Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Report

Frog Pond Cottage Park Place

7252 SW Frog Pond Lane
Wilsonville, Oregon

GeoPacific Engineering, Inc. Project No. 22-6060
November 14, 2023

City of Wilsonville
Exhibit B4 DB23-0004

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November 14, 2023
Project No. 22-6060

Brian Matteoni
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San Jose, CA 95138
Via email: Brian.Matteoni@cbre.com

SUBJECT: GEOTECHNICAL ENGINEERING REPORT
FROG POND COTTAGE PARK PLACE
7252 SW FROG POND LANE
WILSONVILLE, OREGON

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, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-8087, dated May 4, 2022, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

Site Location:	7252 SW Frog Pond Lane Wilsonville, Oregon (Figures 1 and 2)
Developer:	Sullivan Homes 5832 Firestone Court San Jose, CA 95138 Phone: (408) 453-7404
Jurisdictional Agency:	City of Wilsonville, Oregon
Civil Engineer:	AKS Engineering & Forestry, LLC 12965 SW Herman Road, Unit 100 Tualatin, Oregon 97062 Tel (503) 563-6151

2.0 SITE AND PROJECT DESCRIPTION

The subject site is approximately 5 acres in size and located on the south side of SW Frog Pond Lane in the City of Wilsonville, Clackamas County, Oregon (Figure 1). Topography is gently sloping to the west with grades of 5 percent or less. The site is currently occupied by one home and two outbuildings. Vegetation consists primarily of short grasses and sparse trees.

It is our understanding that the site will be developed for 34 lots for single family detached and attached townhomes, new streets, water quality facilities, open space, and associated underground utilities (Figure 2). A grading plan has not been provided for our review; however, we anticipate maximum cuts and fill may be up to 4 feet.

3.0 REGIONAL GEOLOGIC SETTING

The project site is located on the southwestern margin of the Portland West Hills, in the northwest portion of the Tualatin Basin. The Tualatin Basin is an east/west trending structural feature produced by broad regional down warping of the area. Regionally, the subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

The site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley (Yeats et al., 1996; Gannett and Caldwell, 1998). The last of these outburst floods occurred about 10,000 years ago. These deposits typically consist of horizontally layered, micaceous, silt to coarse sand forming poorly-defined to distinct beds less than 3 feet thick.

The Willamette Formation is underlain by the Miocene age (about 14.5 to 16.5 million years ago) Columbia River Basalt Formation, a thick sequence of lava flows that form the crystalline bedrock of Tualatin Valley (Yeats et al., 1996; Gannett and Caldwell, 1998). These basalts are dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. Individual basalt flow units typically range from 25 to 125 feet thick and interflow zones are typically vesicular, scoriaceous, and brecciated, and sometimes include sedimentary rocks. Typically, the upper portion of the basalt is deeply weathered and decomposed to a residual soil consisting of red-brown, clayey silt.

4.0 REGIONAL SEISMIC SETTING

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

4.1 Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills and is approximately 9.7 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is approximately 8.7 miles northeast of the site. The East Bank Fault occurs along the eastern margin of the Willamette River and is located approximately 14.4 miles northeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000). No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

4.2 Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies approximately 10.3 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A recent geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek or Newberg Faults (the faults closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

4.3 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 site-specific explorations for this report were conducted on June 10, 2022. Six exploratory test pits (designated TP-1 through TP-6) were excavated with a medium sized backhoe to depths ranging between 11.5 and 12.5 feet at the approximate locations presented on Figure 2. 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.

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

5.1 Soil Descriptions

Undocumented Fill: Undocumented fill was not encountered in our explorations conducted for this study. We anticipate that areas of undocumented fill may be present outside our test pit locations – especially in the vicinity of the existing structures, driveways, and along the frontage of SW Frog Pond Lane.

Topsoil Horizon: The ground surface in test pits TP-1 through TP-6 was directly underlain by a moderately to highly organic topsoil horizon. The topsoil horizon consisted of brown to dark grayish brown silt (OL-ML) that was loose and contained fine roots throughout. In test pits TP-1 through TP-6, the topsoil extended to a depth of 8 to 15 inches.

