eliminating the need for other remedial measures, such as foundation ties, mat foundations, or deep foundations. To limit disturbance to the existing building, we recommend that a non-vibratory method of ground improvement, such as rigid inclusions, be utilized within 15 horizontal feet from the existing building.

Where rigid inclusions are utilized as ground improvement, we recommend that a layer of crushed aggregate be placed over the rigid inclusions and below the foundation. This layer of compacted crushed aggregate is intended to help transfer loads from the footings to the rigid inclusions and to help avoid stress concentrations. The layer of crushed aggregate should be at least 2.5 feet thick below the foundation elements of the structure.

Provided that the engineered aggregate piers, rigid inclusions, and the crushed aggregate base rock are installed as recommended in this report, it is our opinion that the risks of damage to the proposed structure due to soil liquefaction and cyclic softening will be adequately mitigated.

The recommended ground improvements will also adequately mitigate the risks of damage to the proposed structure due to the presence of soft soils in the static condition and provide an increased allowable bearing capacity.

A sketch illustrating the recommended mitigative measures is shown in Figure 7. The following report sections provide recommendations for site development and construction in accordance with the current applicable codes and local standards of practice.

6.1 Site Response Analysis

We understand that the buildings would classify as “special occupancy,” per ORS 455.447, meaning buildings with a capacity greater than 300 individuals. Due to the proposed occupancy, and risk category of the proposed building, a Seismic Site Hazard Investigation per OSCC 1803.3.2 was performed to obtain seismic design parameters as per ASCE 7-16 and OSSC 2022. Data from shear wave velocity measurements obtained in our ReMi analysis and soil properties from our boring logs indicate that the soil within the upper 100 feet below the ground surface is classified as Site Class D. Since a site response analysis was performed onsite, the determining of exception from Site Class F (ASCE 7-16 20.3.1) due to liquefiable soils is not relevant to this study.

Where site-specific procedure is used to determine the ground motions in accordance with Chapter 21 of ASCE 7-16, OSSC 2022 allows for the site-specific calculated spectra (Sa) and Peak Ground Acceleration (PGA and PGA_M). The site-specific values shall not be taken as less than 80% of the earthquake ground motion parameters per ASCE 7-16, Chapter 11.

The results of this study have shown that a reduction in ground motion parameters is permitted. A site-specific Peak Ground Acceleration (PGA) value of 0.307g (80% of the PGA_M per ASCE 7-16) may be used for design. A reduction of 20% is permitted for certain spectra. However, our study has shown that spectra for some periods are greater than code values. Table 1, presented below, should be referenced for spectra greater than 1 second (also see Figure 6 in the appendix of this report for calculated spectral accelerations in comparison to code-based values). We anticipate that the provided spectral accelerations for periods of up to 5 seconds will be sufficient for the design process of this project. However, if spectral accelerations for periods greater than 5 seconds are needed for design, GeoPacific should be consulted for additional recommendations.

Table 1 – Recommended Design Spectrum Obtained from Site-Response Analysis (See Figure 6)

Period (s)	Spectral Acceleration (g)
0.01	0.228
0.03	0.270
0.04	0.292
0.05	0.316
0.07	0.351
0.11	0.435
0.13	0.465
0.16	0.521
0.18	0.521
0.25	0.521
0.34	0.521
0.40	0.521
0.77	0.521
***1.00**	0.407
***1.98**	0.392
***3.40**	0.175
***4.00**	0.128
***5.00**	0.082

Notes:
1. Design values denoted by "*" are greater than 80% of ground motion accelerations obtained per ASCE 7-16, Chapter 11 (See Figure 6 in the appendix of this report).
2. Linear interpolation may be applied to calculate spectral accelerations between periods.
3. Where MCE_R spectrum is required, it shall be determined by multiplying the design response spectrum by a factor of 1.5 as per ASCE 7-16 section 11.4.7.

6.2 Probabilistic Risk Targeted Bedrock Spectra

The depth to the site class B/C boundary could not be determined under the feasible scope of the current study, therefore the hazard spectra was developed based upon Site Class D. Geologic mapping indicates that the estimated depth to bedrock in the vicinity of the subject site is several hundred feet below the ground surface (O' Connor, 2001). Therefore, the site response model was constructed at a depth of 145 feet below the ground surface using seed motions scaled to a Uniform Hazard Response Spectra (UHRS) provided for Site Class D.

We obtained probabilistic UHRS from the United Stated Geological Survey (USGS) Unified Hazard Tool with spectra ranging from 0 to 5 seconds for Site Class D spectra where the soil shear wave velocity (V_s) is at least approximately 1,000 ft/s or 305 m/s. The probabilistic spectrum is for a 5 percent damped acceleration response and a 2 percent probability of exceedance within a 50-year period, as per ASCE 7-16.

To define the spectra more precisely, values were obtained by logarithmically interpolating between given UHRS values per equation 3-2 of *Site-Specific Ground Motions for Seismic Design of Buildings and Other Structures* (Malhotra 2022). Risk coefficients (C_{Rs} = 0.886, and C_{R1} = 0.866) were applied as per ASCE 7-16, 21.2.1.1 (Method 1). To account for the maximum direction of horizontal response, a rotation factor of 1.1 was applied for short periods less than or equal to 0.2 seconds, 1.3

for periods between 0.2 seconds and 1 second, and 1.5 for long periods of 5 seconds or greater. Risk coefficients and maximum rotation factors were linearly interpolated between denoted periods. The values used to calculate MCE_R utilized for site-response are presented on Table 2 below.

Table 2 – Site Specific MCE_R and Coefficients

Period (sec)	$S_a(V_s=760)$ (g) UHRS	Risk Coefficient $C_{R(Vs=760)}$	Maximum Rotation Factors	$S_a(vs=760)$ (g) MCE_R
0.01	*0.511	0.886	1.10	0.408
0.02	*0.544	0.886	1.10	0.454
0.03	*0.578	0.886	1.10	0.499
0.05	*0.611	0.886	1.10	0.545
0.075	*0.677	0.886	1.10	0.638
0.10	0.758	0.886	1.10	0.757
0.15	*0.839	0.886	1.10	0.878
0.175	*0.984	0.886	1.10	0.894
0.2	1.053	0.886	1.10	0.901
0.25	*1.119	0.886	1.10	0.909
0.3	1.212	0.885	1.11	0.802
0.4	1.231	0.884	1.13	0.759
0.5	1.200	0.881	1.15	0.736
0.75	*1.167	0.879	1.18	0.711
1	1.079	0.872	1.24	0.645
1.5	*0.984	0.866	1.30	0.574
2	0.729	0.866	1.33	0.391
3	0.483	0.866	1.35	0.213
4	0.327	0.866	1.40	0.145
5	0.251	0.866	1.45	0.112

Note: "*" design values were obtained by logarithmic interpolation between USGS deaggregation UHRS values (Dynamic: Conterminous U.S. 2014) per equation 3-2 of *Site-Specific Ground Motions for Seismic Design of Buildings and Other Structures* (Malhotra, 2022)

6.3 Design Acceleration Parameters

Magnitude, rupture distance and other target fault characteristics were obtained from a deaggregation (Dynamic: Conterminous U.S. 2014) generated by the USGS Unified Hazard Tool. These characteristics were used to select ground motions for site-response. Based upon the results of the deaggregation, ground motions at the site are primarily controlled by the Cascadia Subduction Zone events (interface and intraslab) as well as the Mount Angel Fault crustal event. Results of the deaggregation presented on Table 3 were used to select ground motions for the site response analysis.

Table 3 – Deaggregation Contributors to MCE_R (Site Class D)

Event	Cascadia Megathrust (CSZ)	Coastal Deep (CSZ)	Mount Angel Fault (MAF)	Other/Grid
Magnitude	8.48 - 9.10	-	6.63 – 6.75	-
Rupture Distance (km)	71.5 – 122.8	-	0.4 – 3.0	-
Rupture	Subduction	Subduction	Crustal	-
Mechanism	(Interface)	(Intraslab)	(Reverse)	-
% Contribution to Deaggregation	54.4	14.0	8.0	23.6

A total of six recorded bedrock motions were selected to represent the hazard contributions obtained from the deaggregation and experience of the local geology and seismic setting. GeoPacific considered the faulting mechanisms, magnitude, and rupture distance from the deaggregation to select ground motions for analysis. Four motions were selected to represent the CSZ interface subduction event, and the other 2 motions were selected to represent the CSZ intraslab subduction event and the shallow crustal event from the Mount Angel Fault. Once the ground motions were selected, scale factors were applied so that the geometric mean of the ground motions would match the MCE_R spectrum. Table 4 presents the selected ground motions selected and scale factors utilized for site response. Figure 5 in the appendix of this report presents a graph of the scaled ground motions and MCE_R spectral accelerations.

Table 4 – Selected Ground Motions for Site Response

Event	1992 Cape Mendocino, CA	2001 El Salvador	1985 Mexico City, Mexico	2011 Tohoku, Japan	2010 Maule, Chile	2001 Arequipa, Peru
Station	Cape Mendocino (CPM)	Acajutla Cepa (CA)	La Union (UNIO)	Tajiri (MYGH06)	Cerro Santa Lucia (STL)	Moquegua (MOQ)
Component	0	90	N00W	NS	360	NS
Magnitude	7.01	7.7	8	9	8.8	8.4
Rupture Distance (km)	6.96	151.8	*83.9	63.8	64.9	76.7
Vs30 (m/s)	568	Intermediate Intrusive Rock	Meta-Andesite Breccia	593	1411	573
Rupture	Crustal	Subduction	Subduction	Subduction	Subduction	Subduction
Mechanism	(Reverse)	(Intraslab)	(Interface)	(Interface)	(Interface)	(Interface)
Seed Motion $D_{5\text{-}95}$ (sec)	9.7	27.2	24.2	85.5	40.7	36
Seed Motion PGA(g)	1.51	0.10	0.17	0.27	0.24	0.22
Scale Factors Applied for Spectra Matching	**2.00**	**2.10**	**2.00**	**2.06**	**2.00**	**2.10**

Note: Rupture distance denoted by "*" is hypocentral distance.

6.4 Site Response Modelling

Spectral accelerations for seismic design were obtained by performing a site-specific site response utilizing the selected ground motions presented on Table 4. An equivalent linear seismic site response analysis was performed in accordance with ASCE 7-16 Section 21.1. The equivalent linear site response analysis was performed using DEEPSOIL v7.1.70.

The input soil profile used in our model is based upon subsurface information obtained during our site-specific explorations conducted on February 28, 2025. Our site-specific explorations on the site were interpreted to a depth of approximately 145 feet below the ground surface. The site class B/C boundary was not observed within our site-specific explorations or inferred from the ReMi analysis. Geologic mapping indicates that the estimated depth to bedrock in the vicinity of the subject site is several hundred feet below the ground surface. Therefore, the site response model was constructed at a depth of 145 feet below the ground surface using seed motions scaled to a UHRS provided for Site Class D. The soil properties were used to develop a DEEPSOIL model incorporated estimated shear wave velocities and are presented on Table 5 below. The results of the site-response models in comparison with code seismic accelerations are presented on Figure 6.

Table 5 – Soil Input Profile for Site Response

Depth Interval (ft)	Soil Type	Shear Wave Velocity (ft/s)	Modulus Reference Curve
0-15.6	Sandy Silt	494	Darendeli (2001)
15.6-32.1	Sandy Silt	1,388	Darendeli (2001)
32.1-55.9	Silty Sand	575	Seed and Idriss (1970)
55.9-76.8	Very Dense Sand	2,003	Seed and Idriss (1970)
76.8-130.2	Dense Sand	1,097	Seed and Idriss (1970)
130.2-145+	Dense Sand	1,589	Seed and Idriss (1970)

Note: Residual shear strength was utilized as a secondary analysis to account for liquefied soil (Kramer 2015), the greater of the two models was incorporated in recommended design spectra.

6.5 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2025 Statewide GeoHazards Viewer indicates that the site contains areas considered to be at *moderate* risk for soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction typically occurs in loose sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15, and SPT N-Values lower than 15.

The subsurface profile observed within our subsurface explorations, which extended to a maximum depth of 101.5 feet bgs in our explorations, and correlations from the ReMi data provided which were interpreted to a maximum depth of 145 feet bgs, indicate that the site is underlain by very soft to stiff, silt and clay, and medium dense to very dense silty sand and sand. We encountered static groundwater in our explorations at depths of approximately 7 feet bgs in the vicinity of the new building.

The liquefaction potential at the subject site was analyzed for the soil profiles encountered within soil boring B-1 using CLiqSVs version 2.2.1.8, by Geologismiki, and the Boulanger and Idriss method of analysis (2014). The depth of analysis was 100 feet bgs. For design purposes, the groundwater table during an earthquake was estimated to be 7 feet bgs during an earthquake. Using a site-specific peak horizontal ground acceleration of 0.37g (80% of the PGA_m per ASCE 7-16), and an earthquake moment magnitude of 9.1 based upon data obtained from the U.S. Geological Survey (USGS) 2025 Earthquake Hazards Program, the factor of safety was less than 1 for some soil layers, indicating the potential for liquefaction and cyclic softening during an earthquake.

Based upon our analysis of the existing soil profile, potentially liquefiable layers are present underlying the subject site at depths ranging from approximately 7 to 40 feet bgs. Our analysis indicates that the most prevalent liquifiable layers are present in the upper 20 feet of the subsurface profile. Soils meeting the criteria for potentially liquefiable soil layers during an earthquake at this site include the silty sand and sandy silt soils. See the attached exploration logs and results of our liquefaction analyses for more information.

Estimates of anticipated seismically induced settlement at our boring location are summarized on Table 6.

Table 6 - Anticipated Settlement for Design Seismic Event Without Ground Improvement

Exploration Location	Total Seismic Permanent Vertical Deformation (in.), Unimproved	Differential Seismic Permanent Vertical Deformation (in.) Over 20 Horizontal Feet, Unimproved
B-1	3.9	2.0

As a result of soil liquefaction, the soils below the proposed structure could have reduced bearing capacity support, which could result in a bearing capacity failure or leaning of the structure.

In order to mitigate the main geotechnical issues for the proposed building, we recommend that ground improvement be installed to stiff or medium dense soils, which were encountered at a depth of 25 feet below the existing ground surface in soil boring B-1. We recommend that the ground improvement generally consist of engineered aggregate piers installed with displacement methods, so that no casing will be required. In order to lower the risk of damage to the existing structure from vibrations during installation, we recommend that rigid inclusions be utilized for footings within 15 feet of the existing structure.

Where rigid inclusions are utilized as ground improvement, we recommend that a layer of crushed aggregate be placed over the rigid inclusions and below the foundation. This layer of compacted crushed aggregate is intended to help transfer loads from the footings to the rigid inclusions and to help avoid stress concentrations. The layer of crushed aggregate should be at least 2.5 feet thick below the foundation elements of the structure.

The *Structural Foundations* section of this report provides more information on the recommended ground improvements. A sketch illustrating the recommended mitigative measures is shown in Figure 7.

Provided that the engineered aggregate piers, rigid inclusions, and the crushed aggregate base rock are installed as recommended in this report, it is our opinion that the risks of damage to the proposed structure due to soil liquefaction and cyclic softening will be adequately mitigated. We anticipate that the use of ground improvements to an anticipated depth of 25 feet can lower the estimated magnitude of differential settlement to levels of less than 0.5 inches over a horizontal distance of 40 feet.

The recommended ground improvements will also adequately mitigate the risks of damage to the proposed structure due to the presence of soft soils in the static condition and provide an increased allowable bearing capacity, thereby eliminating the need for other remedial measures, such as foundation ties, mat foundations, or deep foundations.

6.6 Other Seismic and Geological Hazards

Additional potential seismic impacts we considered during our geotechnical study include, lateral spreading, fault rupture potential, and other hazards as discussed below:

- **Fault Rupture Potential** – Based on our review of available geologic literature, we are not aware of any mapped active (demonstrating movement in the last 10,000 years) faults on the site. During our field investigation, we did not observe any evidence of surface rupture or recent faulting. Therefore, we conclude that the potential for fault rupture on site is very low.

- **Lateral Spreading** – Lateral spreads involve down-slope movement of large volumes of liquefied soil. Often, layers of non-liquefied soils overlying the liquefied material are also translated down-slope. Lateral spreads generally develop on moderate to gentle slopes and move toward a free face such as a riverbank. Due to the relatively level topography in the vicinity of the subject site, it is our opinion that the risk of lateral spreading is very low and that no special design measures are required to address horizontal displacements due to lateral spreading.

- **Seismically Induced Landslide** – Site grades are flat to gently sloping, with little or no relief across the site. The potential for slope instability and seismically induced landslide on site is considered very low.

- **Effects of Local Geology and Topography** – In our opinion, no additional seismic hazard will occur due to local geology or topography. The site is expected to have no greater seismic hazard than surrounding properties and the Willamette Valley area in general.

6.7 Structural Foundations

As discussed in the *Soil Liquefaction* Analysis section of this report, without ground improvement, we estimate that up to 3.9 inches of seismically induced settlement will occur in the design earthquake even, and that seismically induced differential settlement would be approximately 2.0 inches over a horizontal distance of 20 feet. Since there are layers of very soft to soft native soils and the stiffness of the native soil within 15 feet of the ground surface varies across the footprint of the proposed structure, differential static settlement could also be a concern unless ground improvement is utilized.

Due to concerns about total and differential seismically induced settlement, seismically induced bearing capacity loss, and differential static settlement, we recommend that the ground beneath the proposed structure be improved. We anticipate that the use of ground improvements to stiff or medium dense soils, which were encountered at a depth of 25 feet in soil boring B-1, can lower the estimated magnitude of differential settlement to levels of less than 0.5 inches over a horizontal distance of 40 feet and mitigate the risk of damage to the structure due to loss of bearing capacity support, thereby eliminating the need for other remedial measures, such as foundation ties, mat foundations, or deep foundations.

We recommend that the ground improvement generally consist of engineered aggregate piers installed with displacement methods, so that no casing will be required. In order to lower the risk of damage to the existing structure from vibrations during installation, we recommend that rigid inclusions be utilized for footings within 15 feet of the existing structure.

Where rigid inclusions are utilized as ground improvement, we recommend that a layer of crushed aggregate be placed over the rigid inclusions and below the foundation. This layer of compacted crushed aggregate is intended to help transfer loads from the footings to the rigid inclusions and to help avoid stress concentrations. The layer of crushed aggregate should be at least 2.5 feet thick below the foundation elements of the structure and should extend at least 1 foot horizontally beyond the edges of the foundation. The layer of crushed aggregate should be compacted to at least 95 percent of a maximum dry density determined by a Modified Proctor (ASTM D1557). The layer of crushed rock should consist of 1 ½”-0 crushed aggregate, or an alternative approved by GeoPacific prior to construction.

The Structural Foundations section of this report provides more information on the recommended ground improvements. A sketch illustrating the recommended mitigative measures is shown in Figure 7.

Provided that the engineered aggregate piers, rigid inclusions, and the crushed aggregate base rock are installed as recommended in this report, it is our opinion that the risks of damage to the proposed structure due to soil liquefaction and cyclic softening will be adequately mitigated. We anticipate that the use of ground improvements to an anticipated depth of 25 feet can lower the estimated magnitude of differential settlement to levels of less than 0.5 inches over a horizontal distance of 40 feet.

The recommended ground improvements will also adequately mitigate the risks of damage to the proposed structure due to the presence of soft soils in the static condition and providedowner an increased allowable bearing capacity, thereby eliminating the need for other remedial measures, such as foundation ties, mat foundations, or deep foundations.

For footings bearing on soil that has been improved by the methods described in this report, we anticipate that an allowable soil bearing pressure of up to 4,000 psf may be utilized. The actual soil bearing pressure should be provided by the design-build ground improvement contractor. For planning purposes, a coefficient of subgrade reaction of up to 250 kcf (145 pci) can be utilized for improved soil. Alternative subgrade modulus values may be provided by the design-build ground improvement contractor.

Further descriptions of engineered aggregate piers and rigid inclusion are located in the following sections, *Engineered Aggregate Piers* and *Rigid Inclusions*. For foundation components supported on native soil improved to an anticipated depth of 25 feet below existing grade, we anticipate that seismically induced differential settlement would be less than 0.5 inches measured a horizontal distance of 40 feet.

The recommended maximum allowable bearing pressures may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings. The maximum anticipated total and differential footing movements in the static condition (generally from soil expansion and/or settlement) are 1 inch and ½ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during filling of the water tank, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0.42 may be assumed along the interface between the base of the footing and subgrade soils. Passive earth pressure for buried portions of structures may be calculated using an equivalent fluid weight of 320 pounds per cubic foot (pcf), assuming footings are cast against dense, natural soils or engineered fill. The recommended coefficient of friction and passive earth pressure values do not include a safety factor. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. Loose, wet, or otherwise softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. During our site investigation, we encountered undocumented fill and/or buried topsoil to depth of up to 5.5 feet below the ground surface in some areas of the site. GeoPacific should observe foundation excavations to verify that an appropriate bearing stratum has been reached and that the actual exposed soils are suitable to support the planned foundation loads.

6.7.1 Engineered Aggregate Piers

As previously discussed, the proposed structure may be supported on shallow foundations bearing on native soil that has been improved by the installation of engineered aggregate piers or rigid inclusions. We anticipate that the use of ground improvements to an anticipated depth of 25 feet can lower the estimated magnitude of differential settlement to levels of less than 0.5 inches over a horizontal distance of 40 feet.

An engineered aggregate pier is a vertical column of compacted crushed aggregate. Engineered aggregate piers are formed when lifts of crushed aggregate are introduced into the native soil and compacted using high-energy compaction equipment. The compaction device may be a high-frequency vibratory probe or a vertical tamper. Due to the vibrations caused by the installation of engineered aggregate piers, we recommend that the are not utilized within 15 feet of the existing structure to lower the risk of damage. We recommend that rigid inclusions be utilized for footings in close proximity to the existing structure.

Engineered aggregate pier is a relatively generic term for multiple proprietary and non-propriety ground improvement technologies, such as rammed aggregate piers, vibro-replacement stone columns, vibrated pier, GeoPiers®, Impact Piers®, and others. Engineered aggregate piers are typically designed and installed by a design-build contractor. The design-build contractor would use the subsurface information provided in this geotechnical engineering report.

We strongly recommend that the engineered aggregate piers for this project be installed using displacement methods. Displacement methods involve installing the lifts of compacted rock by pushing a mandrel into the soil to the planned depth and inserting the rock through the mandrel. Displacement methods are advantageous for this site since they do not require drilling open holes, which could have significant caving issues due to shallow groundwater conditions. Displacement methods are also advantageous since they do not generate as much, if any, spoils.

6.7.2 Rigid Inclusions

We recommend that new footings within 15 feet of the existing structure be supported by shallow foundations bearing on a 2.5-foot thick pad of compacted crushed aggregate over rigid inclusions. The rigid inclusions may consist of vertical columns of concrete installed using a mandrel. Rigid inclusions are typically designed and installed by a design-build contractor. The design-build contractor would use the subsurface information provided in this geotechnical engineering report. We anticipate that the use of ground improvements to an anticipated depth of 25 feet can lower the estimated magnitude of differential settlement to levels of less than 0.5 inches over a horizontal distance of 40 feet.

6.8 Site Preparation Recommendations

The areas of proposed structures should be cleared of debris. If encountered, undocumented fill within influence zones of the proposed foundations or other settlement-sensitive improvements, should be completely removed and replaced with engineered fill. Undocumented fill was encountered to depths of up to 5 feet in some of our explorations.

As mentioned above, we encountered undocumented fill to a depth of up to 5 feet below the ground surface in some areas of the site. In hand auger boring HA-3, we encountered buried topsoil to a depth of approximately 5.5 feet. We understand that the properties in the northern portion of the proposed development area, in the vicinity of hand auger borings HA-6 through HA-8, were previously occupied by residential structures. We anticipate that some debris associated with these structures, such as concrete slabs, septic tanks, etc., may be encountered in this area.

Where encountered, undocumented fill material should be completely removed from the influence zone of the proposed structures. In areas where parking and drive areas are proposed and undocumented fill is present, it may be feasible to allow some of the undocumented fill soils to remain in place, pending evaluation by GeoPacific and provided that some vertical settlement and maintenance is acceptable to the owner. If existing soils are to remain in place in parking and drive areas, the upper portion of existing undocumented fill soils in parking and drive areas may need to be ripped and recompacted.

Exposed subgrade soils should be evaluated by the geotechnical engineer. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded scraper or dump truck and potholing with an excavator to evaluate the buried layers of undocumented fill. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe in addition to evaluation by potholing where applicable. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition, over-excavated and replaced with engineered fill (as described below) or stabilized with rock prior to placement of engineered fill. The depth of excavation, if required, should be evaluated by the geotechnical engineer at the time of construction.

Areas proposed for construction of driving areas may need to be reworked and recompacted using standard compaction equipment prior to placement of baserock. We recommend that engineered fill be compacted to project specifications for engineered fill, to at least 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent.

The final depth of soil removal should be determined by the geotechnical engineer or designated representative during site inspection while site preparation/excavation is being performed. Stripped topsoil, demolition debris, and moderately to highly organic fill should be removed from areas proposed for placement of engineered fill. Any remaining topsoil and organic debris should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

If encountered, any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill.

Site earthwork may be impacted by shallow perched groundwater and wet weather conditions. Stabilization of subgrade soils will require aeration and recompaction. If subgrade soils are found to be difficult to stabilize, over-excavation, placement of granular soils, or cement treatment of subgrade soils may be feasible options. GeoPacific should be onsite to observe preparation of subgrade soil conditions prior to placement of engineered fill.

6.9 Engineered Fill

All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Site grading should be conducted in accordance with the requirements outlined in the 2018 International Building Code (IBC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in the section titled *Site Preparation*. Surface soils should then be scarified and recompacted prior to placement of structural fill. Site preparation, soil stripping, and grading activities should be observed and documented by a geotechnical engineer or his representative. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill.

We anticipate that onsite native soils consisting primarily of silt and sand may be suitable for use as engineered fill. We anticipate that some of the existing undocumented fill soils may be suitable for use as engineered fill, if it is generally free of debris. Soils containing greater than 5 percent organic content should not be used as structural fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 12 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

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

6.10 Excavating Conditions and Utility Trench Backfill

We anticipate that onsite soils can generally be excavated using conventional heavy equipment. Bedrock was not encountered within our borings which extended to a depth of approximately 101.5 feet below the ground surface. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926) or be shored. The existing native silt soils to a classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. The existing native silty sand soils to a classify as Type C Soil and temporary excavation side slope inclinations as steep as 1.5H:1V may be assumed for planning purposes. These cut slope inclinations are applicable to excavations above the water table only. Groundwater was encountered within our explorations at dfepths as shallow as 6 feet below the ground surface.

Shallow, perched groundwater may be encountered at the site and should be anticipated in excavations and utility trenches. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

Underground utility pipes should be installed in accordance with the procedures specified in ASTM D2321. We recommend that structural trench and upper 5 feet of drywell backfill be compacted to at least 90 percent of the maximum dry density obtained by the Modified Proctor (ASTM D1557, AASHTO T-180) or equivalent. Initial backfill lift thicknesses for a ¾”-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 100-lineal-foot section of trench.

6.11 Erosion Control Considerations

During our field exploration program, we did not observe soil and topographic conditions which are considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles, fiber rolls, and silt fences. If used, these erosion control devices should remain in place throughout site preparation and construction.

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

6.12 Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and will be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will require expensive measures such as cement treatment or imported granular material to compact areas where fill may be proposed to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent passing the No. 200 sieve. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Geotextile silt fences, straw wattles, and fiber rolls should be strategically located to control erosion.

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

6.13 Concrete Slabs-on-Grade

This section of the report provides recommendations for concrete slabs-on-grade which are non-structural. If structural slabs, such as mat slabs, are proposed GeoPacific should be consulted to provide additional recommendations regarding bearing capacity, modulus of subgrade reaction, and seismic design criteria.

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in the *Site Preparation Recommendations* and *Spread Foundations* sections. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed, and the removal zone backfilled with additional crushed rock.

Exposed subgrade soils, including undocumented fills existing within non-structural slab areas, should be evaluated by the geotechnical engineer. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded dump truck and potholing with an excavator to evaluate the buried layers of undocumented fill. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 150 kcf (87 pci) should be assumed for competent, fine-grained soils anticipated to be present at foundation subgrade elevation following adequate site preparation as described above. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of 8 inches of 1½”-0 crushed aggregate beneath the slab. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 90 percent of its maximum dry density as determined by ASTM D1557 (Modified Proctor) or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside GeoPacific’s area of expertise.

6.14 Permanent Below-Grade Retaining Walls

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

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

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation, level backfill against the wall, and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 6.5H, where H is the total height of the wall.

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

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

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

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3 or 4-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drainpipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

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

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

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

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

6.15 Perimeter and Roof Drains

Construction should include typical measures for controlling subsurface water beneath the structure. Down spouts and roof drains should collect roof water in a system separate from the perimeter drains to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point and storm system well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

Perimeter footing drains should consist of 3 or 4-inch diameter, perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining drain rock. The drainpipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Figure 8 shows a typical detail for perimeter drains. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. In our opinion, footing drains may outlet at the curb, or to the storm drain system where sufficient fall is not available to allow drainage to meet the street. A typical perimeter footing drain detail is shown on Figure 8.

Perimeter drains prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Perimeter drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade.

6.16 Flexible Pavement Design: Private Parking and Drive Areas

We understand that plans for development include on-site parking areas and driving lanes. We anticipate that the driving lanes will be subjected to light traffic loading from daily traffic, trash trucks, delivery vehicles, and occasional emergency vehicles weighing up to 75,000 lbs with point loads up to 12,500 lbs. If the anticipated traffic will be different than assumed, GeoPacific should be consulted to provide updated recommendations. Table 7 presents our recommended minimum dry-weather pavement sections for new flexible pavement construction of new private pavement sections supporting 20 years of vehicle traffic. Pavement design calculations are attached to the appendix of this report.

Table 7: Recommended Minimum Dry-Weather Flexible Pavement Section: Private (20 Years)

Material Layer	Section Thickness (in.)	Compaction Standard
New Asphalt Concrete – Base Course	3.0	91%/ 92% of Max Density per AASHTO T-209
New ¾''-0 Crushed Aggregate Base	2.0	95% of Modified Proctor AASHTO T-180
New 1.5''-0 Crushed Aggregate Subbase	8.0	95% of Modified Proctor AASHTO T-180
Subgrade		95% of Standard Proctor AASHTO T-99 or Approved Equivalent

We reviewed the existing slope located between the parking lot and the tennis court. In this area, grades slope down to the east at an inclination of about 20 percent with a total vertical relief of about 8 feet. The existing slope appears to be sufficiently stable to support the proposed private parking and drive areas. GeoPacific should inspect the soils in the vicinity of the existing slope before or during construction to further evaluate the stability.

6.17 Rigid Pavement Design: Private Parking and Drive Areas

We understand that development at the site may include the construction of Portland Cement Concrete (PCC) drive areas. For the new rigid pavement section, we that the anticipated traffic which may include wheel loads of up to HS-20 loading (up to three axles, maximum 32,000 lbs per axle), and fire trucks weighing up to 75,000 lbs. We assume an anticipated maximum 18-kip ESAL count of approximately 50,000 over a 20-year period. If traffic loading is determined to exceed this estimate, then GeoPacific should be contacted to provide further recommendations.

concrete s

Under these assumptions, our recommended pavement design consists of a steel reinforced PCC slab with a thickness of 7 inches, and a 4,000 psi minimum compressive strength concrete, placed over 8 inches of 1.5-0 inch crushed aggregate compacted to a minimum of 95% relative to ASTM D1557. A single mat of No.4 reinforcing bars should be placed with a maximum spacing of 12-inches each way. The steel reinforcing should be placed to maintain at least 2 inches clearance from the top 3 inches clearance from the bottom, and 3 inches of clearance from the edges. Lap lengths should be a minimum of 40 bar diameters or 20 inches. A maximum joint spacing of 10 feet should be maintained for the PCC concrete. Dowels should be installed across contraction joints. Tolerances of spacing, ties, and clearances, should be constructed in accordance with ACI 318, and the requirements of Chapter 19 of the 2021 IBC. Table 8 presents the recommended minimum section for the proposed rigid pavement. In parking stalls or other lightly loaded areas, it may be feasible to omit reinforcing bars and/or dowels, at the owner's discretion.

Table 8 - Recommended Minimum Dry-Weather Rigid Pavement Section: Private (20 Years)

Material Layer	Section Thickness (in)	Standard
Portland Cement Concrete Pavement 4,000 psi (PCC)	7	Concrete should be sampled and tested per the requirements of ACI 318. 4,000 psi compressive strength at 28 days Air Content 5±1.5 percent. Maximum Slump 5 inches. Reinforcing Steel: Single Mat No. 4 Longitudinal Bars Spaced 12 inches apart each way, Minimum 2-inch clearance on top and 3-inch clearance on bottom and sides. Use epoxy coated, 1.0-inch diameter by 18-inch long smooth circular steel dowl bars at 12-inch spacing along all contraction joints. Bars should be coated with a bond breaker. Maximum transverse joint spacing = 10 feet
1½-0 inch Crushed Aggregate Base	8	95% of Modified Proctor AASHTO T-180
Subgrade	12	95% of Standard Proctor AASHTO T-99 or Approved Equivalent

6.18 Subgrade Preparation for Private Parking and Drive Areas

Subgrade soils should be inspected by GeoPacific prior to the placement of crushed aggregate base for pavement. Typically, a proof roll with a fully loaded water or haul truck is conducted by travelling slowly across the grade and observing the subgrade for rutting, deflection, or movement. Any pockets of organic debris or loose fill encountered during subgrade preparation should be removed and replaced with engineered fill (see Section 6.1, *Site Preparation Recommendations*). We observed undocumented fill and/or buried topsoil to depths of up to 5.5 feet below the ground surface in some areas of the site. It may be feasible to allow some of the undocumented fill soils to remain in place, pending evaluation by GeoPacific and provided that some vertical settlement and maintenance is acceptable to the owner.

We understand that the properties in the northern portion of the proposed development area, in the vicinity of hand auger borings HA-6 through HA-8, were previously occupied by residential structures. We anticipate that some debris associated with these structures, such as concrete slabs, septic tanks, etc., may be encountered in this area.

In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving.

If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project. General recommendations for wet weather pavement sections are provided below.

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

6.19 Wet Weather Construction Pavement Section

This section presents our recommendations for wet weather pavement sections and construction for new pavement sections at the project. These wet weather pavement section recommendations are intended for use in situations where it is not feasible to compact the subgrade soils to project requirements, due to wet subgrade soil conditions, and/or construction during wet weather. Based on our site review, we recommend a wet weather section with a minimum subgrade deepening of 6 to 12 inches to accommodate a working subbase of additional 1½"-0 crushed rock. Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils prior to placement of base rock.

With implementation of the above recommendations, it is our opinion that the resulting pavement section will provide equivalent or greater structural strength than the dry weather pavement section currently planned. However, it should be noted that construction in wet weather is difficult, and the performance of pavement subgrades depend on a number of factors including the weather conditions, the contractor's methods, and the amount of traffic the road is subjected to. There is a potential that soft spots may develop even with implementation of the wet weather provisions recommended in this letter. If soft spots in the subgrade are identified during roadway excavation, or develop prior to paving, the soft spots should be over-excavated and backfilled with additional crushed rock.

During subgrade excavation, care should be taken to avoid disturbing the subgrade soils. Removals should be performed using an excavator with a smooth-bladed bucket. Truck traffic should be limited until an adequate working surface has been established. We suggest that the crushed rock be spread using bulldozer equipment rather than dump trucks, to reduce the amount of traffic and potential disturbance of subgrade soils. Care should be taken to avoid over-compaction of the base course materials, which could create pumping, unstable subgrade soil conditions. Heavy and/or vibratory compaction efforts should be applied with caution. Following placement and compaction of the crushed rock to project specifications (95 percent of Modified Proctor), a finish proof-roll should be performed before paving.

The above recommendations are subject to field verification. GeoPacific should be on-site during construction to verify subgrade strength and to take density tests on the engineered fill, base rock and asphaltic or concrete pavement materials.

6.20 Stormwater Management

We understand that it is desired to include subsurface disposal of stormwater into plans for project development. Based on the results of our infiltration testing, the native Sandy SILT (ML soils exhibit an infiltration rate of 0.25 inches. We observed groundwater at depths of approximately 6 to 7 feet below the ground surface in some of our explorations. Our explorations were conducted in March, which is when groundwater levels are near seasonal highs. According to local well logs, groundwater has been recorded at depths ranging from 10 to 20 feet bgs in the vicinity of the subject site. A separation distance of at least 5 feet is recommended between infiltration systems and the water table. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors.

Stormwater management systems should be constructed as specified by the designer and/or in accordance with the applicable stormwater design codes. Stormwater exceeding storage capacities will need to be directed to a suitable surface discharge location, away from structures. Stormwater management systems may need to include overflow outlets, surface water control measures and/or be connected to the street storm drain system, if available. Evaluating environmental implications of stormwater disposal at this site are beyond the scope of this study.

7.0 UNCERTAINTIES AND LIMITATIONS

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

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

This report should not be relied upon by third parties unless a reliance letter has been issued by GeoPacific specifically to that third party, otherwise the third party should rely upon their own due diligence and geotechnical studies only.

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

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER 92385PE OREGON SEPTEMBER 14, 2021 THOMAS JOHN TORKELSON

Thomas J Torkelson 2025.03.21 16:19:52 -07'00'

RENEWS: June 30, 2026

REGISTERED PROFESSIONAL ENGINEER 105192PE OREGON JULY 09, 2024 ALEXANDRIA B. CAMPBELL

EXPIRES: 12/31/2025

Thomas J. Torkelson, G.E., P.E.
Associate Geotechnical Engineer

Alexandria B. Campbell, P.E.
Staff Engineer

REFERENCES

ASCE Hazard Tool, https://ascehazardtool.org/

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

Beeson, M.H., 1991, Geologic Map of the Portland Quadrangle, Oregon and Clark County: Oregon Department of Geology and Mineral Industries.

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

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

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

Malhotra, P.K., 2022, Site-Specific Ground Motions for Seismic Design of Buildings and Other Structures, American Society of Civil Engineers (ASCE), ISBN 978-0-7844-1596-2, p. 35-87

Oregon Department of Geology and Mineral Industries, Statewide Geohazards Viewer, www.oregongeology.org/hazvu.

O' Connor, J.E., 2001, Origin extent and thickness of Quaternary geologic units in the Willamette Valley, Oregon: United States Geological Survey, Professional Paper 1620, scale 1:250,000.

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

United States Geological Survey, USGS Earthquake Hazards Program Website (earthquake.usgs.gov).

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

Wells, R.E., Haugerud, R.A., Niem, A.R., Niem, W.A., Ma, Lina, Evarts, R.C., O'Connor, J.E., Madin, I.P., Sherrod, D.R., Beeson, M.H., Tolan, T.L., Wheeler, K.L., Hanson, W.B., and Sawlan, M.G., 2020, Geologic map of the greater Portland metropolitan area and surrounding region, Oregon and Washington, U.S. Geological Survey, Scientific Investigations Map SIM-3443.

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

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

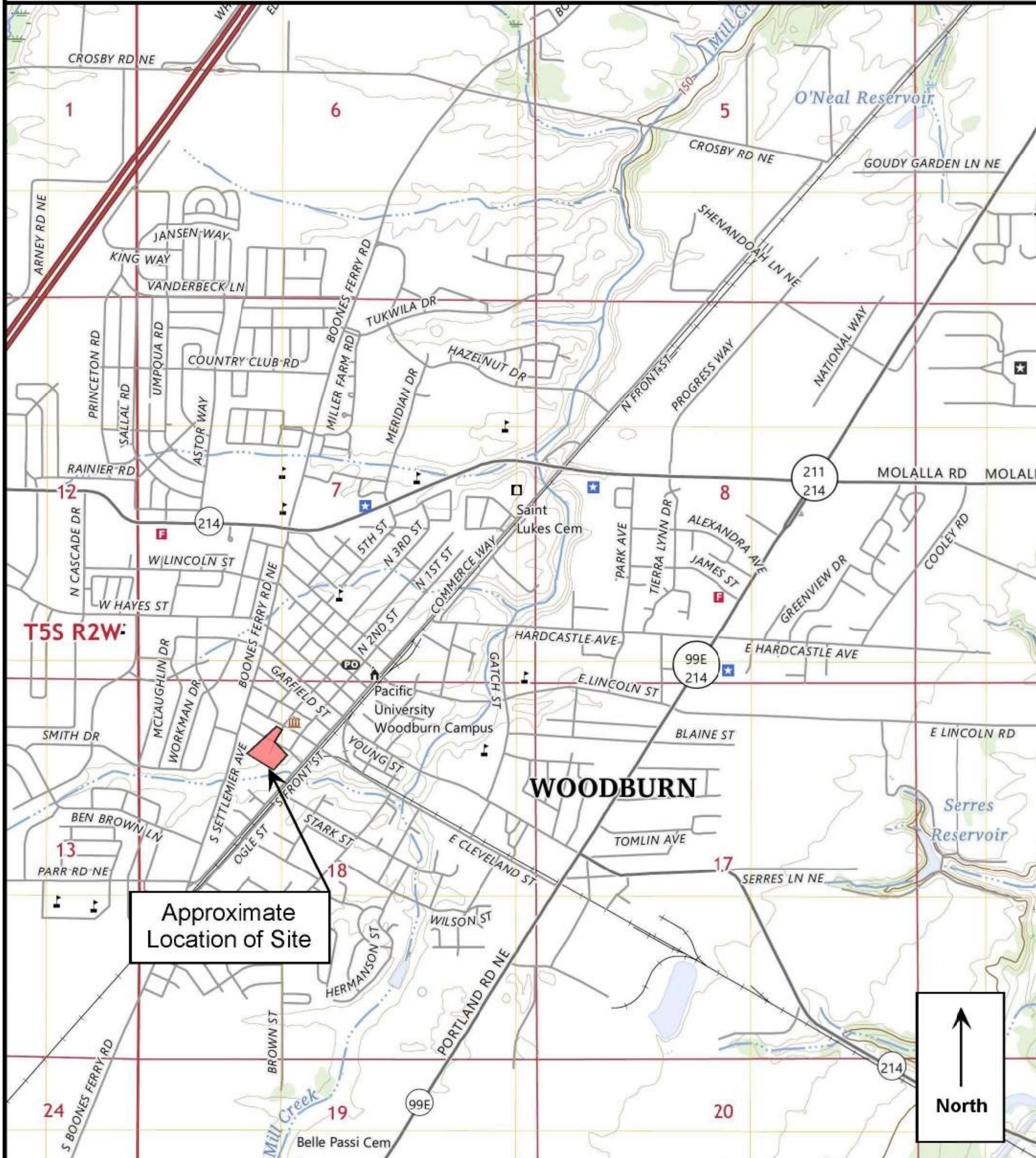
Yeats, R.S., Graven, E.P., Werner, K.S., Goldfinger, C., and Popowski, T., 1996, Tectonics of the Willamette Valley, Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, P. 183-222, 5 plates, scale 1:100,000.

