Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Report

Project Information:	Chinn Property Subdivision GeoPacific Project No. 22-6032 April 20, 2022
Site Location:	SW Murdock Road and SW Mckinley Drive Sherwood, Oregon 97140 Washington County Parcel Number 2S133CB 00600
Client:	Tim Roth JT Roth Construction, Inc. 12600 SW 72nd Ave. Suite 200 Portland, OR 97223 Phone: (503) 639-2639 Email: timr@jtrothinc.com

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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 geologic hazards at the property, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-8058, dated March 28, 2022, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located on the northeast corner of the intersection between McKinley Drive and SW Murdock Road in Sherwood Oregon and consists of Washington County Parcel Number 2S133CB 00600, totaling approximately 3.01 acres in size. Access to the property is provided by a driveway from SW Murdock Road just to the north of the site. Topography onsite is moderately sloping down to the west with site elevations ranging from 300 to 364 feet amsl. The property is currently undeveloped. Vegetation consists primarily of short grasses and medium to large trees throughout the property.

GeoPacific understands that the proposed site development will consist of construction of a new subdivision, with associated new public alleys and streets, private parking areas, stormwater management facilities, and installation of new underground utilities. Based on communication with the client, a retaining wall may be planned for the site. A grading plan has not yet been provided for our review. However, we anticipate that cuts and fills will be on the order of 15 feet or less.

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 site is situated on an uplifted structural block of Columbia River Basalt (Schlicker and Finlayson, 1979). The Columbia River Basalt formation is differentiated into several members. The basalt underlying the subject site is part of the Wanapum Basalt member, which is typically dark gray to black and displays blocky to columnar jointing (Burns et al, 1997). Interflow zones between flows are typically vesicular, scoriaceous, and brecciated, and sometimes include sedimentary rocks. Where highly weathered, the upper portion of the basalt is altered to a distinctive red-brown clayey silt known as laterite or residual soil.

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 reportedly 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 located approximately 13.4 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is located approximately 9.6 miles northeast of the site. The East Bank Fault occurs along the eastern margin of the Willamette River, and is located approximately 14.8 miles northeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000).

According to the USGS Earthquake Hazards Program, the fault was originally mapped as a down-to-the-northeast normal fault but has also been mapped as part of a regional-scale zone of right-lateral, oblique slip faults, and as a steep escarpment caused by asymmetrical folding above a south-west dipping, blind thrust fault. The Portland Hills fault offsets Miocene Columbia River Basalts, and Miocene to Pliocene sedimentary rocks of the Troutdale Formation. No fault scarps on surficial Quaternary deposits have been described along the fault trace, and the fault is mapped as buried by the Pleistocene aged Missoula flood deposits. 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 about 8.0 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault or Newberg Fault; 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.

4.3 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 subsurface explorations for this report were conducted on April 18, 2022. A total of four exploratory test pits (TP-1 through TP-4) were excavated at the site using a Case Extendahoe backhoe excavator with rock teeth provided and operated by Dan Fischer Excavating, to a maximum depth of approximately 12 feet bgs. 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 each test, the test pits were loosely backfilled with onsite soils. The approximate locations of the explorations are indicated 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. Summary exploration logs are attached. The stratigraphic contacts shown on the individual test pit 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: At the locations of our explorations the ground was surfaced primarily with grasses and native plants. A topsoil horizon was observed to be present typically consisting of 8 to 10 inches of organic SILT (OL-ML) containing fine roots.

Residual Soil: Underlying the topsoil layer at the locations of our explorations, we observed Residual Soil. This soil type consisted of reddish brown, medium stiff to stiff, moist, low plasticity, SILT (ML), The Residual Soil extended to depths ranging from 3.0 to 10 feet bgs at the locations of our explorations.

Columbia River Basalt: Underlying the residual soil in all exploration locations, we encountered a zone of weathered basalt which graded into dense, in-tact basalt. All of our test pits were terminated at depths between 5 and 12 feet bgs due to refusal on hard rock. The basalt displayed very soft (R1) to medium hard (R3) consistency.