Willamette Formation: Underlying the topsoil horizon in explorations was clayey silt (ML) belonging to the Willamette Formation. The light brown, clayey silt was generally stiff to very stiff; however, near surface soils in test pits TP-2 and TP-5 had a soft to medium stiff consistency. The clayey silt transitioned to silt below a depth of 2.5 to 6 feet in test pits. The silt transitioned to silt with sand below a depth of 8 to 9 feet in test pits TP-1 through TP-3 and TP-5 and to sandy silt below a depth of 9 feet in test pit TP-4. In test pits, material belonging to the Willamette Formation extended beyond the maximum depth of exploration (11.5 to 12.5 feet).

5.2 Groundwater and Soil Moisture

On June 10, 2022, observed soil moisture conditions were generally moist to wet. Groundwater seepage was encountered in test pits TP-1, TP-2, TP-4, and TP-5 at depths of 2 to 10.5 feet. Discharge was visually estimated at ½ to 2 gallons per minute. Our review of nearby water well logs indicate that static groundwater is present at a depth of approximately 40 to 60 feet below the native ground surface (Oregon Water Resources Department, 2023). 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. If the seasonal fluctuation of the static groundwater table underlying the subject site require detailed understanding, piezometers may be installed and periodically monitored.

6.0 INFILTRATION TESTING

Soil infiltration testing was performed using the pushed pipe infiltration method in test pits TP-1 through TP-4. Soil in the test pits was pre-saturated for a period of over 2 hours. The water level was measured to the nearest tenth of an inch every fifteen minutes to half hour with reference to the ground surface. Falling head infiltration testing continued until rates stabilized. Table 1 presents the results of our falling head infiltration tests.

Table 1. Summary of Infiltration Test Results

Test Pit	Depth (feet)	Soil Type	Infiltration Rate (in/hr)	Hydraulic Head Range (inches)
TP-1	5	Silt (ML)	0	37-38
TP-2	8	Silt (ML)	0	21-22
TP-3	4	Clayey Silt (ML)	0	11-12
TP-4	7	Silt (ML)	0.1	19-20

Due to the presence of fine grained soil conditions, it is our opinion that the site is not suitable for infiltration.

7.0 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. Our explorations indicate the native soils on site are generally stiff to very stiff and are suitable for development utilizing conventional spread footing foundations. The primary geotechnical conditions detrimental to development include:

1. Soft to medium stiff, near surface soils were encountered in test pits TP-2 and TP-5 in the upper 2 to 3 feet. Additional depths of excavation for subgrade preparation and foundations may be required in areas.

2. Shallow, perched groundwater conditions that could make utility trenching difficult, especially in the winter months. Minor caving of the test pit sidewalls was observed in test pit TP-1, TP-2, and TP-5. Adequate shoring (and dewatering) should be maintained.

3. Low permeability soils. Our infiltration testing indicates on site, fine grained soils are not suitable for infiltration of stormwater.

7.1 Site Preparation Recommendations

Areas of proposed construction and areas to receive fill should be cleared of any organic and inorganic debris. Inorganic debris and organic materials from clearing should be removed from the site. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. Depth of stripping of existing topsoil is estimated to be approximately 6 to 9 inches across the majority of the site, however depth of organic soil layers may increase in areas. The final depth of soil removal will be determined because of a site inspection after the stripping/excavation has been performed. Stripped topsoil should be removed from areas proposed for placement of engineered fill. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

If encountered, undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, field drain tiles, etc.) should be completely removed and the excavations backfilled with engineered fill. Field drain tiles should be intercepted at the high end of the site and routed to the storm drain system.

We recommend that areas proposed for placement of engineered fill are scarified to a minimum depth of 12 inches and recompacted prior to placement of structural fill. Prior to placement of engineered fill, the underlying soils be over-excavated, ripped, aerated to optimum moisture content, and recompacted to project specifications for engineered fill as determined by the Modified Proctor (ASTM D1557).

Areas proposed to be left at grade may require additional over-excavation of structural areas in order to reach soils which will provide adequate bearing support for the proposed structures. Site earthwork may be impacted by shallow groundwater. 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.

7.2 Engineered Fill

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

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 2015 International Building Code (IBC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in the *Site Preparation Recommendations* section. 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.

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 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.

7.3 Excavating Conditions and Utility Trench Backfill

Subsurface test pit exploration indicates that, in general, utility trenches can be excavated using conventional heavy equipment such as dozers and trackhoes. Shallow, perched groundwater conditions could cause sidewall caving in excavations and moderate caving was observed in test pits TP-1, TP-2, and TP-4. These conditions could make utility trenching difficult, especially in the winter months, and adequate shoring should be maintained.