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

FIGURES

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

SITE VICINITY MAP



Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Woodburn, Oregon Quadrangle, 2023

Date: 03/04/25

Drawn by: ABC

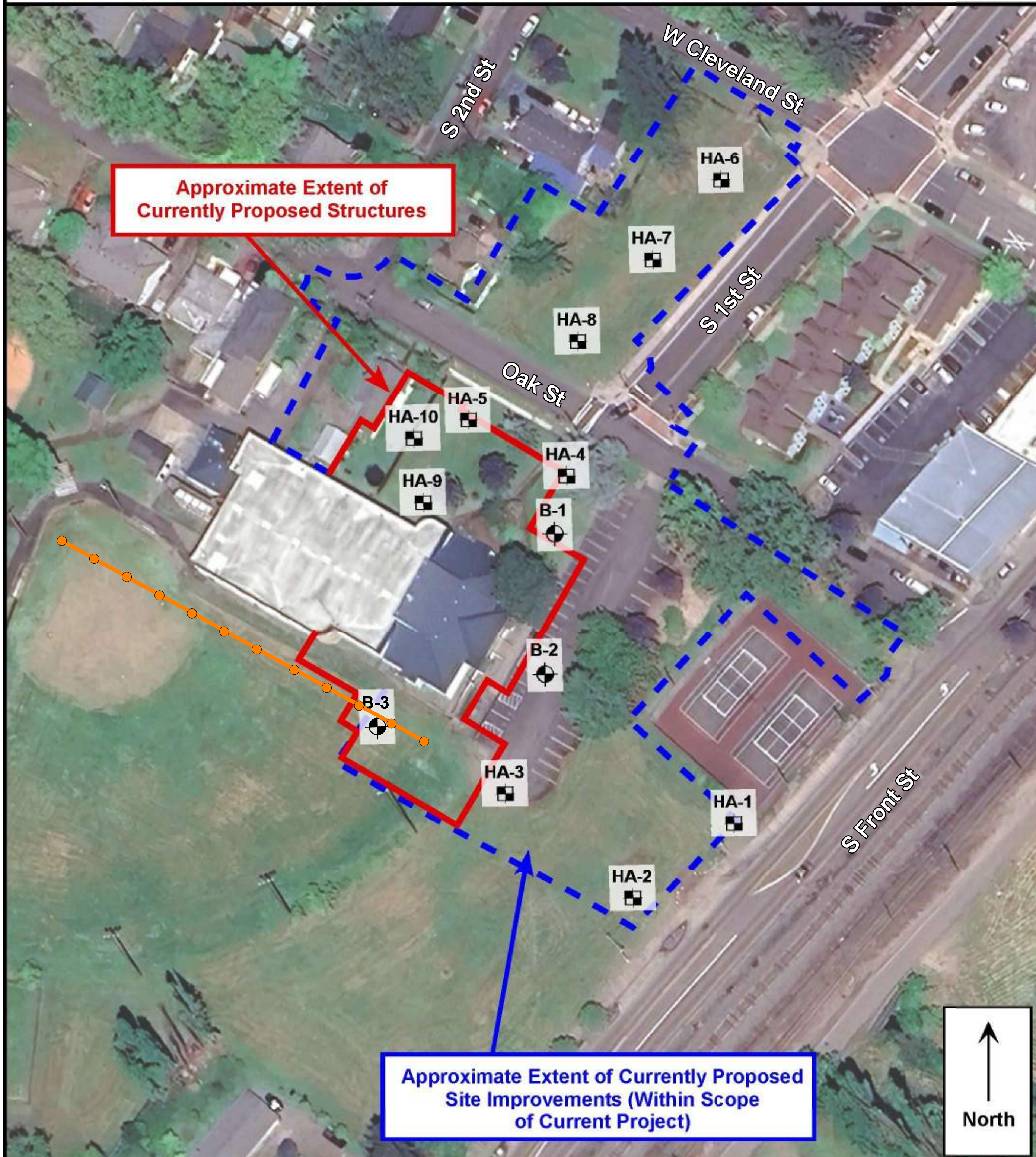
0 2,000

Approximate Scale: 1" = 2,000'

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	FIGURE 1

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

SITE AERIAL PHOTO AND EXPLORATION LOCATIONS



Legend
- Soil Boring Designation and Approximate Location
- Hand Auger Designation and Approximate Location
- ReMi Array Approximate Location

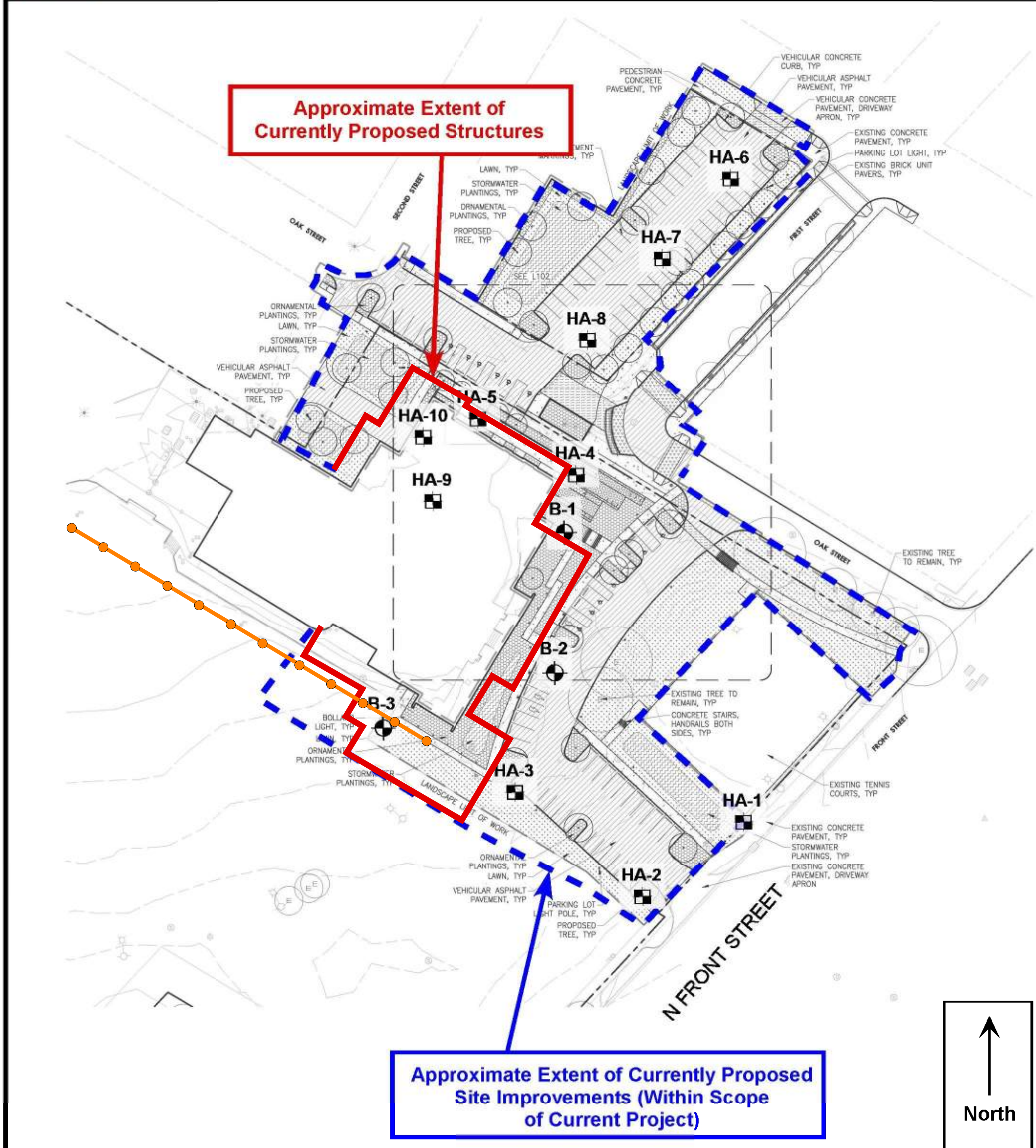
0 100'
APPROXIMATE SCALE 1"=100'

Date: 03/18/25
Drawn by: ABC

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	FIGURE 2

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

SITE PLAN AND EXPLORATION LOCATIONS



Legend
- Soil Boring Designation and Approximate Location
- Hand Auger Designation and Approximate Location
- ReMi Array Approximate Location

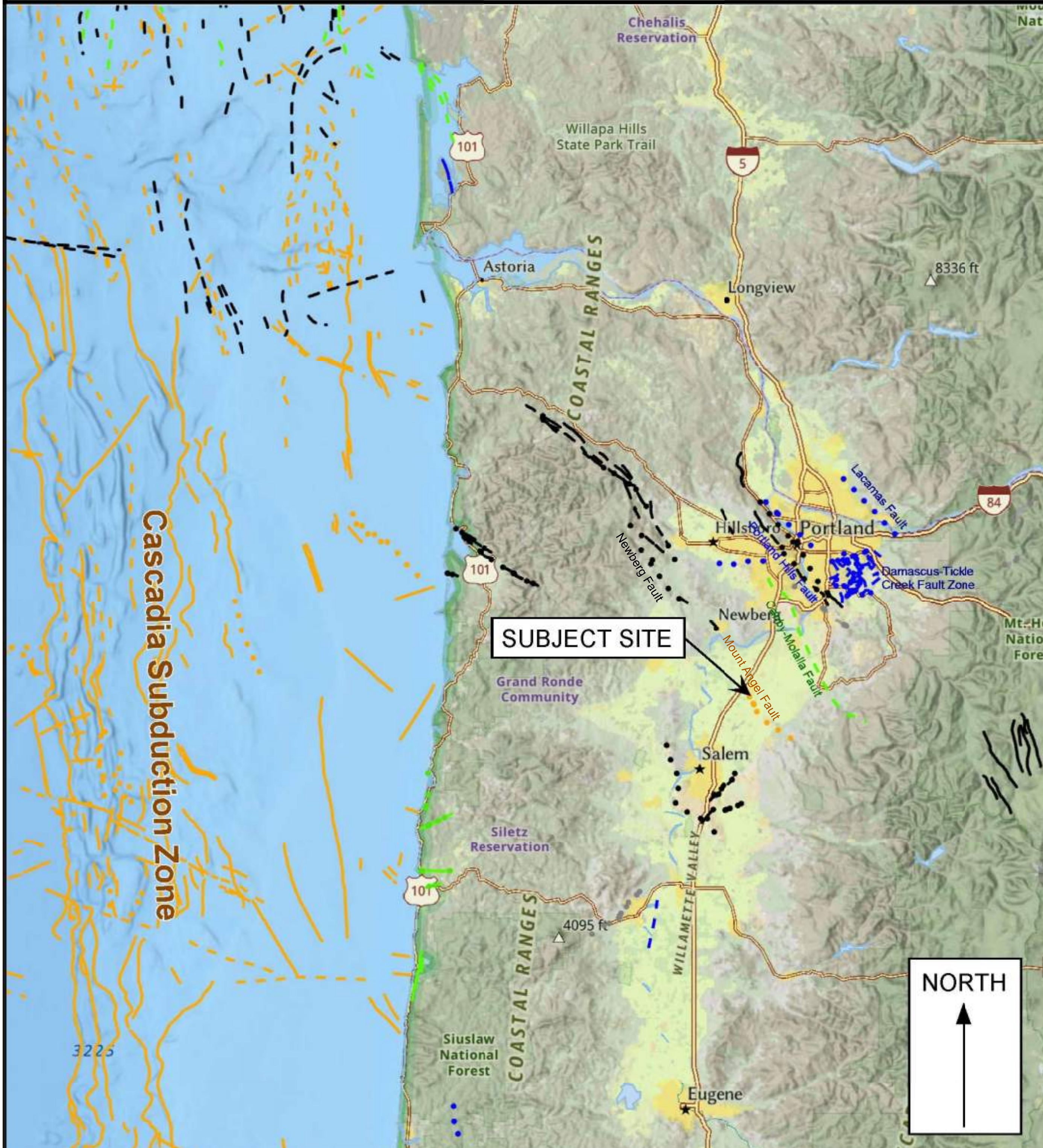
0 100'
APPROXIMATE SCALE 1"=100'

Date: 03/18/25
Drawn by: ABC

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	FIGURE 3

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

QUATERNARY FAULT MAP



Legend

Approximate Scale 1 in = 135,000 feet

Date: 3/14/2025
Drawn by: TJT

Base map: National Geographic, ESRI, Garmin, 2024.

Project: Lucky Lane Reservoir Estacada, Oregon	Project No. 24-6558	FIGURE 4

GEOPACIFIC

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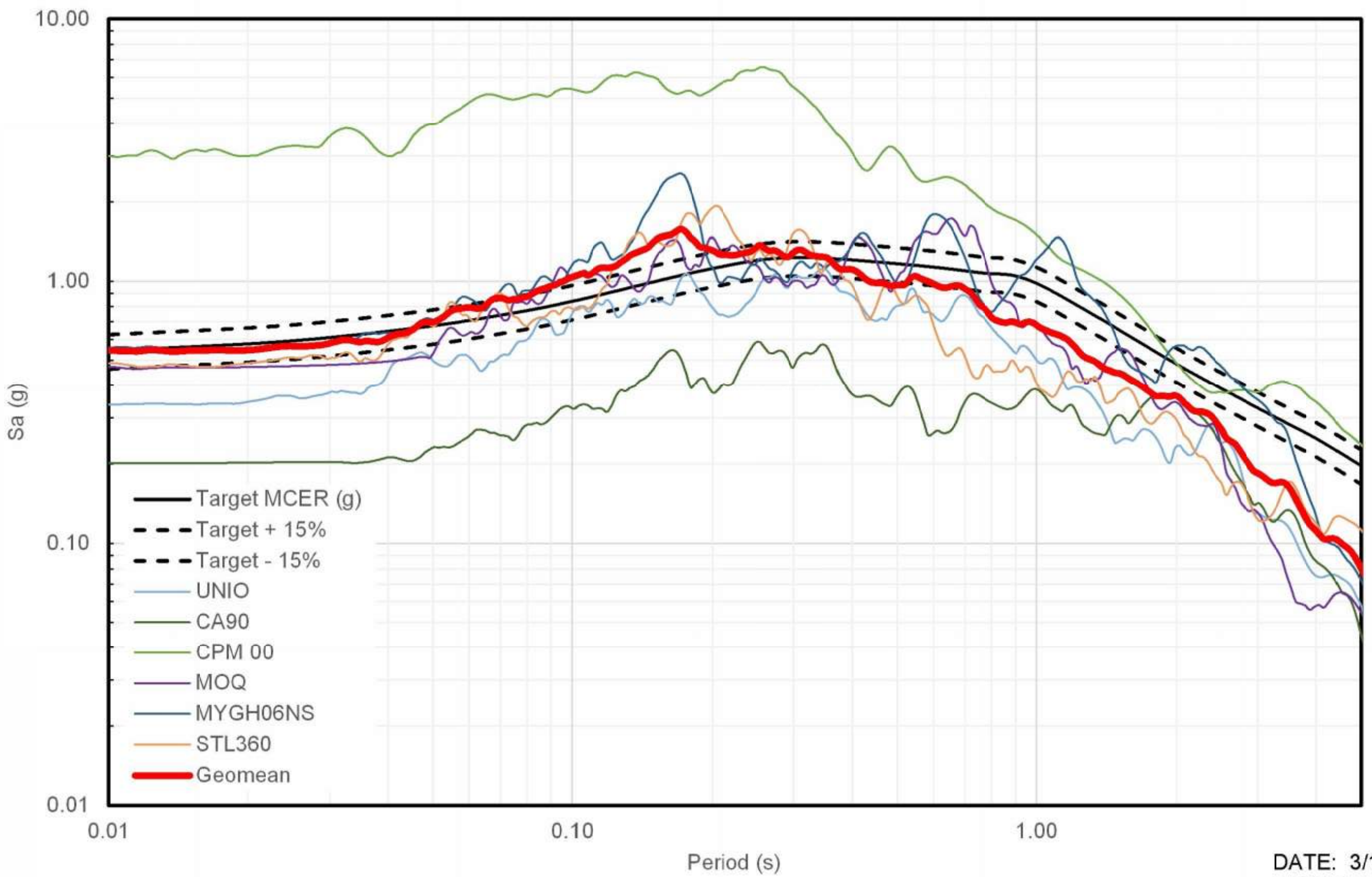
Portland, Oregon 97224
Tel: (503) 598-8445

SPECTRAL MATCHING FOR SITE RESPONSE



DATE: 3/14/25
DRAWN BY: TJT

PROJECT: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	**FIGURE 5**

GEOPACIFIC
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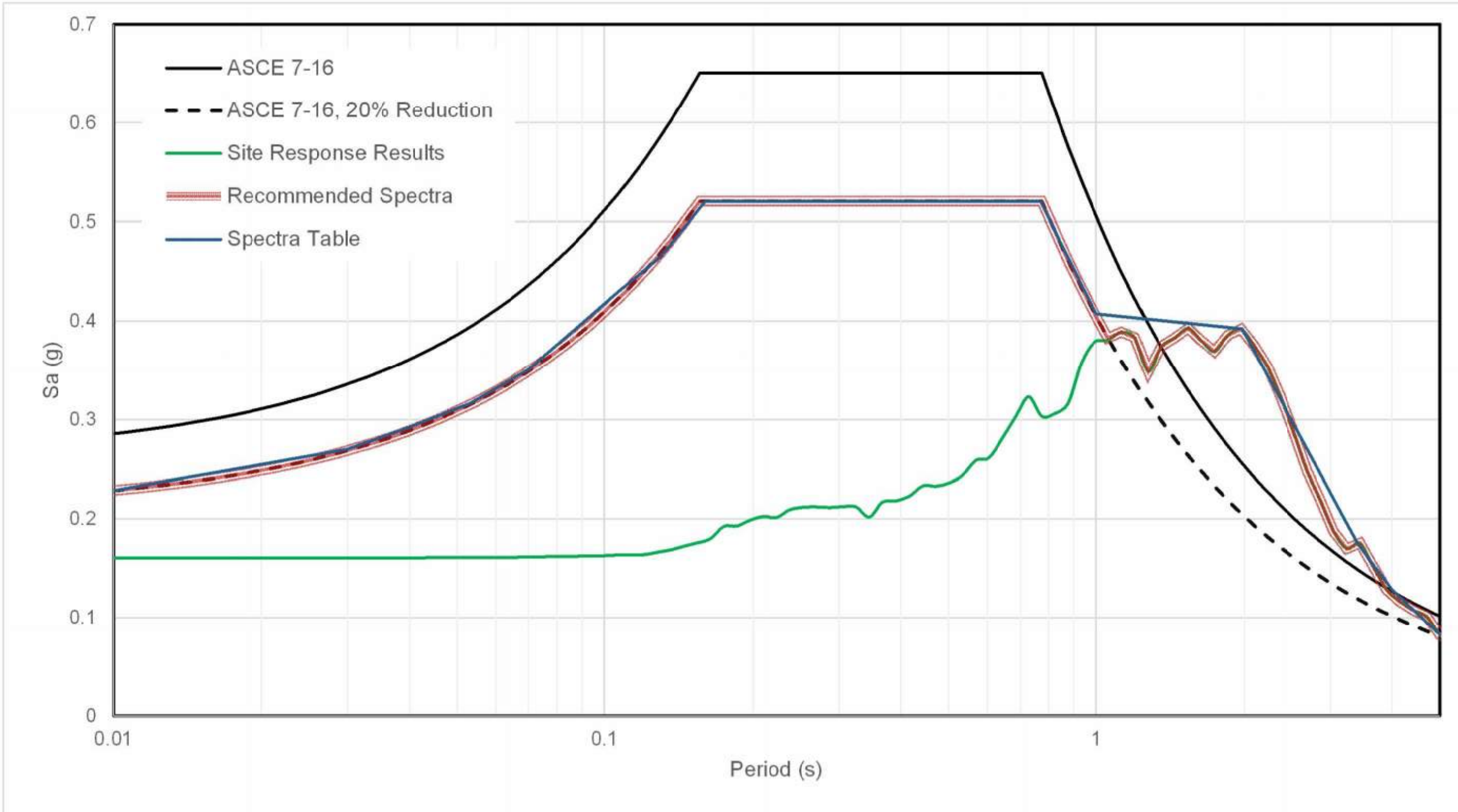
Portland, Oregon 97224
Tel: (503) 598-8445

DESIGN SPECTRUM FROM SITE RESPONSE



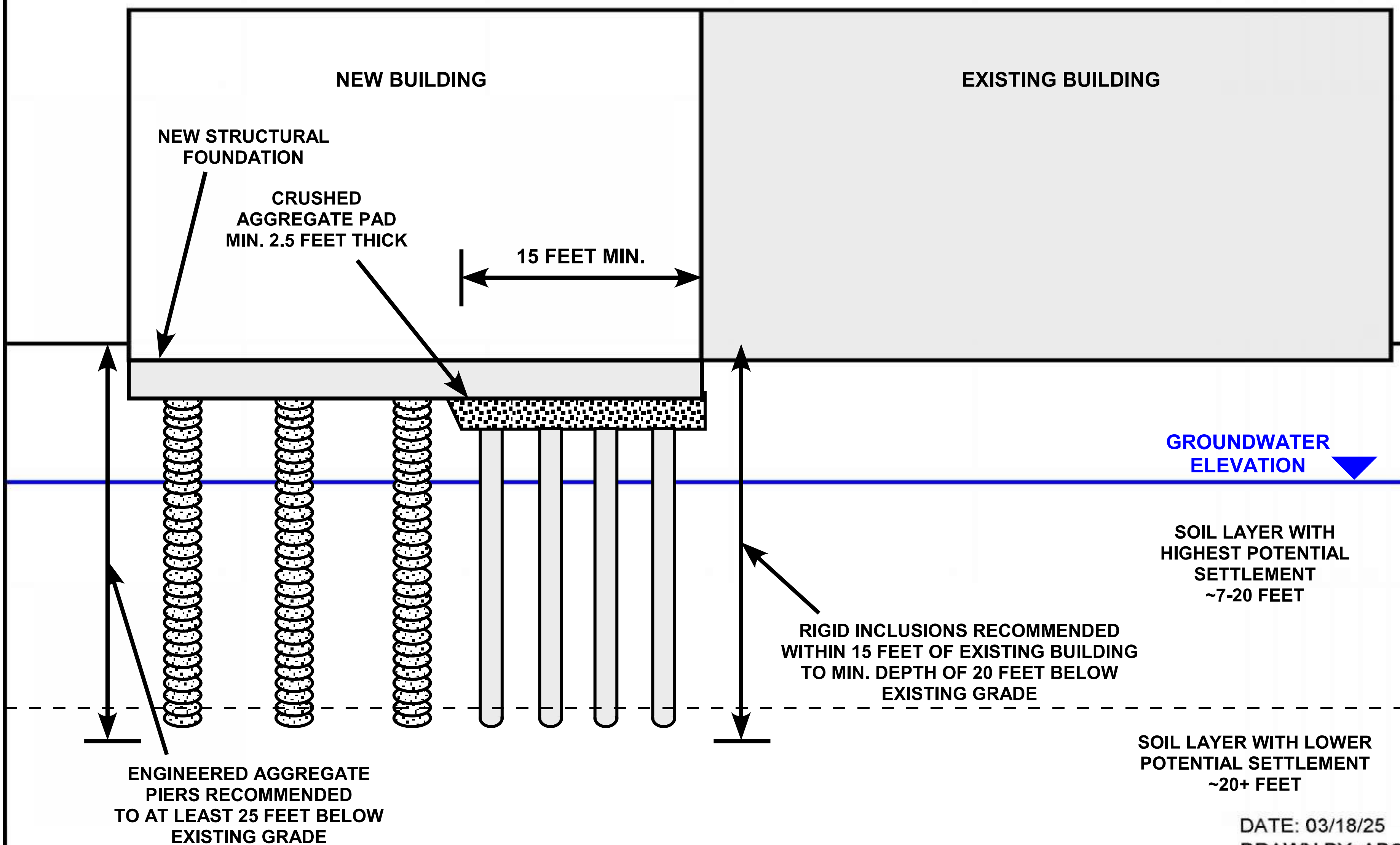
Note: Where MCE_R spectrum is required, it shall be determined by multiplying the design response spectrum by 1.5. ASCE 7-16 section 11.4.7.

DATE: 3/14/25
DRAWN BY: TJT

PROJECT: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	**FIGURE 6**

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

CROSS SECTION SKETCH OF RECOMMENDED MITIGATIVE MEASURES

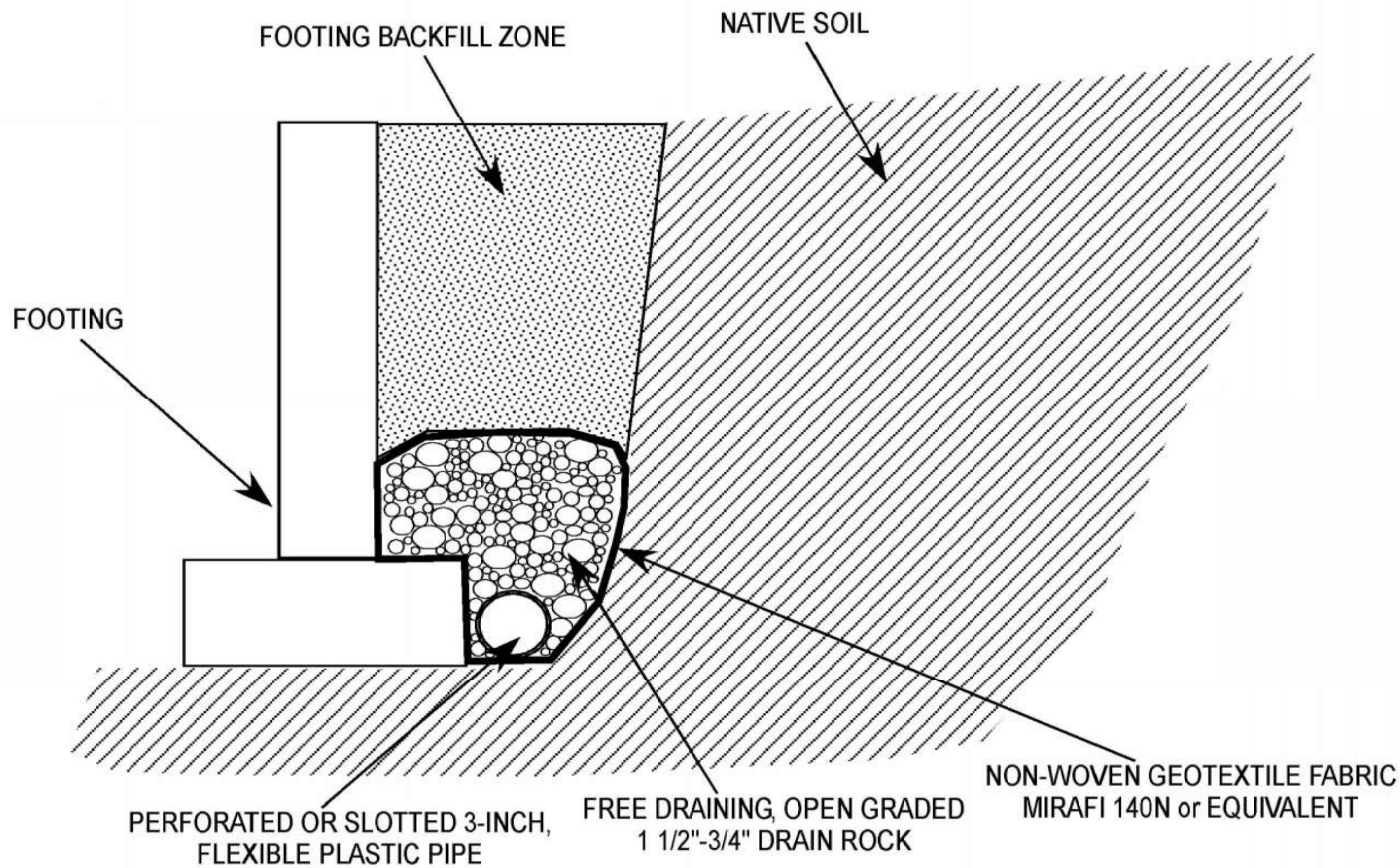


DATE: 03/18/25
DRAWN BY: ABC

PROJECT: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	FIGURE 7

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

TYPICAL PERIMETER FOOTING DRAIN DETAIL



Notes:
1) Drain rock should contain no more than 5 percent fines passing the U.S. No. 200 Sieve.
2) Trench bottom and drain pipe should be sloped to drain to approved discharge location.

Date: 03/06/25
Drawn by: ABC

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6577	FIGURE 8

EXPLORATION LOGS

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

BORING LOG

Project: Woodburn Community Center, Woodburn, Oregon	Project No. 25-6755	Boring Number B-1

Depth (ft)	Sample Type	Blow Counts	N-Value	Moisture Content (%)	Water Bearing Zone	Material Description
	Split-Spoon	10-9-5	14			12-inches-thick SILT with Gravel (ML), dark brown, angular gravel, with some fine roots, stiff, moist (Undocumented Fill)
	Split-Spoon	2-4-3	7	29.8		Sandy SILT (ML), brown, low plasticity, medium stiff, moist (Catastrophic Flood Deposit)
5	Split-Spoon	3-4-4	8	35.5		[Liquid Limit=40; Plasticity Index=11; Percent Passing #200 Sieve=65.7]
	Split-Spoon	2-3-3	6	30.7		Grades to wet
10	Split-Spoon	2-2-3	5	40.7		
15	Split-Spoon	1-2-2	4	43.4		Grades to soft
20	Split-Spoon	2-3-4	7	38.3		Grades to medium stiff [Liquid Limit=23; Plasticity Index=NP; Percent Passing #200 Sieve=69.0]
25	Split-Spoon	6-9-10	19			Silty SAND (SM), brown, low plasticity, with interbedded layer of silt, medium dense, wet (Catastrophic Flood Deposits)
30	Split-Spoon	5-10-14	24	35.1		
35	Split-Spoon	5-6-11	17			Grades to dark blue-gray
40	Split-Spoon	2-6-10	16	36.2		Grades to with more silt and low to moderate plasticity
50	Split-Spoon	3-7-12	19			

Boring log continues on next page

LEGEND

100 to 1,000 g Bag Sample | Split-Spoon | Shelby Tube Sample | Static Water Table at Drilling | Seepage | Water Bearing Zone

Date Drilled: 02.28.2025
Logged By: JN
Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

BORING LOG

Project: Woodburn Community Center
Woodburn, Oregon

Project No. 25-6755

Boring Number **B-1**

Boring log continued from previous page

Depth (ft)	Sample Type	Blow Counts	N-Value	Moisture Content (%)	Water Bearing Zone	Material Description
60	Split-Spoon	4-7-8	15			Silty SAND (SM), dark blue-gray, low to moderate plasticity, medium dense, wet (Catastrophic Flood Deposits)
70	Split-Spoon	15-16-20	36			Poorly Graded SAND (SP), gray and black, dense, wet (Catastrophic Flood Deposit)
80	Split-Spoon	7-19-12	31			
90	Split-Spoon	26-39-30	69			Grades to very dense
100	Split-Spoon	42-28-33	61			

Boring terminated at 101.5 feet

Groundwater level could not be determined due to mud-rotary drilling method

LEGEND

Bag Sample (100 to 1,000 g) | Split-Spoon | Shelby Tube Sample | Static Water Table at Drilling | Seepage | Water Bearing Zone

Date Drilled: 02.28.2025
Logged By: JN
Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

BORING LOG

Project: Woodburn Community Center
Woodburn, Oregon

Project No. 25-6755

Boring Number **B-2**

Depth (ft)	Sample Type	Blow Counts	N-Value	Moisture Content (%)	Water Bearing Zone	Material Description
1						
	Split-Spoon	2-3-2	5			Sandy SILT (ML), blue, low plasticity, medium stiff, moist (Catastrophic Flood Deposit)
5	Split-Spoon	0-1-2	3			Grades to soft and very moist
	Split-Spoon	0-0-1	1		Static Water Table at Drilling; Water Bearing Zone	Grades to very soft and wet
10	Split-Spoon	1-2-3	5		Water Bearing Zone	Grades to medium stiff and with less sand
15	Split-Spoon	1-1-2	3		Water Bearing Zone	Grades to gray-brown, soft, and sandy
						Boring terminated at 16.5 feet
						Static groundwater level at 7 feet
						Boring was left open for 1 hour to observe groundwater conditions
20						

LEGEND

Bag Sample (100 to 1,000 g) · Split-Spoon · Shelby Tube Sample · Static Water Table at Drilling · Seepage · Water Bearing Zone

Date Drilled: 02.28.2025
Logged By: JN
Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

BORING LOG

Project: Woodburn Community Center, Woodburn, Oregon	Project No. 25-6755	Boring Number B-3

Depth (ft)	Sample Type	Blow Counts	N-Value	Moisture Content (%)	Water Bearing Zone	Material Description
1						
	Split-Spoon	1-2-2	4			Sandy SILT (ML), brown, soft, moist (Catastrophic Flood Deposits)
5	Split-Spoon	2-2-2	4			Grades to dark brown and wet
	Split-Spoon	3-2-3	5			Grades to medium stiff
10	Split-Spoon	1-2-1	3			Grades to soft and interbedded with about 4-inch layer of silt with less sand
	Split-Spoon	2-3-2	5			Grades to medium stiff
15	Split-Spoon	4-4-6	10			Grades to stiff
						Boring terminated at 16.5 feet
						Groundwater level could not be determined due to mud-rotary drilling method
20						

LEGEND

Bag Sample (100 to 1,000 g) | Split-Spoon | Shelby Tube Sample | Static Water Table at Drilling | Seepage | Water Bearing Zone

Date Drilled: 02.28.2025
Logged By: JN
Surface Elevation:

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.5						Organic SILT (OL-ML), brown, soft, trace roots, moist, 6-inches (Topsoil Horizon)
0.5–2		100 to 1,000 g				SILT with Gravel (ML), brown, 3/4" sub-angular to angular gravel, stiff, moist to very moist (Undocumented Fill)
2–5		100 to 1,000 g				Sandy SILT (ML), light brown, stiff, slightly micaceous, some orange-gray mottling, moist (Catastrophic Flood Deposits)

Hand Auger Terminated at 5 Feet

Note: No Groundwater Seepage Encountered

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025

Logged By: AJH

Surface Elevation:

GEOPACIFIC 14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-2**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	**Material Description**
0–0.5						Organic SILT (OL-ML), brown, soft, trace roots, moist, 6-inches (Topsoil Horizon)
0.5–3						SILT with Gravel (ML), brown, 3/4" sub-angular to angular gravel, stiff, moist (Undocumented Fill) Grades to with 1.5-inch gravel
3						Practical Refusal on Dense GRAVEL at 3 Feet Note: No Groundwater Seepage Encountered

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025
Logged By: AJH
Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center
Woodburn, Oregon

Project No. 25-6755

Boring No. **HA-3**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1						Poorly Graded Gravel (GP), sparse grass, 3/4" to 1.5" crushed aggregate (Undocumented Fill)
1–5						SILT with Gravel (ML), brown, 3/4" sub-angular to angular gravel, stiff, moist (Undocumented Fill)
4						Grades to with less gravel, trace asphaltic aggregate encountered
5						Organic SILT (OL-ML), dark brown, soft, woody debris, 4" thick (Buried Topsoil Horizon)
6						Sandy SILT (ML), gray, soft, trace roots, very moist, slightly odorous (Catastrophic Flood Deposits)

Hand Auger Terminated at 6.25 Feet

Note: Groundwater Seepage Encountered at 6 Feet

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025

Logged By: AJH

Surface Elevation:

GEOPACIFIC 14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center, Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-4**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.4						Organic SILT (OL-ML), brown, soft, trace roots, moist (Topsoil Horizon)
0.4–5						Sandy SILT (ML), light brown, slightly micaceous, stiff, moist (Catastrophic Flood Deposit)
2.4						0.5-inch tree root encountered
4		Bag Sample (100 to 1,000 g)				Grades to with trace of fine sand
4.5						Infiltration test conducted at approximately 5 feet bgs. Infiltration rate observed as 0.25 inches per hour.
5						Hand Auger Terminated at 5 Feet
						Note: No Groundwater Seepage Encountered

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025

Logged By: AJH

Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center, Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-5**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.4						Organic SILT (OL-ML), brown, soft, trace roots, moist (Topsoil Horizon)
0.4–1.2						Sandy SILT (ML), brown, trace roots, medium stiff, moist (Catastrophic Flood Deposits)
1.5		Bag Sample (100 to 1,000 g)				
2.3						Grades to light brown with some orange-gray mottling, with no roots, stiff, and slightly micaceous
4		Bag Sample (100 to 1,000 g)				
6					Water Bearing Zone (6–10 ft)	Grades to medium stiff and wet
7						Grades to stiff
10						Hand Auger Terminated at 10 Feet

Note: Groundwater Seepage Encountered at 7 Feet

Hand Auger Left Open for 1 Hour to Observe Groundwater Conditions

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

Dates Excavated: 02/28/2025, 03/03/2025

Logged By: AJH

Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center, Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-6**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1		100 to 1,000 g				SILT with Gravel (ML), brown, stiff, moist, sub-angular to angular gravel, trace roots, thin grass mat (Undocumented Fill)
1–1.75						GRAVEL with Sand (GP), brown-gray, dense, sub-angular to angular gravel, concrete debris (Undocumented Fill)
2						Practical Refusal on Dense GRAVEL at 1.75 Feet
						Note: No Groundwater Seepage Encountered
3						
4						
5						
6						
7						
8						
9						
10						

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025
Logged By: AJH
Surface Elevation:

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-7**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0						Organic SILT (OL-ML), brown, soft, trace roots, moist (Topsoil Horizon)
1						Sandy SILT (ML), brown with trace orange staining, trace roots, soft to medium stiff, moist (Catastrophic Flood Deposits)
2						Grades to light brown with some orange-gray mottling, stiff, slightly micaceous
3		100 to 1,000 g				Grades to slight moisture increase
4						
5						
6						Hand Auger Terminated at 5.5 Feet Note: No Groundwater Seepage Encountered
7						
8						
9						
10						

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025
Logged By: AJH
Surface Elevation:

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center
Woodburn, Oregon

Project No. 25-6755

Boring No. **HA-8**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.3						Organic SILT (OL-ML), brown, soft, trace roots, moist (Topsoil Horizon)
0.3–3		100 to 1,000 g (at ~2 ft)				Sandy SILT (ML), brown with trace orange staining, trace roots to 1', soft to medium stiff, moist (Catastrophic Flood Deposit)
3–6						Grades to light brown with some orange-gray mottling, slightly micaceous, and stiff

Hand Auger Terminated at 6 Feet

Note: No Groundwater Seepage Encountered

LEGEND

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

Dates Excavated: 02/28/2025
03/03/2025

Logged By: AJH

Surface Elevation:

GEOPACIFIC

14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

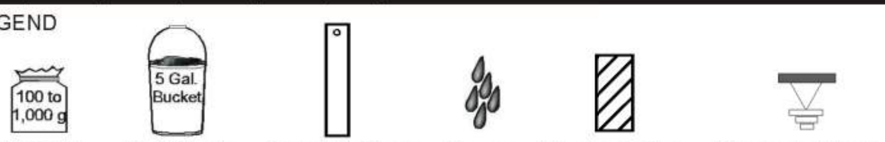
HAND AUGER LOG

Project: Woodburn Community Center Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-9**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.3						Organic SILT (OL-ML), brown, soft, trace roots, moist (Topsoil Horizon)
0.3–5						Sandy SILT (ML), brown with trace orange staining, trace roots to 1', soft to medium stiff, moist (Catastrophic Flood Deposits)
3						Grades to light brown with some orange-gray mottling, slightly micaceous, and stiff
5						Hand Auger Terminated at 5 Feet Note: No Groundwater Seepage Encountered

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

LEGEND



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

Dates Excavated: 02/28/2025
03/03/2025

Logged By: AJH

Surface Elevation:

GEOPACIFIC
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445

HAND AUGER LOG

Project: Woodburn Community Center, Woodburn, Oregon	Project No. 25-6755	Boring No. **HA-10**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0						Organic SILT (OL-ML), brown, soft, trace roots, moist (Topsoil Horizon)
1						Sandy SILT (ML), brown with trace black staining, soft to medium stiff, moist (Catastrophic Flood Deposits)
2						Grades to light brown with some orange-gray mottling, slightly micaceous, and stiff
3		100 to 1,000 g				
4						
5						
6						Grades to medium stiff, increase moisture, decrease in mottling
6.5						Grades to stiff
7						
8						
9						
10						Hand Auger Terminated at 10 Feet
						Note: No Groundwater Seepage Encountered

LEGEND

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

Dates Excavated: 02/28/2025, 03/03/2025
Logged By: AJH
Surface Elevation:

LABORATORY TESTING RESULTS

Project Name:	Woodburn Community Center	Project #:	25-6755
Date Sampled:	2.28.2025	Date Tested:	3.5.2025
Sampled By:	TJT	Tested By:	MTB

Moisture Content

Sample ID:	S25-022
Exploration & Depth:	**B-1 2.5ft**
Tare #:	53
Tare (g):	155.8
Tare + Wet (g):	330.2
Tare + Dry (g):	290.2
Moisture (%):	29.8

Moisture Content

Sample ID:	S25-023
Exploration & Depth:	**B-1 5ft**
Tare #:	F
Tare (g):	548.0
Tare + Wet (g):	935.4
Tare + Dry (g):	833.8
Moisture (%):	35.5

Moisture Content

Sample ID:	S25-024
Exploration & Depth:	**B-1 7.5ft**
Tare #:	50
Tare (g):	156.9
Tare + Wet (g):	395.8
Tare + Dry (g):	339.7
Moisture (%):	30.7

Moisture Content

Sample ID:	S25-025
Exploration & Depth:	**B-1 10ft**
Tare #:	52
Tare (g):	158.3
Tare + Wet (g):	401.2
Tare + Dry (g):	330.9
Moisture (%):	40.7

Moisture Content

Sample ID:	S25-026
Exploration & Depth:	**B-1 15ft**
Tare #:	57
Tare (g):	157.9
Tare + Wet (g):	477.4
Tare + Dry (g):	380.7
Moisture (%):	43.4

Moisture Content

Sample ID:	S25-027
Exploration & Depth:	**B-1 20ft**
Tare #:	E
Tare (g):	546.7
Tare + Wet (g):	781.7
Tare + Dry (g):	716.6
Moisture (%):	38.3