5.2 Groundwater and Soil Moisture

On April 18, 2022, observed soil moisture conditions were generally moist. Groundwater seepage was not encountered during our explorations, which extended to a maximum depth of 12 feet bgs. Based on our review of available well logs from the vicinity of the subject site, static groundwater is expected at depths of 100 to 250 feet bgs, depending on ground surface elevation. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored and may become evident during site grading.

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 concern associated with development at the site the presence of shallow bedrock where underground utilities are proposed. Bedrock was encountered in our explorations at as shallow as 3.5 feet bgs.

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 Preparation Recommendations

Areas of proposed construction and development, and areas to receive fill should be cleared of any organic and inorganic debris, undocumented fill soils, and/or loose stockpiled soils. 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 average approximately 8 to 10 inches.

The final depth of soil removal will be determined during 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.

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

Areas proposed to be left at grade may require additional over-excavation of foundation areas in order to reach soils which will provide adequate bearing support for the proposed foundations. It is unlikely that site earthwork will be impacted by shallow groundwater, however native soils are moisture sensitive and will be difficult to handle during periods of wet weather. Stabilization of subgrade soils will require aeration and re-compaction. 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.

6.2 Engineered Fill

Where incorporated into the project, 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), and 2019 Oregon Structural Specialty Code (OSSC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in Section 6.1, *Site Preparation Recommendations*. Surface soils should be aerated, 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.

Onsite native soils appear to be suitable for use as engineered fill. 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 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Soils should be moisture conditioned to within two percent of optimum moisture. 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.

6.3 Excavating Conditions and Utility Trench Backfill

Refusal on basaltic bedrock was encountered within our subsurface explorations at depths as shallow as 1.5 feet bgs. Excavation below this depth will likely be difficult. The use of rock chippers and other heavy equipment may be needed for the installation of in-ground utilities.

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 soils in the upper 1.5 to 10 feet 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 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 and City of Hillsboro/Washington County standards. We recommend that structural trench backfill be compacted to at least 95 percent of the maximum dry density obtained by the Standard Proctor (ASTM D698, AASHTO T-99) 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.

6.4 Erosion Control Considerations

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.

6.5 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; 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.

6.6 Spread Foundations

We anticipate that the homes will be constructed with typical spread foundations and wood framing, with maximum structural loading on column footings and continuous strip footings on the order of 10 to 35 kips, and 2 to 4 kips respectively.

The proposed structures may be supported on shallow foundations bearing on stiff, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 inches below exterior grade. 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 column or point 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 to achieve a higher allowable bearing capacity. 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 most 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.

6.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in Section 6.1, *Site Preparation Recommendations* and Section 6.6, *Spread Foundations*. 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 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.

6.8 Footing and Roof Drains

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

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

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. 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.

6.9 Permanent Below-Grade Walls

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

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-rest equivalent fluid pressure of 52 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 a passive earth pressure of 320 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

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

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

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3-inch minimum diameter perforated, plastic drainpipe 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. 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.

6.10 Flexible Pavement Design: Public and Private Drives

We understand that development at the site includes a private parking and driveways throughout the site. For analysis and design purposes, we conservatively assume that the native subgrade and engineered fill soils will exhibit a resilient modulus of 4,500 psi, which correlates to a CBR value of 3.

Based upon our understanding of the anticipated traffic which includes light-duty passenger vehicles, medium sized trucks, delivery and pickup vehicles, and occasional emergency vehicles weighing up to 75,000 lbs and point loads up to 2,500 lbs, we estimate an anticipated 18 kip ESAL count of approximately 60,000 over 20 years. Table 1 presents our flexible pavement design input parameters. Table 2 presents our recommended minimum dry weather pavement section for the proposed pavement sections, supporting 20 years of vehicle traffic per City of Sherwood standards.