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

Shallow, perched groundwater and saturated soils may be encountered during the wet weather season and should be anticipated in excavations and utility trenches. We anticipate that dewatering systems consisting of ditches, sumps and pumps would be adequate for control of perched groundwater. Regardless of the dewatering system used, it should be installed and operated such that in-place soils are prevented from being removed along with the groundwater. Trench bottom stabilization, such as one to two feet of compacted crushed aggregate base, may be necessary in deeper 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 and City of Wilsonville standards. We recommend that the upper 4 feet of structural trench backfill be compacted to at least 95 percent of the maximum dry density obtained by the Modified Proctor (ASTM D698) or equivalent. Structural trench backfill below 4 feet should be compacted to at least 90 percent of the maximum dry density obtained by the Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a ¾”-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

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

7.4 Erosion Control Considerations

During our field exploration program, we did not observe soil conditions that are considered to be 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.

7.5 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.
- ➢ 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.

7.6 Spread Foundations

Based upon communication with the client and review of project plans (Figure 2), GeoPacific understands that site development will consist of a subdivision for 34 detached single family and attached townhome lots, new public streets, and associated underground utility installations. It is our understanding that the homes will be constructed with typical spread foundations and crawl spaces. We anticipate that maximum structural loading on column footings and continuous strip footings of the homes will be on the order of 10 to 35 kips, and 4 kips/ft respectively.

The proposed residential structures may likely be supported on shallow foundations bearing on competent undisturbed, low to moderately expansive native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Soft to medium stiff native silt soils were encountered in the upper 2 to 3 feet in test pits TP-2 and TP-5, which were located in the southern portion of the site. Additional depths of excavation for subgrade preparation and foundations may be required in areas. Areas where homes are to be constructed where no engineered fill will be placed should either be prepared as recommended for roadway areas; or the foundation envelopes of the proposed homes should be over-excavated to expose native soils on a lot by lot basis. (See *Site Preparation Recommendations* section).

Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. If soft soil conditions are encountered at footing subgrade elevation, they should be removed and replaced with compacted crushed aggregate.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For loads heavier than 35 kips, the geotechnical engineer should be consulted. If heavier loads than described above are proposed, it may be necessary to over-excavate point load areas and replace with additional compacted crushed aggregate. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

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

Our recommendations are for residential construction incorporating raised wood floors and conventional spread footing foundations. After site development, a Final Soil Engineer's Report should either confirm or modify the above recommendations.

7.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation Recommendations* section. 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.

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 the medium stiff, 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 95 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. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. 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.

7.8 Footing and Roof Drains

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

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

If the proposed structure will have a raised floor, and no concrete slab-on-grade floors are used, perimeter footing drains may be eliminated at the discretion of the geotechnical engineer based on soil conditions encountered at the site and experience with standard local construction practices. Where it is desired to reduce the potential for moist crawl spaces, footing drains may be installed. If concrete slab-on-grade floors are used, perimeter footing drains should be installed as recommended below.

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

7.9 Permanent Below-Grade Foundation Walls

Lateral earth pressures against below-grade foundation 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 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 300 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

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

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

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

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

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

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

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

7.10 Pavement Design

For design purposes, we used an estimated resilient modulus of 6,000 for compacted native soil. Table 2 presents our recommended minimum pavement section for dry weather construction.

Table 2. Recommended Minimum Dry-Weather Pavement Section

Material Layer	Light-duty Public Streets	Compaction Standard
Asphaltic Concrete (AC)	3 in.	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾”-0 (leveling course)	4 in.	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1½”-0	10 in.	95% of Modified Proctor AASHTO T-180
Subgrade	12 in.	95% of Standard Proctor AASHTO T-99

Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill (see *Site Preparation* Section). In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving. If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project.

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

7.11 Wet Weather Construction Pavement Section

This section presents our recommendations for wet weather pavement section 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 City of Wilsonville 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 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.

In some instances, it may be preferable to use Special Treated Base (STB) in combination with over-excavation and increasing the thickness of the rock section. GeoPacific should be consulted for additional recommendations regarding use of STB in wet weather pavement sections if it is desired to pursue this alternative. Cement treatment of the subgrade may also be considered instead of over-excavation. For planning purposes, we anticipate that treatment of the onsite soils would involve mixing cement powder to approximately 6 percent cement content and a mixing depth on the order of 12 to 18 inches.