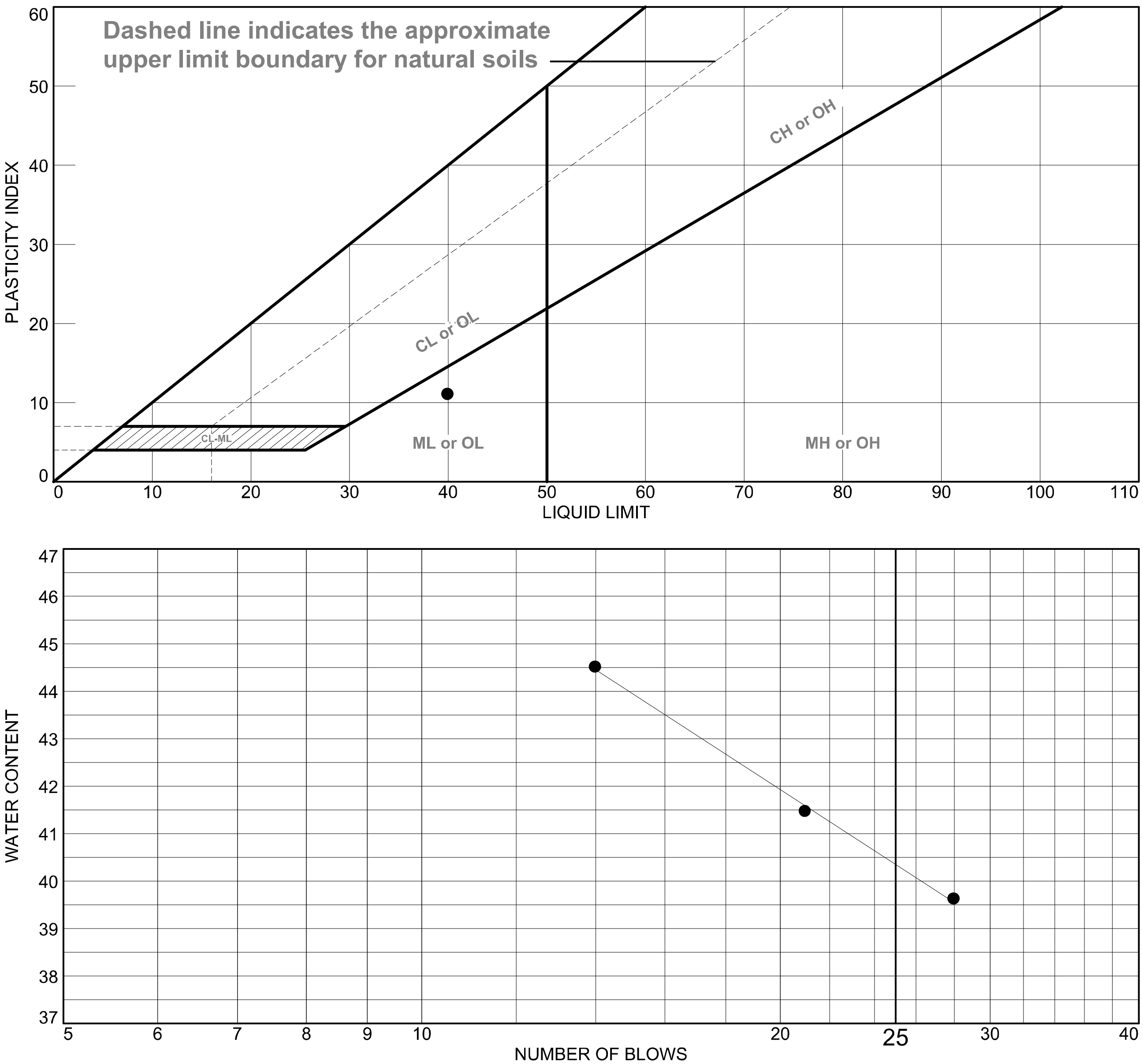
Moisture Content

Sample ID:	S25-028
Exploration & Depth:	**B-1 30ft**
Tare #:	51
Tare (g):	157.5
Tare + Wet (g):	465.3
Tare + Dry (g):	385.3
Moisture (%):	35.1

Moisture Content

Sample ID:	S25-029
Exploration & Depth:	**B-1 40ft**
Tare #:	59
Tare (g):	157.7
Tare + Wet (g):	458.6
Tare + Dry (g):	378.6
Moisture (%):	36.2

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy SILT	40	29	11	100.0	65.7	ML

Project No. 25-6755 **Client:** Public Works Engineering Department

Project: Woodburn Community Center

Location: B-1

Sample Number: S25-023 **Depth:** 5ft

Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: MTB

Particle Size Distribution Report

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.0	34.3	65.7	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1	100.0		
0.75	100.0		
0.5	100.0		
0.375	100.0		
0.25	100.0		
#4	100.0		
#8	100.0		
#10	100.0		
#20	100.0		
#30	100.0		
#40	100.0		
#50	100.0		
#60	99.9		
#100	95.9		
#200	65.7		

* (no specification provided)

Material Description

Sandy SILT

Atterberg Limits (ASTM D 4318)

PL= 29 **LL=** 40 **PI=** 11

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-6(7)

Coefficients

D_{90}= 0.1237 D_{85}= 0.1096 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moistrue Content = 35.5 Percent

Date Received: 2.28.2025 **Date Tested:** 3.6.2025

Tested By: MTB

Checked By: MTB

Title: PM:TJT

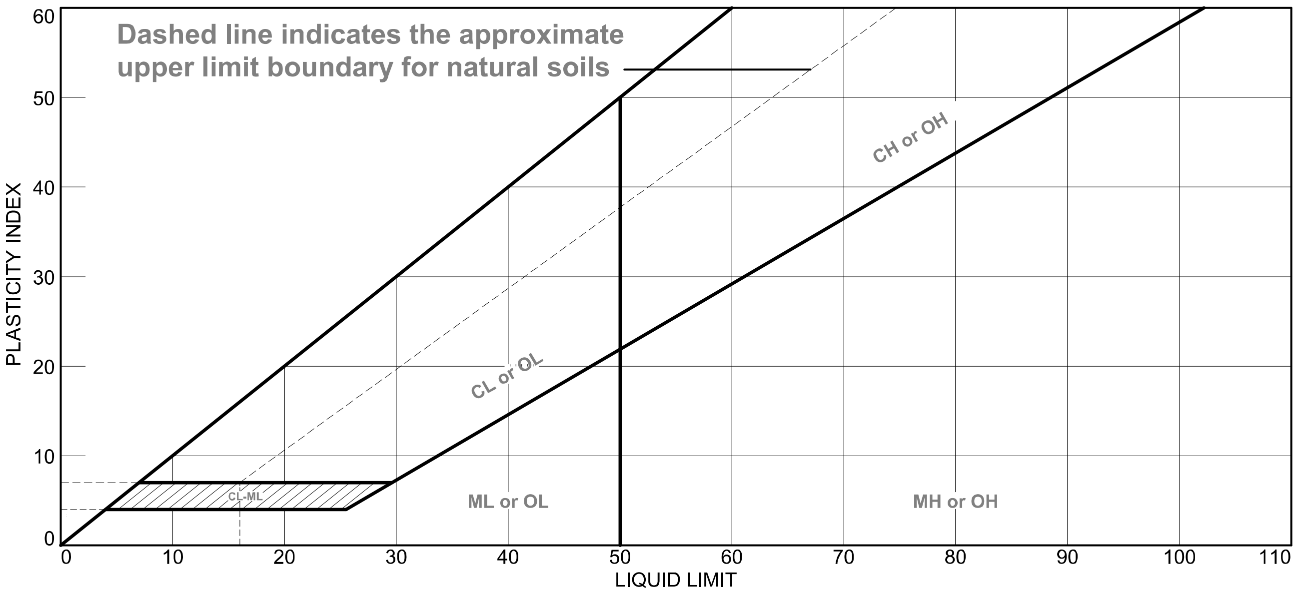
Location: B-1
Sample Number: S25-023 **Depth:** 5ft
Date Sampled: TJT 2.28.25

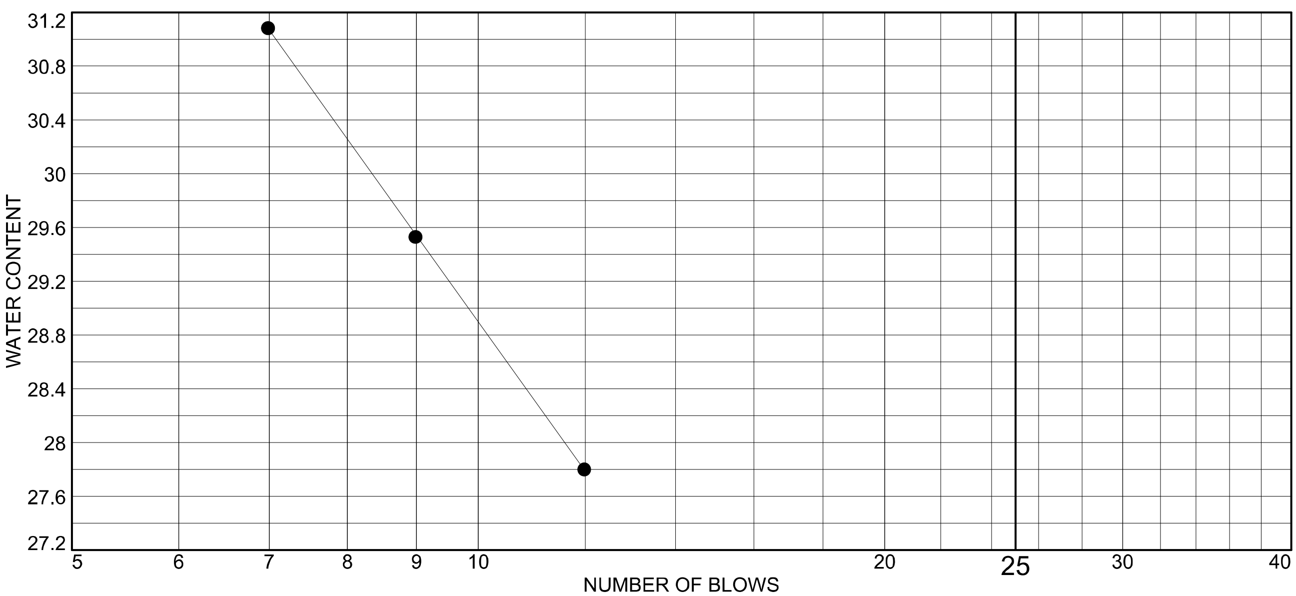
GEOPACIFIC ENGINEERING, INC.

Client: Public Works Engineering Department
Project: Woodburn Community Center

Project No: 25-6755 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy SILT	23	NP	NP	99.6	69.0	ML

Project No. 25-6755 **Client:** Public Works Engineering Department

Project: Woodburn Community Center

Location: B-1
Sample Number: S25-027 **Depth:** 20ft

Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: MTB

Particle Size Distribution Report

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.3	30.6	69.0	

Test Results (AASHTO T 27 & AASHTO T 11)

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1	100.0		
0.75	100.0		
0.5	100.0		
0.375	100.0		
0.25	100.0		
#4	100.0		
#8	99.9		
#10	99.9		
#20	99.8		
#30	99.8		
#40	99.6		
#50	98.9		
#60	98.5		
#100	96.1		
#200	69.0		

* (no specification provided)

Material Description

Sandy SILT

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** 23 **PI=** NP

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(0)

Coefficients

D_{90}= 0.1200 D_{85}= 0.1054 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture Content = 38.3 Percent

Date Received: 2.28.2025 **Date Tested:** 3.6.2025

Tested By: MTB

Checked By: MTB

Title: PM:TJT

Location: B-1
Sample Number: S25-027 **Depth:** 20ft
Date Sampled: TJT 2.28.25

GEOPACIFIC ENGINEERING, INC.

Client: Public Works Engineering Department
Project: Woodburn Community Center

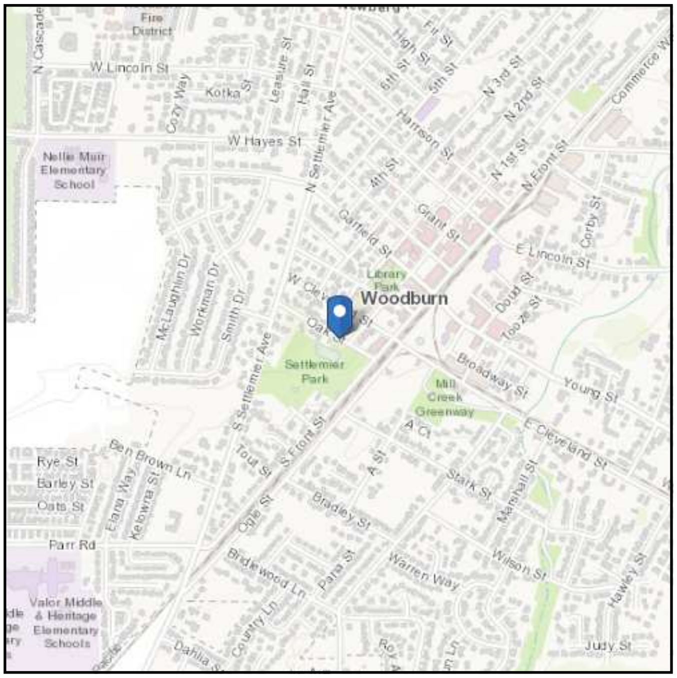
Project No: 25-6755 **Figure**

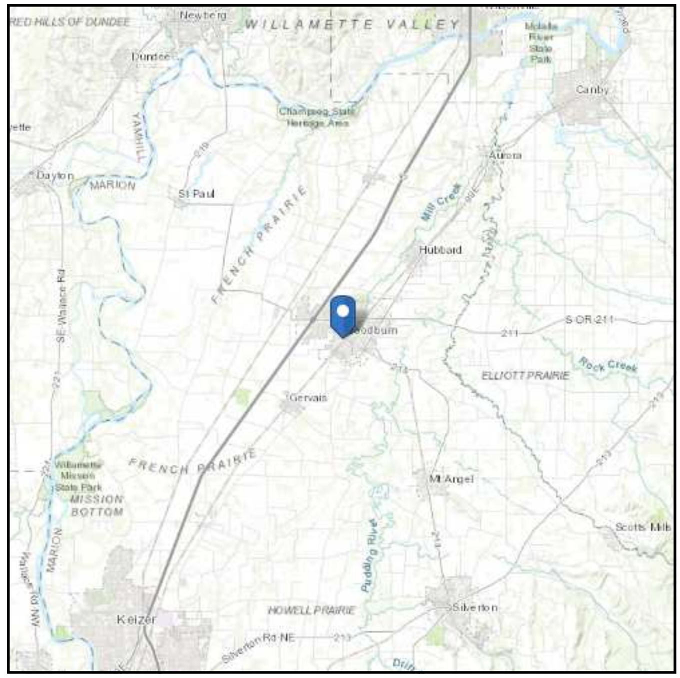
SITE RESEARCH

ASCE Hazards Report

Address:
190 Oak St
Woodburn, Oregon
97071

Standard:	ASCE/SEI 7-16	**Latitude:**	45.141095
Risk Category:	II	**Longitude:**	-122.860084
Soil Class:	D - Stiff Soil	**Elevation:**	184.45116918655796 ft (NAVD 88)





Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	0.838	S_{D1} :	N/A
S_1 :	0.399	T_L :	16
F_a :	1.165	PGA :	0.384
F_v :	N/A	PGA_M :	0.467
S_{MS} :	0.976	F_{PGA} :	1.216
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.651	C_v :	1.219

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Thu Mar 06 2025

Date Source: USGS Seismic Design Maps

The ASCE Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE standard.

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Unified Hazard Tool

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

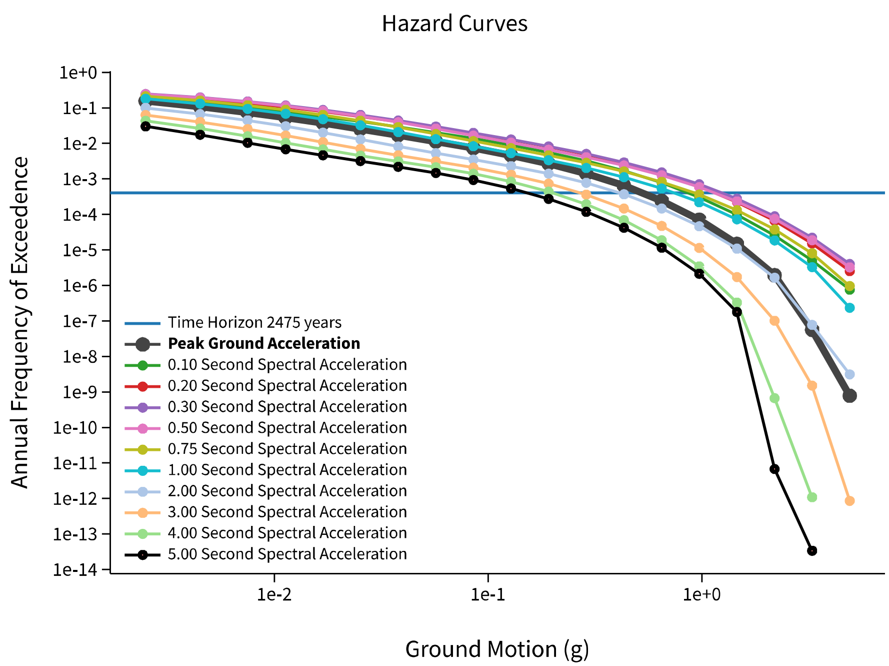
Peak Ground Acceleration

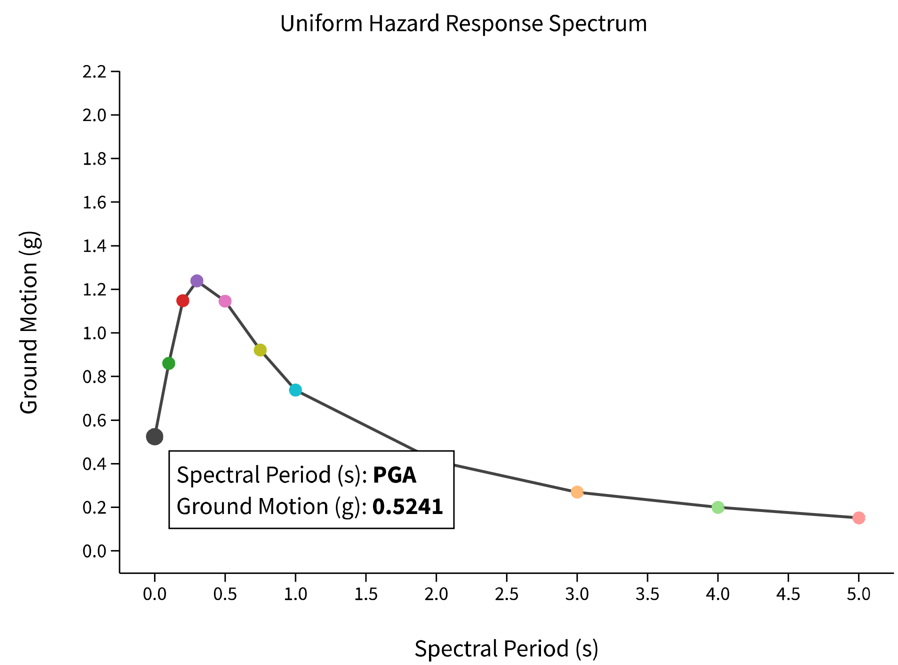
Time Horizon

Return period in years

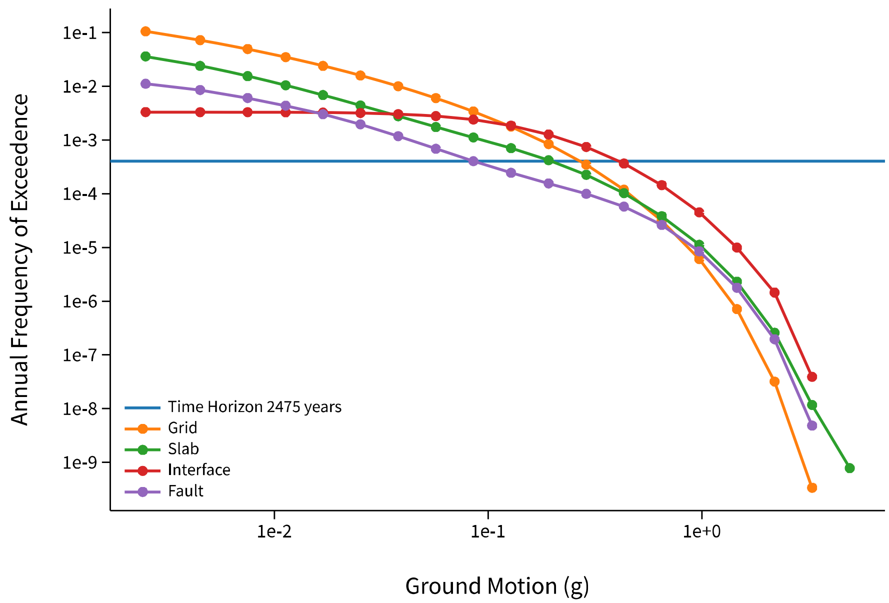
2475

Hazard Curve

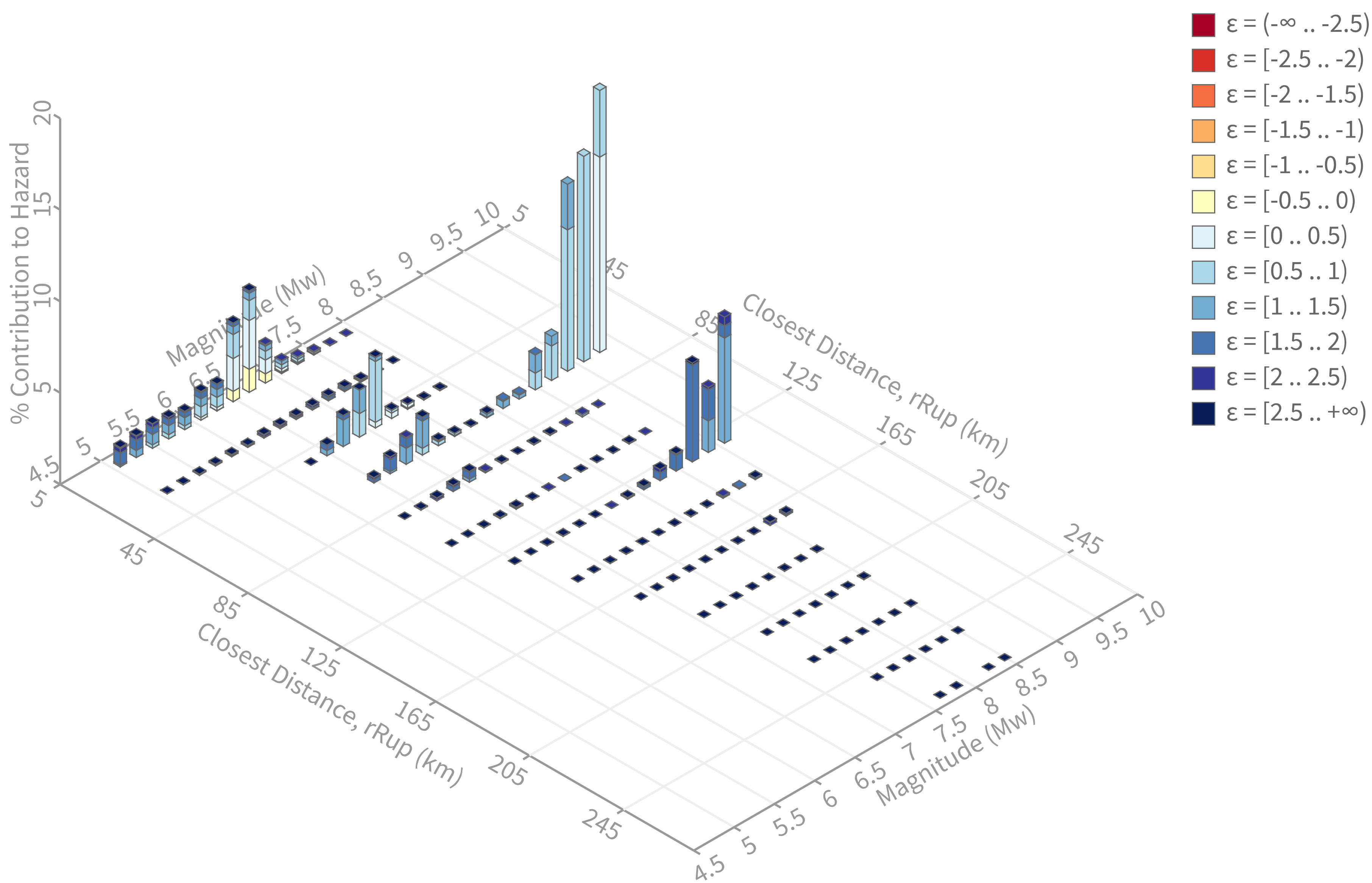




Component Curves for Peak Ground Acceleration



View Raw Data



% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)
ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.52413333 g

Recovered targets

Return period: 2463.1313 yrs
Exceedance rate: 0.00040598729 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.52 %

Mean (over all sources)

m: 8.02
r: 65.19 km
ε_0: 0.99 σ

Mode (largest m-r bin)

m: 9.34
r: 71.46 km
ε_0: 0.48 σ
Contribution: 14.32 %

Mode (largest m-r-ε_0 bin)

m: 9.01
r: 71.42 km
ε_0: 0.72 σ
Contribution: 11.23 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
sub0_ch_bot.in	Interface							34.03
Cascadia Megathrust - whole CSZ Characteristic		71.46	9.11	0.65	123.702°W	45.000°N	256.96	34.03
sub0_ch_mid.in	Interface							11.94
Cascadia Megathrust - whole CSZ Characteristic		122.82	8.93	1.47	124.330°W	45.489°N	289.15	11.94
coastalOR_deep.in	Slab							9.35
coastalOR_deep.in	Slab							4.67
Geologic Model Partial Rupture	Fault							4.56
Mount Angel		3.00	6.63	0.24	122.872°W	45.161°N	336.42	4.31
Geologic Model Full Rupture	Fault							3.43
Mount Angel		0.44	6.75	0.02	122.872°W	45.161°N	336.42	3.17
sub0_ch_top.in	Interface							3.02
Cascadia Megathrust - whole CSZ Characteristic		134.33	8.83	1.65	124.561°W	45.000°N	263.91	3.02
sub2_ch_bot.in	Interface							2.69
Cascadia Megathrust - Goldfinger Case C Characteristic		74.50	8.73	0.96	123.702°W	45.000°N	256.96	2.69
WUSmap_2014_fixSm.ch.in (opt)	Grid							2.58
noPuget_2014_fixSm.ch.in (opt)	Grid							2.58
WUSmap_2014_fixSm.gr.in (opt)	Grid							2.39
noPuget_2014_fixSm.gr.in (opt)	Grid							2.39
sub1_ch_bot.in	Interface							1.51
Cascadia Megathrust - Goldfinger Case B Characteristic		70.91	8.86	0.81	123.702°W	45.000°N	256.96	1.51
sub1_GRb0_bot.in	Interface							1.23
Cascadia floater over southern zone - Goldfinger Case B		75.44	8.48	1.13	123.702°W	45.000°N	256.96	1.23

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

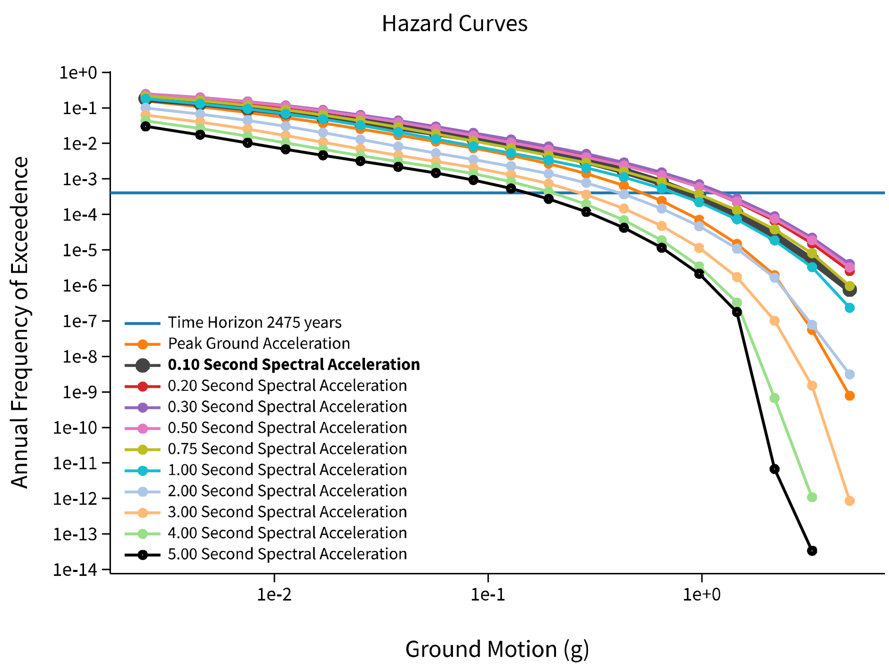
0.10 Second Spectral Acceleration

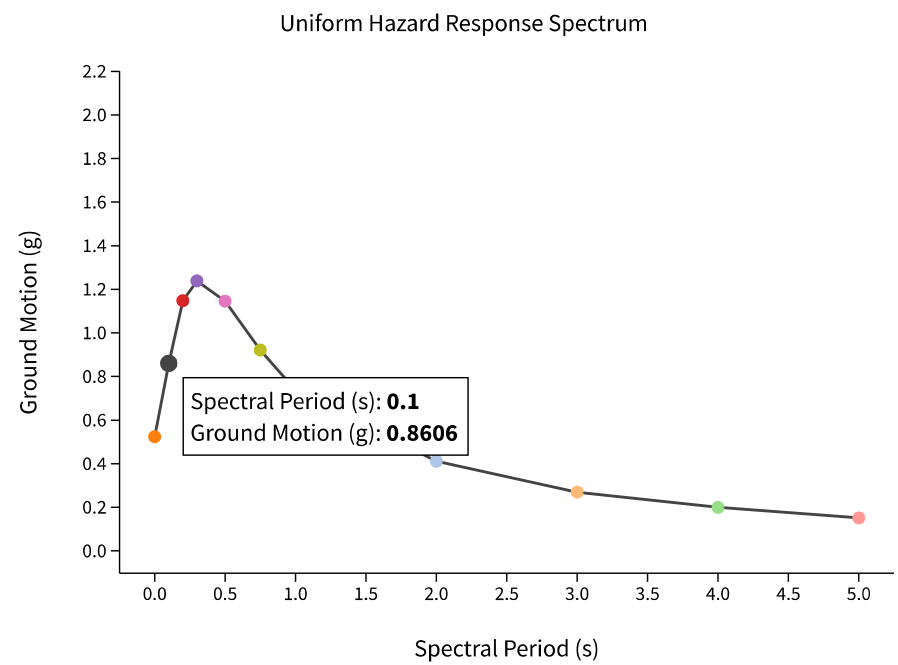
Time Horizon

Return period in years

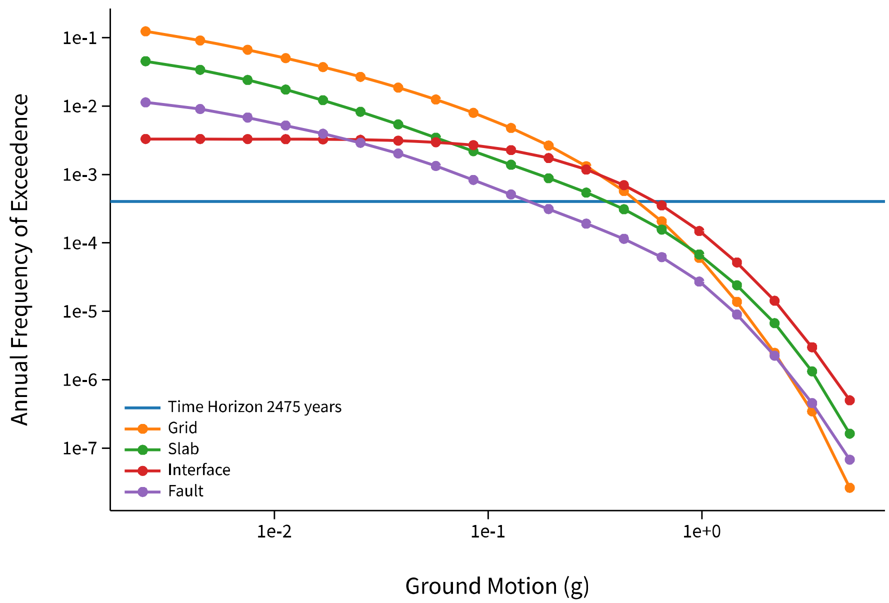
2475

Hazard Curve





Component Curves for 0.10 Second Spectral Acceleration



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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Spectral Period

0.20 Second Spectral Acceleration

Latitude

Decimal degrees

45.141

Time Horizon

Return period in years

2475

Longitude

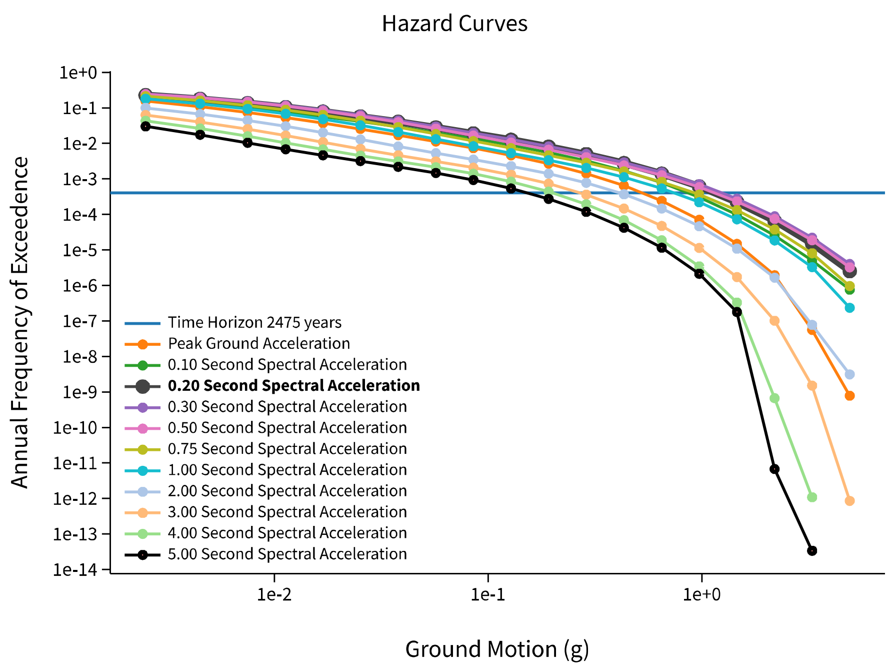
Decimal degrees, negative values for western longitudes

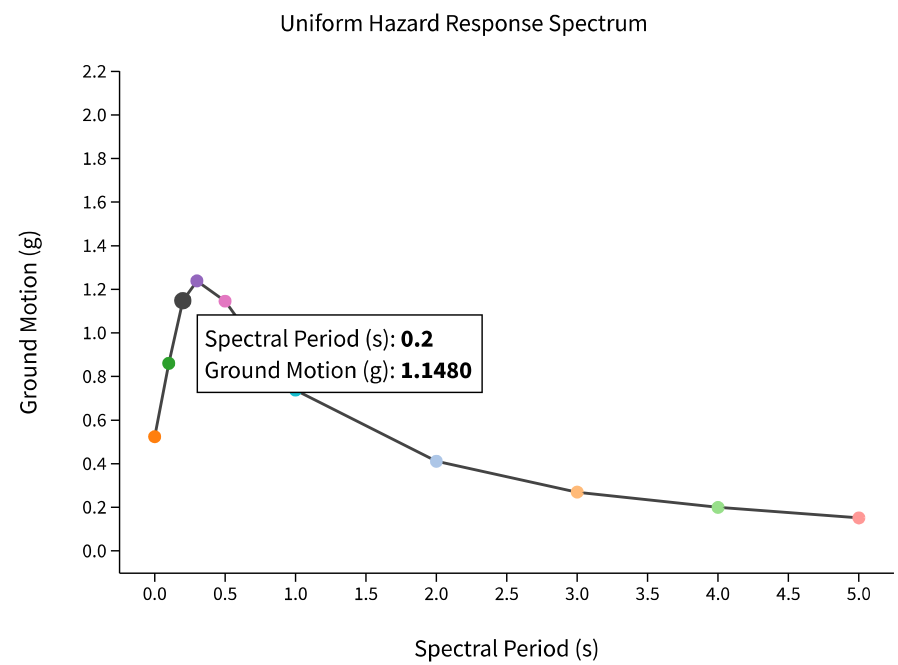
-122.86

Site Class

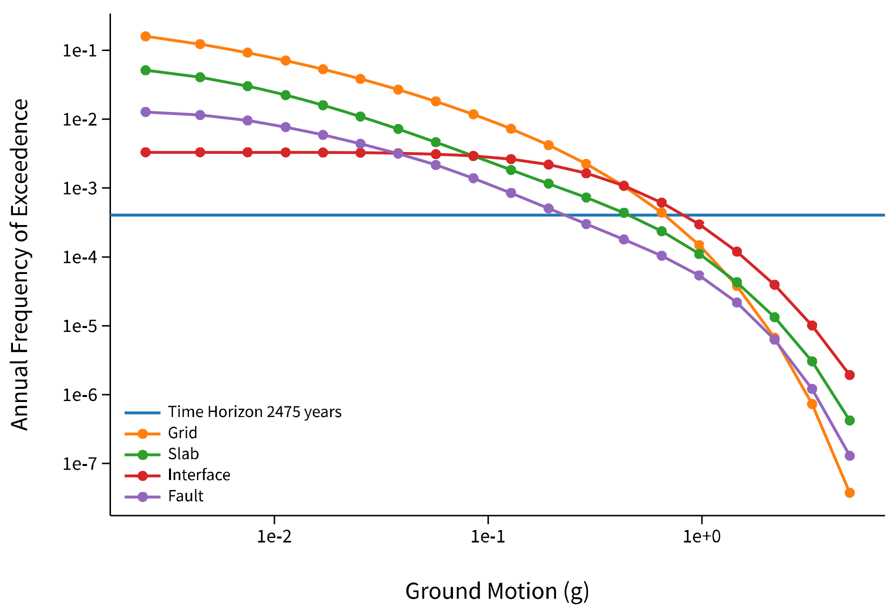
259 m/s (Site class D)