Table 1: Pavement Design Input Parameters

Input Parameter	Design Value (40 Years)
18-kip ESAL Initial Performance Period	60,000
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	80 Percent
Overall Standard Deviation	0.5
Roadbed Soil Resilient Modulus (PSI)	4,000

Table 2: Recommended Minimum Dry-Weather Pavement Section (20 Years)

Material Layer	Section Thickness (in.)	Structural Coefficient	Compaction Standard
New Asphalt Concrete – Base Course	3.0	0.42	91%/ 92% of Max Density per AASHTO T-209
New ¾''-0 Crushed Aggregate Base	2.0	0.10	95% of Modified Proctor AASHTO T-180
New 1.5''-0 Crushed Aggregate Subbase	8.0	0.08	95% of Modified Proctor AASHTO T-180
Subgrade		4,000 PSI	95% of Standard Proctor AASHTO T-99 or equivalent

6.11 Subgrade Preparation

Roadway subgrade soils should be compacted and inspected by GeoPacific prior to the placement of crushed aggregate base for pavement. Typically, a proof roll with a fully loaded water, dump truck 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 ripping or tilling should be removed and replaced with engineered fill (see Section 7.1, Site Preparation). 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 roadway 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 subgrade soils observed in our subsurface exploration, though not necessarily susceptible to erosion, may become slick and difficult to work with in wet weather conditions. General recommendations for wet weather pavement sections are provided below.

During placement of roadway section materials, density testing and proof rolls 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 and a final base aggregate proof roll is conducted within a few days of paving.

6.12 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.

In some instances, it may be preferable to use a subbase material in combination with over-excavation and increasing the thickness of the rock section. GeoPacific should be consulted for additional recommendations regarding use of additional subbase 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 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 pavement materials.

7.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 Statewide GeoHazards Viewer indicates that the site is in an area where *very strong* ground shaking is anticipated during an earthquake. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2018 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2019). We recommend Site Class D be used for design as defined in ASCE 7-16, Chapter 20, and Table 20.3-1. Design values determined for the site using the ATC Hazards by Location 2022 Seismic Design Maps Summary Report are summarized in Table 20 and are based upon observed existing soil conditions.

Table 20: Recommended Earthquake Ground Motion Parameters (ASCE 7-16)

Parameter	Value
Location (Lat, Long), degrees	45.350, -122.826
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.463 g
Short Period, S_s	0.830 g
1.0 Sec Period, S_1	0.391 g
Soil Factors for Site Class D:	
F_a	1.168
* F_v	1.909
SD_s = 2/3 x F_a x S_s	0.646 g
*SD_1 = 2/3 x F_v x S_1	0.497 g
Seismic Design Category	D

* 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) with the assumption that Exception 2 of ASCE 7-16 Chapter 11.4.8 is met per the Structural Engineer. 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.

7.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 Statewide GeoHazards Viewer indicates that the site is in an area considered to be at very low 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.

The subsurface profile observed within our subsurface explorations which extended to a maximum depth of 12 feet bgs, indicated that the site is underlain by medium stiff to stiff, low plasticity silt. These fine-grained materials were underlain by bedrock. We did not encounter groundwater seepage to the maximum exploration depth. Based upon the results of our study, it is our opinion that the risk of soil liquefaction during a seismic event is low, and that no special design measures are needed to address liquefaction.

8.0 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.

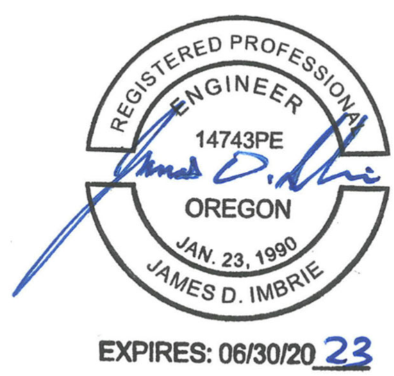
Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the 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



Alexandria B. Campbell, E.I.
Engineering Staff

James D. Imbrie, P.E.
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 (95% of Standard Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Foundation Subgrade Compaction (95% of Modified Proctor)	During Foundation Preparation, Prior to Placement of Reinforcing Steel	Soil Technician/ Geotechnical Engineer	
6	Compaction testing of trench backfill (95% of Standard Proctor)	During backfilling, tested every 4 vertical feet for every 200 linear feet	Soil Technician	
7	Street Subgrade Inspection (95% of Standard Proctor)	Prior to placing base course	Soil Technician	
8	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 linear feet	Soil Technician	
9	Asphalt Compaction (92% Rice Value)	During paving, tested every 100 linear feet	Soil Technician	
10	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