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 risky 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 pavement materials.

8.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (Dogami), Oregon HazVu: 2023 Statewide GeoHazards Viewer indicates that the site is in an area where *severe* ground shaking is anticipated during an earthquake. Single family structures should be designed to resist earthquake loading in accordance with the methodology described in the 2021 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2022). We recommend Site Class D be used for design as defined in ASCE 7, Chapter 20, Table 20.3-1. Design values determined for the site using the Applied Technology Council (ATC) *ASCE7-16 Hazards By Location Online Tool* are summarized in Table 3 and are based upon existing soil conditions.

Table 3. Recommended Earthquake Ground Motion Parameters (ATC 2022)

Parameter	Value
Location (Lat, Long), degrees	45.321, -122.751
Mapped Spectral Acceleration Values (MCE):	
Peak Ground Acceleration PGA_M	0.458
Short Period, S_s	0.82 g
1.0 Sec Period, S_1	0.381 g
Soil Factors for Site Class D:	
F_a	1.172
F_v	*1.919
$SD_s = 2/3 \times F_a \times S_s$	0.641 g
$SD_1 = 2/3 \times F_v \times S_1$	*0.487 g
Residential Seismic Design Category	D

* The F_v value reported in the above table is a straight-line interpolation of mapped spectral response acceleration at 1-second period, S_1 per Table 1613.2.3(2) of OSSC 2019 with the assumption that Exception 2 of ASCE 7-16 Chapter 11.4.8 is met. SD_1 is based on the F_v value. The structural engineer should evaluate exception 2 and determine whether or not the exception is met. If Exception 2 is not met, and the long-period site coefficient (F_v) is required for design, GeoPacific Engineering can be consulted to provide a site-specific procedure as per ASCE 7-16, Chapter 21.

8.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2023 Statewide GeoHazards Viewer indicates that the site is in an area considered to be at *low* to *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 is generally limited to loose, sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15.

For construction of single family structures or townhomes three stories or less, special design or construction measures are not required by code to mitigate the effects of liquefaction. However, GeoPacific may be consulted to perform further study of seismic hazards on the site if desired. We anticipate that our additional explorations on the site for the purpose of evaluating seismic hazards would include at least two cone penetrometer tests.

9.0 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. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

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

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

CERTIFIED
OREGON
ELIZABETH K. RAPP
No. E2190
ENGINEERING GEOLOGIST
exp 2/1/24

REGISTERED PROFESSIONAL ENGINEER
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OREGON
JAN. 23, 1990
JAMES D. IMBRIE
EXPIRES: 06/30/2025

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

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

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CHECKLIST OF RECOMMENDED GEOTECHNICAL TESTING AND OBSERVATION

Item No.	Procedure	Timing	By Whom	Done
1	Preconstruction meeting	Prior to beginning site work	Contractor, Developer, Civil and Geotechnical Engineers	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Soil Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Soil Technician	
4	Compaction testing of engineered fill (90% of Modified Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Retaining Wall Keyway and Subbase	During Excavation	Soil Technician/ Geotechnical Engineer	
6	Retaining Wall Backfill and Geogrid Placement	During Construction	Soil Technician/ Geotechnical Engineer	
7	Compaction testing of trench backfill (95% of Modified Proctor above 4 feet - 90% of Modified Proctor below 4 feet)	During backfilling, tested every 4 vertical feet for every 200 linear feet	Soil Technician	
8	Street Subgrade Inspection (95% of Standard Proctor)	Prior to placing base course	Soil Technician	
9	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 linear feet	Soil Technician	
10	Asphalt Compaction (92% Rice Value)	During paving, tested every 100 linear feet	Soil Technician	
11	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