Hazard Curve





Component Curves for 0.20 Second Spectral Acceleration



View Raw Data

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude
Decimal degrees

45.141

Longitude
Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

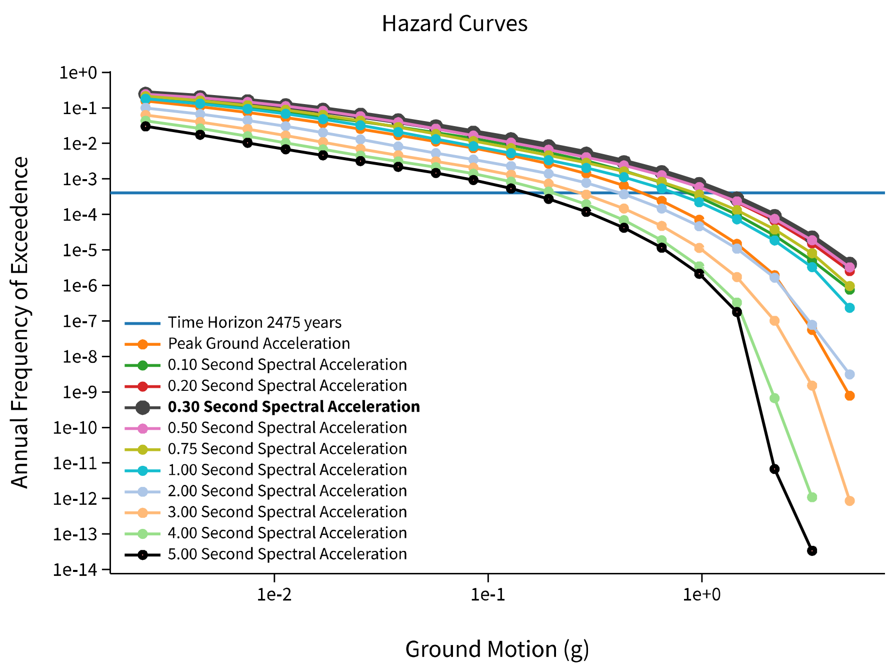
Spectral Period

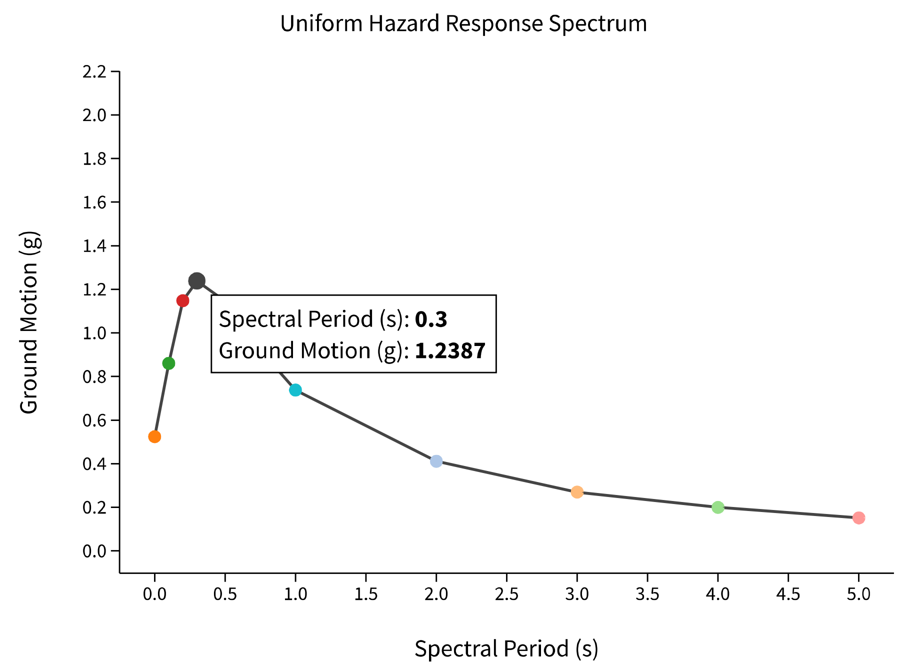
0.30 Second Spectral Acceleration

Time Horizon
Return period in years

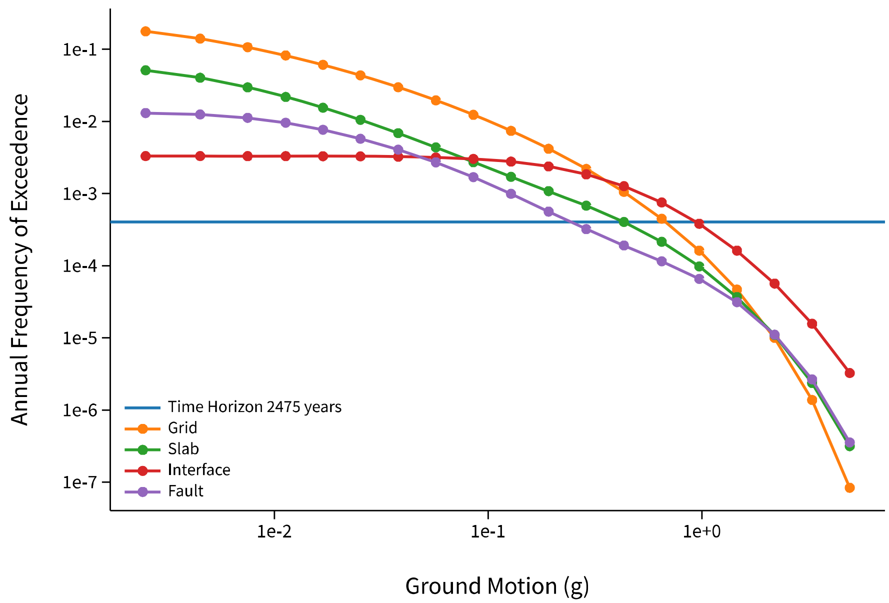
2475

Hazard Curve





Component Curves for 0.30 Second Spectral Acceleration



View Raw Data

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

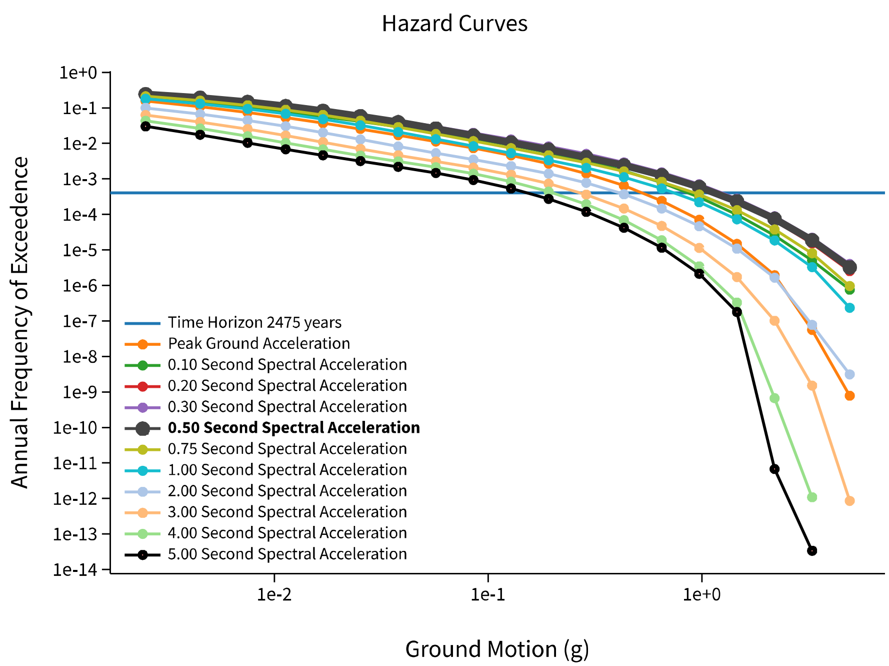
0.50 Second Spectral Acceleration

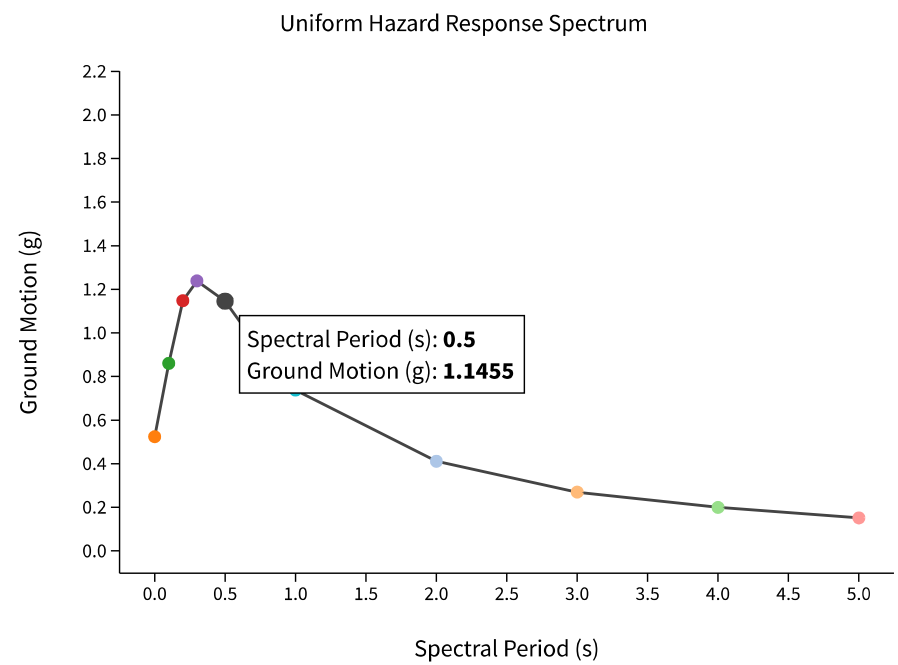
Time Horizon

Return period in years

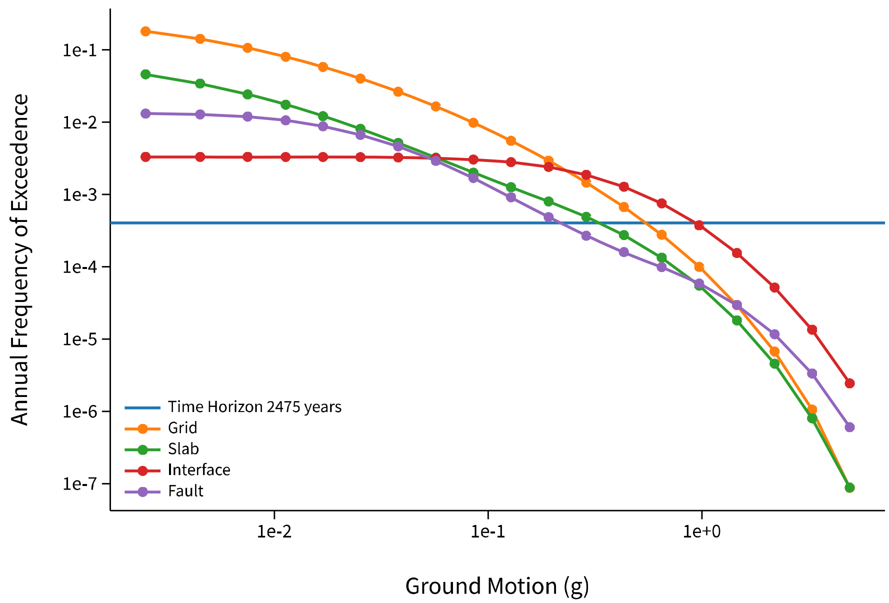
2475

Hazard Curve





Component Curves for 0.50 Second Spectral Acceleration



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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Spectral Period

0.75 Second Spectral Acceleration

Latitude

Decimal degrees

45.141

Time Horizon

Return period in years

2475

Longitude

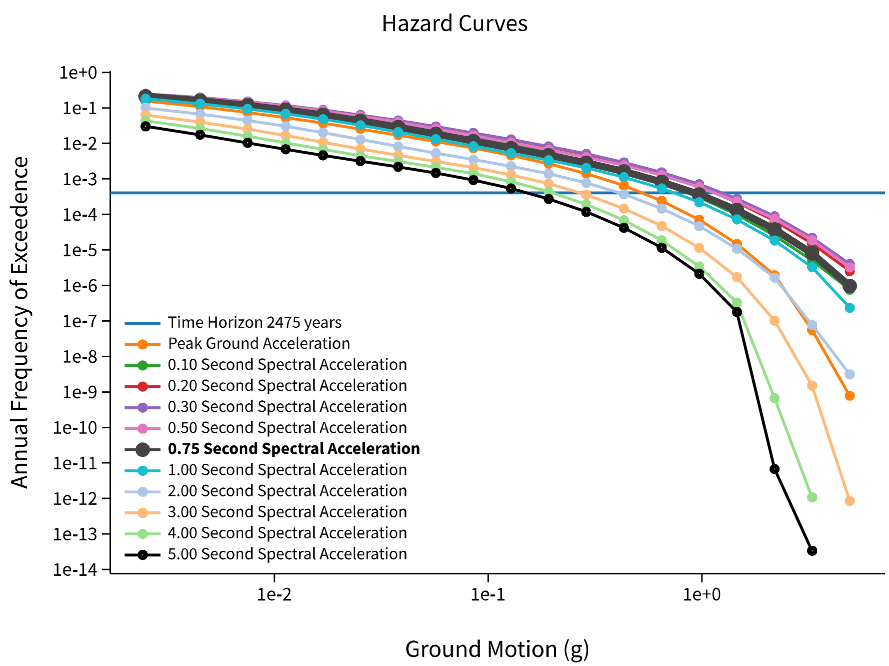
Decimal degrees, negative values for western longitudes

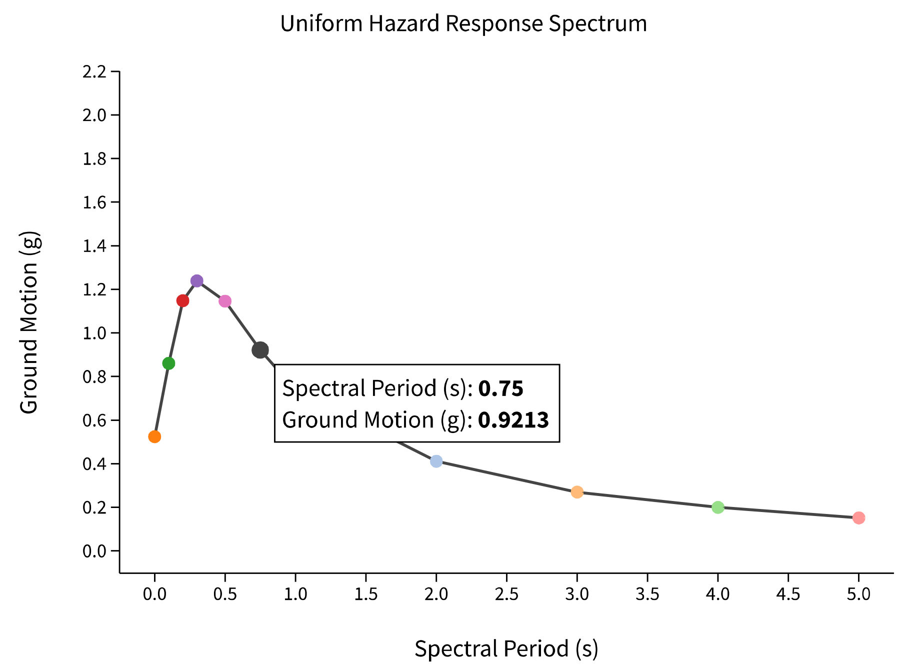
-122.86

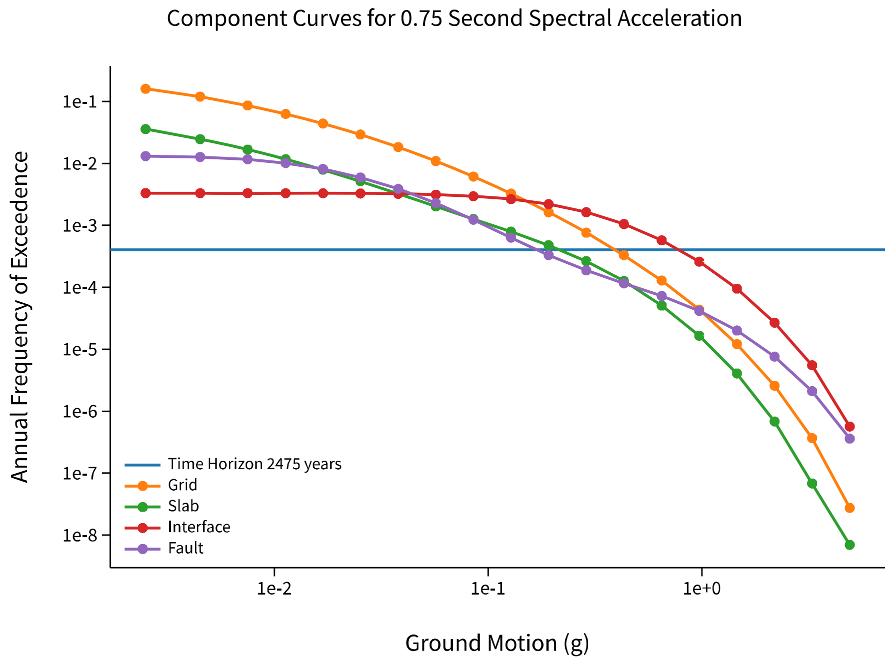
Site Class

259 m/s (Site class D)

Hazard Curve







[View Raw Data]

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

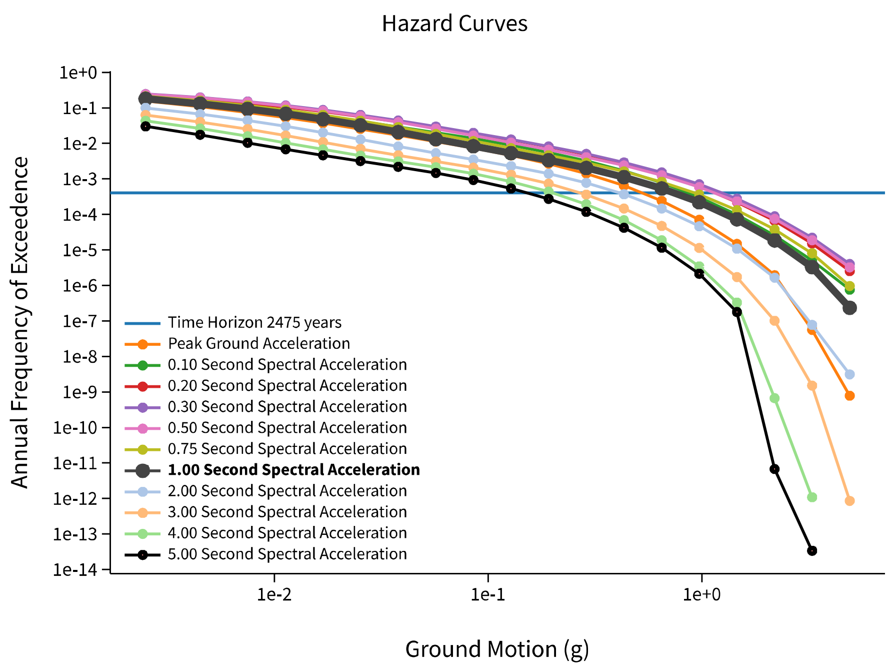
1.00 Second Spectral Acceleration

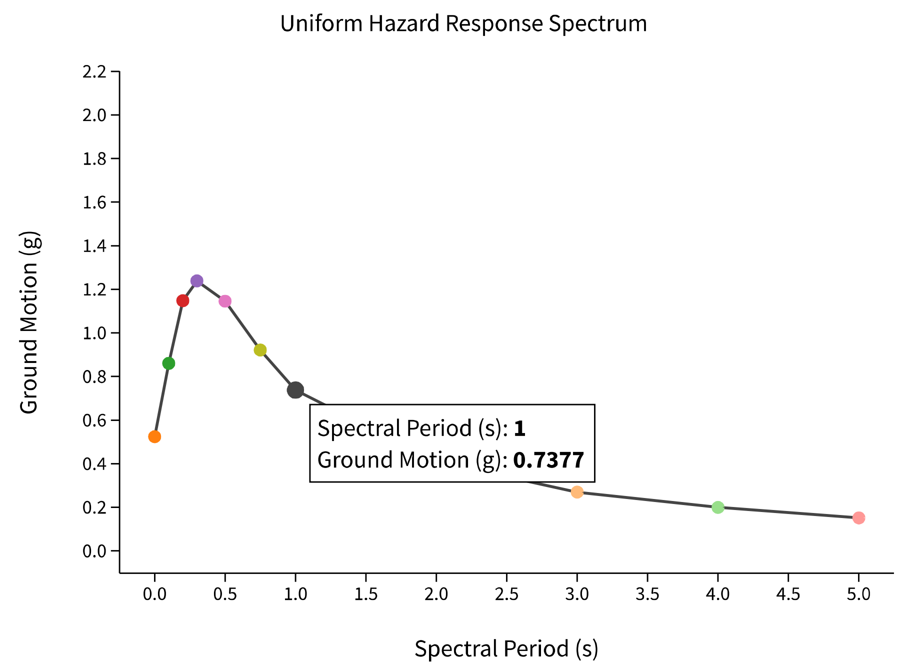
Time Horizon

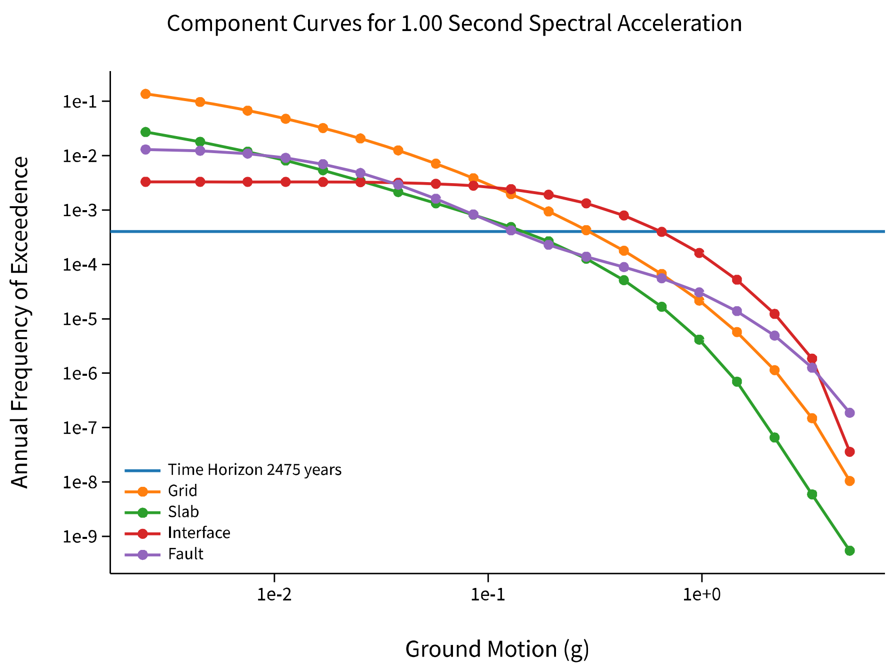
Return period in years

2475

Hazard Curve







View Raw Data

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

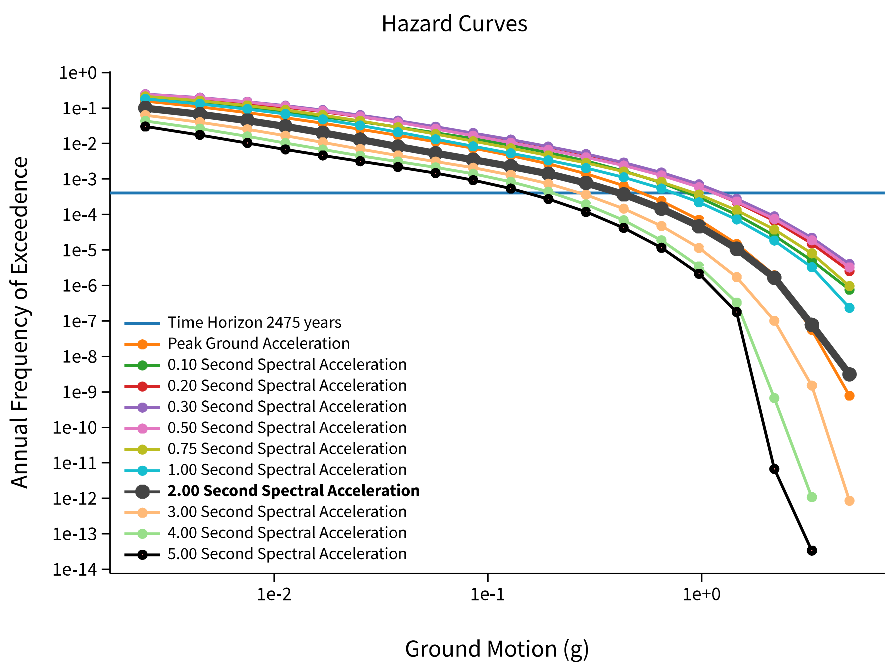
2.00 Second Spectral Acceleration

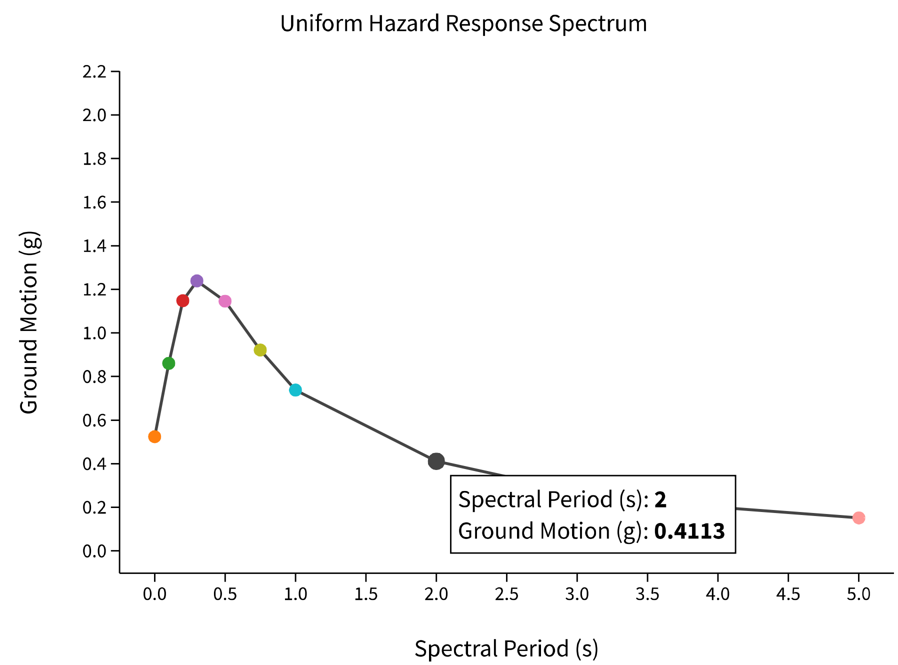
Time Horizon

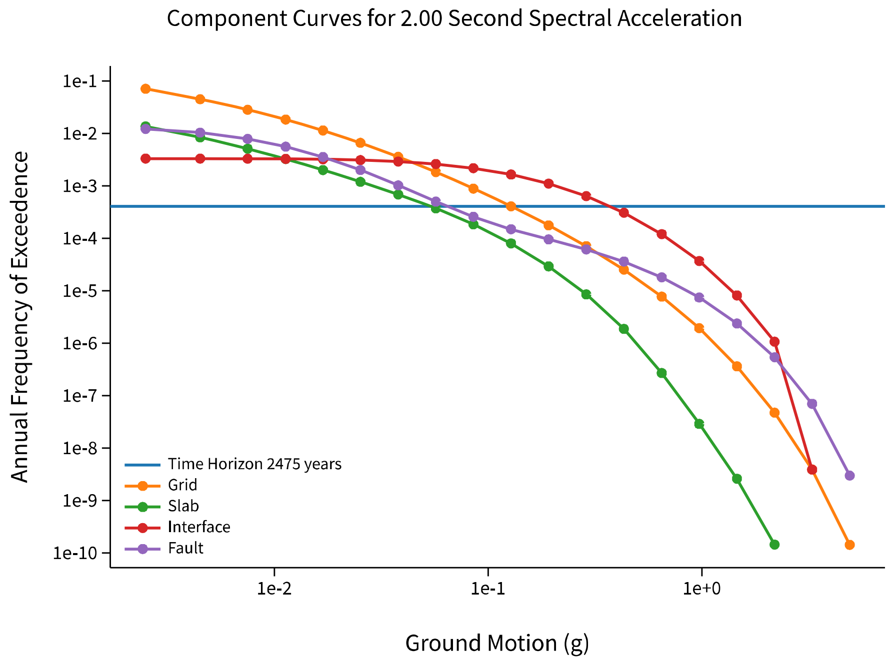
Return period in years

2475

Hazard Curve







[View Raw Data]

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Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

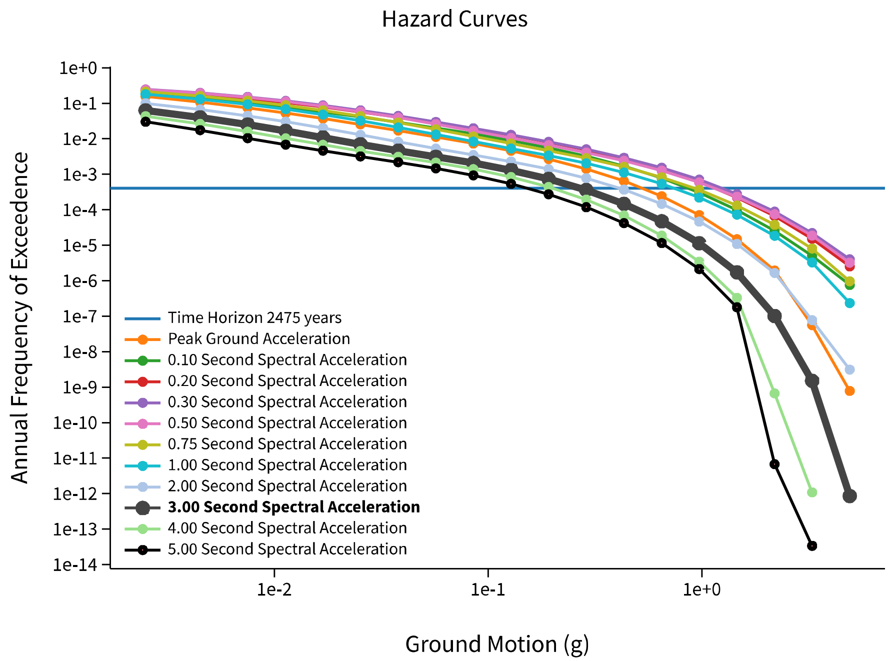
3.00 Second Spectral Acceleration

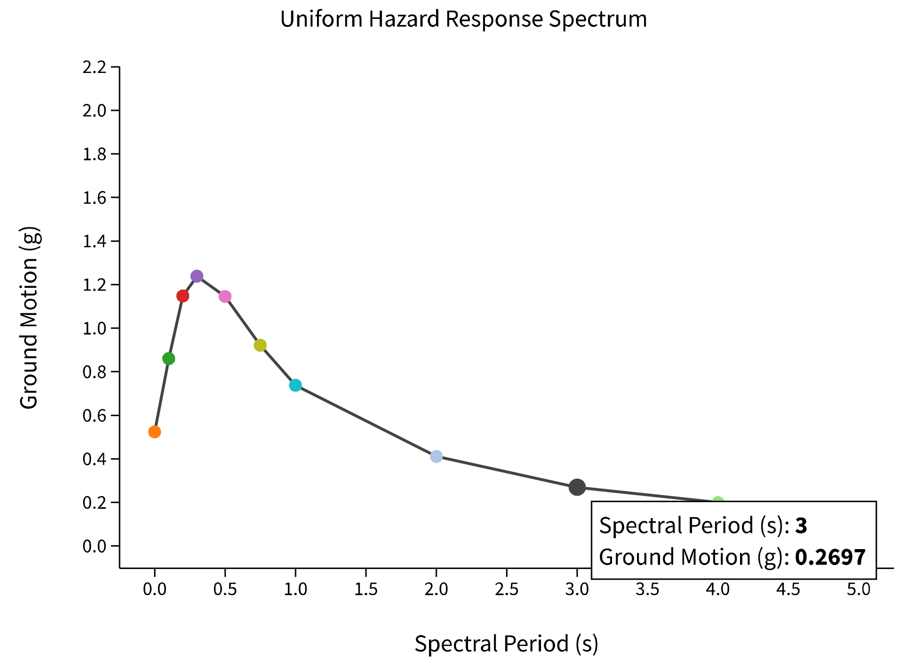
Time Horizon

Return period in years

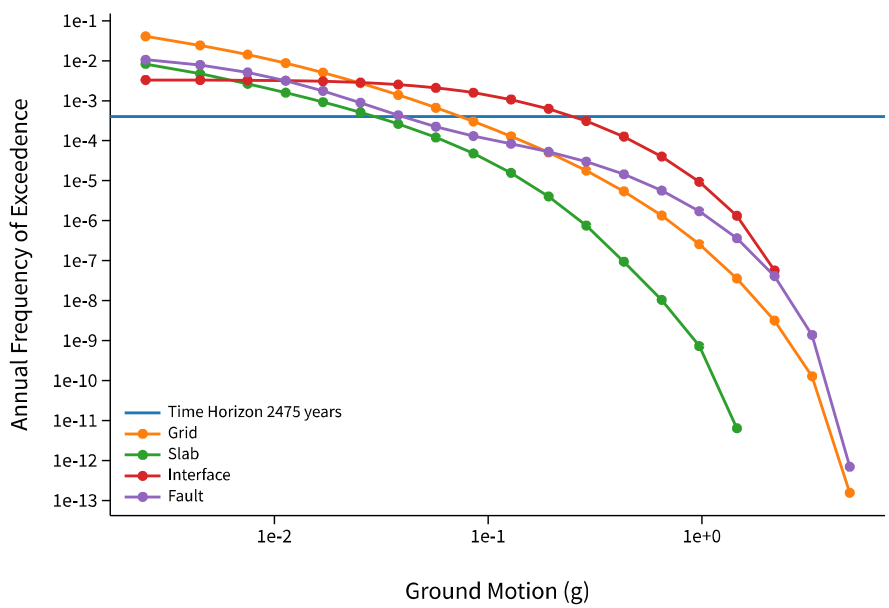
2475

Hazard Curve





Component Curves for 3.00 Second Spectral Acceleration



View Raw Data

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

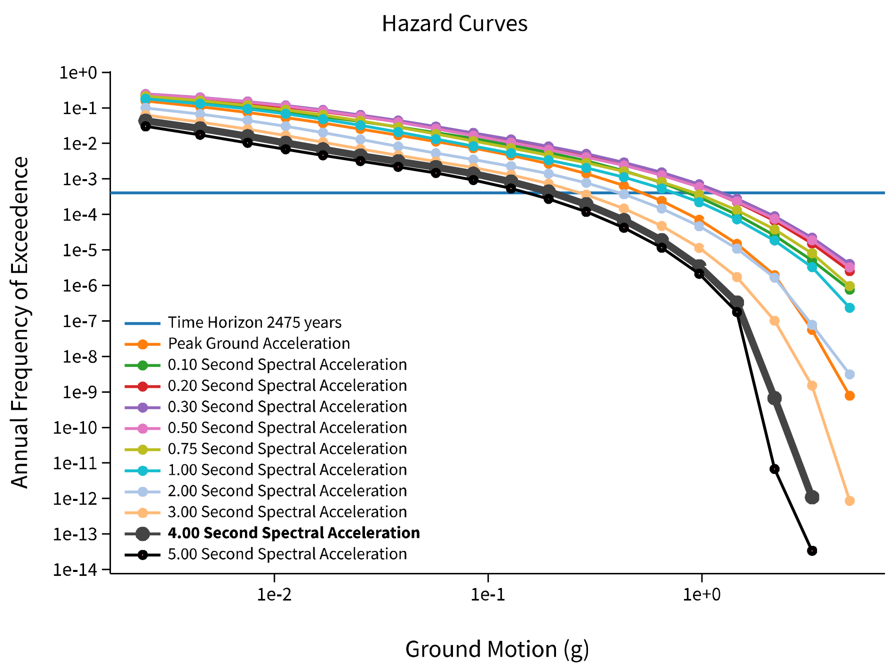
4.00 Second Spectral Acceleration

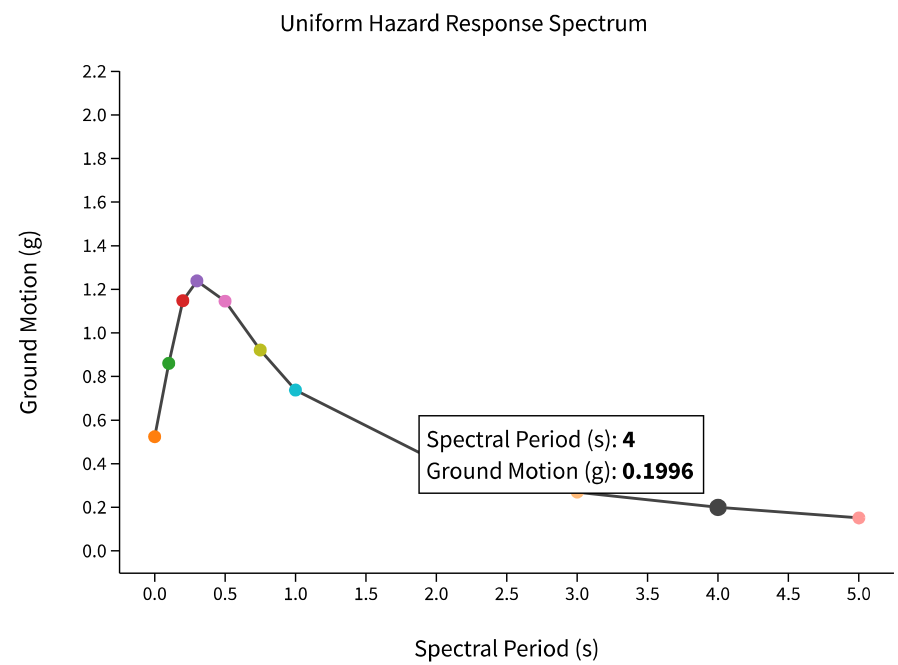
Time Horizon

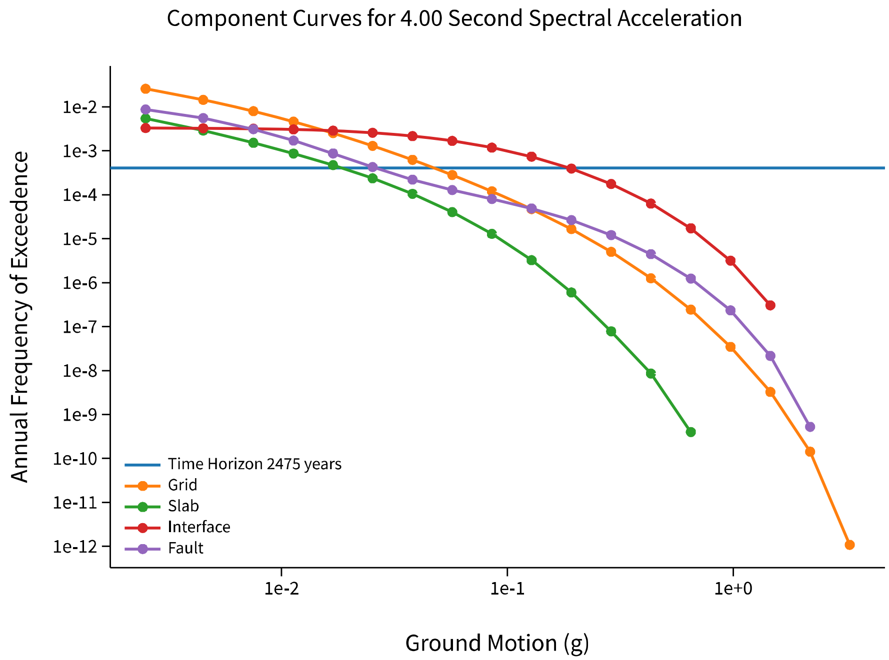
Return period in years

2475

Hazard Curve







View Raw Data

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (unknown)

Latitude

Decimal degrees

45.141

Longitude

Decimal degrees, negative values for western longitudes

-122.86

Site Class

259 m/s (Site class D)

Spectral Period

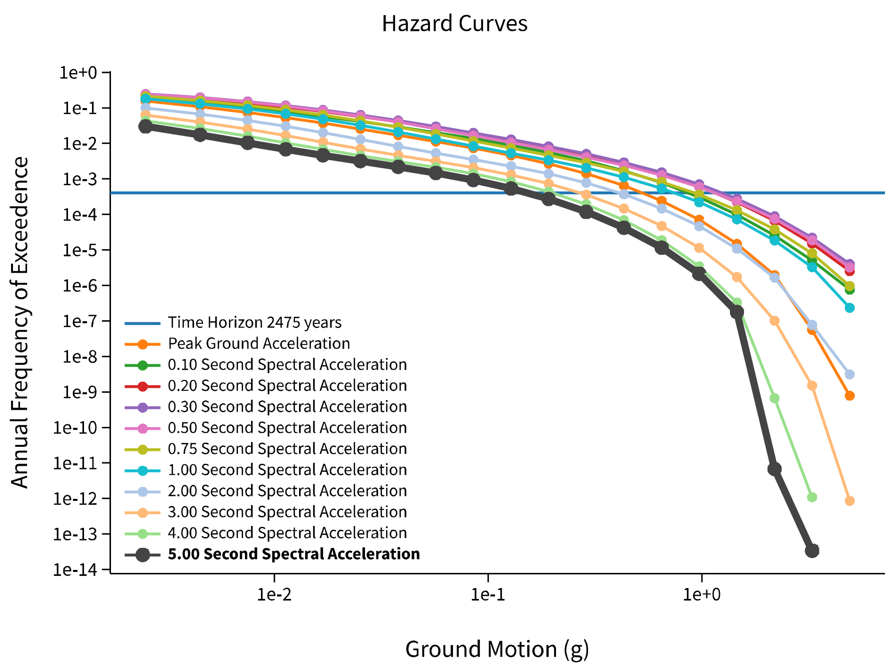
5.00 Second Spectral Acceleration

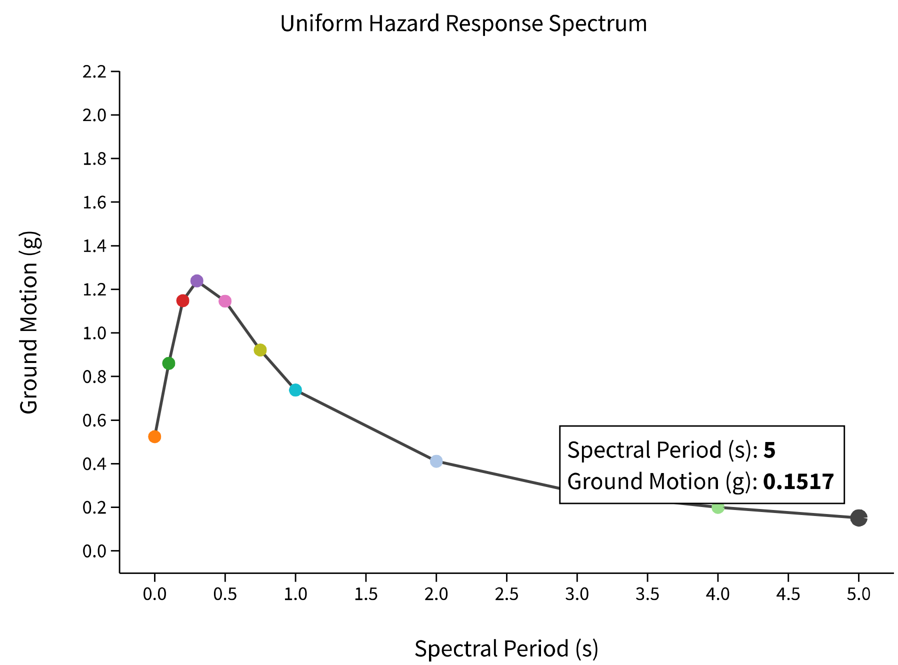
Time Horizon

Return period in years

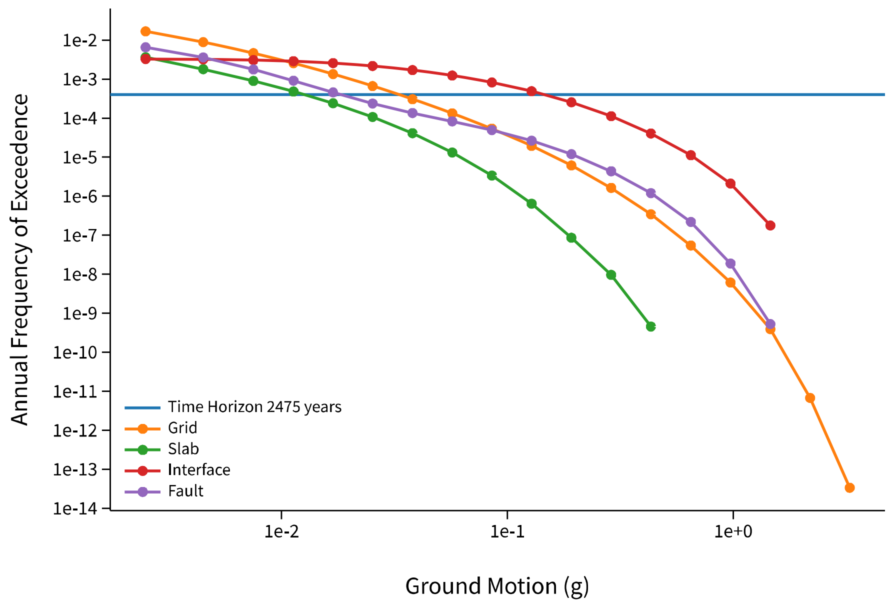
2475

Hazard Curve





Component Curves for 5.00 Second Spectral Acceleration



View Raw Data

SHEAR WAVE REFRACTION MICROTREMOR ANALYSIS (REMI)

Array Location

Datum:	No Datum given
Site Name:	**Woodburn Community**
Center:	0.0, 0.0
Geophone 1:	0.0, 0.0
Geophone12:	0.0, 0.0

Results

Vs100:	**855 ft/s**
Site Class:	D
ASCE:	ASCE 7-16
Depth:	**144.7 ft**

Survey Parameters

Geophone Count:	12
Geophone Spacing:	26.3 ft
Array Length:	289 ft
Survey Date:	02282025
Performed By:	GeoPacific
Analysis By:	TJT/ABC
Analysis Date:	Fri Mar 07 14:33:27 PST 2025

Narrative

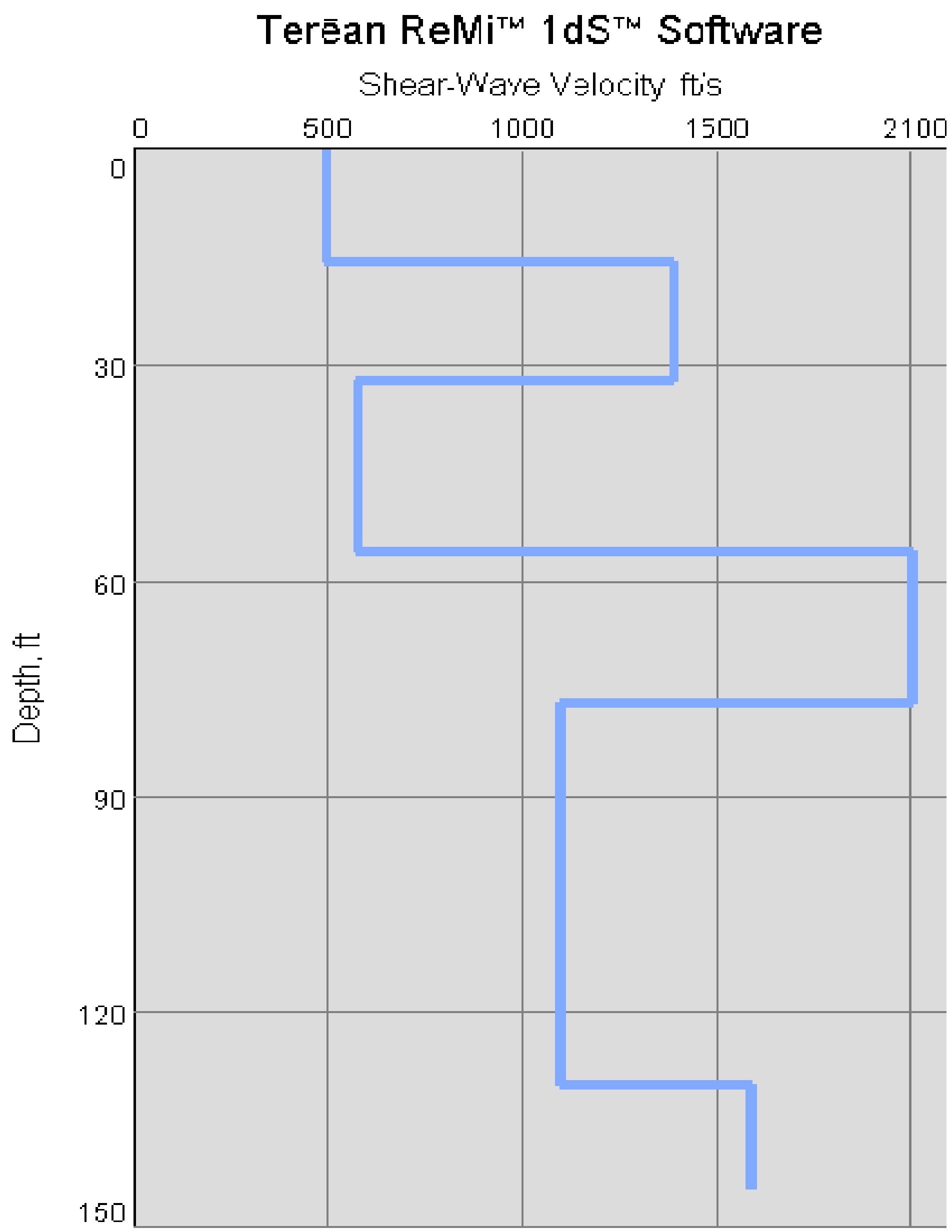
SHEAR WAVE VELOCITIES AND SEISMIC SITE CLASS

GeoPacific ’s geophysical site evaluation included subsurface seismic imaging and earthquake ground shaking potential evaluation using Terēan’s VsSurf ReMi™ seismic data processing software. Seismic surveys were performed to determine depth to bedrock and the seismic site class per ASCE 7-16 using the weighted-average soil shear wave velocity for the upper 100 feet (Vs100). The surveys were performed by recording active and/or ambient (passive) seismic sources. The seismic recording array for these surveys consisted of 12, 4.5 Hz geophones at 26.3 ft spacing, for a total survey length of 289 feet. Noise generated by 6lb hammer blows placed at array ends during data acquisition while ambient noise was generated from traffic along the nearby roads. The seismic data were acquired using a ReMiDAQ™ 4-12 channel seismograph, while data was processed using Terēan's ReMi™ software (terean.com/products). Survey results indicate a weighted-average soil shear wave velocity of the upper 100 feet (Vs100) of 855 ft/s. This results in a designation of a Seismic Site Class D according to Table ASCE 7-16.