FIGURES

GEOPACIFIC
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Portland, Oregon 97224
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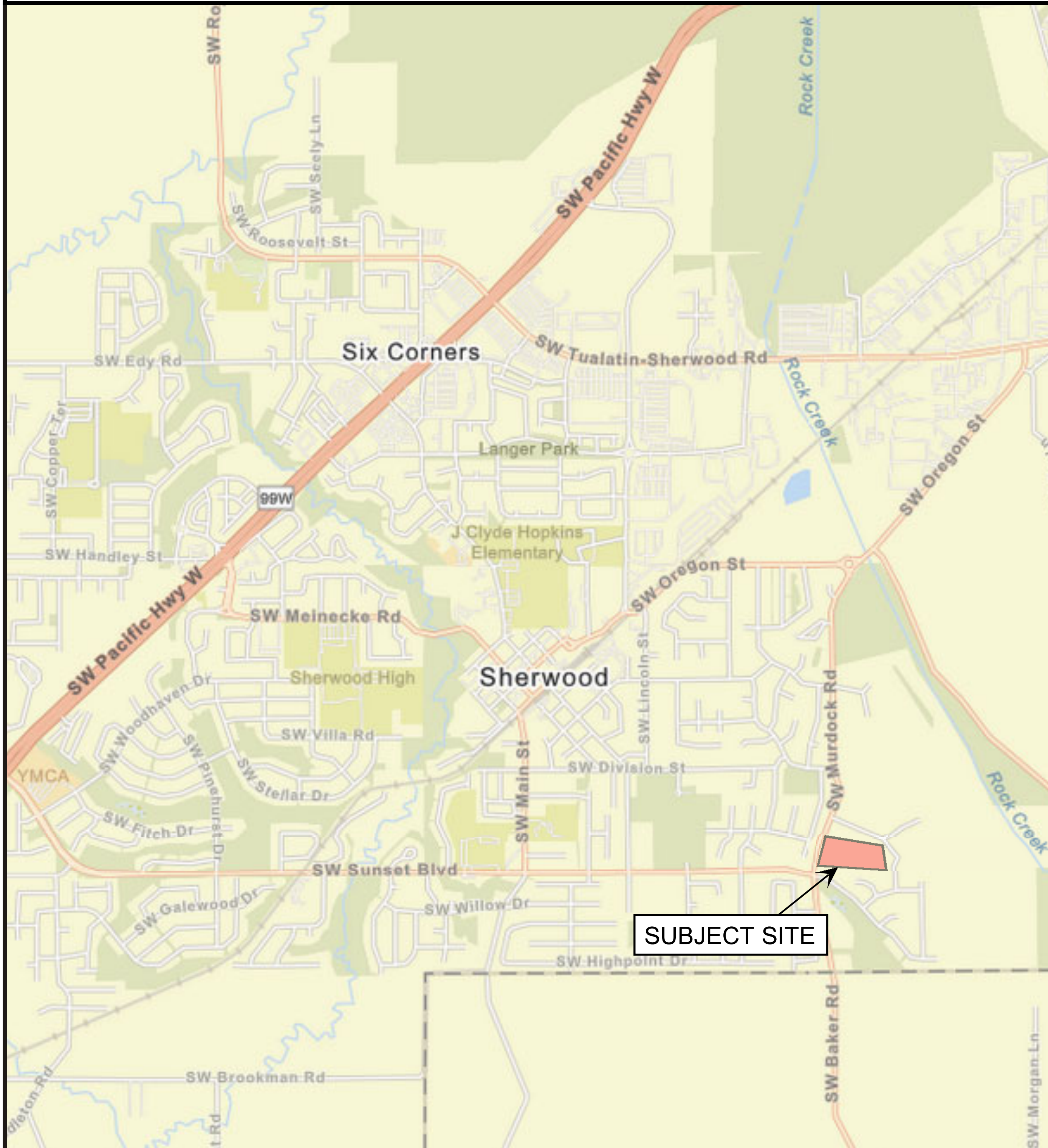
SITE VICINITY MAP