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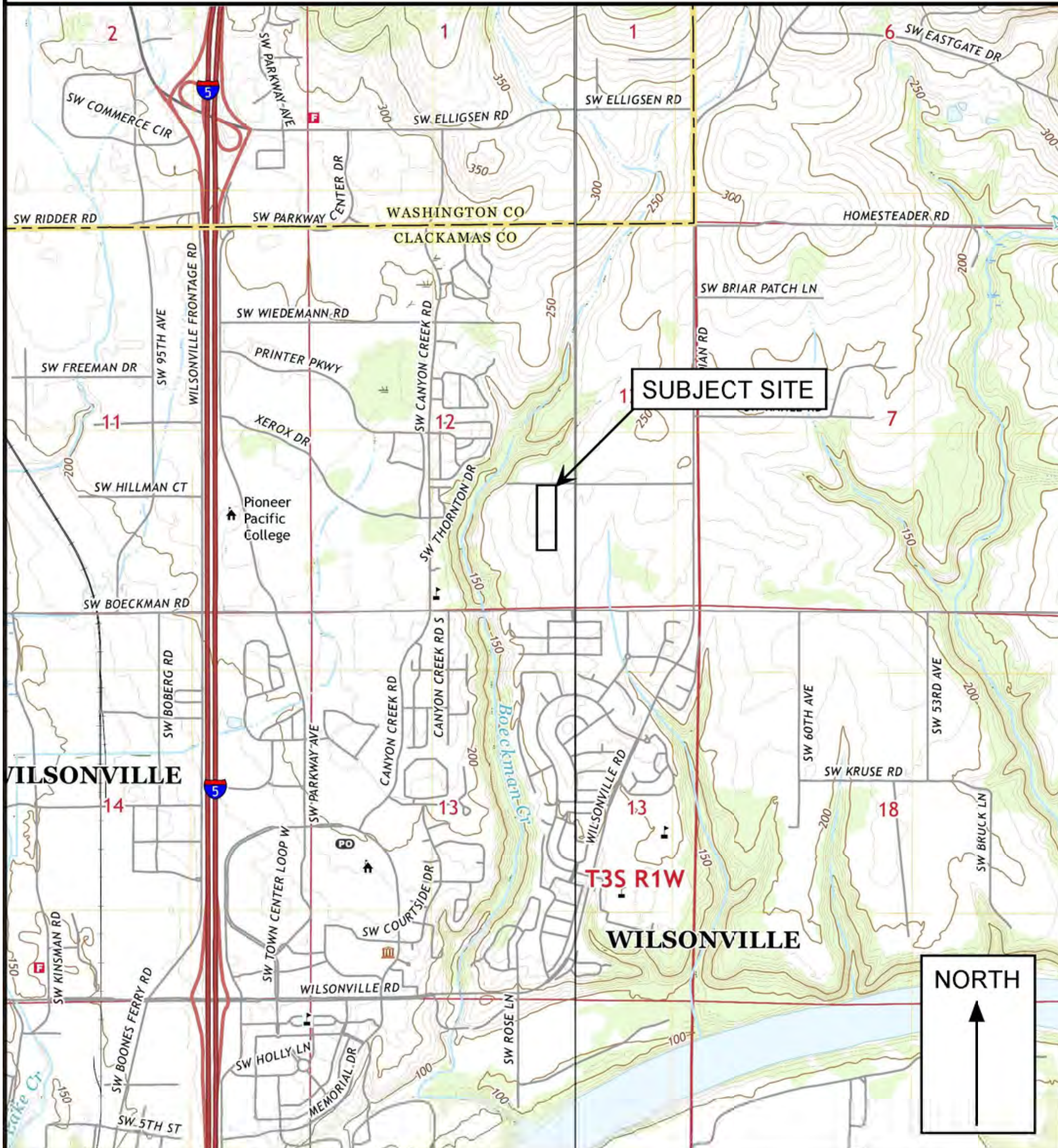
VICINITY MAP



Legend

Approximate Scale 1 in = 2,000 feet

Date: 11/14/2023
Drawn by: EKR

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

Project: Frog Pond Cottage Park Place Wilsonvile, Oregon	Project No. 22-6060	FIGURE 1

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SITE PLAN AND EXPLORATION LOCATIONS

Legend

TP-1
Test Pit Designation and Approximate Location

Date: 11/14/2023
Drawn by: EKR

0 150'
APPROXIMATE SCALE 1"=150'

Project: Frog Pond Cottage Park Place Wilsonville, Oregon	Project No. 22-6060	FIGURE 2

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TEST PIT LOG

Project: Frog Pond Cottage Park Place, Wilsonville, Oregon	Project No. 22-6060	Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1	1.5					Moderately organic SILT (OL-ML), dark gray brown, fine roots throughout, loose, wet (Topsoil Horizon)
1–5	2.5, 2.5, 2.0				Seepage (2.5–3.5 ft)	Stiff to very stiff, clayey SILT (ML), light brown, micaceous, trace black staining, strong orange and gray mottling, minor caving of sidewall, moist to wet (Willamette Formation)
5–12.5						Stiff, SILT (ML), light brown, micaceous, subtle orange and gray mottling, moist (Willamette Formation) With trace fine grained sand below 8 feet.