Woodburn Community
25-6755

Shear-Wave Velocity Report
Terēan VsSurf ReMi™ 2.1 Software

Depth (ft)	Vs (ft/s)
0.0	494
15.6	494
15.6	1,388
32.1	1,388
32.1	575
55.9	575
55.9	2,003
76.8	2,003
76.8	1,097
130.2	1,097
130.2	1,589
144.7	1,589

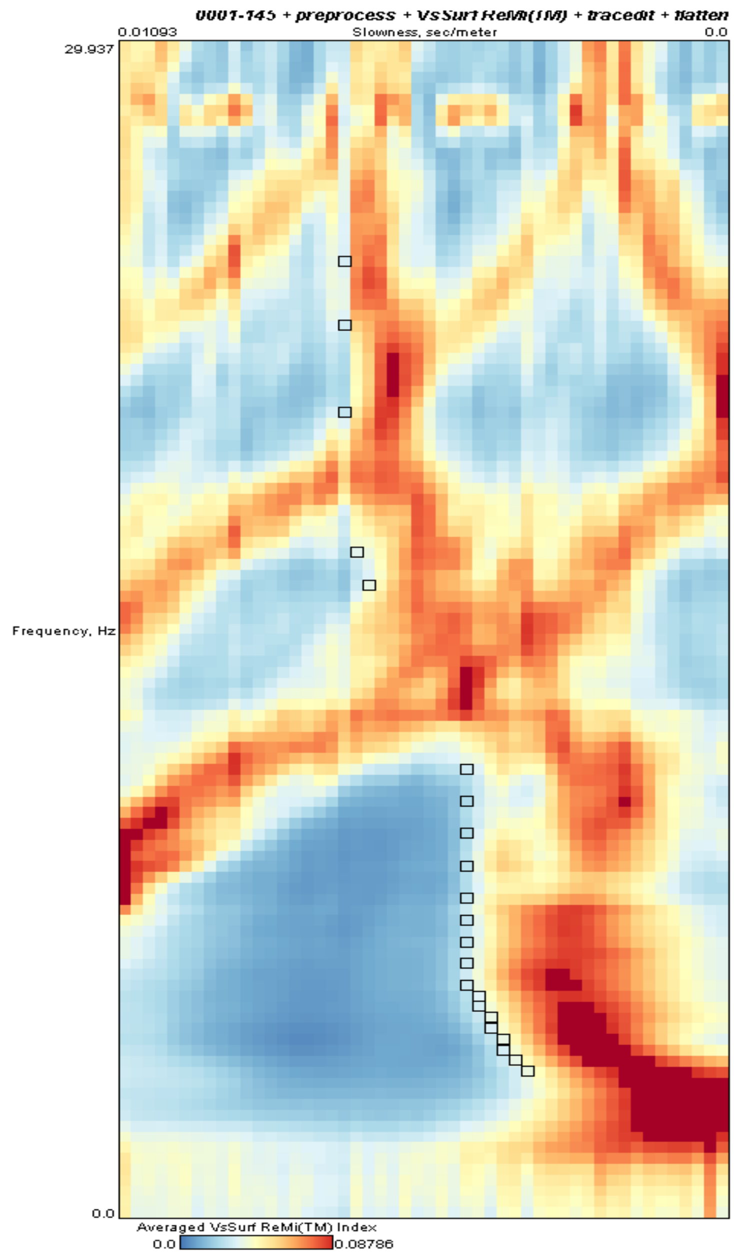


Woodburn Community
25-6755

Shear-Wave Velocity Report
Terēan VsSurf ReMi™ 2.1 Software

Frequency (Hz)	Slowness (s/m)
24.444	0.00699
22.796	0.00699
20.599	0.00699
17.028	0.00678
16.204	0.00656
11.535	0.00481
10.711	0.00481
9.888	0.00481
9.064	0.00481
8.240	0.00481
7.690	0.00481
7.141	0.00481
6.592	0.00481
6.042	0.00481
5.768	0.00459
5.493	0.00459
5.218	0.00437
4.944	0.00437
4.669	0.00415
4.394	0.00415
4.120	0.00393
3.845	0.00371

Seismic File:	
Pre-Processing:	preprocess
Surveyed Geophones:	No
Max Frequency, Hz:	30
Min Velocity, m/s:	91



LIQUEFACTION ANALYSIS

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland OR 97232

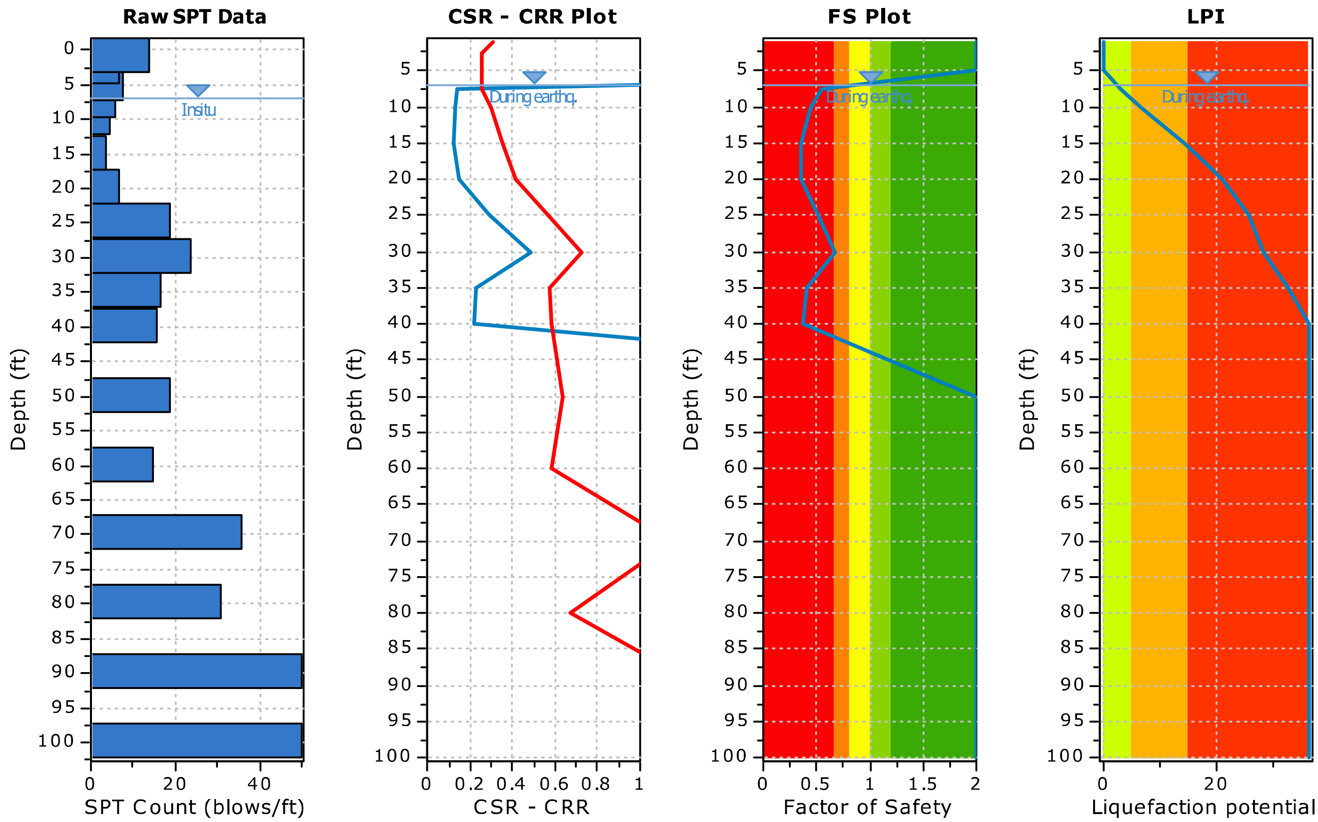
SPT BASED LIQUEFACTION ANALYSIS REPORT

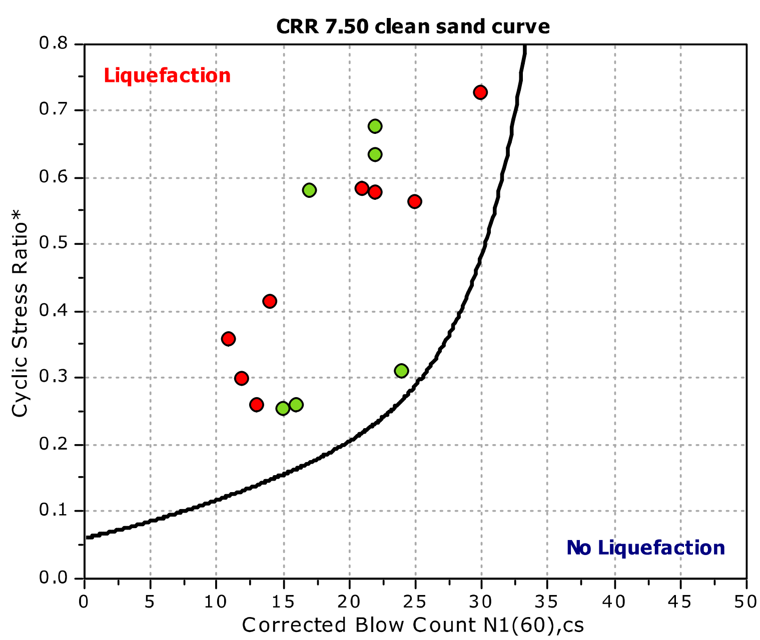
Project title : 25-66755 Woodburn Community Center **SPT Name: SPT #1**

Location : Woodburn

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	7.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	7.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	9.10
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.37 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		

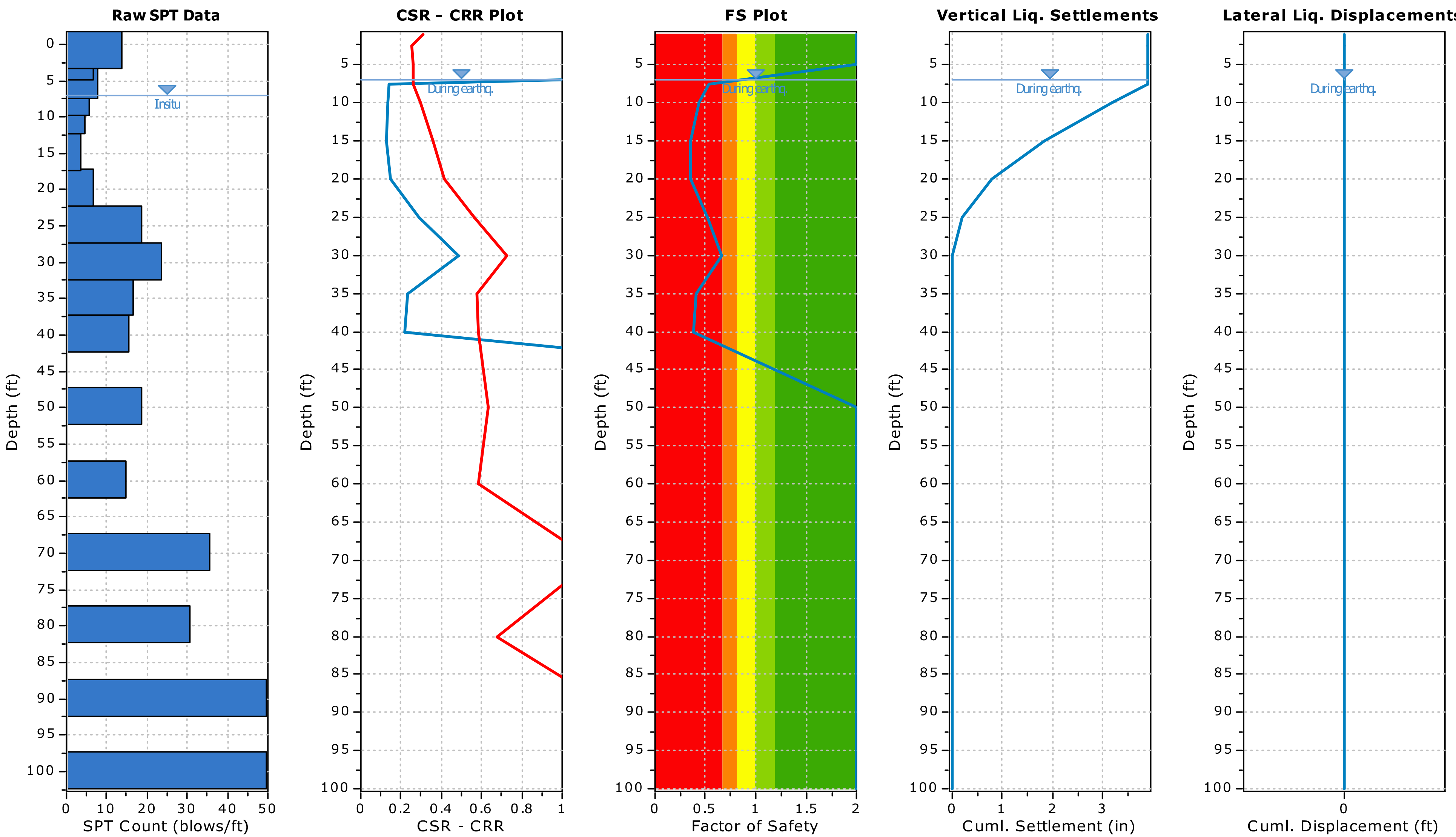




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



References

- Ronald D. Andrus, Hossein Hayati, Nisha P. Mohanan, 2009. Correcting Liquefaction Resistance for Aged Sands Using Measured to Estimated Velocity Ratio, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 6, June 1

- Boulanger, R.W. and Idriss, I. M., 2014. CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES. DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING COLLEGE OF ENGINEERING UNIVERSITY OF CALIFORNIA AT DAVIS

- Dipl.-Ing. Heinz J. Priebe, Vibro Replacement to Prevent Earthquake Induced Liquefaction, Proceedings of the Geotechnique - Colloquium at Darmstadt, Germany, on March 19th, 1998 (also published in Ground Engineering, September 1998), Technical paper 12-57E

- Robertson, P.K. and Cabal, K.L., 2007, Guide to Cone Penetration Testing for Geotechnical Engineering. Available at no cost at http://www.geologismiki.gr/

- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 127, October, pp 817-833

- Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180

- Zhang, G., Robertson. P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 130, No. 8, 861 -871

- Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 124, No. 4, 364-368

- R. Kayen, R. E. S. Moss, E. M. Thompson, R. B. Seed, K. O. Cetin, A. Der Kiureghian, Y. Tanaka, K. Tokimatsu, 2013. Shear - Wave Velocity–Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 139, No. 3, March 1

PHOTOGRAPHIC LOG

Location of B-1, Facing South

Split Spoon Sample from B-3, 12.5'

Appendix C

Conveyance Calculations – (in final version of report)

Appendix D

Plans & Details

WOODBURN COMMUNITY CENTER
WOODBURN, OREGON

GENERAL NOTES

1. SURVEY PROVIDED BY S&F LAND SERVICES, DATED 04/01/2020. ELEVATIONS ARE BASED ON MARION COUNTY VERTICAL DATUM ESTABLISHED PER BENCH MARK NO. T13, P10 RD 0246 LOCATED AT THE BENCHMARK AT THE INTERSECTION OF LINCOLN AND NORTH FIRST STREET WITH AN ELEVATION OF 187.52, NAVD 88. SURVEY HAS PROVIDED CONVERSTION FROM NAVD 88 TO NGVD 29 IS -3.37.
2. CONSTRUCTION LAYOUT (ALL ACTUAL LINES AND GRADES) SHALL BE STAKED BY A PROFESSIONAL SURVEYOR, REGISTERED IN THE STATE OF OREGON, BASED ON COORDINATES, DIMENSIONS, BEARINGS, AND ELEVATIONS, AS SHOWN, ON THE PLANS.
3. PROJECT CONTROL SHALL BE FIELD VERIFIED AND CHECKED FOR RELATIVE HORIZONTAL POSITION PRIOR TO BEGINNING CONSTRUCTION LAYOUT. [SEE SHEET C0.2A FOR PROJECT CONTROL INFORMATION.]
4. PROJECT CONTROL SHALL BE FIELD VERIFIED AND CHECKED FOR RELATIVE VERTICAL POSITION BASED ON THE BENCHMARK STATED HEREON, PRIOR TO BEGINNING CONSTRUCTION LAYOUT.
5. WHEN DIMENSIONS AND COORDINATE LOCATIONS ARE REPRESENTED - DIMENSIONS SHALL HOLD OVER COORDINATE LOCATION. NOTIFY THE CIVIL ENGINEER OF RECORD IMMEDIATELY UPON DISCOVERY OF ANY DISCREPANCIES.
6. BUILDING SETBACK DIMENSIONS FROM PROPERTY LINES SHALL HOLD OVER ALL OTHER CALLOUTS. PROPERTY LINES AND ASSOCIATED BUILDING SETBACKS SHALL BE VERIFIED PRIOR TO CONSTRUCTION LAYOUT.
7. CONTRACTOR SHALL PRESERVE AND PROTECT FROM DAMAGE ALL EXISTING MONUMENTATION DURING CONSTRUCTION. THE CONTRACTOR IS RESPONSIBLE FOR COORDINATING AND PAYING FOR THE REPLACEMENT OF ANY MONUMENTS DAMAGED OR REMOVED DURING CONSTRUCTION. NEW MONUMENTS SHALL BE REESTABLISHED BY A LICENSED SURVEYOR.
8. SOME SITE DEMOLITION AND UTILITY RELOCATION HAS BEEN PERFORMED. SURVEY MAY NOT BE COMPLETE OR ACCURATE. CONTRACTOR TO VERIFY EXISTING SITE CONDITIONS PRIOR TO CONSTRUCTION. CONTRACTOR SHALL BRING ANY DISCREPANCIES TO THE ATTENTION OF THE ENGINEER PRIOR TO BEGINNING CONSTRUCTION.
9. CONTRACTOR TO REFERENCE SOILS REPORT FOR THE SITE SOILS CONDITIONS.
10. ALL CONSTRUCTION AND MATERIALS SHALL CONFORM TO THESE PLANS, THE PROJECT SPECIFICATIONS AND THE APPLICABLE REQUIREMENTS OF THE 2024 OREGON STANDARD SPECIFICATIONS FOR CONSTRUCTION AND REQUIREMENTS OF THE CITY OF WOODBURN.
11. THE COMPLETED INSTALLATION SHALL CONFORM TO ALL APPLICABLE FEDERAL, STATE, AND LOCAL CODES, ORDINANCES AND REGULATIONS. ALL PERMITS, LICENSES AND INSPECTIONS REQUIRED BY THE GOVERNING AUTHORITIES FOR THE EXECUTION AND COMPLETION OF WORK SHALL BE SECURED BY THE CONTRACTOR PRIOR TO COMMENCING CONSTRUCTION.
12. ATTENTION: OREGON LAW REQUIRES YOU TO FOLLOW RULES ADOPTED BY THE OREGON UTILITY NOTIFICATION CENTER. THOSE RULES ARE SET FORTH IN OAR 952-001-0010 THROUGH OAR 952-001-0090. YOU MAY OBTAIN COPIES OF THE RULES BY CALLING THE CENTER. (NOTE: THE TELEPHONE NUMBER FOR THE OREGON UTILITY NOTIFICATION CENTER IS (503) 232-1987). EXCAVATORS MUST NOTIFY ALL PERTINENT COMPANIES OR AGENCIES WITH UNDERGROUND UTILITIES IN THE PROJECT AREA AT LEAST 48 BUSINESS-DAY HOURS, BUT NOT MORE THAN 10 BUSINESS DAYS PRIOR TO COMMENCING AN EXCAVATION, SO UTILITIES MAY BE ACCURATELY LOCATED.
13. THE LOCATION OF EXISTING UNDERGROUND UTILITIES SHOWN ON THE PLANS ARE FOR INFORMATION ONLY AND ARE NOT GUARANTEED TO BE COMPLETE OR ACCURATE. CONTRACTOR SHALL VERIFY ELEVATIONS, PIPE SIZE, AND MATERIAL TYPES OF ALL UNDERGROUND UTILITIES PRIOR TO COMMENCING WITH CONSTRUCTION AND SHALL BRING ANY DISCREPANCIES TO THE ATTENTION OF KPFF CONSULTING ENGINEERS, 72 HOURS PRIOR TO START OF CONSTRUCTION TO PREVENT GRADE AND ALIGNMENT CONFLICTS.
14. THE ENGINEER OR OWNER IS NOT RESPONSIBLE FOR THE SAFETY OF THE CONTRACTOR OR HIS CREW. ALL O.S.H.A. REGULATIONS SHALL BE STRICTLY ADHERED TO IN THE PERFORMANCE OF THE WORK.
15. TEMPORARY AND PERMANENT EROSION CONTROL MEASURES SHALL BE IMPLEMENTED. THE CONTRACTOR SHALL ADHERE TO CITY OF WOODBURN DRAINAGE MANUAL AND DEQ MANUAL AND BEST MANAGEMENT PRACTICES FOR MINIMUM EROSION CONTROL MEASURES. THE ESC FACILITIES SHOWN IN THESE PLANS ARE THE MINIMUM REQUIREMENTS FOR ANTICIPATED SITE CONDITIONS. DURING THE CONSTRUCTION PERIOD, ESC FACILITIES SHALL BE UPGRADED AS NEEDED FOR UNEXPECTED STORM EVENTS AND TO ENSURE THAT SEDIMENT AND SEDIMENT LADEN WATER DO NOT LEAVE THE SITE.
16. THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING ALL ROADWAYS, KEEPING THEM CLEAN AND FREE OF CONSTRUCTION MATERIALS AND DEBRIS, AND PROVIDING DUST CONTROL AS REQUIRED.
17. TRAFFIC CONTROL SHALL BE PROVIDED BY THE CONTRACTOR THROUGHOUT CONSTRUCTION. CONTRACTOR SHALL PROVIDE A TRAFFIC CONTROL PLAN TO THE CITY OF WOODBURN FOR REVIEW AND APPROVAL PRIOR TO COMMENCING CONSTRUCTION.
18. CONTRACTOR SHALL MAINTAIN ALL UTILITIES TO THE WOODBURN COMMUNITY CENTER DURING OPERATING HOURS. CHECK WITH THE CITY OF WOODBURN FOR MORE REQUIREMENTS.
19. THE CONTRACTOR SHALL BE RESPONSIBLE FOR COORDINATING AND SCHEDULING ALL WORK WITH THE OWNER.
20. NOTIFY CITY INSPECTOR 72 HOURS BEFORE STARTING WORK. A PRECONSTRUCTION MEETING WITH THE CITY'S REPRESENTATIVE, THE CITY'S ARCHITECT, THE CITY'S ENGINEER, AND THE CONTRACTOR SHALL BE REQUIRED.

CONSTRUCTION NOTES

GENERAL

1. ACTUAL LINES AND GRADES SHALL BE STAKED BY A PROFESSIONAL SURVEYOR, REGISTERED IN THE STATE OF OREGON, BASED ON DIMENSIONS, ELEVATIONS AND BEARINGS AS SHOWN ON THE PLANS.
2. SUBGRADE AND TRENCH BACKFILL SHALL BE COMPACTED TO AT LEAST 95% OF THE MAXIMUM DRY DENSITY AS DETERMINED BY ASTM D-698. FLOODING OR JETTING THE BACKFILLED TRENCHES WITH WATER IS NOT PERMITTED.
3. SPECIAL INSPECTION REQUIRED FOR ALL COMPACTION TESTING.

DEMOLITION

1. THE CONTRACTOR SHALL BE RESPONSIBLE FOR DEMOLITION AND DISPOSAL OF EXISTING AC, CURBS, SIDEWALKS AND OTHER SITE ELEMENTS WITHIN THE SITE AREA IDENTIFIED IN THE PLANS.
2. EXCEPT FOR MATERIALS INDICATED TO BE STOCKPILED OR TO REMAIN ON OWNER'S PROPERTY, CLEARED MATERIALS SHALL BECOME CONTRACTOR'S PROPERTY, REMOVED FROM THE SITE, AND DISPOSED OF PROPERLY.
3. ITEMS INDICATED TO BE SALVAGED SHALL BE CAREFULLY REMOVED AND DELIVERED STORED AT THE PROJECT SITE AS DIRECTED BY THE OWNER.
4. ALL LANDSCAPING, PAVEMENT, CURBS AND SIDEWALKS, BEYOND THE IDENTIFIED SITE AREA, DAMAGED DURING THE CONSTRUCTION SHALL BE REPLACED TO THEIR ORIGINAL CONDITION OR BETTER.
5. CONCRETE SIDEWALKS SHOWN FOR DEMOLITION SHALL BE REMOVED TO THE NEAREST EXISTING CONSTRUCTION JOINT.
6. SAWCUT STRAIGHT MATCHLINES TO CREATE A BUTT JOINT BETWEEN THE EXISTING AND NEW PAVEMENT.

UTILITIES

1. ADJUST ALL INCIDENTAL STRUCTURES, MANHOLES, VALVE BOXES, CATCH BASINS, FRAMES AND COVERS, ETC. TO FINISHED GRADE.
2. CONTRACTOR SHALL ADJUST ALL EXISTING AND/OR NEW FLEXIBLE UTILITIES (WATER, TV, TELEPHONE, ELEC., ETC.) TO CLEAR ANY EXISTING OR NEW GRAVITY DRAIN UTILITIES (STORM DRAIN, SANITARY SEWER, ETC.) IF CONFLICT OCCURS.
3. CONTRACTOR SHALL COORDINATE WITH PRIVATE UTILITY COMPANIES FOR THE INSTALLATION OF OR ADJUSTMENT TO GAS, ELECTRICAL, POWER AND TELEPHONE SERVICE.
4. BEFORE BACKFILLING ANY SUBGRADE UTILITY IMPROVEMENTS CONTRACTOR SHALL SURVEY AND RECORD MEASUREMENTS OF EXACT LOCATION AND DEPTH AND SUBMIT TO ENGINEER AND OWNER.
5. ALL WORK TO CONFORM TO THE 2023 OREGON PLUMBING SPECIALTY CODE

STORM AND SANITARY

1. CONNECTIONS TO EXISTING STORM AND SANITARY SEWERS SHALL CONFORM TO THE 2018 OREGON STANDARD SPECIFICATIONS FOR CONSTRUCTION, SECTION 00490, "WORK ON EXISTING SEWERS AND STRUCTURES".
2. BEGIN LAYING STORM DRAIN AND SANITARY SEWER PIPE AT THE LOW POINT OF THE SYSTEM, TRUE TO GRADE AND ALIGNMENT INDICATED WITH UNBROKEN CONTINUITY OF INVERT. THE CONTRACTOR SHALL ESTABLISH LINE AND GRADE FOR THE STORM AND SANITARY SEWER PIPE USING A LASER.
3. ACTUAL LINES AND GRADES SHALL BE STAKED BY A QUALIFIED SURVEYOR, BASED ON COORDINATES, DIMENSIONS AND BEARINGS INDICATED ON THE PLANS. CONTRACTOR SHALL RETAIN A SURVEYOR LICENSED IN THE STATE OF OREGON.
4. ALL ROOF DRAIN AND CATCH BASIN LEADERS SHALL HAVE A MINIMUM SLOPE OF 2 PERCENT UNLESS NOTED OTHERWISE IN THE PLANS.
5. ALL HORIZONTAL CONNECTIONS TO THE SANITARY OR STORM SEWERS SHALL BE OF THE 'WYE' BRANCH TYPE.

WATER

1. ALL WATER AND FIRE PROTECTION PIPE SHALL HAVE A MINIMUM 36-INCH COVER TO THE FINISH GRADE.
2. ALL WATER AND FIRE PRESSURE FITTINGS SHALL BE PROPERLY RESTRAINED WITH THRUST BLOCKS PER DETAIL.
3. ALL WATER MAIN / SANITARY SEWER CROSSINGS SHALL CONFORM TO THE OREGON STATE HEALTH DEPARTMENT REGULATIONS, CHAPTER 333.

EARTHWORKS

1. CONTRACTOR SHALL PREVENT SEDIMENTS AND SEDIMENT LADEN WATER FROM ENTERING THE STORM DRAINAGE SYSTEM.
2. TRENCH BEDDING AND BACKFILL SHALL BE AS SHOWN ON THE PIPE BEDDING AND BACKFILL DETAIL. THE PROJECT SPECIFICATIONS AND AS REQUIRED IN THE SOILS REPORT. FLOODING OR JETTING THE BACKFILLED TRENCHES WITH WATER WILL NOT BE PERMITTED.

PAVING

1. SEE LANDSCAPE PLANS FOR SIDEWALK FINISHING AND SCORING PATTERNS.

SEPARATION STATEMENT

ALL WATER MAIN CROSSINGS SHALL CONFORM TO THE OREGON STATE HEALTH DEPARTMENT, CHAPTER 333. WATER MAINS SHALL CROSS OVER SANITARY SEWERS WITH A 18" MINIMUM CLEARANCE BETWEEN OUTSIDE DIAMETERS OF PIPE WITH ALL PIPE JOINTS EQUIDISTANT FROM CROSSING. HORIZONTAL SEPARATION BETWEEN WATER MAINS AND SANITARY SEWERS IN PARALLEL INSTALLATIONS SHALL BE 10'. MAINTAIN 12" MINIMUM VERTICAL DISTANCE FOR ALL OTHER UTILITY CROSSINGS AND 12" HORIZONTAL PARALLEL DISTANCE. IN CASES WHERE IT IS NOT POSSIBLE TO MAINTAIN THE MINIMUM 10' HORIZONTAL SEPARATION, THE WATER MAIN SHALL BE LAID ON A SEPARATE SHELF IN THE TRENCH 18" INCHES ABOVE THE SEWER.

MATERIAL NOTES

1. GENERAL: MATERIALS SHALL BE NEW. THE USE OF MANUFACTURER'S NAMES, MODELS, AND NUMBERS IS INTENDED TO ESTABLISH STYLE, QUALITY, APPEARANCE, AND USEFULNESS. PROPOSED SUBSTITUTIONS WILL REQUIRE WRITTEN APPROVAL FROM ARCHITECT AND OWNER PRIOR TO INSTALLATION.
2. STORM AND SANITARY SEWER PIPING SHALL BE PVC PIPE, DUCTILE IRON PIPE, REINFORCED CONCRETE PIPE (RCP), OR HIGH DENSITY POLYETHYLENE (HDPE) PIPE CONFORMING TO THE PROJECT SPECIFICATIONS; AS INDICATED IN THE PLANS.
3. PRIVATE WATER MAINS 4-INCH DIAMETER AND LARGER SHALL BE DUCTILE IRON PIPE OR PVC PIPE CONFORMING TO THE PROJECT SPECIFICATIONS; AS INDICATED IN THE PLANS.
4. PRIVATE WATER LINES 3-INCH DIAMETER AND SMALLER SHALL BE TYPE K COPPER TUBING OR SCHEDULE 40 PVC CONFORMING TO THE PROJECT SPECIFICATIONS; AS INDICATED IN THE PLANS.
5. CONCRETE FOR CURBS, SIDEWALK AND DRIVEWAYS SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 3,500 PSI AT 28 DAYS.

ABBREVIATIONS

AC	ASPHALT CONCRETE	P/L	PROPERTY LINE
AD	AREA DRAIN	PC	POINT OF CURVATURE
APPROX	APPROXIMATE	PCC	POINT OF COMPOUND CURVATURE
B	BOLLARD	PCR	POINT OF CURB RETURN
BLDG	BUILDING	PED	PEDESTRIAN
BOW	BACK OF WALK	PM	PARKING METER
BS	BOTTOM OF SWALE	POC	POINT ON CURVE
	BOTTOM OF STAIR	PP	POWER POLE
BW	BOTTOM OF WALL	PRC	POINT OF REVERSE CURVATURE
CB	CATCH BASIN	PT	POINT OF TANGENT
CL	CENTERLINE	P.U.E	PUBLIC UTILITY EASEMENT
CO	CLEANOUT	PVC	POLYVINYL CHLORIDE
CONC.	CONCRETE	PVMT	PAVEMENT
COTG	CLEANOUT TO GRADE	PVT	PRIVATE
CP	CONTROL POINT	R	RIM
Δ	DELTA	RD	ROOF DRAIN
D/W	DRIVEWAY	R.O.W	RIGHT-OF-WAY
DIA.,Ø	DIAMETER	S	SLOPE (FT/FT)
DIP	DUCTILE IRON PIPE	SD	STORM DRAIN
E	EASTING	SDMH	STORM DRAIN MANHOLE
EXIST./EX	EXISTING	SHT	SHEET
FDC	FIRE DEPARTMENT CONNECTION	SS	SANITARY SEWER
FF	FINISH FLOOR ELEVATION	SSMH	SANITARY SEWER MANHOLE
FG	FINISH GRADE	ST	STREET
FH	FIRE HYDRANT	STA	STATION
FL	FLOWLINE	STD	STANDARD
FND	FOUNDATION	S/W	SIDEWALK
G	GUTTER	TC	TOP OF CURB
GB	GRADE BREAK	TD	TRENCH DRAIN
GL	GAS LINE	TG	TOP OF GROUND
GV	GATE VALVE	TP	TOP OF PAVEMENT
H	HEIGHT	TRANS.	TRANSFORMER
HCP	HANDICAP PARKING SPACE	TS	TOP OF STAIR
HP	HIGH POINT	TW	TOP OF WALL
IE	INVERT ELEVATION		TOP OF WALK
INV	INVERT	TYP	TYPICAL
IRR.	IRRIGATION	UG	UNDERGROUND
LP	LIGHT POLE	UGE	UNDERGROUND ELECTRIC
MH	MANHOLE	W	WATER
MIN	MINIMUM	W/	WITH
N	NORTHING	WCR	WHEEL CHAIR RAMP
OF	OUTFALL	WM	WATER METER
OVH/OH	OVERHEAD	WV	WATER VALVE

NOTICE TO EXCAVATORS:
ATTENTION: OREGON LAW REQUIRES YOU TO FOLLOW RULES ADOPTED BY THE OREGON UTILITY NOTIFICATION CENTER. THOSE RULES ARE SET FORTH IN OAR 952–001–0010 THROUGH OAR 952–001–0090. YOU MAY OBTAIN COPIES OF THE RULES BY CALLING THE CENTER.
(NOTE: THE TELEPHONE NUMBER FOR THE OREGON UTILITY NOTIFICATION CENTER IS (503)–232–1987).

POTENTIAL UNDERGROUND FACILITY OWNERS

Call before you dig.

Oregon811 UTILITY NOTIFICATION CENTER or 800-332-2344

EMERGENCY TELEPHONE NUMBERS

CENTURYLINK	1–800–573–1311
COMCAST	1–800–391–3000
NW NATURAL GAS	503–226–4211 x4313
PACIFICORP	877–508–5088
PBOT MAINTENANCE	503–823–1700
PGE	503–464–7777
PORTLAND WATER BUREAU	503–823–4874
VERIZON	1–800–VERIZON

MAP FROM: 2025 MICROSOFT© 2025 TOMTOM©

VICINITY MAP

SCALE: NTS

PROJECT CONTACTS

OWNER:
CITY OF WOODBURN
CITY OF WOODBURN 270 MONTGOMERY ST
WOODBURN, OREGON 97071
TEL: 503-XXX-XXXX
CONTACT: JESSE CUOMO

ARCHITECT:
OPSIS
975 SE MAIN ST.
PORTLAND, OREGON 97214
TEL: 646-306-4120
CONTACT: JUSTINE BANDA

CIVIL ENGINEER/PROJECT MANAGER:
KPFF CONSULTING ENGINEERS
111 SW FIFTH AVENUE, SUITE 2600
PORTLAND, OREGON 97204
TEL: 503-542-3840
CONTACT: JOSH LIGHTHIPE

LANDSCAPE ARCHITECT:
LANGO HANSEN LANDSCAPE ARCHITECTS
1100 NW GLISAN #3A,
PORTLAND, OREGON 97209
TEL: 971-380-3580
CONTACT: KYLE TRULEN

SHEET INDEX

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F: 503.274.4681
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PRELIMINARY NOT FOR CONSTRUCTION

Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

Project Adress:
190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No. Revision Date

Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
COVER SHEET

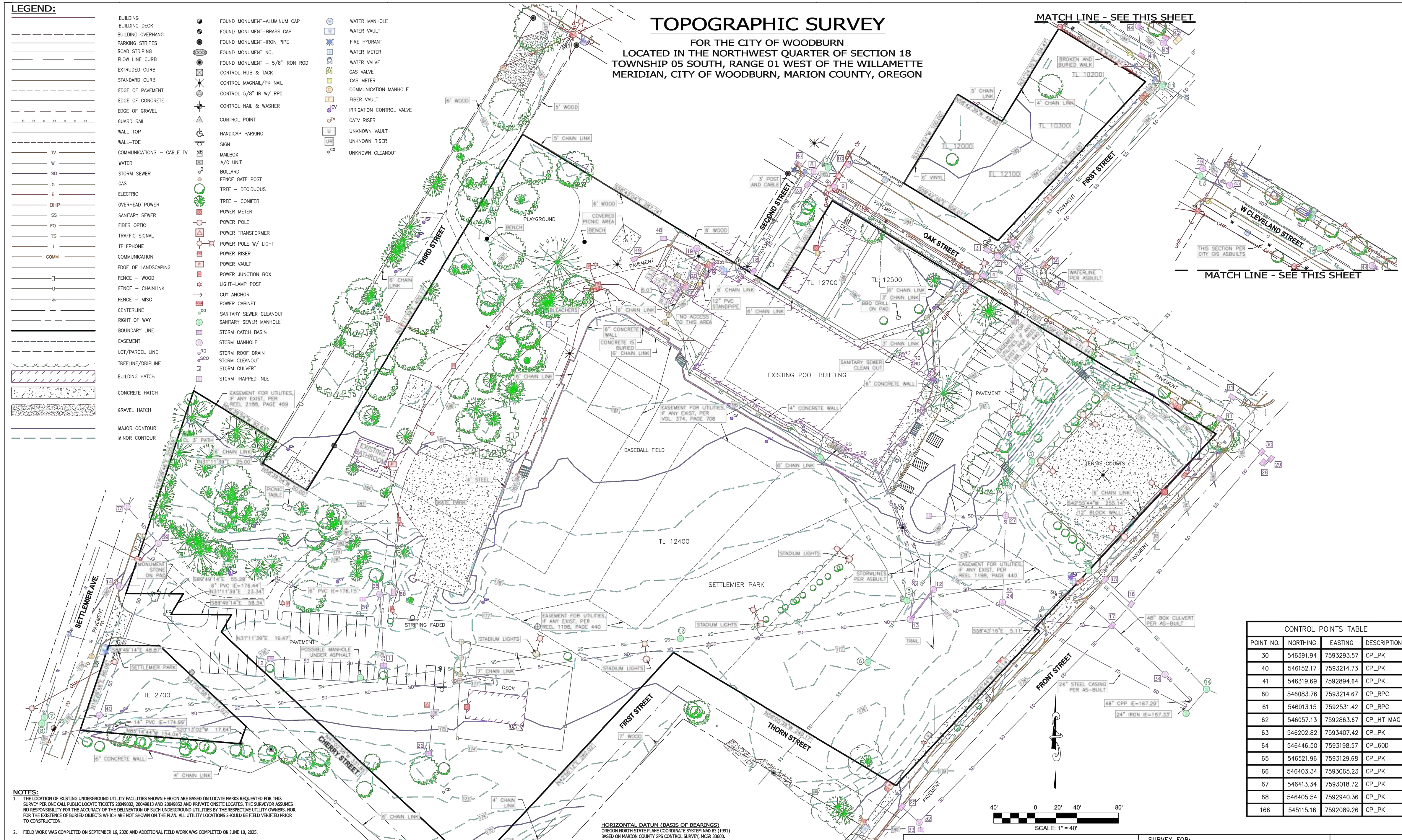
Sheet No.
C0.1

Job No.
4773-01

File: N:\cp\2019\1900192-woodburn-comm-ctr\CAD\PLOT\1900192-01-COVER.dwg TAB:C0.1
Plotted: 8/1/25 at 4:53pm By: JCollinson

TOPOGRAPHIC SURVEY

FOR THE CITY OF WOODBURN
LOCATED IN THE NORTHWEST QUARTER OF SECTION 18
TOWNSHIP 05 SOUTH, RANGE 01 WEST OF THE WILLAMETTE
MERIDIAN, CITY OF WOODBURN, MARION COUNTY, OREGON



SCALE: 1" = 40'

CONTROL POINTS TABLE			
POINT NO.	NORTHING	EASTING	DESCRIPTION
30	546391.94	7593293.57	CP_PK
40	546152.17	7593214.73	CP_PK
41	546319.69	7592894.64	CP_PK
60	546083.76	7593214.67	CP_RPC
61	546013.15	7592531.42	CP_RPC
62	546057.13	7592863.67	CP_HT MAG
63	546202.82	7593407.42	CP_PK
64	546446.50	7593198.57	CP_60D
65	546521.96	7593129.68	CP_PK
66	546403.34	7593065.23	CP_PK
67	546413.34	7593018.72	CP_PK
68	546405.54	7592940.36	CP_PK
166	545115.16	7592089.26	CP_PK

NOTES:

1. THE LOCATION OF EXISTING UNDERGROUND UTILITY FACILITIES SHOWN HEREON ARE BASED ON LOCATE MARKS REQUESTED FOR THIS SURVEY PER ONE CALL PUBLIC LOCATE TICKETS 20049802, 20049813 AND 20049852 AND PRIVATE ONSITE LOCATES. THE SURVEYOR ASSUMES NO RESPONSIBILITY FOR THE ACCURACY OF THE DELINEATION OF SUCH UNDERGROUND UTILITIES BY THE RESPECTIVE UTILITY OWNERS, NOR FOR THE EXISTENCE OF BURIED OBJECTS WHICH ARE NOT SHOWN ON THE PLAN. ALL UTILITY LOCATIONS SHOULD BE FIELD VERIFIED PRIOR TO CONSTRUCTION.