Base map: Dogami Hazvu, USGS, Accessed 04/20/2022
Date: 04/20/2022
Drawn by: ABC

North

Project: Chinn Property Subdivision Sherwood, Oregon	Project No. 22-6032	FIGURE 1

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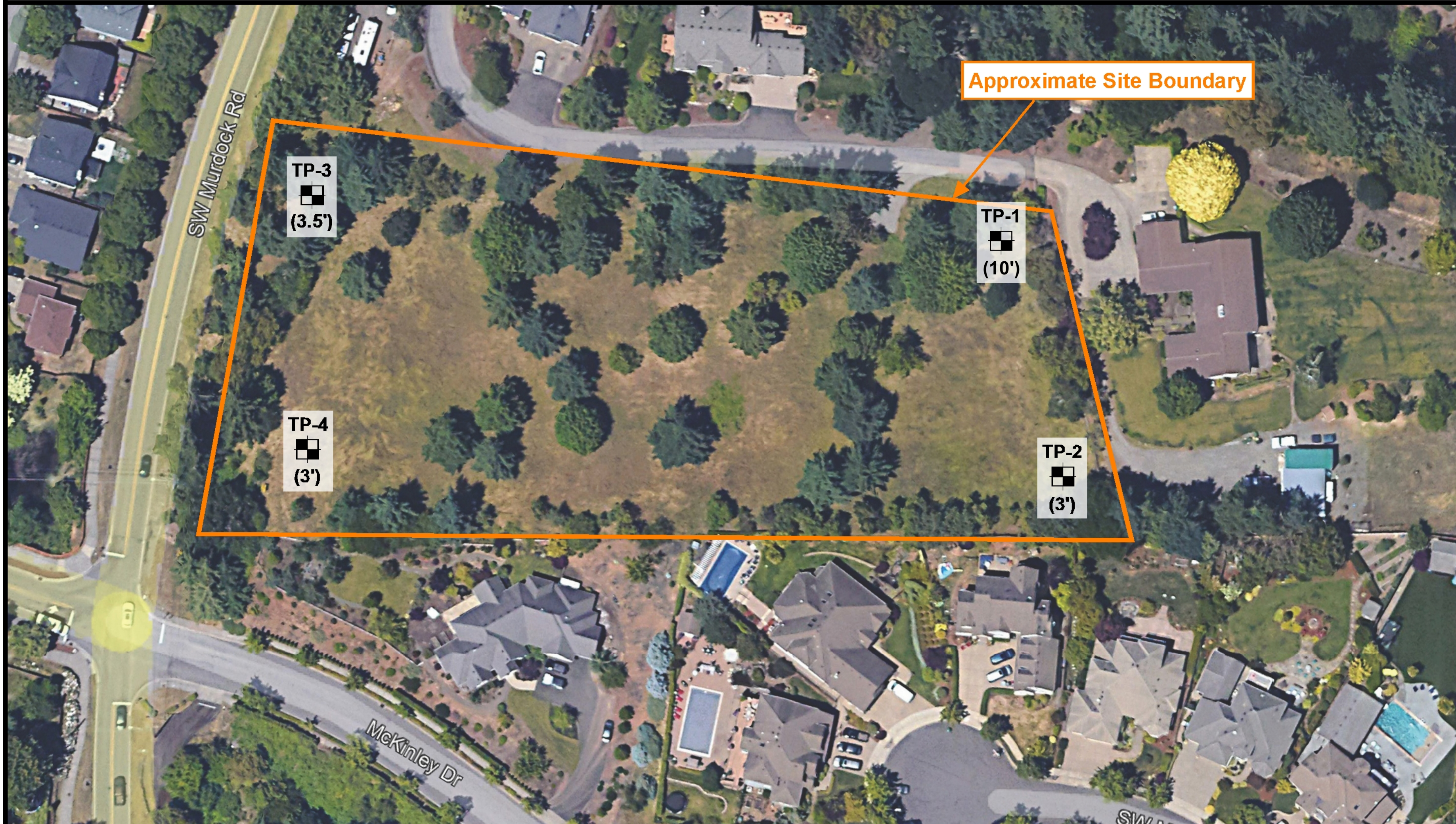
SITE AERIAL AND EXPLORATION LOCATIONS