Test Pit Terminated at 12.5 Feet.

Note: Groundwater seepage encountered at 2.5 to 3.5 feet.
Discharge visually estimated at 2 gallons per minute.

LEGEND

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

Date Excavated: 6/10/2022
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: Frog Pond Cottage Park Place, Wilsonville, Oregon	Project No. 22-6060	Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1	0.5 (1 ft)					Highly organic SILT (OL-ML), dark brown, fine roots throughout, 3 to 4 inch thick root mat, loose, moist (Topsoil Horizon)
1–2.5	0.5 (2 ft)				Seepage (2–2.5 ft)	Soft to medium stiff, clayey SILT (ML), light brown, micaceous, trace black staining, strong orange and gray mottling, wet (Willamette Formation)
2.5–12.5	3.5 (3 ft); 2.5 (4 ft)					Stiff to very stiff, SILT (ML), light brown, micaceous, subtle orange and gray mottling, minor sidewall caving below 5 feet, moist (Willamette Formation) With trace fine grained sand and strong mottling below 9 feet.

Test Pit Terminated at 12.5 Feet.

Note: Groundwater seepage encountered at 2 to 2.5 feet.
Discharge visually estimated at 1 gallon per minute.

LEGEND

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

Date Excavated: 6/10/2022
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: Frog Pond Cottage Park Place Wilsonville, Oregon	Project No. 22-6060	Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.9						Moderately organic SILT (OL-ML), dark brown, fine roots throughout, 3 to 4 inch thick root mat, loose, moist (Topsoil Horizon)
1	1.0					
2	2.5					Stiff to very stiff, clayey SILT (ML), light brown, micaceous, trace roots to 2.5 feet, strong orange and gray mottling, moist (Willamette Formation)
3	4.0					
4	3.0					
5						
6						Stiff, SILT (ML), light brown, micaceous, subtle orange and gray mottling, trace black staining, moist (Willamette Formation)
7						
8						
9						With trace fine grained sand below 9 feet.
10						
11						
12						
13						Test Pit Terminated at 12.5 Feet.
14						Note: No seepage or groundwater encountered.
15						
16						
17						

LEGEND

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

Date Excavated: 6/10/2022
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: Frog Pond Cottage Park Place Wilsonville, Oregon	Project No. 22-6060	Test Pit No. **TP-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–~0.8						Moderately organic SILT (OL-ML), brown, abundant roots throughout, loose, moist (Topsoil Horizon)
1	2.5					Stiff to very stiff, clayey SILT (ML), light brown, micaceous, trace roots to 3.5 feet, trace black staining, strong orange and gray mottling, moist (Willamette Formation)
2	3.5					
3	4.0					
4	4.5					
5						
6						Stiff to very stiff, SILT (ML), light brown, micaceous, subtle orange and gray mottling, moist (Willamette Formation)
7						
8						
9						Stiff, sandy SILT (ML), light brown, micaceous, subtle orange and gray mottling, minor sidewall caving, moist to wet (Willamette Formation)
10					Seepage	
11						
12–17						Test Pit Terminated at 11.5 Feet. Note: Groundwater seepage encountered at 10 to 10.5 feet. Discharge visually estimated at 1 gallon per minute.

LEGEND

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

Date Excavated: 6/10/2022
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: Frog Pond Cottage Park Place Wilsonville, Oregon	Project No. 22-6060	Test Pit No. **TP-5**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Water Bearing Zone	Material Description
0–1						Moderately to highly organic SILT (OL-ML), dark brown, fine roots throughout, loose, moist (Topsoil Horizon)
1	1.0					
2	1.0				Seepage	Medium stiff, clayey SILT (ML), light brown, micaceous, trace black staining, strong orange and gray mottling, moist to wet (Willamette Formation)
3	2.0					
4	2.5					
5						Stiff to very stiff, SILT (ML), light brown, micaceous, subtle orange and gray mottling, trace black staining, moist (Willamette Formation)
6						
7						
8						With trace fine grained sand below 8 feet.
9						
10						
11						
12						
13						Test Pit Terminated at 12.5 Feet.
14						Note: Groundwater seepage encountered at 2 to 2.5 feet. Discharge visually estimated at 1/2 gallon per minute.
15						
16						
17						