2. FIELD WORK WAS COMPLETED ON SEPTEMBER 16, 2020 AND ADDITIONAL FIELD WORK WAS COMPLETED ON JUNE 10, 2025.

3. THE LOCATION OF EXISTING UNDERGROUND UTILITY FACILITIES SHOWN HEREON ARE BASED ON THE FIELD MEASUREMENTS BETWEEN UTILITY STRUCTURES WITHIN THE PROJECT AREA. THE SURVEYOR ASSUMES NO RESPONSIBILITY FOR THE ACCURACY OF THE DELINEATION OF SUCH UNDERGROUND UTILITIES BY THE RESPECTIVE UTILITY OWNERS/BUILDERS, NOR FOR THE EXISTENCE OF BURIED OBJECTS WHICH ARE NOT SHOWN ON THE PLAN. DUE TO THE HAZARDOUS NATURE AND APPLICABLE OSHA REQUIREMENTS REGARDING CONFINED SPACES, IT IS S&F LAND SERVICES POLICY TO NOT SEND FIELD STAFF INTO UTILITY MANHOLES OR CONFINED SPACES TO RETRIEVE DEPTH AND SIZE INFORMATION. INFORMATION SHOWN HEREON IS SUBJECT TO AN UNCERTAINTY IN ACCURACY DEPENDING ON DEPTH, SIZE, FLOW, AND CONSTRUCTION OF MANHOLES. THE SURVEYOR HAS NOT PHYSICALLY LOCATED THE UNDERGROUND UTILITY LINES BETWEEN STRUCTURES

4. EASEMENTS SHOWN HEREON ARE PER AMERITITLE STATUS OF RECORD TITLE REPORT, TITLE NUMBER: 356016AM, EFFECTIVE FEBRUARY 28, 2020.

5. EXISTING TAX LOT LINES ARE NOT SHOWN HEREON, LINES ARE SHOWN WITH PRE-CONSTRUCTION RECORD OF SURVEY AND WITHIN THE PROVIDED DRAWING FILES.

HORIZONTAL DATUM (BASIS OF BEARINGS)

OREGON NORTH STATE PLANE COORDINATE SYSTEM NAD 83 (1991)
BASED ON MARION COUNTY GPS CONTROL SURVEY, MCSR 33600.
DISTANCES SHOWN HEREON ARE GROUND DISTANCES, INTERNATIONAL FEET, SCALED ABOUT CONTROL POINT NO 2. TO CONVERT TO GRID DISTANCES MULTIPLY BY THE COMBINED FACTOR OF 0.9998893832.

VERTICAL DATUM

NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
BASED ON NATIONAL GEODETIC SURVEY (NGS) BENCHMARK (BM) DESIGNATION T13, PID RD0246 WITH AN ELEVATION OF 187.52 FEET. BM SET AT THE INTERSECTION OF WEST LINCOLN AND NORTH FIRST STREETS, IN TOP OF THE SOUTHWEST END OF THE FOURTH STEP AT THE MAIN ENTRANCE OF AN ABANDONED BUILDING (FORMER CITY HALL BUILDING).

THE SHIFT FROM NAVD88 TO NGVD29 IS -3.37 FEET.

2004902–TOPO.dwg

S&F Land Services
PORTLAND, VANCOUVER, BEND, SEASIDE
4905 SW SCHOLLS FERRY RD.
PORTLAND, OR 97225
(503) 345–0328
WWW.SFLANDS.COM
EMAIL: INFO@SFLANDS.COM

DATE	JOB NO.	FIELD	DRAWN	CHECKED
JUNE 18, 2025	2020–049–02	AJ	TLB	MJF

SURVEY FOR:
WOODBURN COMMUNITY CENTER
CITY OF WOODBURN
LOCATED IN THE
NORTHWEST 1/4 OF S18,
T5S, R1W, OF THE W.M.
CITY OF WOODBURN, MARION COUNTY, OREGON
SHEET 1 OF 2

REGISTERED
PROFESSIONAL
LAND SURVEYOR
OREGON
JUNE 08, 2009
MATTHEW J. FAULKNER
75618LS
RENEWS: 12/31/25

opsis

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S&F LAND SERVICES 503.345.0328

Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

Project Adress:
190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No.	Revision	Date

Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
EXISTING CONDITIONS

Sheet No.
C0.2A

Job No.
4773-01

opsis

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TOPOGRAPHIC SURVEY

FOR THE CITY OF WOODBURN
LOCATED IN THE NORTHWEST QUARTER OF SECTION 18
TOWNSHIP 05 SOUTH, RANGE 01 WEST OF THE WILLAMETTE
MERIDIAN, CITY OF WOODBURN, MARION COUNTY, OREGON

VICINITY MAP
(NOT TO SCALE)

STORM DRAINAGE TABLE

No.	Description	Elevation
1	STM CB RIM:	175.61'
	IE 8" RCP IN (N):	172.48'
	IE 12" RCP IN (W):	172.49'
	IE 12" RCP OUT (S):	172.47'
2	STM CB RIM:	175.64'
	IE 12" RCP IN (N) STUB:	172.90'
	IE 12" RCP OUT (E):	172.67'
3	STM CB RIM:	183.12'
	IE 8" PVC OUT (SE):	179.97'
4	STM CB RIM:	182.81'
	IE 8" PVC OUT (NE):	180.01'
5	STM MH RIM:	183.10'
	IE 8" CPP IN (SW):	179.82'
	IE 8" PVC IN (NW):	
	IE 8" STEEL IN (NE):	
	IE 10" RCP OUT (SE):	
	CENTERLINE CHANNEL:	179.23'
6	STM MH RIM:	183.44'
	IE 10" RCP IN (NW):	179.04'
	IE 8" PVC IN (E):	178.23'
	IE 8" PVC IN (SE):	178.23'
	IE 18" RCP IN (NE):	177.01'
	IE 18" RCP OUT (S):	176.96'
7	STM CB RIM:	182.68'
	IE 8" STEEL OUT (SW):	179.44'
8	STM MH RIM:	185.31'
	IE 8" PVC IN (NE):	180.99'
	IE 8" PVC IN (SE):	180.75'
	IE 8" PVC IN (SW):	180.82'
	IE 8" PVC OUT (NW):	180.75'
9	STM CB RIM:	184.27'
	IE 8" PVC IN (NE):	181.66'
	IE 8" PVC OUT (NW):	180.65'
10	STM CB RIM:	184.37'
	IE 8" PVC OUT (NE):	181.85'
11	STM CB RIM:	181.03'
	IE 12" CONC IN (NE):	178.53'
	IE 12" CONC OUT (SE):	178.03'
12	STM MH RIM:	176.94'
	SUMP:	166.79'
	(STANDING WATER)	
13	STM MH RIM:	176.45'
	SUMP:	167.45'
	(STANDING WATER)	
14	STM CB RIM:	179.78'
	IE 8" PVC OUT (E):	176.87'
15	STM CB RIM:	177.06'
	IE 12" CPP OUT (SE):	175.36'
16	STM CB RIM:	177.04'
	IE 12" CPP OUT (NW):	174.24'
17	STM CB RIM:	177.23'
	IE 12" PVC IN (SW):	172.17'
	SUMP:	167.93'
	(CAN'T SEE LOWER PIPES)	
18	STM CB RIM:	185.66'
	IE 8" PVC IN (NW):	181.79'
	IE 8" PVC OUT (NE):	181.46'
19	STM MH RIM:	186.29'
	IE 6" PVC IN (SW):	182.53'
	IE 8" PVC IN (NW):	182.53'
	IE 8" PVC OUT (SE):	182.53'
20	STM CB RIM:	185.67'
	IE 6" PVC OUT (NW):	182.73'
21	STM CB RIM:	185.67'
	IE 6" PVC OUT (NE):	182.60'
22	STM CB RIM:	172.02'
	IE 8" STEEL OUT (N):	170.61'
23	STM CB RIM:	185.10'
	IE 8" PVC IN (SW):	180.89'
	IE 8" PVC OUT (NE):	180.89'
	SUMP:	179.55'
24	STM MH RIM:	177.70'
	IE 18" CONC (N):	168.41'
	(CAN'T SEE PIPES DUE TO OFFSET)	
25	STM CB RIM:	179.43'
	IE 10" CPP OUT (E):	176.85'
26	STM MH RIM:	181.34'
	IE 10" CONC IN (SW):	172.91'
	IE 10" IRON OUT (SE):	172.87'
27	STM MH RIM:	178.69'
	IE 18" CONC IN (N):	169.46'
	IE 18" CONC OUT (S):	169.46'
	(LOOKS ABANDONED)	
28	STM CB RIM:	180.90'
	IE 8" CPP OUT (NE):	178.00'
29	STM CB (ROUND) RIM:	181.09'
	IE 8" CPP IN (SW):	177.71'
	IE (NE):	CAPPED
	IE 12" PVC IN (NW):	175.78'
	IE 12" CONC OUT (SW):	175.53'
30	STM CB (ROUND) RIM:	181.43'
	IE 12" CONC (NE):	176.22'
	IE 12" CONC IN (NW):	176.05'
	IE 12" CONC OUT (SW):	176.02'
31	STM CB RIM:	181.50'
	UNABLE TO MEASURE	
	DUE TO PARKED CAR	
32	STM MH RIM:	179.64'
	IE 10" CPP (SE):	173.88'
	IE 10" CPP IN (NW):	173.83'
	IE 10" CPP OUT (NE):	173.63'
33	STM CB RIM:	179.40'
	IE 10" CPP OUT (NW):	176.00'
34	STM MH RIM:	176.32'
	IE 48" CONC IN (NW):	167.67'
	IE 48" CPP OUT (SE):	167.67'
35	STM CB RIM:	182.62'
	IE 8" PVC IN (NE):	178.62'
	IE 8" PVC OUT (NW):	178.62'
	SUMP:	177.12'
36	STM CB RIM:	182.70'
	IE 8" PVC OUT (W):	179.88'
	SUMP:	179.35'
37	STM MH RIM:	181.64'
	IE 10" RCP (NE):	178.38'
	IE 10" RCP IN (NW):	178.54'
	IE 10" RCP OUT (SE):	178.34'
38	STM MH RIM:	177.12'
	IE 4" IRON (W):	173.16'
	IE 6" IRON IN (E):	172.57'
	IE 10" RCP OUT (S):	170.66'
39	STM MH RIM:	179.96'
	IE 12" CONC IN (W):	176.82'
	IE 12" CONC OUT (S):	176.77'
40	STM MH RIM:	177.87'
	IE 12" PVC IN (W):	174.99'
	IE 12" PVC IN (N):	172.87'
	IE 12" PVC OUT (S):	172.74'
41	STM CB RIM:	183.67'
	IE 8" PVC OUT (N):	
	SUMP:	179.17'
42	STM CB RIM:	183.56'
	IE 10" PVC OUT (SW):	180.31
	SUMP:	179.06'
43	STM MH RIM:	183.87'
	IE 10" PVC IN (NE):	179.57'
	IE 10" PVC IN (NW):	178.05'
	IE 10" PVC OUT (SE):	178.05'
44	STM MH RIM:	184.09'
	IE 8" PVC IN (S):	178.01'
	IE 8" PVC IN (NE):	178.29'
	IE 10" PVC IN (NW):	177.95'
	IE 10" PVC OUT (SE):	177.95'
	(REVERSE FLOW)	
45	STM MH RIM:	184.82'
	IE 8" PVC IN (NE):	178.58'
	IE 10" PVC IN (NW):	178.45'
	IE 10" PVC OUT (SE):	178.41'
46	STM MH RIM:	185.19'
	IE 10" RCP IN (NE):	178.79'
	IE RCP" PVC IN (W):	181.05'
	IE 10" PVC OUT (SE):	178.72'
47	STM CB RIM:	185.06'
	IE 10" PVC OUT (SE):	181.06'
	SUMP:	179.56'
48	STM CB RIM:	186.02'
	IE 10" PVC IN (SW):	183.02'
	IE 10" PVC OUT (SE):	183.02'
49	STM CB RIM:	185.95'
	IE 10" PVC IN (SW):	183.55'
	IE 10" PVC OUT (NE):	183.55'
50	STM CB RIM:	176.10'
	(FULL OF DEBRIS)	
51	STM CB RIM:	175.60'
	IE 4" PVC OUT (NE):	173.88'

SANITARY SEWER TABLE

No.	Description	Elevation
1	SAN MH RIM:	182.44'
	IE 10" RCP IN (NW):	172.38'
	IE 10" RCP IN (NE):	172.34'
	IE 10" RCP OUT (SW):	172.30'
2	SAN MH RIM:	185.18'
	IE 10" RCP IN (NW):	173.68'
	IE 10" RCP IN (NE):	173.69'
	IE 10" RCP OUT (SE):	173.62'
3	SAN MH RIM:	176.83'
	IE 15" PVC IN (NE):	171.30'
	IE 15" PVC OUT (SW):	170.94'
4	SAN MH RIM:	183.76'
5	SAN MH RIM:	177.37'
	IE 12" RCP IN (SW):	164.84'
	IE 18" RCP IN (W):	163.72'
	IE 8" RCP IN (NW):	173.01'
	IE 15" PVC IN (NE):	170.13'
	IE 18" RCP OUT (E):	163.77'
6	SAN MH RIM:	178.15'
	IE 8" RCP IN (SW):	168.31'
	IE 8" RCP OUT (NE):	168.28'
7	SAN MH RIM:	177.92'
	IE 10" RCP IN (N):	169.66'
	IE 10" RCP OUT (S):	169.22'
8	SAN MH RIM:	177.88'
	IE 10" RCP IN (N):	168.84'
	IE 12" RCP IN (N):	166.05'
	IE 12" RCP OUT (S):	166.00'
10	SAN MH RIM:	176.76'
	IE 15" CONC IN (W):	163.26'
	IE 15" CONC OUT (SE):	163.06'
11	SAN MH RIM:	183.86'
	IE 8" PVC IN (SW):	174.72'
	IE 10" PVC IN (NW):	174.62'
	IE 8" PVC IN (NE):	174.35'
	IE 12" PVC OUT (SE):	174.32'
12	SAN MH RIM:	176.43'
	IE 18" CONC IN (E):	164.12'
	IE 18" CONC OUT (W):	164.14'
14	SAN MH RIM:	175.46'
	IE 8" CONC IN (SW):	164.16'
	IE 15" CONC IN (NW):	162.82'
	IE 15" CONC OUT (SE):	162.82'
15	SAN MH RIM:	183.19'
	IE 8" RCP IN (NW):	172.23'
	IE 8" RCP IN (SW):	172.13'
	IE 8" RCP OUT (NE):	172.05'
16	SAN MH RIM:	184.52'
	IE 8" RCP IN (NW):	176.17'
	IE 10" RCP OUT (SE):	175.64'
17	SAN MH RIM:	185.18'
	IE 8" RCP IN (NW):	176.29'
	IE 8" RCP OUT (SE):	176.21'

S&F Land Services
PORTLAND, VANCOUVER, BEND, SEASIDE
4905 SW SCHOLLS FERRY RD.
PORTLAND, OR 97225
(503) 345-0328
WWW.SFLANDS.COM EMAIL: INFO@SFLANDS.COM

2004902-TOPO.dwg

DATE	JOB NO.	FIELD	DRAWN	CHECKED
JUNE 18, 2025	2020-049-02	AJ	TLB	MJF

SURVEY FOR:
WOODBURN COMMUNITY CENTER
CITY OF WOODBURN
LOCATED IN THE
NORTHWEST 1/4 OF S18,
T5S, R1W, OF THE W.M.
CITY OF WOODBURN, MARION COUNTY, OREGON
SHEET 2 OF 2

REGISTERED
PROFESSIONAL
LAND SURVEYOR
OREGON
JUNE 08, 2009
MATTHEW J. FAULKNER
75618LS
RENEWS: 12/31/25

File: N:\c\p\2019\1900192-woodburn-comm-ctr\CAD\PLOT\1900192-02-EXC.dwg TAB:C0.2B
Plotted: 8/1/25 at 4:54pm By: JCollinson

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Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

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190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No.	Revision	Date

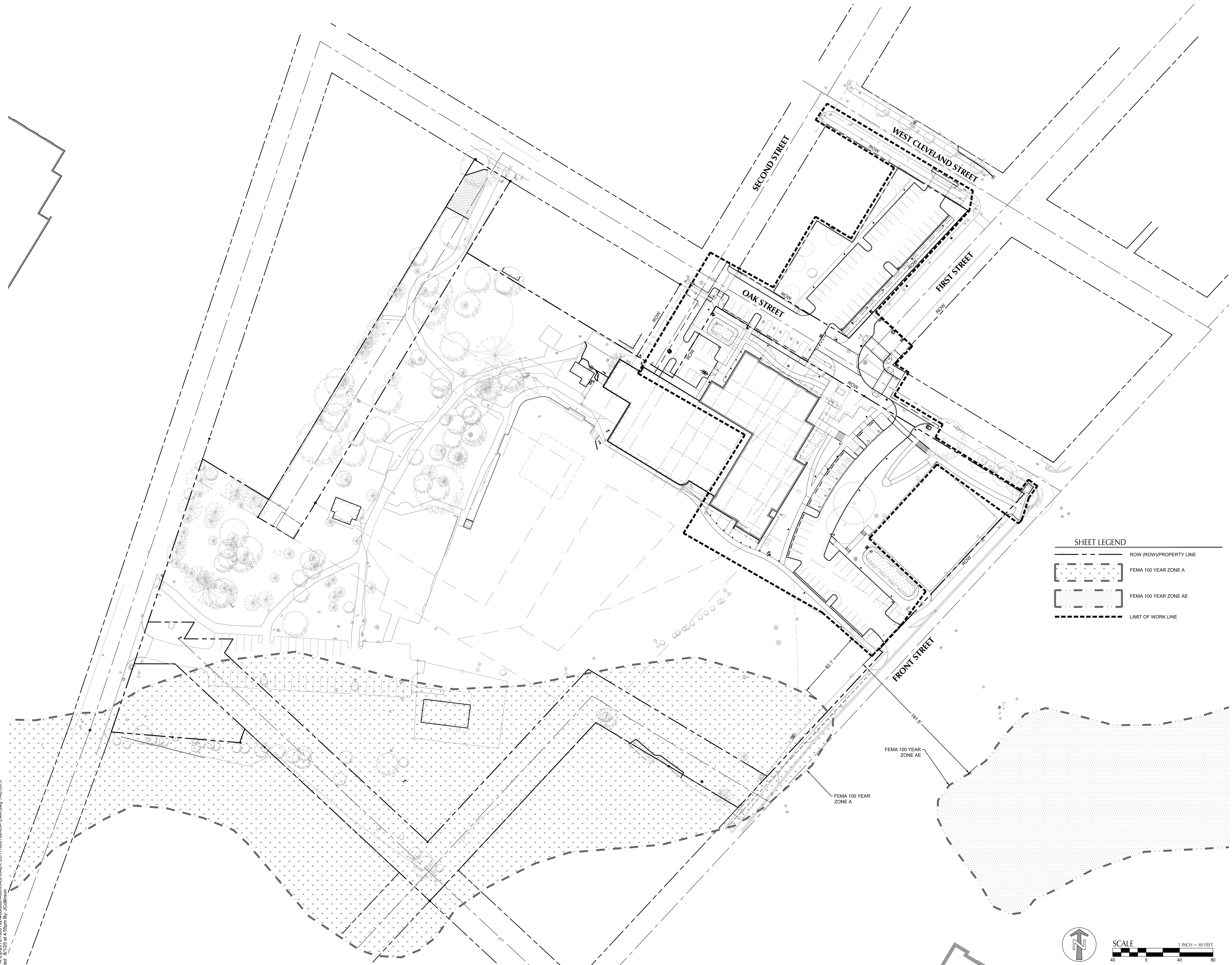
Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
EXISTING CONDITIONS

Sheet No.
C0.2B

Job No.
4773-01



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Key Plan

Revisions to Sheet

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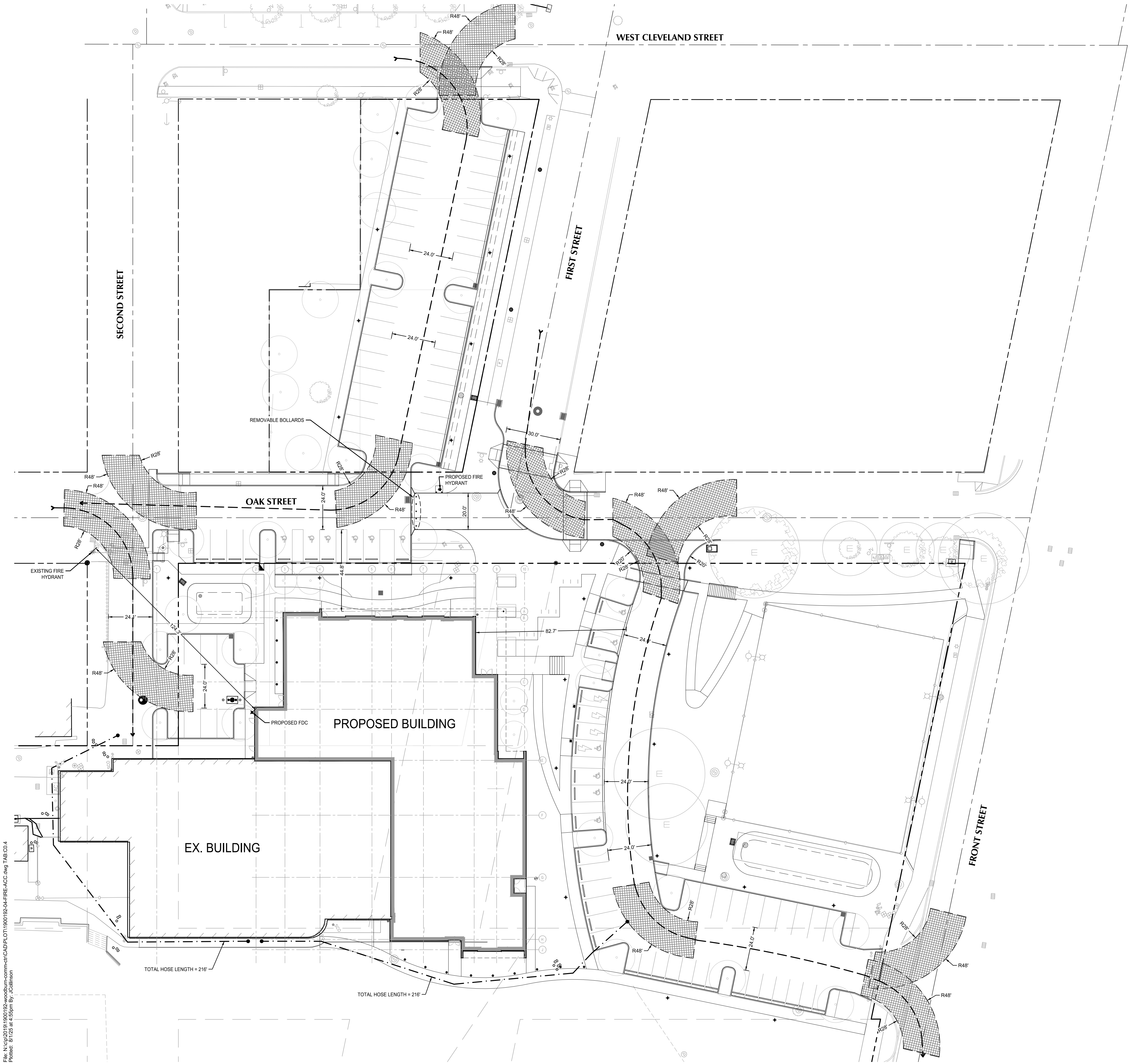
Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
FEMA OVERLAY MAP

Sheet No.
C0.3

Job No.
4773-01



SHEET NOTES

FIRE APPARATUS ACCESS ROADS SHALL BE IN ACCORDANCE WITH THE 2022 OREGON FIRE CODE AND ALL OTHER APPLICABLE REQUIREMENTS OF THE INTERNATIONAL FIRE CODE.

1. FLOW TEST RESULTS

   104 HYDRANT TESTED ON 5/08/2025

   STATIC: 56 PSIG
   RESIDUAL: 47 PSIG
   @ 20 PSI: 3769 GPM

3. CURBS IN FIRE ACCESS ZONES SHALL BE PAINTED RED WITH WHITE "NO PARKING FIRE LANE" STENCIL AT A MINIMUM OF 20' ON CENTER. TEXT SHALL BE 1 INCH WIDE BY 6 INCHES HIGH PER OFC 503.3.

UTILITY LABEL LEGEND

STRUCTURE TYPE

CALLOUT	DESCRIPTION	DETAIL REF.
FDC	FIRE DEPARTMENT CONNECTION	
FH	FIRE HYDRANT (PRIVATE OR PUBLIC AS NOTED)	

SHEET LEGEND

FIRE APPARATUS PATH OF TRAVEL

HOSE PULL, LENGTH AS NOTED.

28' RADIUS INSIDE AND 48' RADIUS OUTSIDE ON FIRE ACCESS ROAD

SCALE 1 INCH = 20 FEET
20 0 20 40

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Revisions to Sheet

No.	Revision	Date

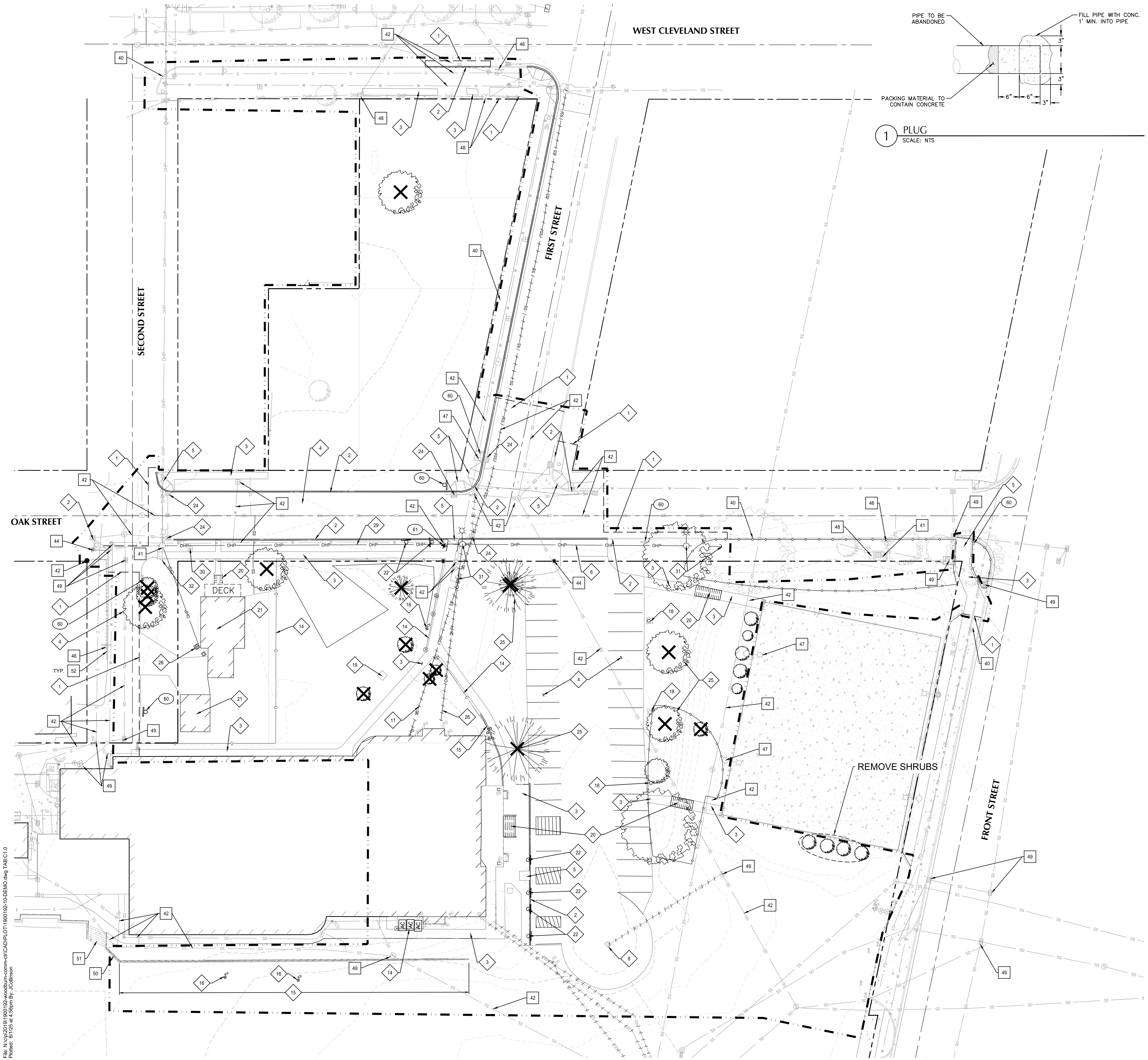
Status: RCWOD

Date: 08.08.2025

Sheet Title
FIRE ACCESS PLAN

Sheet No.
C0.4

Job No.
4773-01



1 PLUG
SCALE: NTS

SHEET NOTES

1. CONTRACTOR MAY STAGE WITHIN LIMITS OF DEMOLITION.
2. REMOVE ALL SITE COMPONENTS AND RECYCLE COMPONENTS AS REQUIRED IN THE SPECIFICATIONS.
3. GENERAL DEMOLITION PERMIT SHALL BE SECURED BY THE CONTRACTOR.
4. ALL TRADE LICENSES AND PERMITS NECESSARY FOR THE PROCUREMENT AND COMPLETION OF THE WORK SHALL BE SECURED BY THE CONTRACTOR PRIOR TO COMMENCING DEMOLITION.
5. THE CONTRACTOR SHALL PRESERVE AND PROTECT FROM DAMAGE ALL EXISTING RIGHT-OF-WAY SURVEY MONUMENTATION DURING DEMOLITION. THE CONTRACTOR IS RESPONSIBLE FOR COORDINATING AND PAYING FOR THE REPLACEMENT BY A LICENSED SURVEYOR OF ANY DAMAGED OR REMOVED MONUMENTS.
6. PROTECT ALL ITEMS ON ADJACENT PROPERTIES AND IN THE RIGHT OF WAY INCLUDING BUT NOT LIMITED TO SIGNAL EQUIPMENT, PARKING METERS, SIDEWALKS, STREET TREES, STREET LIGHTS, CURBS, PAVEMENT AND SIGNS. CONTRACTOR SHALL BE RESPONSIBLE FOR RESTORING ANY DAMAGED ITEMS TO ORIGINAL CONDITION.
7. PROTECT STRUCTURES, UTILITIES, SIDEWALKS, AND OTHER FACILITIES IMMEDIATELY ADJACENT TO EXCAVATIONS FROM DAMAGES CAUSED BY SETTLEMENT, LATERAL MOVEMENT, UNDERMINING, WASHOUT AND OTHER HAZARDS.
8. SAWCUT STRAIGHT LINES IN SIDEWALK, CURB, AND PAVEMENT, AS NECESSARY.
9. CONTRACTOR IS RESPONSIBLE TO CONTROL DUST AND MUD DURING THE DEMOLITION PERIOD, AND DURING TRANSPORTATION OF DEMOLITION DEBRIS. ALL STREET SURFACES OUTSIDE THE CONSTRUCTION ZONE MUST BE KEPT CLEAN.
10. ALL EXPOSED PORTIONS OF UNDERGROUND UTILITIES TO BE ABANDONED SHALL BE PLUGGED PER DETAIL 1/########.
11. ALL TREE REMOVAL, SALVAGE, AND PROTECTION TO BE FOUND ON LANDSCAPE ARCHITECT PLANS.

X DEMOLITION KEY NOTES

1 SAWCUT LINE
2 REMOVE CONCRETE CURB.
3 REMOVE CONCRETE SIDEWALK
4 REMOVE ASPHALT PAVEMENT AND CRUSHED ROCK SUBGRADE.
5 REMOVE CONCRETE ADA RAMP.
6 REMOVE CONCRETE DRIVEWAY.
7 REMOVE TENNIS COURT CONCRETE SLAB.
8 REMOVE AREA DRAIN
9 REMOVE STORM SEWER LINE
10 REMOVE STORM SEWER MANHOLE
11 REMOVE SANITARY SEWER LINE (UNDER PROPOSED BUILDING)
12 REMOVE SANITARY SEWER MANHOLE
13 REMOVE BLOCK WALL
14 REMOVE CHAIN LINK FENCE
15 REMOVE CONCRETE RETAINING WALL
16 REMOVE IRRIGATION PIPING AND CONTROL VALVES
17 REMOVE OVERHEAD STADIUM LIGHTS AND FOUNDATION
18 REMOVE LIGHTING STRUCTURE AND FOUNDATION
19 REMOVE RECREATIONAL FACILITY AND CONCRETE PAD
20 REMOVE STAIRS
21 REMOVE HOUSE AND FOUNDATION
22 REMOVE SIGN
23 REMOVE BOLLARD
24 REMOVE STORM INLET
25 REMOVE EXISTING TREE PER LANDSCAPE PLAN
26 REMOVE EXISTING ELECTRIC FACILITIES
27 REMOVE EXISTING GAS FACILITIES
28 REMOVE EXISTING CONCRETE STRUCTURE
29 REMOVE OVERHEAD UTILITIES
30 REMOVE EXISTING WATER METER
31 REMOVE POLE AND GUY WIRE
32 RELOCATE GUY WIRE

X PROTECTION KEY NOTES

40 PROTECT CURB AND SIDEWALK.
41 PROTECT ELECTRIC FACILITY.
42 PROTECT UNDERGROUND UTILITIES.
43 PROTECT EXISTING BUILDING.
44 PROTECT MONUMENTATION.
45 PROTECT EXISTING FENCE.
46 PROTECT EXISTING STREET SIGN.
47 PROTECT EXISTING LIGHT AND POLE.
48 PROTECT EXISTING OVERHEAD ELECTRIC POLE.
49 PROTECT EXISTING UTILITY STRUCTURE.
50 PROTECT RETAINING WALL.
51 PROTECT STAIRS.
52 PROTECT BOLLARDS

X SALVAGE KEY NOTES

60 SALVAGE SIGN AND RETURN TO CITY AT DESIGNATED LOCATION.
61 SALVAGE & RELOCATE EXISTING HYDRANT

SHEET LEGEND

PROPERTY LINE
DEMOLITION/WORK LIMITS (SHOWN OFFSET FOR CLARITY)
SAWCUT LINE
REMOVE OR ABANDON UTILITY LINE IN PLACE
TREE PROTECTION
REMOVE TREE

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Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

Project Adress:
190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No.	Revision	Date

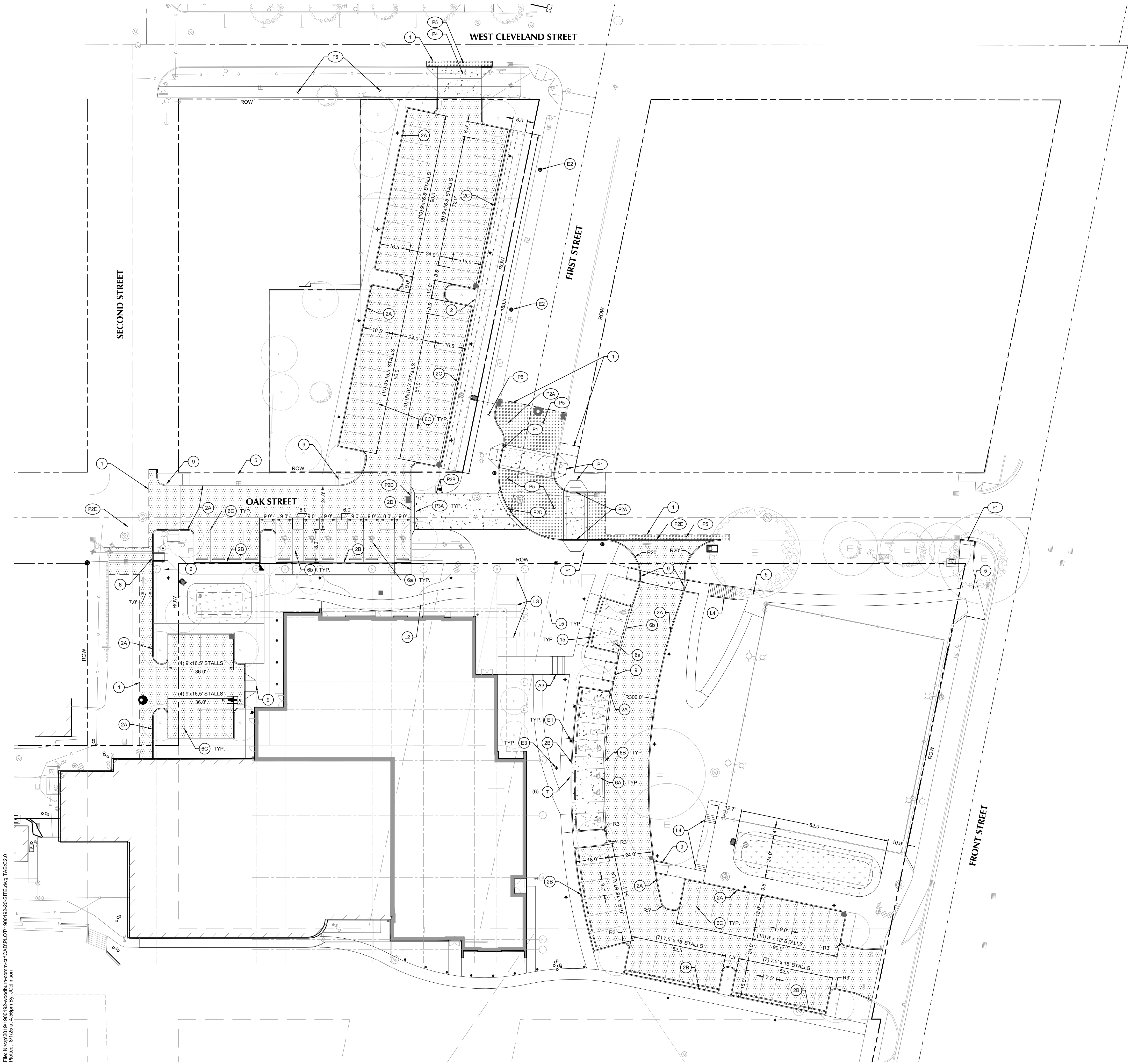
Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
DEMOLITION PLAN

Sheet No.
C1.0

Job No.
4773-01



SHEET NOTES

1. ALL DIMENSIONS ARE TO FACE OF CURB OR FACE OF WALL.
2. ALL SIDEWALK PAVEMENT JOINTS SHALL BE CONSTRUCTED PER DETAIL 15/C5.0.
3. PROPOSED FRONTAGE IMPROVEMENTS IN ROW SHALL BE CONSTRUCTED PER CITY OF WOODBURN AND/OR ODOT CONSTRUCTION STANDARDS AND DETAILS.
4. SLOPES PROVIDED ON SLOPE ARROW ARE FOR REFERENCE ONLY.
5. LANDINGS ON ACCESSIBLE ROUTES SHALL NOT EXCEED 2% IN ANY DIRECTION.
6. ALL ACCESSIBLE ROUTES SHALL COMPLY WITH CURRENT ADA ACCESSIBILITY GUIDELINES FOR BUILDING AND FACILITIES (ADAAG).