TP-1
Test Pit Designation and Approximate Location

(3.5') Approximate Depth to Observed Bedrock

APPROXIMATE SCALE
(FEET)
50 0 50 100

Drawn by: ABC
Date: 04/20/2022

North

Project: Chinn Property Subdivision Sherwood, Oregon	Project No. 22-6032	FIGURE 2

EXPLORATION LOGS

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TEST PIT LOG

Project: Chinn Property Subdivision
Sherwood, Oregon

Project No. 22-6032

Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Torvane Shear (tons/ft²)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
							Organic SILT (OL-ML), brown, moist, low plasticity, fine roots extend to approximately 8 inches (Topsoil)
							SILT (ML), brown to reddish brown, medium stiff, moist, low plasticity (Residual Soil)
1	1.5						
2	2.0						Grades to stiff and with weathered rock fragments
3	2.5						Boulders encountered
							BASALT, very soft to soft (RO-R1), gray, fractured, gently sloping bedding, damp to moist (Columbia River Basalt)
4	2.5						
5							
6							
7							
							Test pit terminated at 7.5 feet bgs due to refusal on apparent bedrock
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

Date Excavated: 04/18/2022

Logged By: ABC

Surface Elevation: 357 feet

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Portland, Oregon 97224
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TEST PIT LOG

Project: Chinn Property Subdivision Sherwood, Oregon	Project No. 22-6032	Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Torvane Shear (tons/ft²)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
							Organic SILT (OL-ML), brown, moist, low plasticity, fine roots extend to approximately 8 inches (Topsoil)
							SILT (ML), brown to reddish brown, stiff, moist, low plasticity (Residual Soil)
1	2.0						
2	2.0						
3	3.0						
4	3.0						
5							
6							
7							
8							Grades to with weathered rock fragments
9							
10							BASALT, very soft to soft (RO-R1), gray, fractured, gently sloping bedding, damp to moist (Columbia River Basalt)

Test pit log continues on next page

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: 04/18/2022
Logged By: ABC
Surface Elevation: 364 feet

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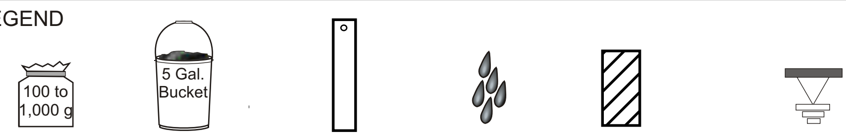
TEST PIT LOG

Project: Chinn Property Subdivision Sherwood, Oregon	Project No. 22-6032	Test Pit No. **TP-2 Cont.**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Torvane Shear (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	**Material Description**
11 12							*Test pit log continued from previous page*
13 14 15 16 17 18 19 20							Test pit terminated at 12 feet bgs due to refusal on apparent bedrock

LEGEND



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

Date Excavated: 04/18/2022
Logged By: ABC
Surface Elevation: 364 feet

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TEST PIT LOG

Project: Chinn Property Subdivision, Sherwood, Oregon	Project No. 22-6032	Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Torvane Shear (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0							Organic SILT (OL-ML), brown, moist, low plasticity, fine roots extend to approximately 8 inches (Topsoil)
1							SILT (ML), brown to reddish brown, medium stiff, moist, low plasticity (Residual Soil)
2							Grades to with weathered rock fragments
3							BASALT, very soft to soft (RO-R1), gray, fractured, gently sloping bedding, damp to moist (Columbia River Basalt)
4							
5							Test pit terminated at 5 feet bgs due to refusal on apparent bedrock
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