LEGEND

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

Date Excavated: 6/10/2022
Logged By: B. Rapp
Surface Elevation:

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TEST PIT LOG

Project: Frog Pond Cottage Park Place Wilsonville, Oregon	Project No. 22-6060	Test Pit No. **TP-6**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.7						Moderately to highly organic SILT (OL-ML), dark brown, fine roots throughout, loose, moist (Topsoil Horizon)
1	2.5					
2	3.0					Stiff to very stiff, clayey SILT (ML), light brown, micaceous, trace roots, strong orange and gray mottling, moist (Willamette Formation)
3	3.0					
4	3.0					
5						
6						
7						Stiff, SILT (ML), light brown, micaceous, subtle orange and gray mottling, trace black staining, moist (Willamette Formation)
8						
9						
10						
11						
12						Test Pit Terminated at 12 Feet. Note: No seepage or groundwater encountered.
13						
14						
15						
16						
17						

LEGEND

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

Date Excavated: 6/10/2022
Logged By: B. Rapp
Surface Elevation:

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MAINTENANCE OF HILLSIDE HOMESITES AND SLOPES

All homes and slopes require a certain level of maintenance for general upkeep and to preserve the overall integrity of structures and land. Hillside homesites and slopes require some additional maintenance because they are subject to natural slope processes, such as runoff, erosion, shallow soil sloughing, soil creep, perched groundwater, etc. If not properly controlled, these processes could adversely affect your or neighboring properties. Although surface processes are usually only capable of causing minor damage, if left unattended, they could possibly lead to more serious instability problems. Slumps are common and unpredictable and should be considered part of standard slope maintenance.

The primary source of problems on hillsides is uncontrolled surface water runoff and blocked groundwater seepage which can erode, saturate, and weaken soil. Therefore, it is important that drainage and erosion control features be implemented on the property, and that these features be maintained in operative condition (unless changed on the basis of qualified professional advice). By employing simple precautions, you can help properly maintain your hillside site and avoid most potential problems. The following is an abbreviated list of common Do's and Don'ts recommended for maintaining hillside homesites and slopes – including those within open spaces.

Do List

1. Make sure that roof rain drains are connected to the street, local storm drain system, or transported via enclosed conduits or lined ditches to suitable discharge points away from structures and improvements. In no case, should rain drain water be discharged onto slopes or in an uncontrolled manner. Energy dissipation devices should be employed at discharge points to help prevent erosion.
2. Check your roof drains, gutters, and spouts to make sure that they are clear. Roofs are capable of producing a substantial flow of water. Blocked gutters, etc., can cause water to pond or run off in such a way that erosion or adverse oversaturation of soil can occur.
3. Make sure that drainage ditches and/or berms are kept clear throughout the rainy season. If you notice that a neighbor's ditches are blocked such that water is directed onto your property or in an uncontrolled manner, politely inform them of this condition.
4. Locate and check all drain inlets, outlets, and weep holes from foundation footings, retaining walls, driveways, etc. on a regular basis. Clean out any of these that have become clogged with debris.
5. Watch for wet spots on the property. These may be caused by natural seepage or indicate a broken or leaking water or sewer line. In either event, professional advice regarding the problem should be obtained followed by corrective action, if necessary.
6. Do maintain the ground surface adjacent to lined ditches so that surface water is collected in the ditch. Water should not be allowed to collect behind or flow under the lining.

Don't List

1. Do not change the grading or drainage ditches on the property without professional advice. You could adversely alter the drainage pattern across the site and cause erosion or soil movement.
2. Do not allow water to pond on the property. Such water will seep into the ground causing unwanted saturation of soil.
3. Do not allow water to flow onto slopes in an uncontrolled manner. Once erosion or oversaturation occurs, damage can result quickly or without warning.
4. Do not let water pond against foundations, retaining walls or basements. Such walls are typically designed for fully-drained conditions.
5. Do not connect roof drainage to subsurface disposal systems unless approved by a geotechnical engineer.
6. Do not irrigate in an unreasonable or excessive manner. Regularly check irrigation systems for leaks. Drip systems are preferred on hillsides.

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