(X) KEY NOTES

#	DESCRIPTION	DETAIL REF.
1	SAWCUT	
2A	CONCRETE CURB	2a/C5.0
2B	CONCRETE CURB AND GUTTER	2b/C5.0
2C	THICKENED CURB AND GUTTER	2c/C5.0
3A	REMOVABLE BOLLARD	
3B	STANDARD BOLLARD	
5	SIDEWALK	1c/C5.0
6a	ADA PARKING STALLS AND STRIPING	2/C5.0
6b	'NO PARKING" ZONE STRIPING	3/C5.0
6c	4" WIDE WHITE STRIPE	
7	ADA PARKING SIGN	4/C5.0
8	DETECTABLE WARNING	8/C5.0
9	CURB RAMP	10/C5.0

(X) ELECTRICAL

(SHOWN FOR REFERENCE, SEE ELECTRICAL PLANS)

#	DESCRIPTION	DETAIL REF.
E1	ELECTRICAL CHARGER	
E2	STREET LIGHT	
E3	ON-SITE LIGHTING	

(X) LANDSCAPE

(SHOWN FOR REFERENCE, SEE LANDSCAPE PLANS)

#	DESCRIPTION	DETAIL REF.
L1	CONCRETE WALL	
L2	CONCRETE WALKWAY/PLAZA	
L3	SEAT WALL	
L4	STAIRS AND HANDRAIL	
L5	BIKE RACKS	

(X) PUBLIC KEY NOTES

(SEE SHEET NOTE 3)

#	DESCRIPTION	DETAIL REF. (ODOT OR COW)
P1	PERPENDICULAR CURB RAMP	RD910
P2A	TYPE ' C' CURB	4100-2
P2B	FLUSH CURB	
P2C	TYPE 'A' CURB & GUTTER	4100-2
P2D	MOUNTABLE CURB	RD700
P2E	TYPE 'A' CURB AT DRIVEWAY	4100-4
P3A	REMOVABLE BOLLARD	RD130
P3B	STANDARD BOLLARD	RD130
P4	DRIVEWAY APPROACH	4150-1
P5	SURFACE FULL DEPTH REPAIR	4220-1
P6	SIDEWALKS	4150-8

SHEET LEGEND

- PROPERTY LINE
- CONCRETE PAVEMENT
  - DRIVE AISLES — 1C/C5.1
  - PARKING STALLS — 1D/C5.1
- ASPHALT PAVEMENT
  - DRIVE AISLES — 1B/C5.1
  - PARKING STALLS — 1B/C5.1
- STORMWATER BASIN — 5/C5.1

SCALE 1 INCH = 20 FEET
20 0 20 40

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Key Plan

Revisions to Sheet

No.	Revision	Date

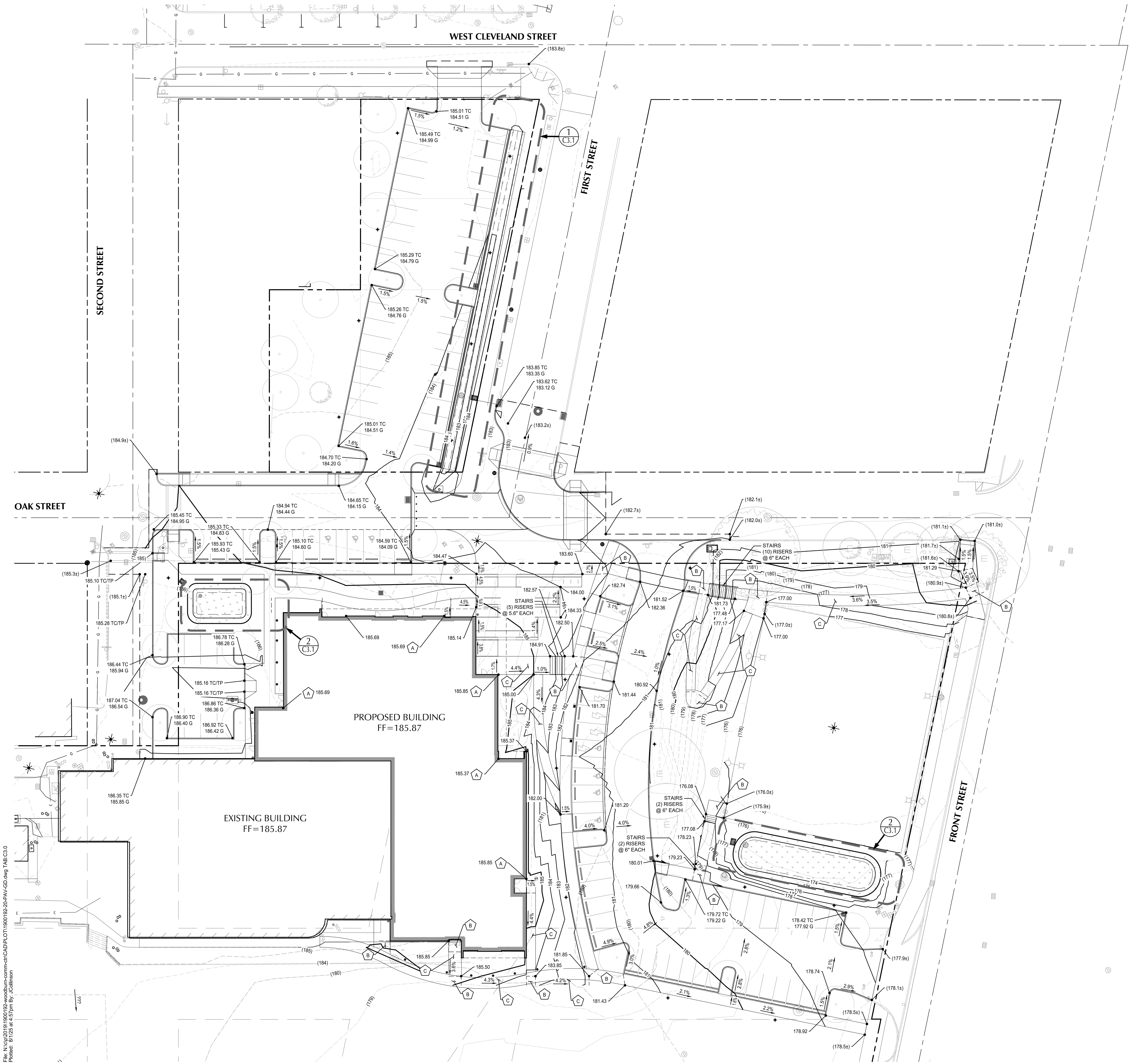
Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
SITE PLAN

Sheet No.
C2.0

Job No.
4773-01



SHEET NOTES

1. ALL DIMENSIONS ARE TO FACE OF CURB OR FACE OF WALL.
2. ALL SIDEWALK PAVEMENT JOINTS SHALL BE CONSTRUCTED PER DETAIL 15/C5.0.
3. PROPOSED FRONTAGE IMPROVEMENTS IN RIGHT-OF-WAY SHOWN FOR REFERENCE ONLY. TO BE PERMITTED UNDER SEPARATE PUBLIC WORKS PERMIT.
4. SLOPES PROVIDED ON SLOPE ARROW ARE FOR REFERENCE ONLY.
2. CROSS-SLOPES ON ACCESSIBLE ROUTES ARE DESIGNED AT 1.5% MAX. AND SHALL COMPLY WITH CURRENT ADA ACCESSIBILITY GUIDELINES FOR BUILDING AND FACILITIES (ADAAG)
3. WALKWAYS ARE DESIGNED TO NOT REQUIRE HANDRAILS, UNLESS NOTED OTHERWISE. THEREFORE WALKWAYS WITH SLOPES STEEPER THAN 4.5% AND LESS THAN 7.8% SHALL NOT EXCEED 0.5' RISE.

X GRADING KEY NOTES

NOTE	DESCRIPTION
A	LANDING AT DOOR. TYPICAL GRADING SHALL BE: TOP OF CONCRETE DOOR = FFE MINUS 0.02FT. SLOPE CONCRETE 1.5% MAX. (UNO) AWAY FROM BLDG.
B	LANDING ZONE. ACCESSIBLE ROUTE THROUGH THIS ZONE REQUIRES NO MORE THAN 2% SLOPE IN ANY DIRECTION.
C	SLOPING WALKWAY DESIGNED TO BE ACCESSIBLE WITH RUNNING SLOPE < 5% (DESIGN INTENT 4.5% MAX.) AND CROSS-SLOPE < 2% (DESIGN INTENT 1.5% MAX).
D	WALKWAY RAMP DESIGNED TO BE ACCESSIBLE WITHOUT HANDRAILS. RISE SHALL BE < 0.5', RUNNING SLOPE < 8% (DESIGN INTENT 7.8% MAX.) AND CROSS-SLOPE < 2% (DESIGN INTENT 1.5% MAX).
E	TOP OF GROUND ALONG BUILDING SHALL BE 6" BELOW FINISHED FLOOR AND SHALL SLOPE AWAY AT 2% MIN FOR 3' MIN. , UNLESS NOTED OTHERWISE.
F	WALKWAY RAMP WITH HANDRAILS EACH SIDE. RISE BETWEEN LANDINGS SHALL BE 2.5' MAX, RUNNING SLOPE <8% (DESIGN INTENT 7.8% MAX.) AND CROSS-SLOPE <2% (DESIGN INTENT 1.5% MAX.)

GRADING LABEL LEGEND

CALLOUT	DESCRIPTION
X.X%	GRADING SLOPE AND DIRECTION (DOWNHILL)
XX.XX	SPOT ELEVATION
XX	DESCRIPTION LISTED BELOW. NO DESCRIPTION MEANS TP OR TG
BOS	BOTTOM OF SWALE
BOW	BACK OF WALK
BS	BOTTOM OF STEP
BW	BOTTOM OF WALL
EG	EXISTING GRADE
FF	FINISHED FLOOR
FL	FLOW LINE
G	GUTTER
HP	HIGH POINT
LP	LOW POINT
RIM	RIM OF STRUCTURE
TC	TOP OF CURB
TG	TOP OF GROUND
TP	TOP OF PAVEMENT
TS	TOP OF STEP
TW	TOP WALL
(XXX.X±)	EXISTING GRADE (MATCH WHERE APPLICABLE)

SHEET LEGEND

	PROPERTY LINE
	GRADE BREAK
(49)	EX. CONTOUR MINOR
(50)	EX. CONTOUR MAJOR
49	CONTOUR MINOR (FG)
50	CONTOUR MAJOR (FG)

SCALE 1 INCH = 20 FEET
20 0 20 40

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Key Plan

Revisions to Sheet

No.	Revision	Date

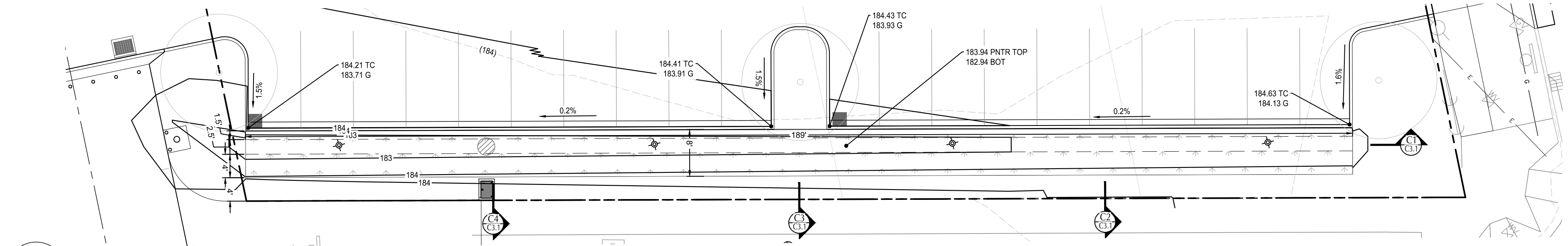
Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
GRADING PLAN

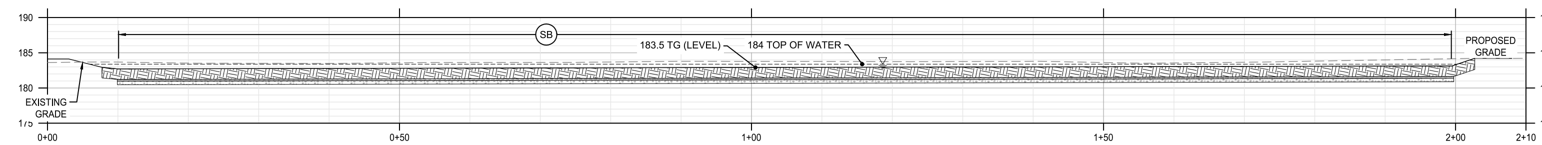
Sheet No.
C3.0

Job No.
4773-01



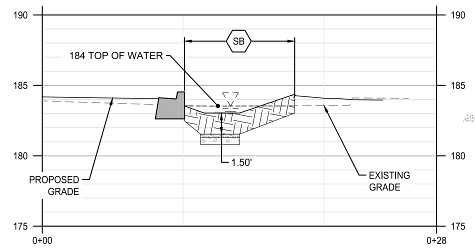
ENLARGEMENT 1 - STORM BASIN "C"

SCALE: 1" = 10'



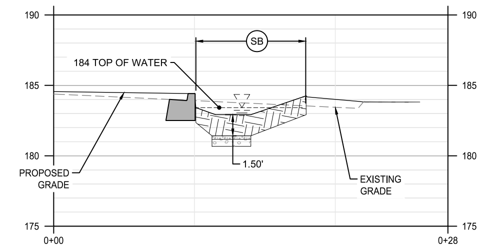
C1 SECTION

SCALE: HORZ: 1" =10'
VERT: 1" = 10'



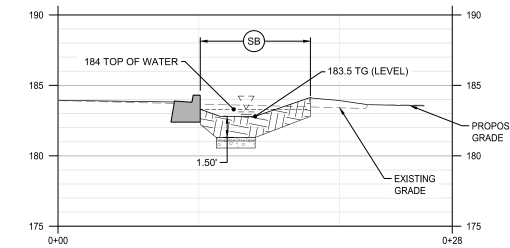
C2 SECTION

SCALE: HORZ: 1" =10'
VERT: 1" = 10'



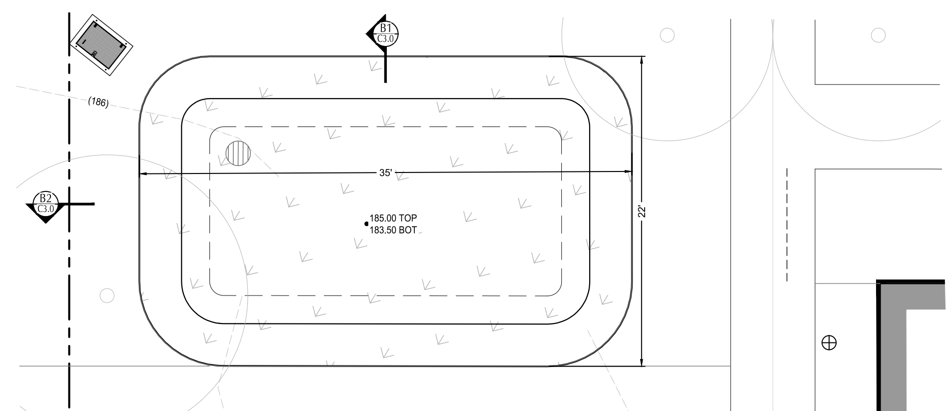
C3 SECTION

SCALE: HORZ: 1" =10'
VERT: 1" = 10'



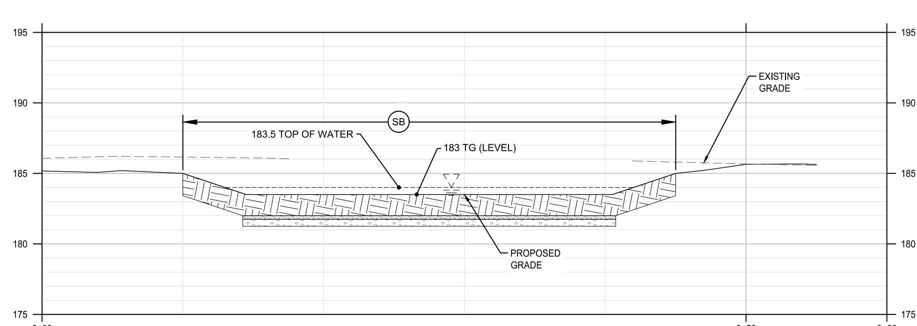
C4 SECTION

SCALE: HORZ: 1" =10'
VERT: 1" = 10'



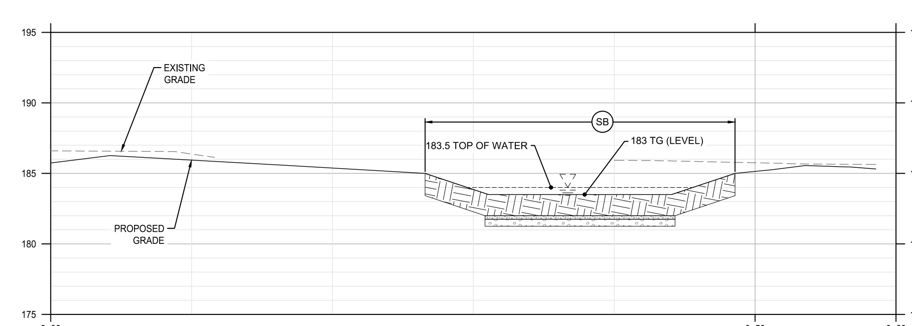
ENLARGEMENT 2 - STORM BAIN "B"

SCALE: 1" = 5'



B1 SECTION

SCALE: HORZ: 1" =10'
VERT: 1" = 10'



B2 SECTION

SCALE: HORZ: 1" =10'
VERT: 1" = 10'

SHEET NOTES

1. ALL DIMENSIONS ARE TO FACE OF CURB OR FACE OF WALL.
2. ALL SIDEWALK PAVEMENT JOINTS SHALL BE CONSTRUCTED PER DETAIL 15/C5.0.
3. PROPOSED FRONTAGE IMPROVEMENTS IN RIGHT-OF-WAY SHOWN FOR REFERENCE ONLY. TO BE PERMITTED UNDER SEPARATE PUBLIC WORKS PERMIT.
4. SLOPES PROVIDED ON SLOPE ARROW ARE FOR REFERENCE ONLY.
5. LANDINGS ON ACCESSIBLE ROUTES SHALL NOT EXCEED 2% IN ANY DIRECTION.
6. ALL ACCESSIBLE ROUTES SHALL COMPLY WITH CURRENT ADA ACCESSIBILITY GUIDELINES FOR BUILDING AND FACILITIES (ADAAG).

X KEY NOTES

NOTE	DESCRIPTION	DETAIL REF.
SB	STORM BASIN (ID AS SHOWN)	5/C5.1

GRADING LABEL LEGEND

CALLOUT	DESCRIPTION
X.X%	GRADING SLOPE AND DIRECTION (DOWNHILL)
XX.XX	SPOT ELEVATION
XX	DESCRIPTION LISTED BELOW. NO DESCRIPTION MEANS TP OR TG
BOS	BOTTOM OF SWALE
BOW	BACK OF WALK
BS	BOTTOM OF STEP
BW	BOTTOM OF WALL
EG	EXISTING GRADE
FF	FINISHED FLOOR
FL	FLOW LINE
G	GUTTER
HP	HIGH POINT
LP	LOW POINT
RIM	RIM OF STRUCTURE
TC	TOP OF CURB
TG	TOP OF GROUND
TP	TOP OF PAVEMENT
TS	TOP OF STEP
TW	TOP WALL
(XXX.X±)	EXISTING GRADE (MATCH WHERE APPLICABLE)

SHEET LEGEND

	PROPERTY LINE
	GRADE BREAK
(49)	EX. CONTOUR MINOR
(50)	EX. CONTOUR MAJOR
49	CONTOUR MINOR (FG)
50	CONTOUR MAJOR (FG)

SCALE 1 INCH = 20 FEET
20 0 20 40

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PRELIMINARY NOT FOR CONSTRUCTION

Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

Project Adress:
190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No.	Revision	Date

Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
GRADING PLAN ENLARGEMENT & SECTIONS

Sheet No.
C3.1

Job No.
4773-01

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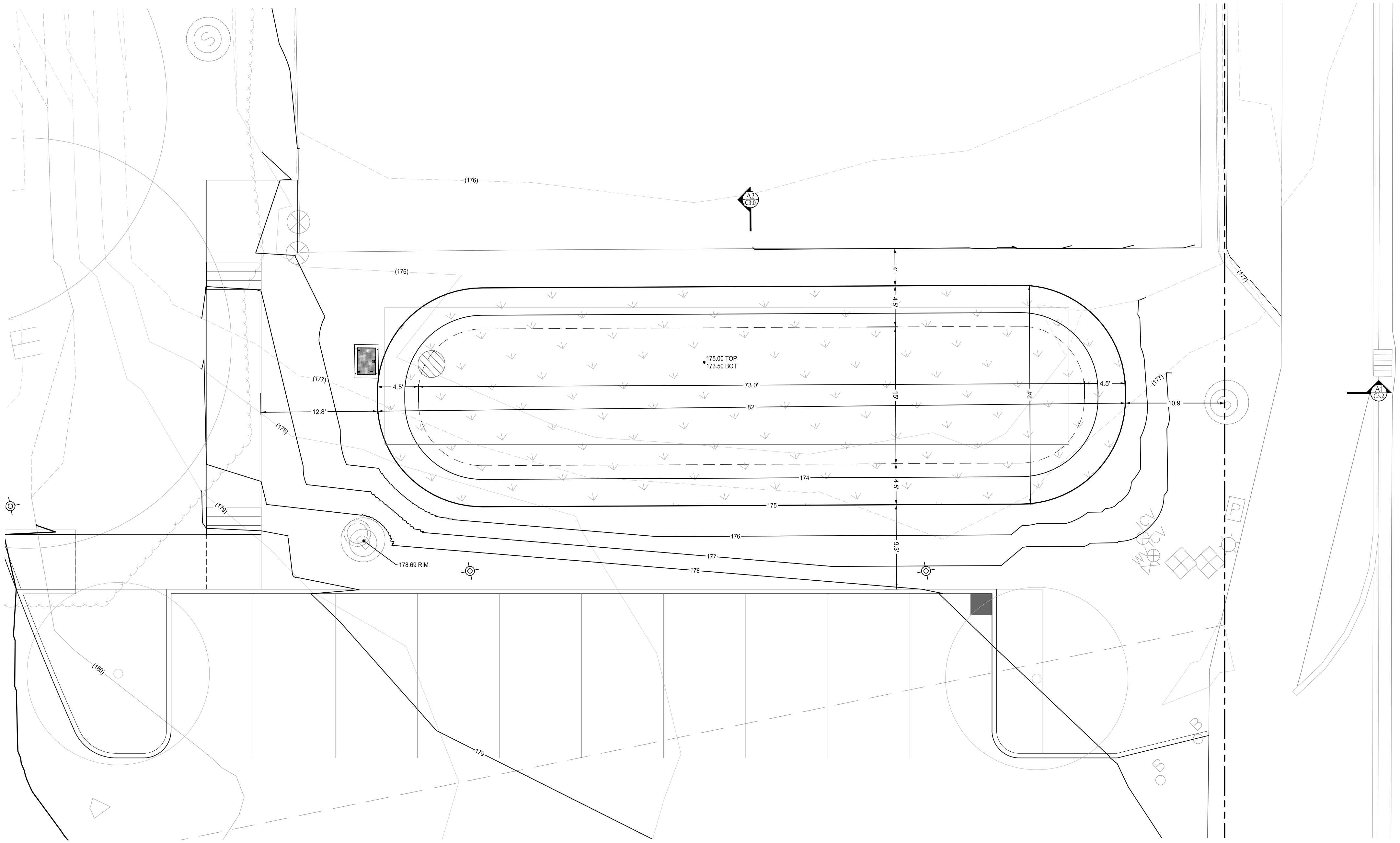
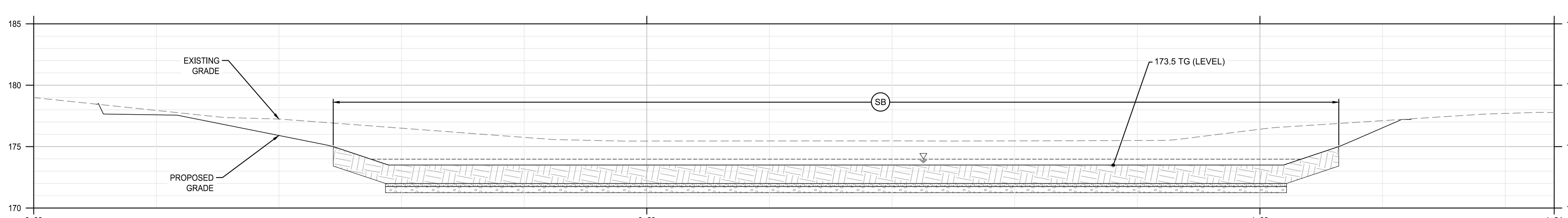


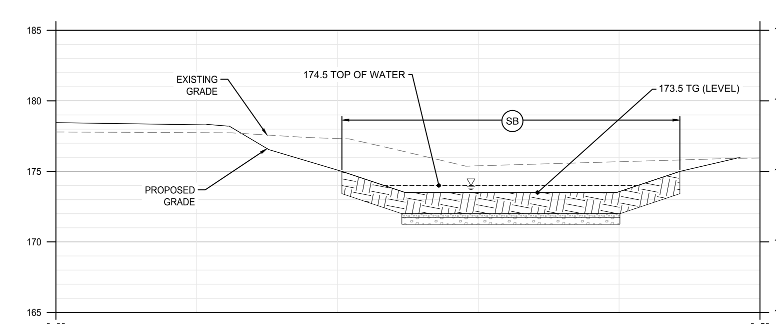
ENLARGEMENT 3 - STORM BASIN "A"

SCALE: 1" = 5'



A1 SECTION

SCALE: HORZ: 1" =5'
VERT: 1" = 5'



A2 SECTION

SCALE: HORZ: 1" =5'
VERT: 1" = 5'

SHEET NOTES

1. ALL DIMENSIONS ARE TO FACE OF CURB OR FACE OF WALL.
2. ALL SIDEWALK PAVEMENT JOINTS SHALL BE CONSTRUCTED PER DETAIL 15/C5.0.
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6. ALL ACCESSIBLE ROUTES SHALL COMPLY WITH CURRENT ADA ACCESSIBILITY GUIDELINES FOR BUILDING AND FACILITIES (ADAAG).

X KEY NOTES

NOTE	DESCRIPTION	DETAIL REF.
SB	STORM BASIN (ID AS SHOWN)	5/C5.1

GRADING LABEL LEGEND

CALLOUT	DESCRIPTION
X.X%	GRADING SLOPE AND DIRECTION (DOWNHILL)
XX.XX	SPOT ELEVATION
XX	DESCRIPTION LISTED BELOW. NO DESCRIPTION MEANS TP OR TG
BOS	BOTTOM OF SWALE
BOW	BACK OF WALK
BS	BOTTOM OF STEP
BW	BOTTOM OF WALL
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RIM	RIM OF STRUCTURE
TC	TOP OF CURB
TG	TOP OF GROUND
TP	TOP OF PAVEMENT
TS	TOP OF STEP
TW	TOP WALL
(XXX.Xx)	EXISTING GRADE (MATCH WHERE APPLICABLE)

SHEET LEGEND

	PROPERTY LINE
	GRADE BREAK
(49)	EX. CONTOUR MINOR
(50)	EX. CONTOUR MAJOR
49	CONTOUR MINOR (FG)
50	CONTOUR MAJOR (FG)

SCALE 1 INCH = 20 FEET
20 0 20 40

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Key Plan

Revisions to Sheet

No.	Revision	Date

Status: **RCWOD**

Date: **08.08.2025**

Sheet Title

GRADING PLAN ENLARGEMENT & SECTIONS

Sheet No. **C3.2**

Job No. **4773-01**

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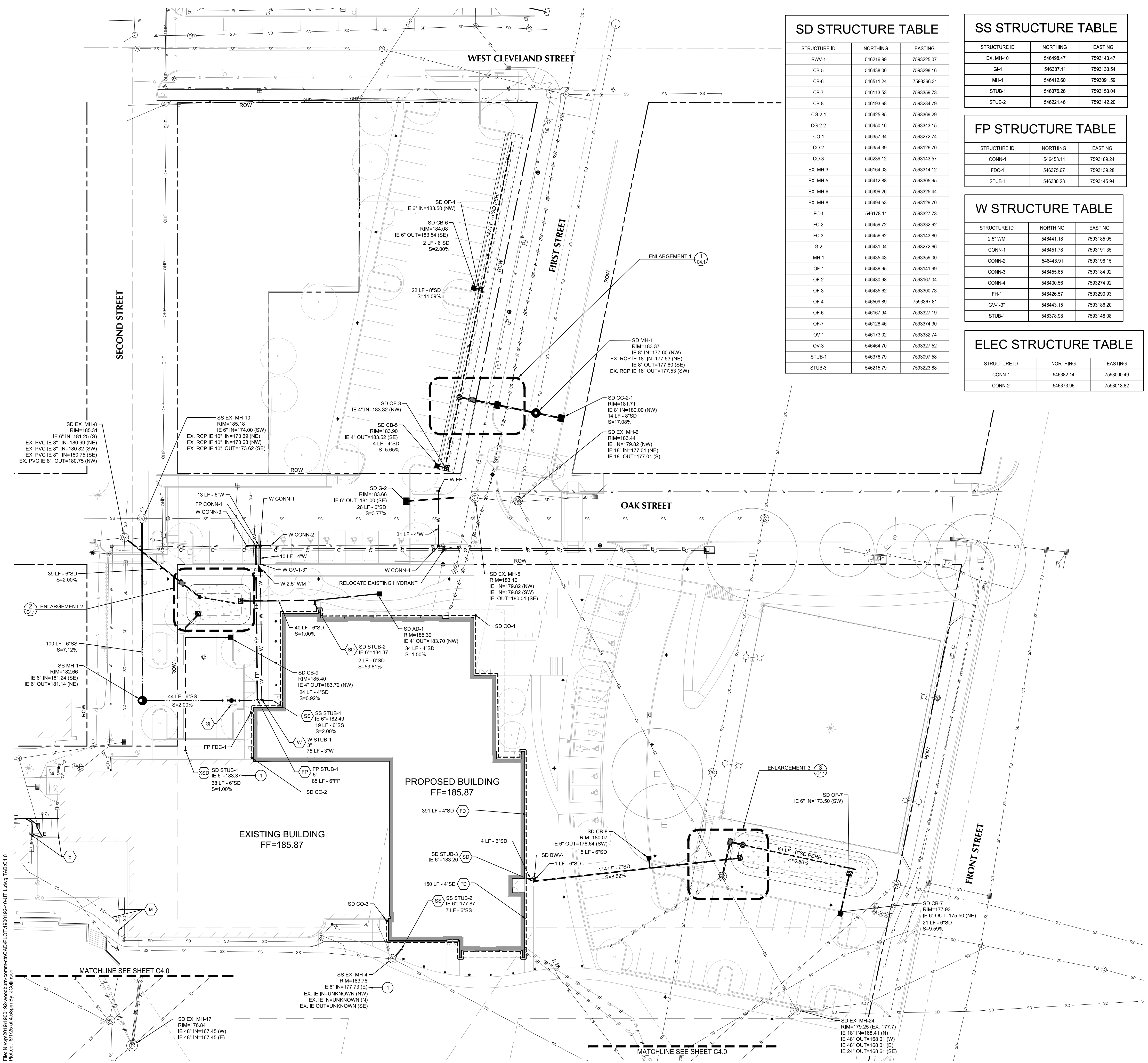

SD STRUCTURE TABLE

STRUCTURE ID	NORTHING	EASTING
BWV-1	546216.99	7593225.07
CB-5	546438.00	7593298.16
CB-6	546511.24	7593366.31
CB-7	546113.53	7593359.73
CB-8	546193.68	7593284.79
CG-2-1	546425.85	7593369.29
CG-2-2	546450.16	7593343.15
CO-1	546357.34	7593272.74
CO-2	546354.39	7593126.70
CO-3	546239.12	7593143.57
EX. MH-3	546164.03	7593314.12
EX. MH-5	546412.88	7593305.95
EX. MH-6	546399.26	7593325.44
EX. MH-8	546494.53	7593129.70
FC-1	546178.11	7593327.73
FC-2	546459.72	7593332.92
FC-3	546456.62	7593143.80
G-2	546431.04	7593272.66
MH-1	546435.43	7593359.00
OF-1	546436.95	7593141.99
OF-2	546430.98	7593167.04
OF-3	546435.62	7593300.73
OF-4	546509.89	7593367.81
OF-6	546167.94	7593327.19
OF-7	546128.46	7593374.30
OV-1	546173.02	7593332.74
OV-3	546464.70	7593327.52
STUB-1	546376.79	7593097.58
STUB-3	546215.79	7593223.88

SS STRUCTURE TABLE

STRUCTURE ID	NORTHING	EASTING
EX. MH-10	546498.47	7593143.47
GI-1	546387.11	7593133.54
MH-1	546412.60	7593091.59
STUB-1	546375.26	7593153.04
STUB-2	546221.46	7593142.20

FP STRUCTURE TABLE

STRUCTURE ID	NORTHING	EASTING
CONN-1	546453.11	7593189.24
FDC-1	546375.67	7593139.28
STUB-1	546380.28	7593145.94

W STRUCTURE TABLE

STRUCTURE ID	NORTHING	EASTING
2.5" WM	546441.18	7593185.05
CONN-1	546451.78	7593191.35
CONN-2	546448.91	7593196.15
CONN-3	546455.65	7593184.92
CONN-4	546400.56	7593274.92
FH-1	546426.57	7593290.93
GV-1-3"	546443.15	7593186.20
STUB-1	546378.98	7593148.08

ELEC STRUCTURE TABLE

STRUCTURE ID	NORTHING	EASTING
CONN-1	546382.14	7593000.49
CONN-2	546373.96	7593013.82

SHEET NOTES

1. PRIOR TO CONSTRUCT FIELD VERIFY LOCATIONS, SIZE AND IE OF ALL EXISTING UTILITIES AT PROPOSED POINTS OF CONNECTION OR WHERE NOTED, NOTIFY ENGINEER DISCREPANCIES.
2. ON-SITE PIPE BEDDING AND BACKFILL FOR ALL UTILITIES SHALL BE DONE PER DETAIL 1/C5.1.
3. OFF-SITE (IN ROW) PIPE BEDDING AND TRENCH BACKFILL AND SURFACING PER CITY STD DWG P-100 AND P-101.
4. STRUCTURES LOCATIONS ARE BASED ON CENTER OF STRUCTURE.
5. INSTALL TRUST BLOCK ON FIRE AND WATER LINES PER CITY STD DWG 5000-1, 5000-2, AND 5000-3
6. INSTALL UTILITIES PER CITY STD DWG 5000-6 AND 6200-4.
7. INSTALLATION OF WATER MAIN AIR RELEASE VALVES TO BE DONE PER CITY STD DWG 5050-3.
8. INSTALLATION OF SAMPLING TAP ASSEMBLY TO BE PER CITY STD DWG 5100-1.
9. INSTALLATION OF SANITARY SEWER SERVICE PER CITY STD DWG 6200-3.

(X) KEY NOTES

1 CONNECT TO EXISTING PIPE. SEE SHEET NOTE 1.

2 CUT-IN-TEE CONNECTION TO EX PUBLIC WATER LINE. COORDINATE TEMPORARY SHUT DOWN WITH CITY PUBLIC WORKS.

(X) UTILITY KEY NOTES

NOTE	DESCRIPTION	DETAIL REF.
ADJ	ADJUST RIM TO PROPOSED ELEVATION.	
E	CONNECT 1" CONDUIT, SEE AQUATIC PLANS	
FD	FOUNDATION DRAIN	
S	CONNECT TO WASTE LINE. SEE PLUMBING PLANS FOR CONTINUATION. SIZE AS NOTED.	
SD	CONNECT TO STORM DRAIN/ROOF DRAIN. SEE PLUMBING PLANS FOR CONTINUATION. SIZE AND IE AS NOTED	
W	CONNECT TO COLD WATER SYSTEM. SEE PLUMBING PLANS FOR CONTINUATION. SIZE AS NOTED.	
!!	UTILITY CROSSING. PROVIDE 12" MIN. CLEARANCE, U.N.O.	
M	MAINTENANCE EXISTING UTILITY PIPE AND CONFIRM FUNCTIONAL.	

UTILITY LABEL LEGEND

STRUCTURE LABEL

XX XX-XX
RIM=
IE IN = XX.X
IE OUT = XX.X

UTILITY TYPE (SD=STORM DRAINAGE, SS=SANITARY SEWER, W=WATER, FP=FIRE PROTECTION)
STRUCTURE TYPE CALLOUT
ID NUMBER (WHERE APPLICABLE)
STRUCTURE INFO (WHERE APPLICABLE)

PIPE LABEL

XXLF - XX" XX
S=X.XX%

UTILITY LENGTH
UTILITY SIZE
UTILITY TYPE
SLOPE (WHERE APPLICABLE)

STRUCTURE TYPE

CALLOUT	DESCRIPTION	DETAIL REF.
BWV	BACKWATER VALVE	
CB	CATCH BASIN	
CO	CLEANOUT TO GRADE	6200-1
CG-2	CONCRETE INLET	RD366
DCBA	DOUBLE CHECK BACKFLOW ASSEMBLY	1/C5.2
DCVA	DOUBLE CHECK VALVE ASSEMBLY	5070-2
FC	FLOW-CONTROL VAULT	1/C5.1
FDC	FIRE DEPARTMENT CONNECTION	(SEE OTHERS)
FH	FIRE HYDRANT	5070-1
G-2	CONCRETE INLET	RD364
GV	GATE VALVE	5050-2
HB	HORIZONTAL BEND	
MH	MANHOLE	6510-3
OV	OVERFLOW INLET	3/C5.1
STUB	STUB	
TD	TRENCH DRAIN	4 & 5 / C5.1
TEE	TEE CONNECTION	
W	WATERLINE	
WM	WATER METER	6/C5.2

SHEET LEGEND

STORMWATER PLANTER/BASIN 5/C5.1
STORM PIPE
PERIMETER FOUNDATION DRAIN
TRENCH DRAIN
MANHOLE
OVERFLOW INLET
BACKWATER VALVE
CATCH BASIN
OUTFALL
PIPE REMOVAL

SCALE 1 INCH = 20 FEET
20 0 20 40

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Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

Project Adress:
190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No.	Revision	Date

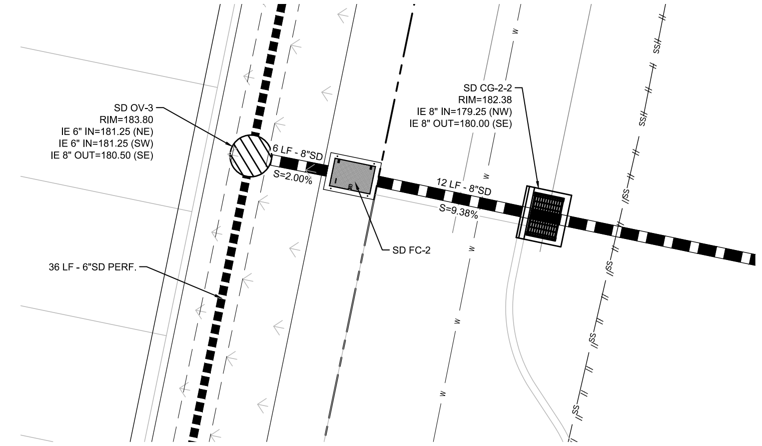
Status: **RCWOD**

Date: **08.08.2025**

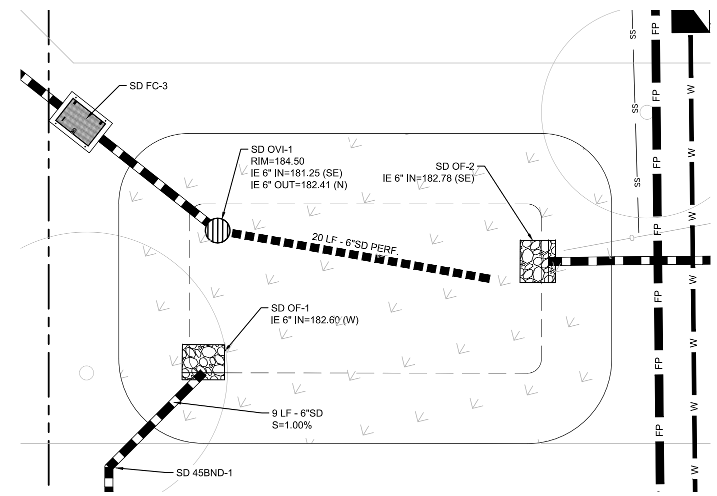
Sheet Title
UTILITY PLAN

Sheet No.
C4.0

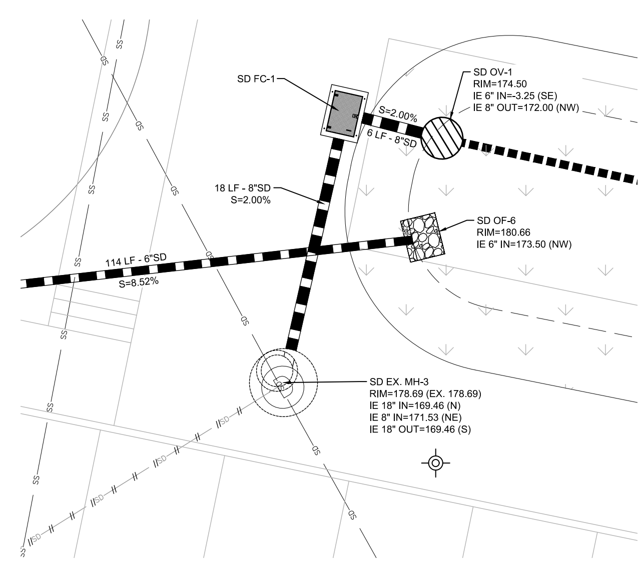
Job No.
4773-01



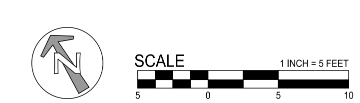
1 ENLARGEMENT 1
SCALE: 1" = 5'



2 ENLARGEMENT 2
SCALE: 1" = 5'



3 ENLARGEMENT 3
SCALE: 1" = 5'



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Sheet Title
UTILITY PLAN ENLARGEMENT

Sheet No.
C4.1

Job No.
4773-01

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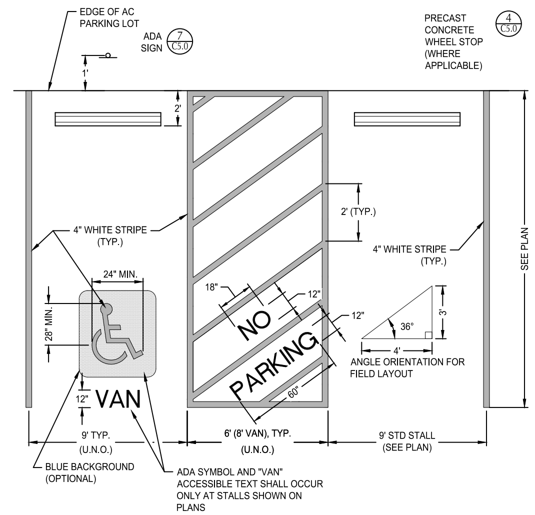
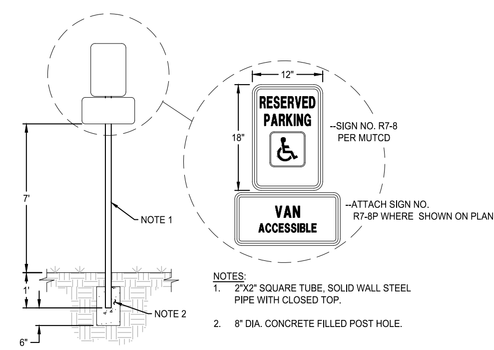
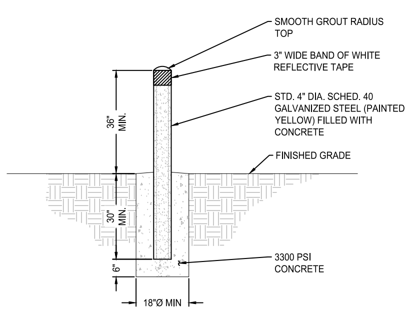


6 TYPICAL PARKING LAYOUT
SCALE: NTS

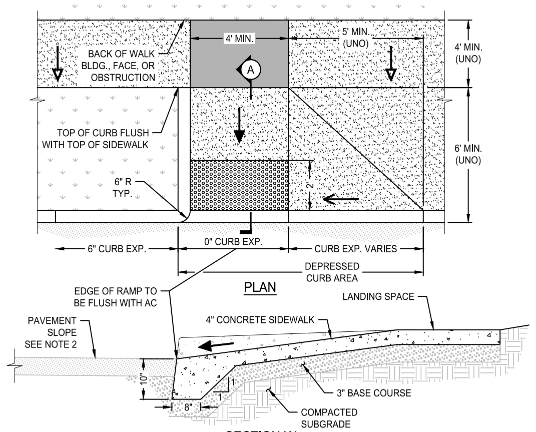


NOTES:
1. 2"X2" SQUARE TUBE, SOLID WALL STEEL PIPE WITH CLOSED TOP.
2. 8" DIA. CONCRETE FILLED POST HOLE.