Date Excavated: 04/18/2022
Logged By: ABC
Surface Elevation: 294 feet

GEOPACIFIC
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Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Chinn Property Subdivision, Sherwood, Oregon	Project No. 22-6032	Test Pit No. **TP-4**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Torvane Shear (tons/ft²)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0							Organic SILT (OL-ML), brown, moist, low plasticity, fine roots extend to approximately 8 inches (Topsoil)
							SILT (ML), brown to reddish brown, medium stiff, moist, low plasticity (Residual Soil)
1	2.5						
2	3.5						
							Grades to with weathered rock fragments
3	2.5						
							BASALT, very soft to soft (RO-R1), gray, fractured, gently sloping bedding, damp to moist (Columbia River Basalt)
4							
5							Test pit terminated at 5 feet bgs due to refusal on apparent bedrock
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

Date Excavated: 04/18/2022
Logged By: ABC
Surface Elevation: 316 feet

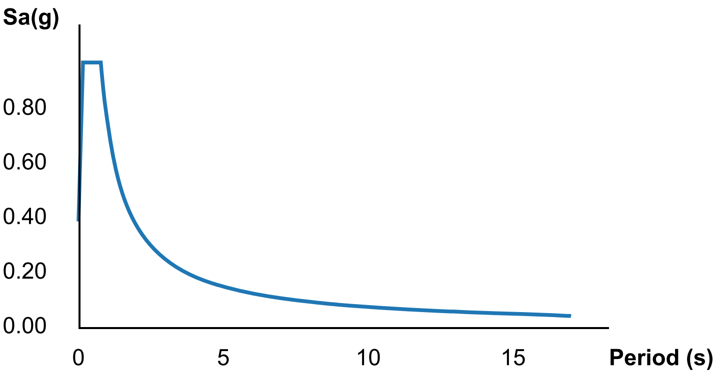
SITE RESEARCH

Hazards by Location

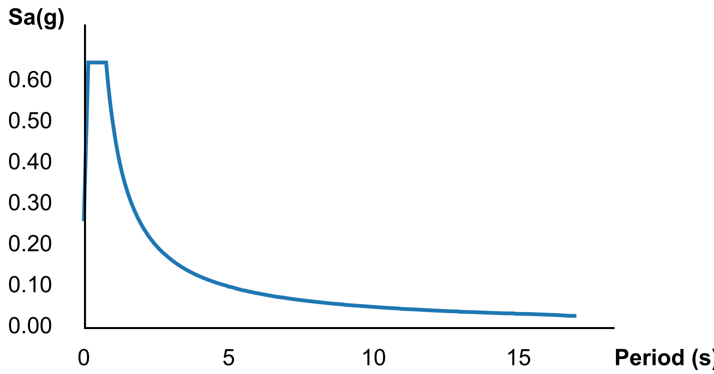
Search Information

Coordinates:	45.35013873757507, -122.82568442618103
Elevation:	333 ft
Timestamp:	2022-04-20T15:47:02.506Z
Hazard Type:	Seismic
Reference Document:	NEHRP-2015
Risk Category:	II
Site Class:	D

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.83	MCE_R ground motion (period=0.2s)
S_1	0.391	MCE_R ground motion (period=1.0s)
S_{MS}	0.97	Site-modified spectral acceleration value
S_{M1}	* 0.746	Site-modified spectral acceleration value
S_{DS}	0.646	Numeric seismic design value at 0.2s SA
S_{D1}	* 0.497	Numeric seismic design value at 1.0s SA

* See Section 11.4.7

▾Additional Information

Name	Value	Description
SDC	* D	Seismic design category
F_a	1.168	Site amplification factor at 0.2s
F_v	* 1.909	Site amplification factor at 1.0s

CR_S	0.884	Coefficient of risk (0.2s)
CR_1	0.865	Coefficient of risk (1.0s)
PGA	0.379	MCE_G peak ground acceleration
F_{PGA}	1.221	Site amplification factor at PGA
PGA_M	0.463	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.83	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.939	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.391	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.452	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.602	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.7

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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PHOTOGRAPHIC LOG

Test Pit TP-1 Soil Profile – Typical Soil Profile