7 ADA PARKING SIGN
SCALE: NTS



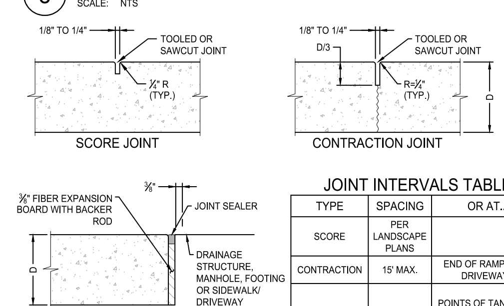
8 PIPE BOLLARD (4" DIA)
SCALE: NTS



NOTES:
1. SEE PLAN FOR PROJECT SPECIFIC DIMENSIONS.
2. MAXIMUM GRADE BREAK FROM RAMP TO PAVEMENT SHALL BE 11%. MAXIMUM COUNTERSLOPE OF ADJOINING GUTTERS AND ROAD SURFACES IMMEDIATELY ADJACENT TO THE CURB RAMP SHALL NOT BE STEEPER THAN 5.0%.
3. WHERE THE LANDING SPACE IS CONSTRAINED AT THE BACK OF WALK, THE MIN. LEVEL AREA SHALL BE 4.0' X 5.0'. THE 5.0' DIMENSION SHALL BE PROVIDED IN THE DIRECTION OF THE RAMP RUN.
4. GRADE BREAKS AT THE TOP AND BOTTOM OF CURB RAMP RUNS SHALL BE PERPENDICULAR TO THE DIRECTION OF THE RAMP RUN.

LEGEND
- RAMP SLOPE: 7.5% MAX. SLOPE (8.3% MAX. FINISHED SURFACE)
- CROSS SLOPE: 1.5% MAX. SLOPE (2.0% MAX. FINISHED SURFACE)
- RAMP WING: 9.5% MAX. SLOPE (10.0% MAX. FINISHED SURFACE)
- LANDING SPACE: MIN. LEVEL AREA 4.0' X 4.0', A 2.0% MAX. FINISHED SURFACE IN ANY DIRECTION IS CONSIDERED LEVEL, SEE NOTE 3
- SIDEWALK: 4.5% MAX. RUNNING SLOPE (5.0% MAX. FINISHED SURFACE),1.5% MAX. CROSS SLOPE (2.0% MAX. FINISHED SURFACE)
- DETECTABLE WARNING SURFACE, SEE X/CX.XX
- LANDSCAPING

10 CURB RAMP - TYPE 1
SCALE: NTS



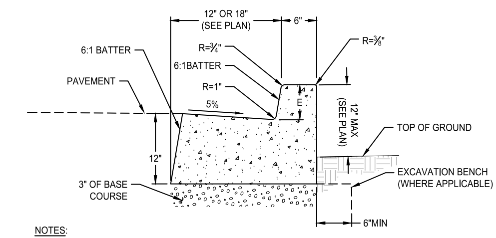
JOINT INTERVALS TABLE

TYPE	SPACING	OR AT...
SCORE	PER LANDSCAPE PLANS	
CONTRACTION	15' MAX.	END OF RAMPS AND DRIVEWAYS
EXPANSION/ ISOLATION	200' *	POINTS OF TANGENCY AND AT ENDS OF EACH DRIVEWAY OR OTHER FIXED OBJECTS

* MONOLITHIC CURB AND SIDEWALK SHALL BE 45' MAX.

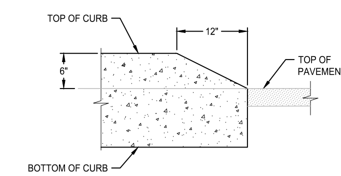
NOTES:
1. CONSTRUCTION COLD JOINTS MAY BE USED IN PLACE OF CONTRACTION JOINTS.
2. PROVIDE MEDIUM BROOM FINISH WITH NO TOOL MARKS.

9 CONCRETE PAVEMENT JOINTS
SCALE: NTS

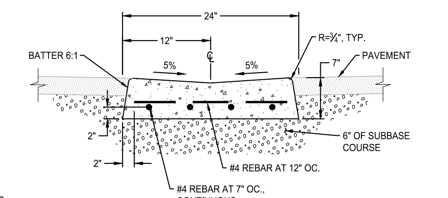


NOTES:
1. CURB EXPOSURE 'E' = 6", TYP. VARY AS SHOWN ON PLANS OR AS DIRECTED.
2. CONSTRUCT CONTRACTION JOINTS AT 15' MAX. SPACING AND AT RAMPS. CONSTRUCT EXPANSION JOINTS AT 200' MAX SPACING AT POINTS OF TANGENCY AND AT ENDS OF EACH DRIVEWAY.
3. TOPS OF ALL CURBS SHALL SLOPE TOWARD THE ROADWAY AT 2% UNLESS OTHERWISE SHOWN OR AS DIRECTED.
4. DIMENSIONS ARE NOMINAL AND MAY VARY TO CONFORM WITH CURB MACHINE AS APPROVED BY THE ENGINEER.

2C THICKENED CURB AND GUTTER
SCALE: NTS

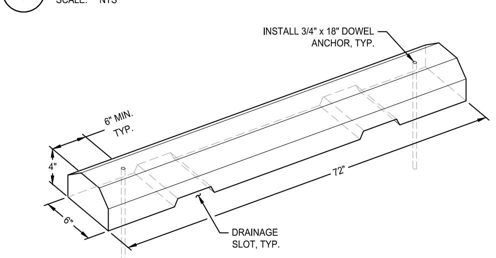


2D CONCRETE CURB - ENDING
SCALE: NTS



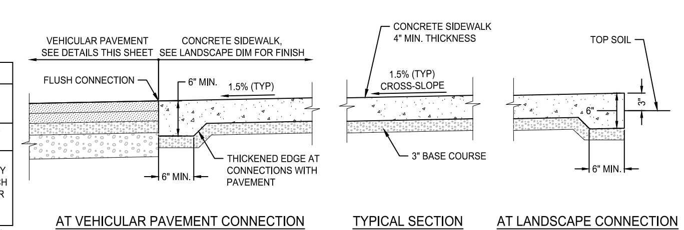
NOTES:
1. CONSTRUCT JOINTS PER DETAIL 9/C6.1 AND AT INTERVALS AND TYPES SHOWN ON LANDSCAPE PLANS.
2. REFER TO LANDSCAPE PLANS AND SPECIFICATIONS FOR ALL CONCRETE SCORING AND FINISH.

3 VALLEY GUTTER
SCALE: NTS



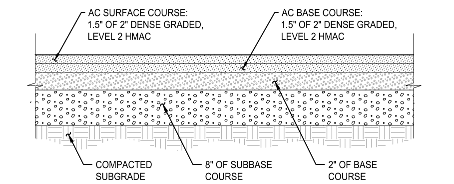
NOTES:
1. DIMENSIONS ARE ACTUAL AND MAY CONFORM TO SITE CONDITIONS APPROVED BY LANDSCAPE.

4 PRECAST CONCRETE WHEEL STOP
SCALE: NTS



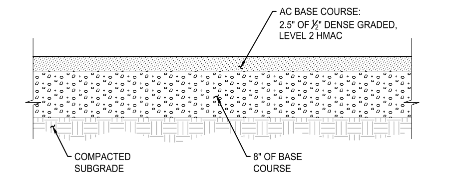
NOTES:
1. REFER TO LANDSCAPE PLANS AND SPECIFICATIONS FOR ALL CONCRETE SIDEWALK SCORING AND FINISH.
2. CONSTRUCT CONTRACTION JOINTS AT 15' MAX. SPACING. CONSTRUCT EXPANSION JOINTS AT 40' MAX SPACING.

5 CONCRETE SIDEWALK SECTION
SCALE: NTS

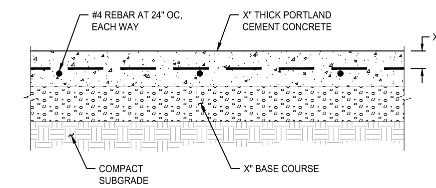


NOTE: REFER TO GEOTECH REPORT FOR ADDITIONAL INFORMATION AND ADDITIONAL MEASURES DURING WET WEATHER INSTALLATION.

1A HEAVY ASPHALT PAVEMENT SECTION
SCALE: NTS

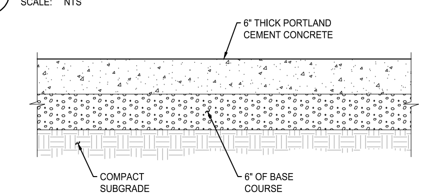


1B STANDARD ASPHALT PAVEMENT SECTION
SCALE: NTS



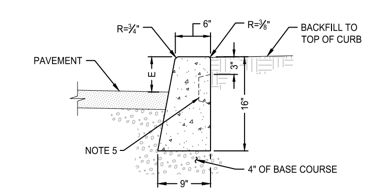
NOTES:
1. JOINTS:
 - CONSTRUCT CONTRACTION JOINTS AT 15' MAX. SPACING AND AT RAMPS.
 - CONSTRUCT EXPANSION JOINTS AT 200' MAX. SPACING AT POINTS OF TANGENCY AND AT ENDS OF EACH DRIVEWAY.
2. PROVIDE MEDIUM TO COARSE BROOM FINISH.

1C REINFORCED CONCRETE PAVEMENT SECTION
SCALE: NTS



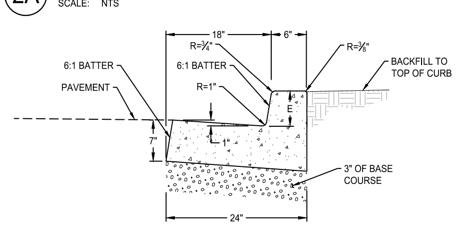
NOTES:
1. CONSTRUCT JOINTS PER DETAIL 9/C6.1 AND AT INTERVALS AND TYPES SHOWN ON LANDSCAPE PLANS.
2. REFER TO LANDSCAPE PLANS AND SPECIFICATIONS FOR ALL CONCRETE SCORING AND FINISH.

1D CONCRETE PAVEMENT SECTION
SCALE: NTS



NOTES:
1. CURB EXPOSURE 'E' = 6", TYP. VARY AS SHOWN ON PLANS OR AS DIRECTED.
2. CONSTRUCT JOINTS PER DETAIL 9/C6.1 AND AT INTERVALS AND TYPES SHOWN ON LANDSCAPE PLANS.
3. REFER TO LANDSCAPE PLANS AND SPECIFICATIONS FOR ALL CONCRETE SCORING AND FINISH.
4. DIMENSIONS ARE NOMINAL AND MAY VARY TO CONFORM WITH CURB MACHINE AS APPROVED BY THE ENGINEER.
5. WHERE CONCRETE SIDEWALK IS USED, INSTALL KEYWAY INTO CURB AS SHOWN.

2A CONCRETE CURB - STANDARD
SCALE: NTS



NOTES:
1. CURB EXPOSURE 'E' = 6", TYP. VARY AS SHOWN ON PLANS OR AS DIRECTED.
2. CONSTRUCT JOINTS PER DETAIL 9/C6.1 AND AT INTERVALS AND TYPES SHOWN ON LANDSCAPE PLANS.
3. REFER TO LANDSCAPE PLANS AND SPECIFICATIONS FOR ALL CONCRETE SCORING AND FINISH.
4. DIMENSIONS ARE NOMINAL AND MAY VARY TO CONFORM WITH CURB MACHINE AS APPROVED BY THE ENGINEER.

2B CONCRETE CURB AND GUTTER
SCALE: NTS

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Key Plan

Revisions to Sheet

No.	Revision	Date

Status: RCWOD

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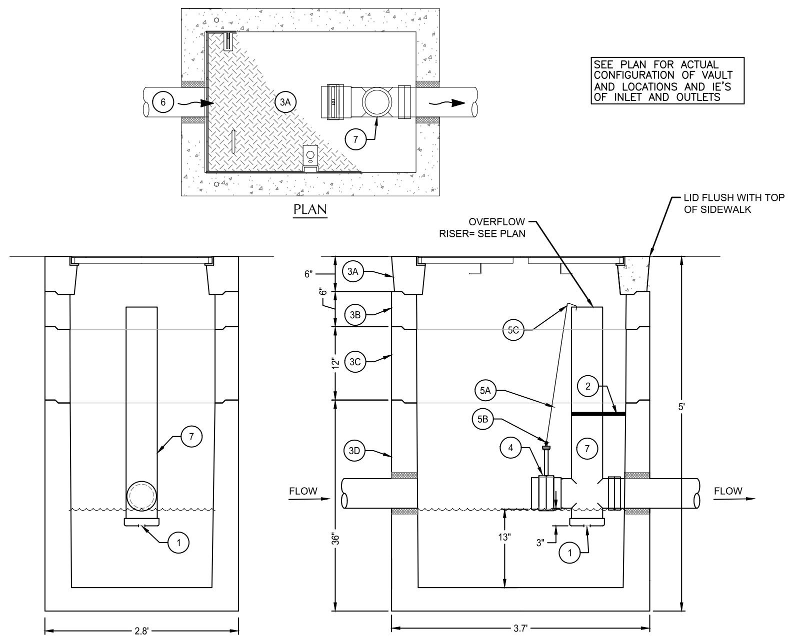
Sheet Title
CIVIL DETAILS

Sheet No.
C5.0

Job No.
4773-01

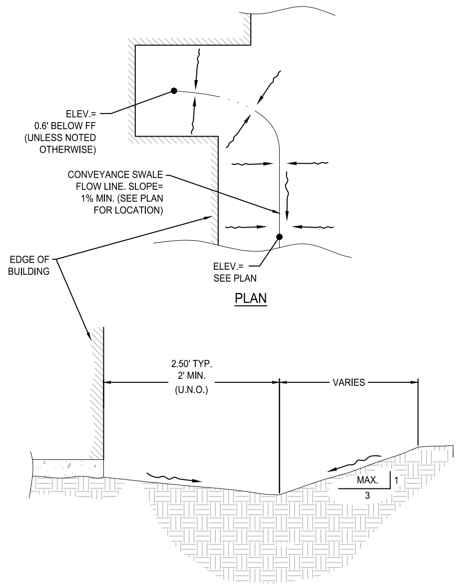


KEY NOTES

1 FLOW CONTROL ORIFICE=1.0" (DRILL IN CENTER OF PVC CAP)
2 ATTACHED RISER PIPE TO VAULT WALL WITH INSTALL S/S PIPE BRACKET AND ANCHOR BOLTS
3A 2436P GALVANIZED "NON-SLIP" DOOR W/LOCKING LATCH. ORIENT LID AS SHOWN.
3B 233-6-INCH RISER BY OLD CASTLE OR APPROVED EQUAL.
3C 233-12-INCH RISER BY OLD CASTLE OR APPROVED EQUAL.
3D 233-SOLID WALL VAULT BY OLD CASTLE OR APPROVED EQUAL.
4 6" KNIFE GATE VALVE BY VALTERRA WITH STAINLESS PADDLE OR APPROVED EQUAL.
5A USE 3/8" S.S. ROD FOR PULL ROD TO OPEN AND CLOSE KNIFE GATE VALVE.
5B BEND SMALL LOOP ON BOTTOM OF ROD. ATTACH BOTTOM OF ROD TO KNIFE GATE HANDLE W/ 12 GAUGE COPPER WIRE.
5C CREATE SMALL HOOK HANDLE AT TOP OF ROD THAT HOOKS ON TOP OF OVERFLOW RISER.
6 8" PIPE CONNECTED TO SUB-SURFACE DETENTION GALLERY
7 6" DIA. PVC FLOW CONTROL STRUCTURE. CONSTRUCT AS SHOWN.

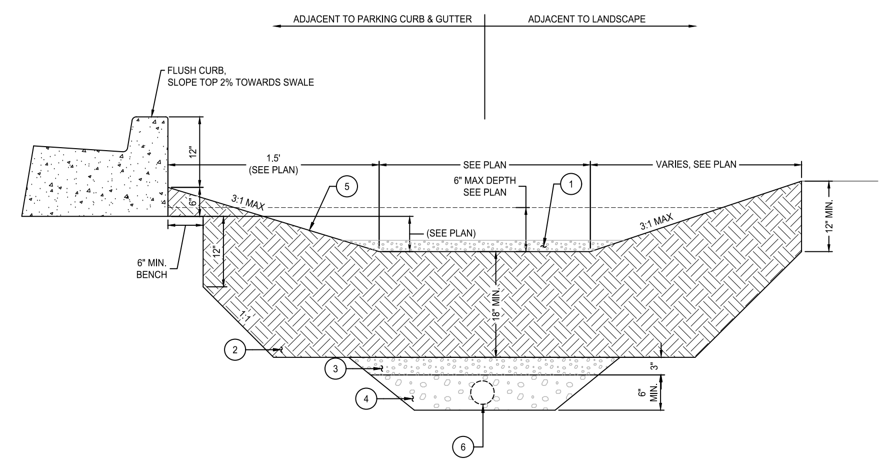
6 FLOW-CONTROL VAULT

SCALE: NTS



4 CONVEYANCE SWALE

SCALE: NTS

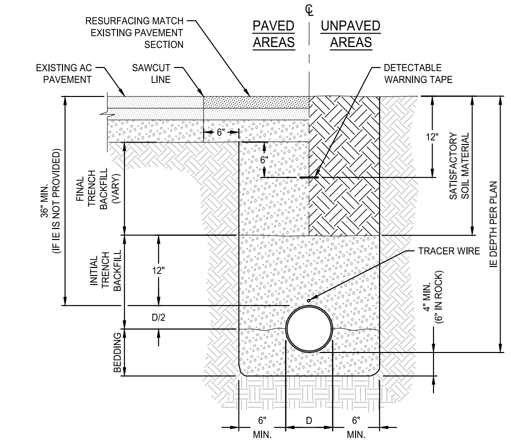


KEY NOTES

1 INSTALL 2" THICK LAYER OF PEA GRAVEL OR OTHER NON-FLOATING MULCH. (CONFIRM TYPE WITH LANDSCAPE)
2 STORMWATER FACILITY GROWING MEDIA PER SPECS.
3 DRAINAGE LENS COURSE (3/4" - NO. 4 OPEN GRADED AGGREGATE).
4 DRAINAGE FILL PER SPECS.
5 PLANTING SEE LANDSCAPE PLANS.
6 4" PVC PERF. PIPE, ORIENT WITH HOLES FACING DOWN.

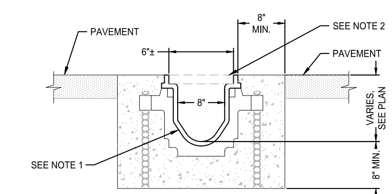
5 STORM BASIN

SCALE: NTS



1 TYPICAL PIPE BEDDING AND BACKFILL

SCALE: NTS

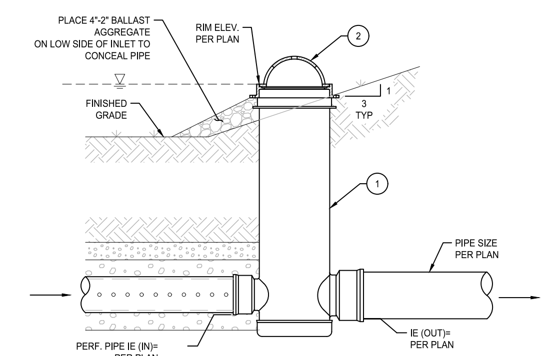


NOTES:

1. TRENCH DRAIN SHALL BE NEUTRAL-SLOPED 6" WIDE ZURN OR ACO TRENCH DRAIN OR APPROVED EQUAL.
2. TRENCH DRAINS GRATE SHALL BE LOCKABLE HEAVY DUTY TRENCH GRATE - CLASS C.
3. TRENCH SYSTEM SHALL BE INSTALLED PER MANUFACTURER'S INSTRUCTIONS.

2 TRENCH DRAIN - 6 INCH WIDE

SCALE: NTS



KEY NOTES

1 12" NYLOPLAST DRAIN BASIN, OR APPROVED EQUAL
2 12" LIGHT DUTY DOMED GRATE MODEL 1299CGD BY ADS, OR APPROVED EQUAL

3 OVERFLOW INLET

SCALE: NTS

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C5.1

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4773-01

File: N:\c\p\2019\1900192-woodburn-comm-ctr\CAD\PLOT\1900192-40&50-UTIL-DTLS.dwg TAB:C5.1
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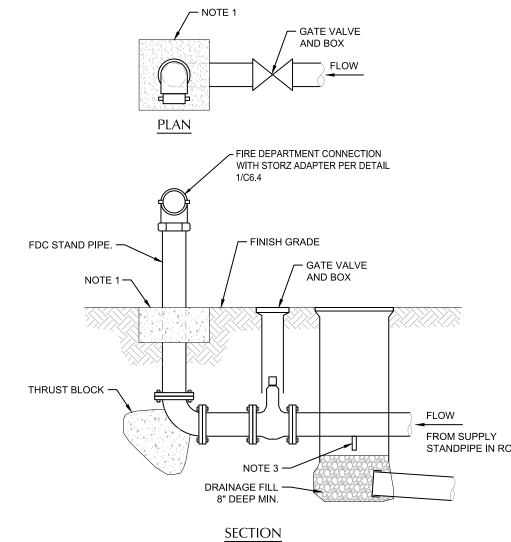
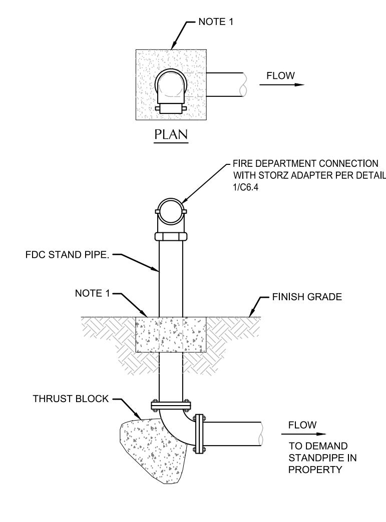


NOTES:
1. CONCRETE ANCHOR PAD TO BE 18"x18"x6" THICK, UNLESS NOTED OTHERWISE. ELIMINATE IF INSTALLED IN CONCRETE PAVED AREA.
2. USE FLANGE OR THREADED FITTINGS.
3. CONTRACTOR SHALL PROVIDE SINGLE CHECK VALVE AND BALL DRIP VALVE IN ACCESSIBLE LOCATION AT LOW POINT.

4 FIRE DEPARTMENT DRY STAND PIPE CONNECTION DEMAND SIDE

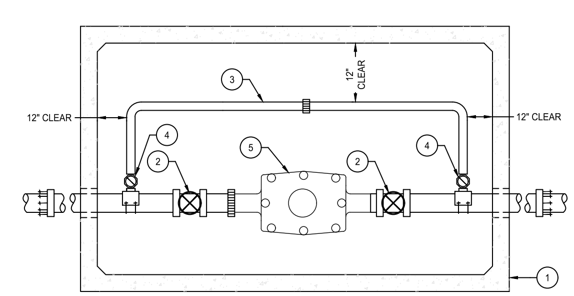
SCALE: NTS



NOTES:
1. CONCRETE ANCHOR PAD TO BE 18"x18"x6" THICK, UNLESS NOTED OTHERWISE. ELIMINATE IF INSTALLED IN CONCRETE PAVED AREA.
2. USE FLANGE OR THREADED FITTINGS.

5 FIRE DEPARTMENT DRY STAND PIPE CONNECTION SUPPLY SIDE

SCALE: NTS

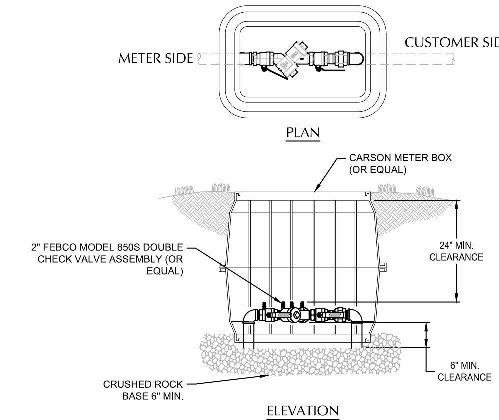


X KEY NOTES

1 CONCRETE VAULT. SIZED FOR CLEARANCES WITH DOUBLE HATCH
2 RESILIENT WEDGE GATE VALVE, FLANGED WITH NON-RISING STEM
3 2" BYPASS LINE
4 2" CORPORATION STOP (BALL TYPE)
5 4" COMPOUND WATER METER

6 4" WATER METER

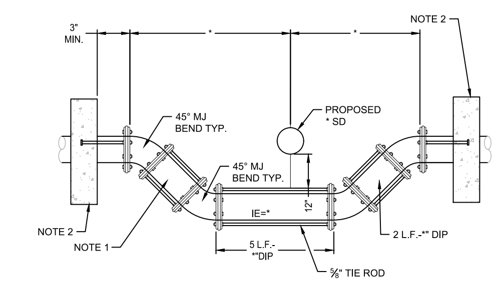
SCALE: NTS



NOTE:
INSTALLATION SHOWN IS ONLY A SUGGESTION. THE DISTANCE FROM BOTTOM OF DEVICE TO FINISH GRADE, FREEZE PROTECTION, AND CLEARANCE FOR TESTING & REPAIR ARE THE MAJOR CONSIDERATIONS FOR INSTALLATION. PLUGS TO BE INSTALLED IN TEST COCKS OF BELOW GROUND INSTALLATIONS (NO DISSIMILAR METALS). IF FREEZE PROTECTION IS PROVIDED, THE 24" MIN CLEARANCE MAY BE REDUCED.

1 DOUBLE CHECK BACKFLOW ASSEMBLY

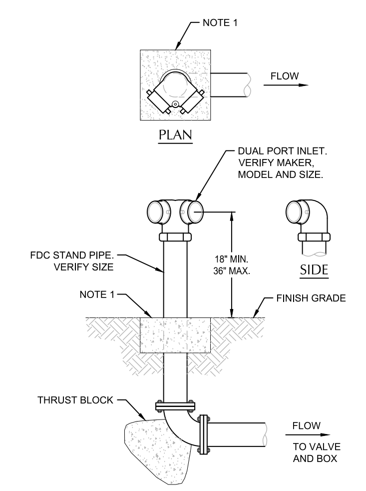
SCALE: NTS



NOTES:
1. DIP MJ SPOOL. * L.F.-* DIP CONTRACTOR TO VERIFY LENGTH OF * DIP NECESSARY TO MEET WATERLINE CLEARANCE TYP.
2. CONC. ANCHOR BLOCK W/ TIE BOLTS, DUCT LUGS OR STEEL PLATE AS APPROVED

2 TYPICAL WATERLINE UTILITY CROSSING

SCALE: NTS



NOTES:
1. CONCRETE ANCHOR PAD TO BE 12"x12"x6" THICK, UNLESS NOTED OTHERWISE. ELIMINATE IF INSTALLED IN CONCRETE PAVED AREA.
2. USE FLANGE OR THREADED FITTINGS.
3. CONTRACTOR SHALL PROVIDE SINGLE CHECK VALVE AND BALL DRIP VALVE IN ACCESSIBLE LOCATION INSIDE DDCV VAULT. COORDINATE WITH PLUMBING.

3 FIRE DEPARTMENT CONNECTION (FDC) DUAL PORT

SCALE: NTS

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Project Owner:
City Of Woodburn Oregon

Project Name:
Woodburn Community Center

Project Adress:
190 Oak Street
Woodburn, OR 97071

Key Plan

Revisions to Sheet

No.	Revision	Date

Status: **RCWOD**

Date: **08.08.2025**

Sheet Title
CIVIL DETAILS

Sheet No.
C5.2

Job No.
4773-01

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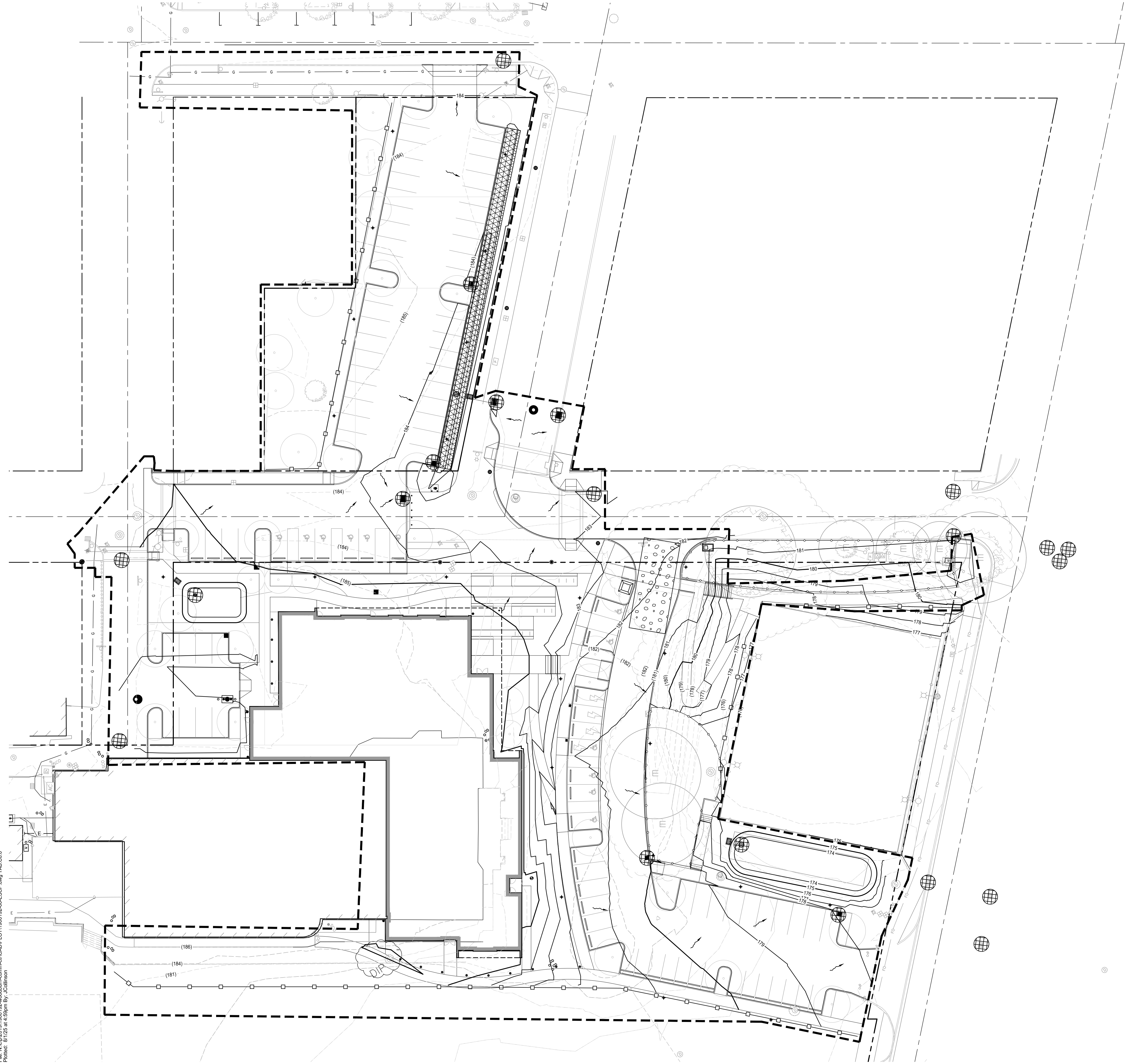

SHEET NOTES

1. EROSION CONTROL MEASURES SHALL BE INSTALLED PRIOR TO THE START OF WORK.
2. TEMPORARY EROSION CONTROL MEASURES SHALL BE IMPLEMENTED AS SHOWN. ADHERE TO CITY OF PORTLAND EROSION CONTROL GUIDELINES AND SPECIFICATION SECTION 015713 TEMPORARY EROSION AND SEDIMENT CONTROL. THE MEASURES SHOWN HERE ARE THE MINIMUM REQUIREMENTS FOR ANTICIPATED CONDITIONS. UPGRADE FACILITIES AS NEEDED TO ENSURE THAT SEDIMENT AND SEDIMENT LADEN WATER DO NOT LEAVE THE SITE.
3. MAINTAIN ALL ROADWAYS, KEEPING THEM CLEAN AND FREE OF CONSTRUCTION MATERIALS AND DEBRIS. PROVIDE DUST CONTROL AS NEEDED.
4. PREVENT SEDIMENT AND SEDIMENT LADEN WATER FROM ENTERING THE STORM DRAINAGE SYSTEM.

SHEET LEGEND

PROPERTY LINE

49 EX. CONTOUR MINOR

50 EX. CONTOUR MAJOR

49 PROP. CONTOUR MINOR

50 PROP. CONTOUR MAJOR

EXTENT OF WORK

SEDIMENT CONTROL FENCE. PLACE AT PROPERTY LINES, UNO (SHOWN OFFSET FOR CLARITY). 1 C7.0

INLET PROTECTION 2 C7.0

TREE PROTECTION FENCE 3 C7.0

PROTECT PROPOSED STORMWATER BASIN (SEE STORMWATER BASIN NOTES)

CONSTRUCTION ENTRANCE 4 C7.0

CONCRETE WASH

DIRECTION FLOW LINES

SCALE 1 INCH = 40 FEET

40 0 40 80

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Sheet Title
EROSION AND SEDIMENT CONTROL PLAN

Sheet No.
C6.0

Job No.
4773-01

File: N:\c\p\2019\1900192-woodburn-comm-ctr\CAD\PLOT\1900192-60-ESCP.dwg TAB:C6.0
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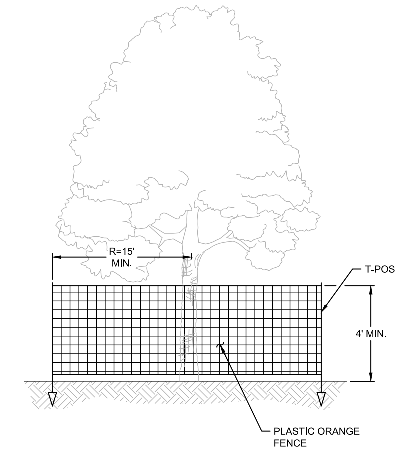


NOTES:

1. TEMPORARY FENCE SHALL BE PLACED AT 15' MIN RADIUS FROM TRUNK OF TREE TO BE SAVED. FENCE SHALL COMPLETELY ENCIRCLE TREE(S). INSTALL FENCE POSTS USING T-POST ONLY. AVOID DRIVING POSTS OR STAKES INTO MAJOR ROOTS.

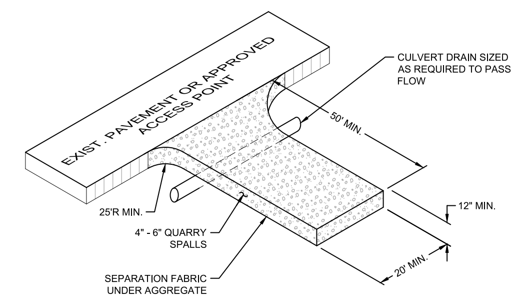
2. TREATMENT OF ROOTS EXPOSED DURING CONSTRUCTION: FOR ROOTS OVER 1" IN DIAMETER DAMAGED DURING CONSTRUCTION. MAKE A CLEAN STRAIGHT CUT TO REMOVE DAMAGED PORTION OF ROOT. ALL EXPOSED ROOTS SHALL BE TEMPORARILY COVERED WITH DAMP BURLAP TO PREVENT DRYING, AND COVERED WITH SOILS AS SOON AS POSSIBLE.

3. WORK WITHIN PROTECTION FENCE SHALL BE DONE MANUALLY. NO STOCKPILING OF MATERIALS, VEHICULAR TRAFFIC, OR STORAGE OF EQUIPMENT OR MACHINERY SHALL BE ALLOWED WITHIN THE LIMITS OF THE FENCING.

4. WITHIN CLEARING/GRADING LIMITS OR AT THE EDGE OF THE CLEARING/GRADING LIMITS, TREE PROTECTION MAY BE INSTALLED AROUND GROUPS OF TREES.

3 TREE PROTECTION - CONSTRUCTION FENCE

SCALE: NTS

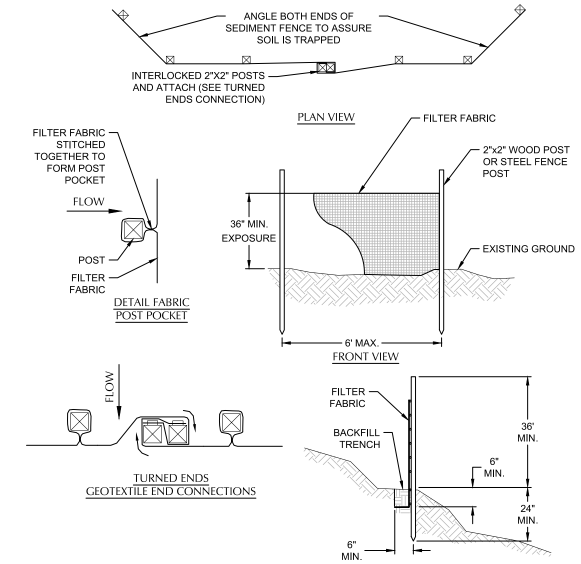


NOTES:

1. THE ENTRANCE SHALL BE MAINTAINED IN A CONDITION THAT WILL PREVENT TRACKING OR FLOWING OF SEDIMENT ONTO PUBLIC RIGHT-OF-WAYS. THIS MAY REQUIRE TOP DRESSING, REPAIR AND/OR CLEAN OUT OF ANY MEASURES USED TO TRAP SEDIMENT.
2. WHEN NECESSARY, WHEELS SHALL BE CLEANED PRIOR TO ENTRANCE ONTO PUBLIC RIGHT-OF-WAY.
3. WHEN WASHING IS REQUIRED, IT SHALL BE DONE ON AN AREA STABILIZED WITH CRUSHED STONE THAT DRAINS INTO AN APPROVED SEDIMENT TRAP OR SEDIMENT BASIN.
4. WHERE RUNOFF CONTAINING SEDIMENT LADEN WATER IS LEAVING THE SITE VIA THE CONSTRUCTION ENTRANCE, OTHER MEASURES SHALL BE IMPLEMENTED TO DIVERT RUNOFF THROUGH AN APPROVED FILTERING SYSTEM.
5. DIMENSIONS: 50' LONG BY 20' WIDE 3-6" CLEAN ROCK, GOVERNING AUTHORITY MAY REQUIRE GEOTEXTILE FABRIC TO PREVENT SUB-SOIL PUMPING.

4 CONSTRUCTION ENTRANCE

SCALE: NTS



NOTES:

1. THE FILTER FABRIC SHALL BE (36" MIN. WIDTH) PURCHASED IN A CONTINUOUS ROLL CUT TO THE LENGTH OF THE BARRIER TO AVOID USE OF JOINTS. WHEN JOINTS ARE NECESSARY, FILTER CLOTH SHALL BE SPLICED TOGETHER ONLY AT A SUPPORT POST, WITH A MINIMUM 6-INCH OVERLAP, AND BOTH ENDS SECURELY FASTENED TO THE POST, OR OVERLAP 2"x2" POSTS AND ATTACH AS SHOWN ON DETAIL SHEET.

2. THE FILTER FABRIC FENCE SHALL BE INSTALLED TO FOLLOW THE CONTOURS WHERE FEASIBLE. THE FENCE POSTS SHALL BE SPACED A MAXIMUM OF 6-FEET APART AND DRIVEN SECURELY INTO THE GROUND A MINIMUM OF 24-INCHES.

3. THE FILTER FABRIC SHALL HAVE A MINIMUM VERTICAL BURIAL OF 6-INCHES. ALL EXCAVATED MATERIAL FROM FILTER FABRIC FENCE INSTALLATION, SHALL BE BACKFILLED AND COMPACTED, ALONG THE ENTIRE DISTURBED AREA.

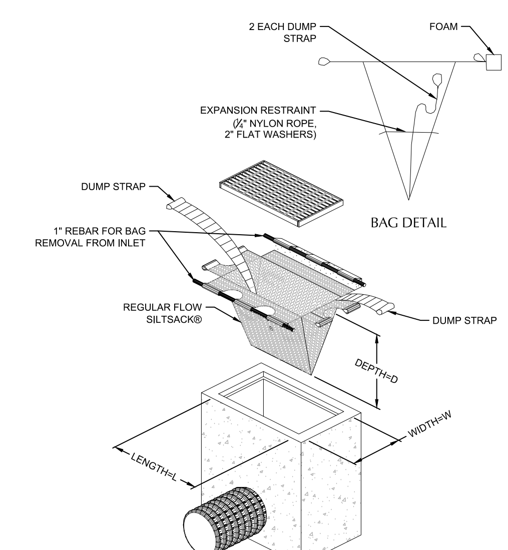
4. STANDARD OR HEAVY DUTY FILTER FABRIC SHALL HAVE MANUFACTURED STITCHED LOOPS FOR 2"x2" POST INSTALLATION. STITCHED LOOPS WITH STAKES SHALL BE INSTALLED ON THE DOWN-HILL SIDE OF THE SLOPED AREA.

5. FILTER FABRIC FENCES SHALL BE REMOVED WHEN THEY HAVE SERVED THEIR USEFUL PURPOSE, BUT NOT BEFORE THE UP-SLOPE AREA HAS BEEN PERMANENTLY PROTECTED AND STABILIZED.

6. FILTER FABRIC FENCES SHALL BE INSPECTED BY CONTRACTOR IMMEDIATELY AFTER EACH RAINFALL AND AT LEAST DAILY DURING PROLONGED RAINFALL. ANY REQUIRED REPAIRS SHALL BE MADE IMMEDIATELY.

1 SEDIMENT FENCE

SCALE: NTS



NOTE:

1. DO NOT USE HIGH FLOW INSERT BAGS.

2 INLET SEDIMENT PROTECTION

SCALE: NTS

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EROSION AND SEDIMENT CONTROL DETAILS

Sheet No.
C7.0

Job No.
4773-01

Appendix E

Operations and Maintenance Manual (In final version of Report)

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